SILVER JUBILEE LECTURE
ON
APPLICATION OF SOIL MECHANICS WITH PARTICULAR REFERENCE TO PROBLEMS IN INDIA

by

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MR. CHAIRMAN, LADIES AND GENTLEMEN :

I am deeply grateful for the great honour bestowed on me by selecting me to deliver this lecture.

When the subject of this talk was proposed, I started wondering what should be covered in such a talk to such a distinguished gathering. Discussions with friends brought out that in such a lecture it will not be possible to refer to all the experiences by Soil Engineers in India or to discuss the technical aspects of various projects and soil problems with which I have been associated. They all agreed that it should be descriptive in nature briefly outlining some of the problems of interest to our soils and foundation engineers.

Today we are celebrating the Silver Jubilee of our Society and it may not be out of place to find out how Soil Mechanics has grown in India during these years and how important a role Soil Mechanics is playing in the Civil Engineering profession. Due to my intimate association with the growth of Soil Mechanics in the Calcutta area, it would perhaps be appropriate for me to discuss this aspect and describe the problems being repeatedly encountered there. This also happens to be an area where all aspects of Soil Mechanics can be most extensively used in Civil Engineering construction.

Growth of Soil Mechanics in the Calcutta Area

Prior to 1940, construction in Calcutta region was mostly confined up to 5-storey residential or office buildings. The only heavy structures were constructed for the construction of Calcutta Dock, which involved sinking of large numbers of wells and constructing retaining structures on top. There were cracks and shift of one of these retaining structures when the water-table in the Dock was lowered, but by and large the structures have stood the test of time[1].

Of the various monuments which have been constructed in this area, Victoria Memorial Hall is the heaviest one. Although there is no outward manifestation of any damage, it is reported that the structure settled more than 0.3 m (1 ft) during its construction.

Of interest to the Soil Mechanics people today is a 1·75 m diameter tunnel for cable lines constructed across the River Hooghly at a depth of 25 to 28 m from the ground level and 8 m below the deepest river bed. This tunnel is more than 400 m long and it took about 3 years (1929-32) to

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complete. For constructing this tunnel the Engineers sunk two bore holes, one on each bank and it showed identical stratification on both sides of the river with a 8 to 10 m thick layer of stiff clay below 23 m depth.

The first known application of the principles of Soil Mechanics for designing civil engineering structure took place in Calcutta around 1938 for the design and construction of the Howrah Bridge. The bore holes and various in situ tests enabled pre-determination of foundation levels for this structure. The bearing pressure at the foundation level, 27 m at Howrah side and 33 m at Calcutta side, was 1.0 kg/cm². The estimated settlement of the main piers was about 14 cm on Calcutta side and about 19 cm on Howrah side. Actual measurement agreed very well with the predicted values(2).

After the Second World War. the interest began to be shown in using Soil Mechanics and a large number of laboratories in the colleges and research organisations were set up. Under the able leadership of Dr. K.L. Rao, the lead was taken by Central Water and Power Commission in its application in the field, particularly in the design and construction of earthen dams. In the later part of the forties, work also started on stabilisation of soil for road and other work. Some excellent work has been done in these fields and our expertise is considered to be as advanced as can be found in Europe and America. It took some time before it started to be used for the construction of foundations of buildings and other structures, mainly due to the absence of organisations for doing deep sub-soil investigations on a scientific basis.

Around 1956 in the Calcutta Port area, some deep seated slips took place involving part of a building on the bank of the River Hooghly. Following this a renowned soil investigation contractor from the U.K. was employed to sink a series of deep bore holes in the place where deep slips took place and in other parts of the dock as well. These bore holes were sunk by shell and auger technique and various S.P.T. and vane tests were conducted along with recovery of undisturbed soil samples. The results were used to analyse the slips and to evolve remedial measures together with the selection of various techniques for supporting the structures on the river banks. All these solutions have been adopted and till todate there is no sign of distress in the buildings or further failure of the bank slope. In many respects this should be considered to be a real turning point and beginning of the application of soil mechanics in areas other than earthen dams and stabilisation of soil.

Around this time, in 1957, a major sub-soil exploraiton was undertaken at the Durgapur Steel Project site by a local organisation with the author as in-charge of this unit which subsequently developed into the first commercial Soil Mechanics unit in India capable of undertaking deep sub-soil explorations with up-to-date techniques.

The investigation carried out for CPC in 1956 revealed remarkable uniformity of thickness and properties of the various strata in the port area. Around 1960 a copy of the report interested some people of the Soil Mechanics profession and attempts were made to correlate and understand the nature of sub-soil conditions of Calcutta. Between 1960 and 1964 sub-soil exploration by the use of shell and auger technique was introduced on a wide scale. Deep bore holes were sunk at proposed multistoreyed building sites as well as in the River Hooghly for a second bridge across this river. These information together with geological studies, particularly that by Dr. Coulson, indicated remarkable uniformity of the deposits, down to great depth in and around Calcutta, over a large area[3,4,5].

The picture was contrary to the prevalent notion that Calcutta area is a part of the recent deltaic land formation and that the sub-soil stratification will be extremely erratic. It revealed that below the mean sea level to at least a depth of 30.5 m (100 ft) from the surface, the deposits have taken place in a large inland fresh water lake. The River Hooghly has cut through this lake and is younger than the deposits below the mean sea level. There has been a number of changes in the course of this river, which is basically bringing down sand and silts. In the normal Calcutta deposit seven distinct strata have been identified. The original lake deposit consisting of seven distinct strata has been termed as normal Calcutta soil, whereas that brought by the River Hooghly and its tributaries is called Hooghly River deposit. Though the stratification in and around Calcutta is uniform over wide areas, some erratic deposits have been found in many places along the various buried channels of the Ganga, part of which is now known as the River Hooghly.

In the fifties some tall buildings were constructed. Many of these showed distress and foundation problems. Except sinking some wash bore holes occasionally, no efforts were made to understand these foundation problems. When the Calcutta sub-soil picture was evolved the main sources of foundation problems became very apparent. At that time, some of us spent quite sometime to examine many of these problems and induced Central Building Research Institute, Roorkee, to undertake a series of field experiments to evolve sound solutions of the problems. In spite of great efforts by CBRI and some soil engineers in Calcutta area, we are still finding widespread use of erroneous solutions for foundations in this area. Because of its importance it is felt that it may not be out of place to describe some of these.

Foundation Problems in Calcutta Area

The following problems are likely to be of interest to all of us here :

(1) *Multistoreyed buildings more than 5 storeys high:*

(*a*) On raft

(*b*) On short piles

(*c*) On long piles.

(2) Damage to adjacent buildings:

(*a*) Due to settlement

(*b*) Due to negative friction on long piles

(*c*) Due to vibration caused by driven piling

(*d*) Due to bored piling.

(3) Problems in recently reclaimed areas.

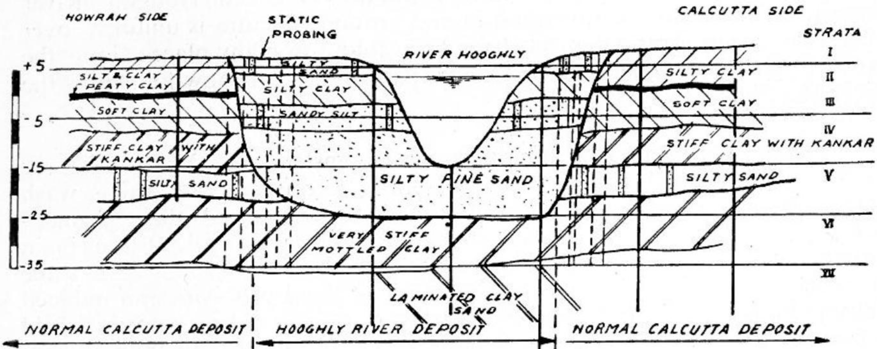
(4) Excavation for basements and sewers.

(5) High embankments and retaining structures.

MULTISTOREYED BUILDINGS MORE THAN 5 STOREYS HIGH

The study of the sub-soil will show that the formation above the mean sea level is generally a desiccated mass with low coefficient of compressibility. This deposit formed by the River Hooghly during flood time has undergone some action of desiccation by the sun and hence has attained characteristics of firm soil. Thickness of the formation is 5 to 6 m which corresponds to the maximum tidal variation in this area. Immediately underlying this desiccated crust, there exists a thin layer of peaty clay followed by soft to very soft clay to a depth of 13 m (42 ft). In most places there is an abrupt transition from this soft clay to a firm to stiff clay which is underlaid by stiff sandy clay or sand below (Figure 1).

Over the years many load tests have been conducted on the upper crust and its allowable bearing capacity has been *found to be about 10 tonnes/m²*. Buildings in Calcutta up to 5 storeys high are built on strip footings with



a) TYPICAL SOIL PROFILE ACROSS HOOGHLY RIVER AT CALCUTTA

	STRATA	L.L.	P.L.	N M.C	C kg/cm²	φ	MV cm²/TONNE	N BLOW/m
+5m								
CL	SILTS AND CLAY 1	VARIES WIDELY		31%	0.49	×	13.9	22.9–39.4
±0	PEATY CLAY 2	DO			0.35	0°	46.4	6.6–16.4
CL	SOFT CLAY 3	>45	20 TO 28	30 TO 50	0.35	0°	32.5	
-7								
CH	STIFF CLAY WITH KANKAR 4	50-22	19-25	20-30	0.7	0°	9.2	22.9–45.9
-15								
SM	SILTY SAND 5	VARIES WIDELY					9.2	16.4–164
-22								
CH	VERY STIFF MOTTLED CLAY 6	60-80	20-30	25 TO 35	Bh 1.75	0°	9.2	49.2–82

b) TYPICAL BORE LOG WITH SOIL CHARACTERISTICS

FIGURE 1.

this bearing capacity and by and large these structures have shown no distress. Figure 2 explains the reasons for satisfactory performance and small settlement of these buildings resting on strip footings near the surface.

Foundation problems start when the buildings are more than 5-storey and the bulb of pressure imposes heavy stress on the soft clay existing below 5 m depth. For buildings more than 8 storeys, the load becomes more than 10 tonnes/m² (1·0 ton/ft²) and the following solutions are normally adopted :

(1) Complete raft
(2) Isolated footings on short piles
(3) Isolated footings on long piles.

Figure 2 shows that a 15 m wide raft placed at 1.5 m depth will settle about 24 cm (9.5 in.) under 10 tonnes/m² load. For a 10-storey building the settlement will normally exceed 30 cm (12 in.)([6]).

In Calcutta short timber piles, 5 to 6 m long, are extensively used under building foundations. There is a notion that a ground capable of carrying 10 tonnes/m² load will safely carry an additional 5 tonnes/m² load when these timber piles are used at 75 cm centres. With this increased carrying capacity, 8 to 10-storey buildings are often founded on isolated footings on timber piles. Figure 3 shows that a 15 m wide raft on such timber piles is likely to settle even more than the same raft without timber piles. When such buildings are founded on isolated footings on timber piles with higher intensity of load, the soft clay existing below 5 m depth will be stressed even more and consequently there will be more settlement of the building.

The plan dimensions of these buildings are generally small and with the columns, floor slabs and walls, these act as very rigid structures. Consequently, the structures behave reasonably well in spite of 25 to 37 cm (10 to 15 in.) settlement. The major problem is that most of these buildings go out of verticality. Such 8 to 10-storey buildings going out of verticality by 15 to 30 cm (6 to 12 in.) are very common.

The use of long piles for supporting tall structures is a satisfactory solution. In most places these are founded in Stratum V which consists of dense silty sand or stiff to hard sandy clay. So far many buildings have been constructed on such piles and all of them, including a 21-storey building, has been reported to be behaving well. However, it is imperative that some settlement studies are made on a number of such buildings, as well as studies are made to determine the soundness of these long driven cast in situ piles when large number of these are driven at close spacing. The fact that the cellular Howrah Bridge foundations imposing a net load of 10 tonnes/m² on Stratum VI have settled more than 15 cm indicates that 15 to 20-storey structures are also likely to settle even more than 15 cm. So long as these structures behave as rigid units, this order of settlement may not be detrimental to the structure itself, but when a narrow and long structure is involved, this aspect must be duly considered.

DAMAGE TO ADJACENT BUILDINGS

This is being caused by a number of causes as under

(*a*) *Due to Settlement of the New Structure*

It has been shown earlier that 8 to 10-storey buildings whether on

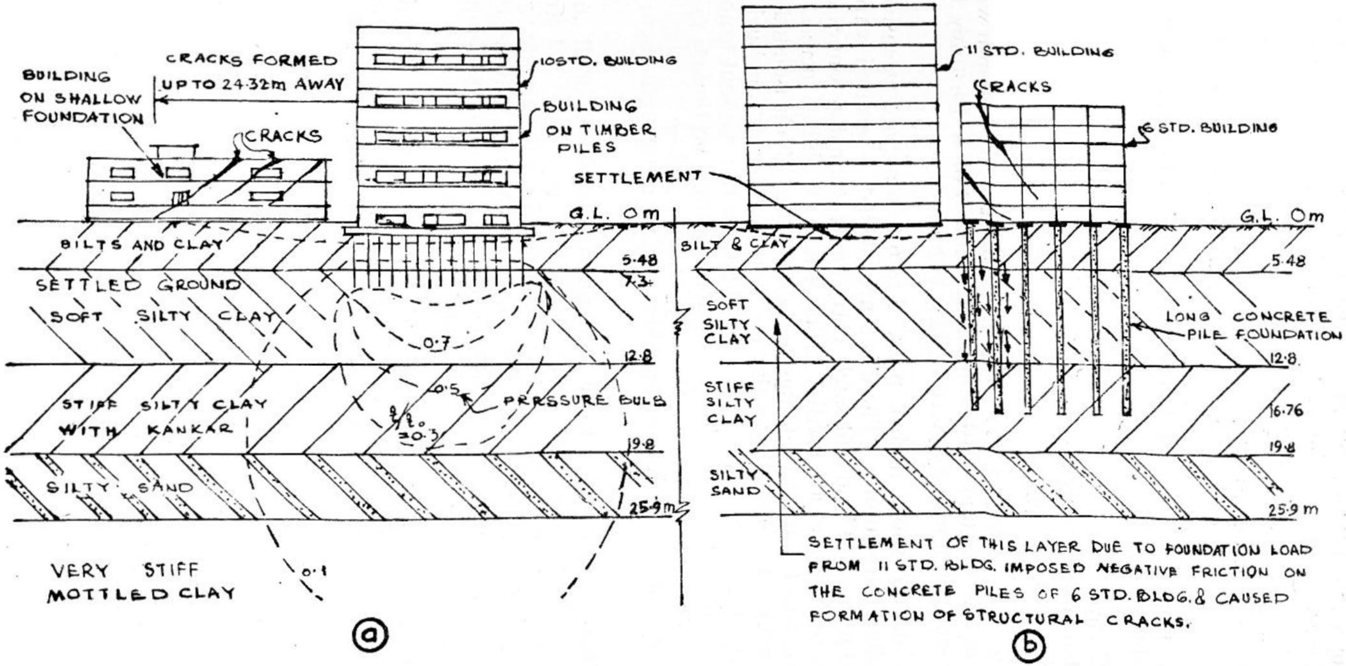


FIGURE 2 : Examples of foundations in Calcutta.

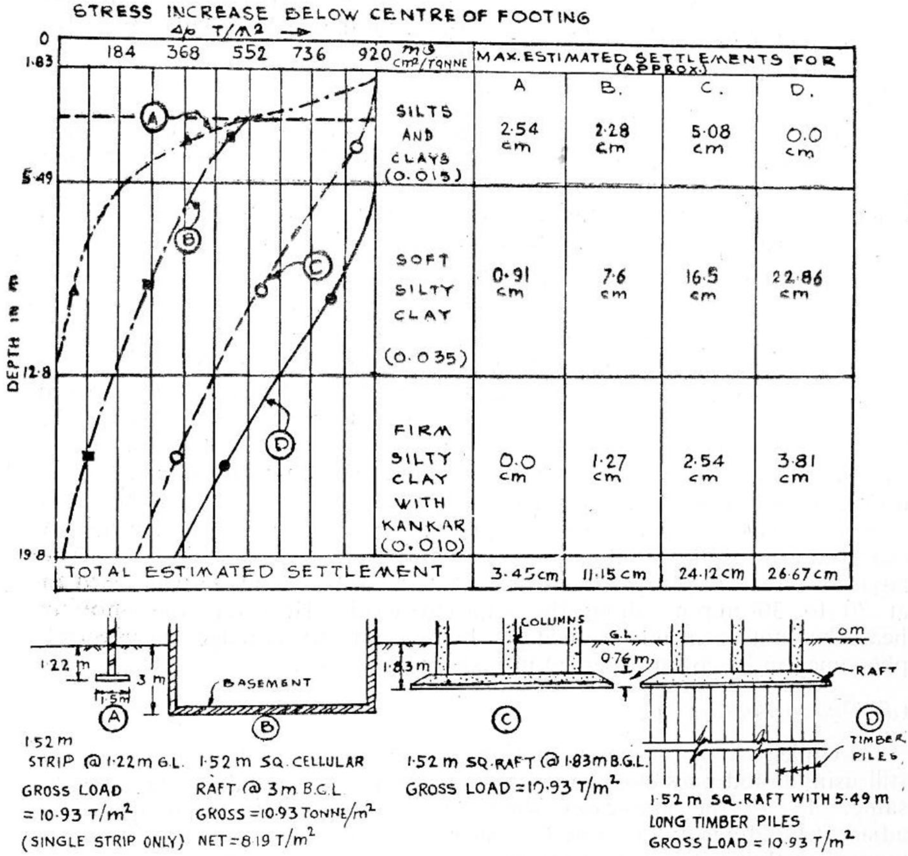


FIGURE 3 : Comparison of settlements for different types of foundations in Calcutta.

raft or short timber piles will settle 25 to 37 cm (10 to 15 in.). Under this magnitude of settlement the adjacent ground also settles appreciably, severely damaging the nearby structures. Judging by the cracks on the adjacent buildings, the settlement crater extends even 15 to 25 m from the new building.

In Calcutta there are innumerable examples of extensive damages to adjacent structures caused due to settlement of new buildings but still this defective practice is showing no signs of coming to an end. There is also no municipal law which can control such practice.

(b) Due to Negative Friction Caused on Piles

If the adjacent building is on short timber piles, the settlement of the new building will cause the adjacent building to settle as if it is not on piles. On the other hand when adjacent structure is on long concrete piles resting into Stratum IV, then heavy negative friction will be generated on these at the time of the settlement of the new building. This negative friction can exceed 50.8 tonnes (50 tons) on a 40.6 cm (16 in.) dia. pile. The effect of this pronounced negative friction can be very severe on the structure.

In recent years one such case has come to light, where the building on piles has developed wide cracks and severe structural damage.

(*c*) *Due to Pile Driving Operations*

The vibrations generated during driving operations do cause damage to surrounding buildings if these are old brick buildings. Vibrations generally cause the weak plasters to spall off and occasionally develop some cracks. Practically no damage is caused in the buildings if these are of good construction with cement mortar.

In an effort to solve this problem, CBRI made some studies near piling sites. They observed the effect of the pile driving on ground vibration by a simple and effective technique. Beakers, partially filled with water, were placed at the plinth and window *sill of ground floors of an* adjacent building when pile driving was in progress. It was observed that virtually no vibration was transmitted to the buildings when the hammers were dropped by less than 15 cm during driving of casing up to 14 m from the surface, corresponding to the lower part of the soft clay in normal Calcutta deposit. The drop of hammer can be increased thereafter without causing any appreciable increase in the vibration. It was further observed that if piling is done in this manner then the vibration caused to adjacent structures is even less than that caused by loaded trucks moving at 20 to 30 m.p.h. along the adjacent road. However, the effect of heave has not been fully studied yet, but so far no damage due to this phenomenon has been observed in Calcutta.

(*d*) *Due to Bored Piling*

Under a very erroneous notion, some engineers have used and are still using bored piles close to existing buildings to avoid damage to the same. In all the five cases which have come to light, damage to the adjacent buildings due to bored piling has been found to be substantially more than that would have happened if driven piles were used even without precautionary measures mentioned above.

Figure 4 indicates the phenomenon that takes place during bored piling operation in normal Calcutta deposit. When the bore hole is extended ahead of the casing through soft clay below 5 m depth, the soil squeezes into the open hole causing appreciable subsidence to the surrounding ground, particularly when a number of these piles are involved.

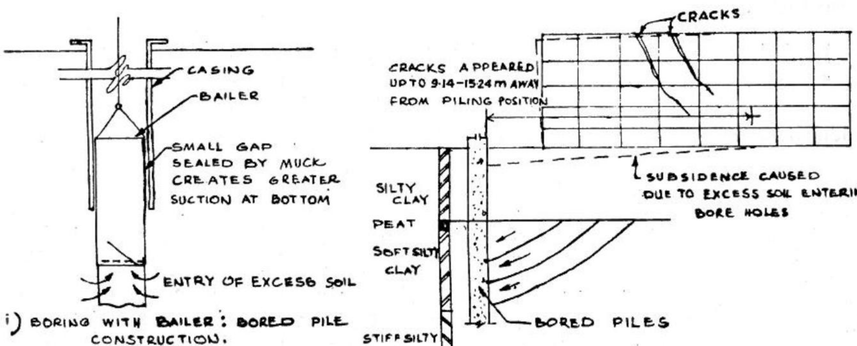


FIGURE 4 : Hazards due to entry of excess soil during boring for construction of bored piles.

It is difficult to drive 45.7 to 50.8 cm (18 to 20 in.) diameter casing ahead of the boring with normal bored piling equipment and in spite of every effort, the loss of ground from adjacent area due to the boring operation cannot be prevented. Records are available which show damages to buildings even at a distance of 15 to 20 m from the piling line. In spite of repeated occurrence of such examples, even now bored piling by normal boring method, is still being recommended by many engineers in this region.

PROBLEMS IN RECENTLY RECLAIMED AREAS

A large area (about 8 sq km) has recently been reclaimed in the North-Eastern part of Calcutta, by hydraulically filling with Hooghly River sand. The area consisted of large fishery ponds separated by bunds. The thickness of the sand fill varies considerably, the average is about 1.75 m thick. Investigations revealed that below this sand fill, pond bottom material consisting of soft to very soft clay extends to a depth of 3.5 to 4.5 m. This is underlain by normal Calcutta stratification at comparable elevations. It became clear that the upper 5 to 6 m thick desiccated crust occurring in most part of Calcutta is non-existent here and that heavy settlement will take place even in 2 to 3-storey buildings.

It was estimated that a 2-storey building may settle more than 15 cm where the thickness of the sand fill is about 1.75 m thick and a normal 4-storey residential building may settle even 30 to 40 cm (12 to 15 in.).

In 1967 a housing society wanted to build 24 Nos. 4-storey residential buildings each having a covered area of about 185.8 sq m (2,000 sq ft). Original project estimate was done on the basis of using open foundations. When the sub-soil information became available, the use of open foundation was ruled out. It was then estimated that use of concrete pile foundation will add about 25 percent to the cost of the building. As an alternative to this, use of sand drain and pre-loading was considered, but the cost of normal sand drain was such that it was still going to add about 18 percent to the cost of the buildings. It was at this time that I invented Sandwicks which drastically reduced the cost of sand drains, and the cost of Sandwicks and pre-loading added a mere 6 percent to the cost of the housing project(7).

Installation of sandwicks and pre-loading was done in a very systematic pattern and the maximum settlement for a uniformly distributed load of about 7.65 $tonnes^1/m^2$ (0·70 ton/ft^2) at the surface was about 58.4 cm (23 in.), with a differential settlement of about 17.8 cm (7 in.). The building project has been completed in 1970. None of the buildings showed more than 1.3 cm ($\frac{1}{2}$ in.) of settlement, which also took place during the construction period only. Sandwicks were used at 1·5 m centres and complete settlement took place within 32 days. Contrary to the prevailing notion, the use of vertical drainage channel and pre-loading and subsequent construction was done at a faster rate than what would have been required if normal driven concrete piles were used, (8) (Figure 5).

In an adjacent area experiment was carried out by putting on load equivalent to 2-storey building. Settlement was observed over a period of 600 days when a maximum settlement of more than 20 cm was recorded. This clearly indicates that estimated settlement was quite reasonable(9).

It is often mentioned that residential buildings normally coming up in Calcutta are relatively small in dimension and with the walls and columns act as a rigid unit and hence the differential settlement cannot be to that extent to cause damage to the structures. Whilst this has great validity,

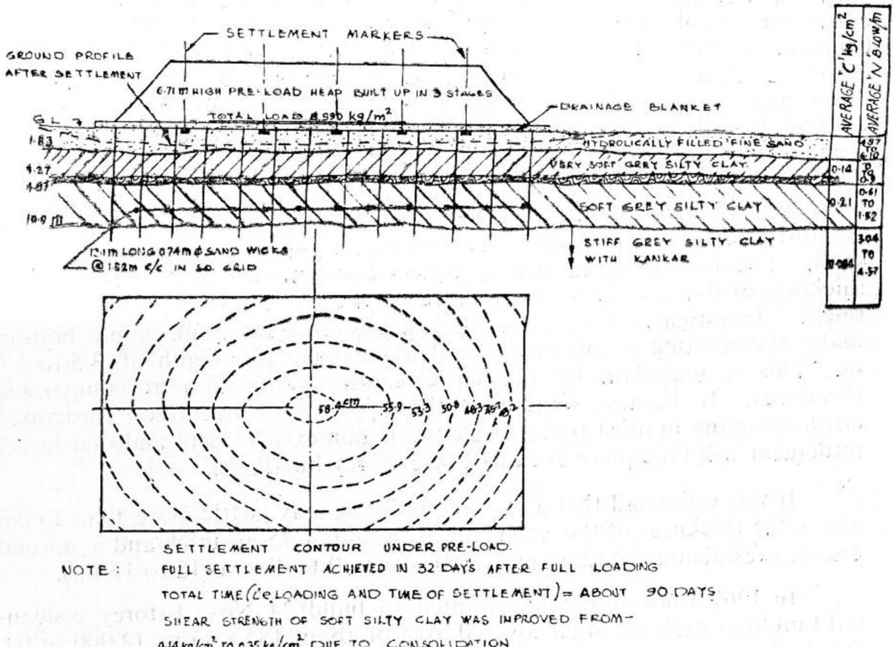


FIGURE 5 : Pre-consolidation of soft clay layer at salt-lake, Calcutta by sandwick and pre-loading technique.

such buildings going out of alignment when the settlement is 25.4 to 38.1 cm (10 to 15 in.) cannot often be prevented. Although some remedial measures can be undertaken by pre-loading suitable parts of building when it goes out of verticality, however, it is doubtful if any owner would like to see a progressive settlement of his building over 3 to 5 years, of a magnitude which was indicated by the above pre-loading projects.

An interesting feature emerged from this study that if this upper sand could have been 3 to 3.5 m thick instead of an average of 1.75 m, then the sand carpet would have enabled 3 to 4-storey buildings to be founded on open foundations near the surface without any pile or ground treatment.

The major drawback of sand drain and pre-loading is the pre-loading part of the work. Engineers are now directing their attention to the possibility of using 12 to 14 m long under-reamed concrete piles or long timber piles.

EXCAVATION OF BASEMENTS AND SEWERS

The sub-soil up to a depth of 6 m has been formed geologically in recent years by the action of the River Hooghly (Adi Ganga). This horizon consists of desiccated silts and clays with large number of layers or laminations or lenses of sand. During the dry season the water-table in the sand below Stratum IV go down more than 3 m (10 ft), but in the residential areas, the leakages through the water and sewer pipes, keep the ground saturated with water almost throughout the year. This results in inflow of water even in shallow excavations. Where the lenses or layers of sand are thick, excavation often becomes a serious problem.

In the dry season excavations up to about 3 m (10 ft) depth can be made vertically without any support if the ground does not contain any sand layer. Deeper excavations require some form of support.

Two modes of support are used extensively :

(1) Driving timber piles side by side, and

(2) Using timber planks side by side.

Timber piles are driven before starting excavation and are generally extended 2.4 to 3 m (8 to 10 ft) below excavation level. They act as cantilever piles. Timber piles are used more in places where strutting cannot be done. Timber planks are used along with strutting. These are driven down by manual sledge hammers and cannot be driven much below the bottom of the excavation. Both the methods have got a great weakness in that gaps are formed between planks or timber piles. These gaps allow ingress of soil along with water often causing heavy subsidence of the surrounding ground. Control of water is done by open pumping from the excavation. This is often not a very satisfactory method and results in a very poor concreting work.

In general the procedure being followed for deep sewer pipe construction is far from satisfactory. This is a field where Soil Engineers have not shown interest so far, but it should be of interest to the profession.

For excavations deeper than 5 m heave of the bottom should be duly considered. Failure to consider this led to the movement of ground which even pushed forward and deflected a large number of reinforced concrete piles at a site in Calcutta. Cracks on the ground showed the soil up to a distance of 30 m was moving when the excavation was about 8 m deep.

Because of the difficulties associated with excavations deeper than 3 m, deep basements are not popular in this area.

Calcutta Metropolitan Transport Project (Railways)(10)

Any talk about the application of Soil Mechanics in India today cannot be complete without some reference to this project. About 16 km length of underground construction will be required for this first underground Railway Project in India. The work will involve 10 to 12 m deep and up to 20 m wide excavation along the existing main roads in the most heavily congested areas of Calcutta. In many places the excavation will be within a few metres of existing multi-storeyed buildings, many of these are on shallow foundations near the surface.

Uptil now soil mechanics has been most extensively used on this project to select the types of construction that will be most economical and practicable with locally available expertise. Soil mechanics has also been used to identify the various construction problems and how to solve them. This is a project where soil engineers should play a most dominant role. Underground railways have been constructed in many cities around the world and their experience will be of great help to this project, but there are few problems which will require to be solved in a most ingenious way. A most extensive and thorough sub-soil exploration has been undertaken. Ultimately bore holes will be sunk at 30 m spacing. The construction will be undertaken by "cut and cover" method over almost 15 km length. About 1.0 km length will be tunnel under compressed air.

The sub-soil consists of normal Calcutta deposit along almost 13 km length and the Hooghly River deposit consisting 20 to 30 m thick formation of silty sand, almost from the surface, along the remaining 3 km length.

In the normal Calcutta deposit, the bottom of the excavation will be in the soft clay stratum and the proximity of adjacent structures will require adequate strutted support. The sheet piles or diaphragm walls will have to penetrate sufficiently deep to prevent bottom heave and for this purpose these may have to penetrate 7 to 8 m below 10 m deep excavation. Special method of construction will have to be evolved to reduce the bending moment on sheet pile or diaphragm wall with such deep penetration in soft clay. Over a substantial length of the alignment the silty sand of Stratum V will exist close to the bottom of excavation. This will require hydrostatic pressure in this stratum to be reduced by 5 to 7 m to prevent failure by bursting. The construction will require the hydrostatic pressure to be kept reduced over 3 to 6 months or more at any particular location. This will generate heavy settlement of the surrounding ground unless some effective system of re-charging is made to keep the water-table high outside the excavation (Figures 6 & 7).

In the areas with deep sand deposit, extensive heavy de-watering will be involved. At a few locations at the junction of the normal Calcutta soil and the river deposits, the soil consists of silts which may require stabilisation by vacuum well point or by electro-osmosis.

In the area where tunnel will be required, the soft clay will squeeze in the tunnel unless balanced by at least 2·0 kg/cm^2 pressure.

All along the underground section, the method of construction and estimates of the various structural sections must be absolutely safe. Failure of the supporting structure during construction or subsidence of ground during bottom heave, etc., may cause serious damage or even collapse of adjacent buildings.

In addition to the repeated occurrence of foundation problems mentioned above, there has also been heavy settlement and slip failure of high embankments and lateral movement of high abutments on long concrete piles. All these are related to railway overbridge and similar flyover structures[11].

From the discussions it will be appreciated that various foundation problems encountered in Calcutta area are all amenable for proper solution by the principles of Soil Mechanics and hence that area offers considerable scope to Soil Engineers.

Apart from Calcutta area, I had opportunities to examine a very large number of foundation problems in different parts of India.

It is not possible to discuss in such a short time all these foundation and soil problems with which I had been directly associated or many more problems which are being efficiently dealt with by other Soil Engineers in India. I would, however, like to mention only a few which I feel would be of importance to all of us here,

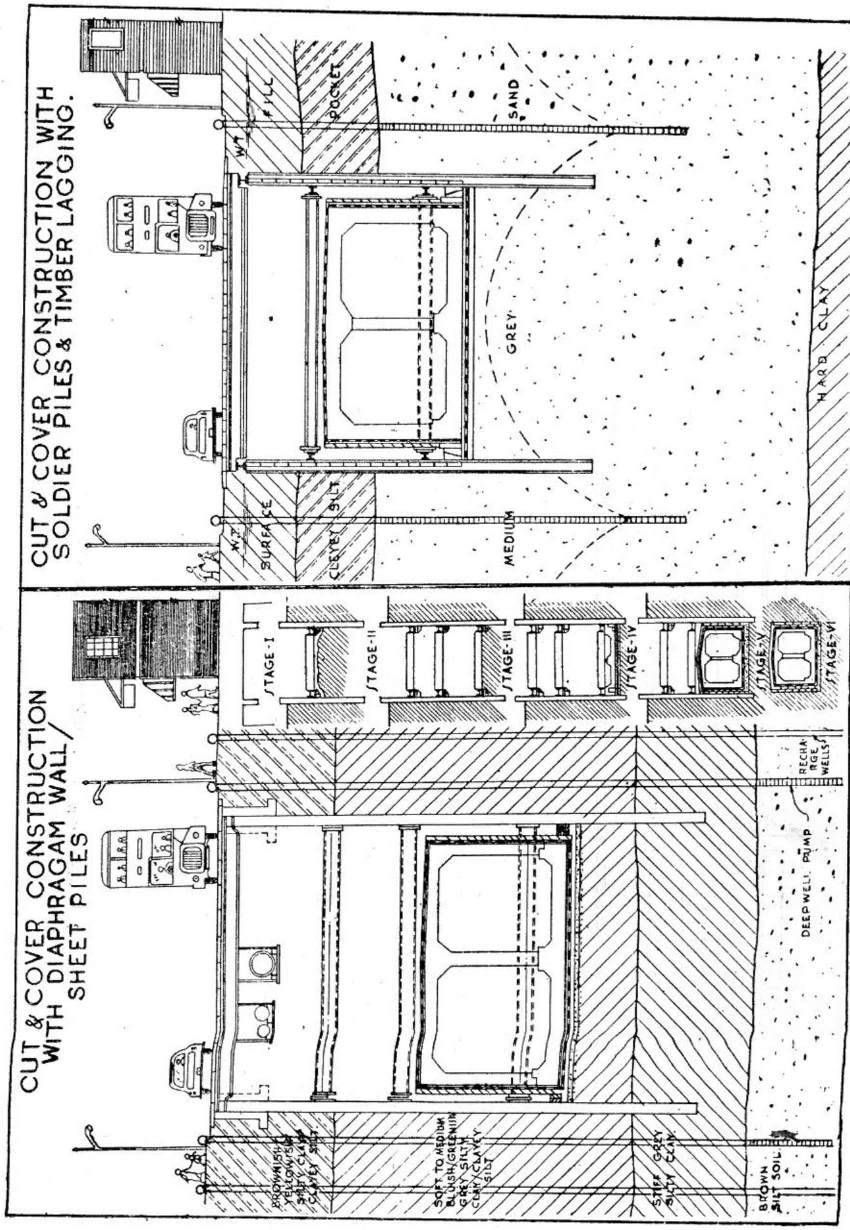


FIGURE 6.

NEGATIVE FRICTION IN PILES

Few years back Vizag Port and surrounding areas were a marshy tidal land. Over the last 10 years most of this land has been raised above high sea level. This has involved filling up 2 to 4 m over a very large area. Many areas were raised up by sand dredged from sea, whilst other areas by clay obtained by cutter suction dredgers and dumped on marshy area.

The sub-soil over bulk of the area is 2 to 4 m of fill overlying 15 m thick layer of very soft to soft clay. Underlying the soft clay there is a 5 m

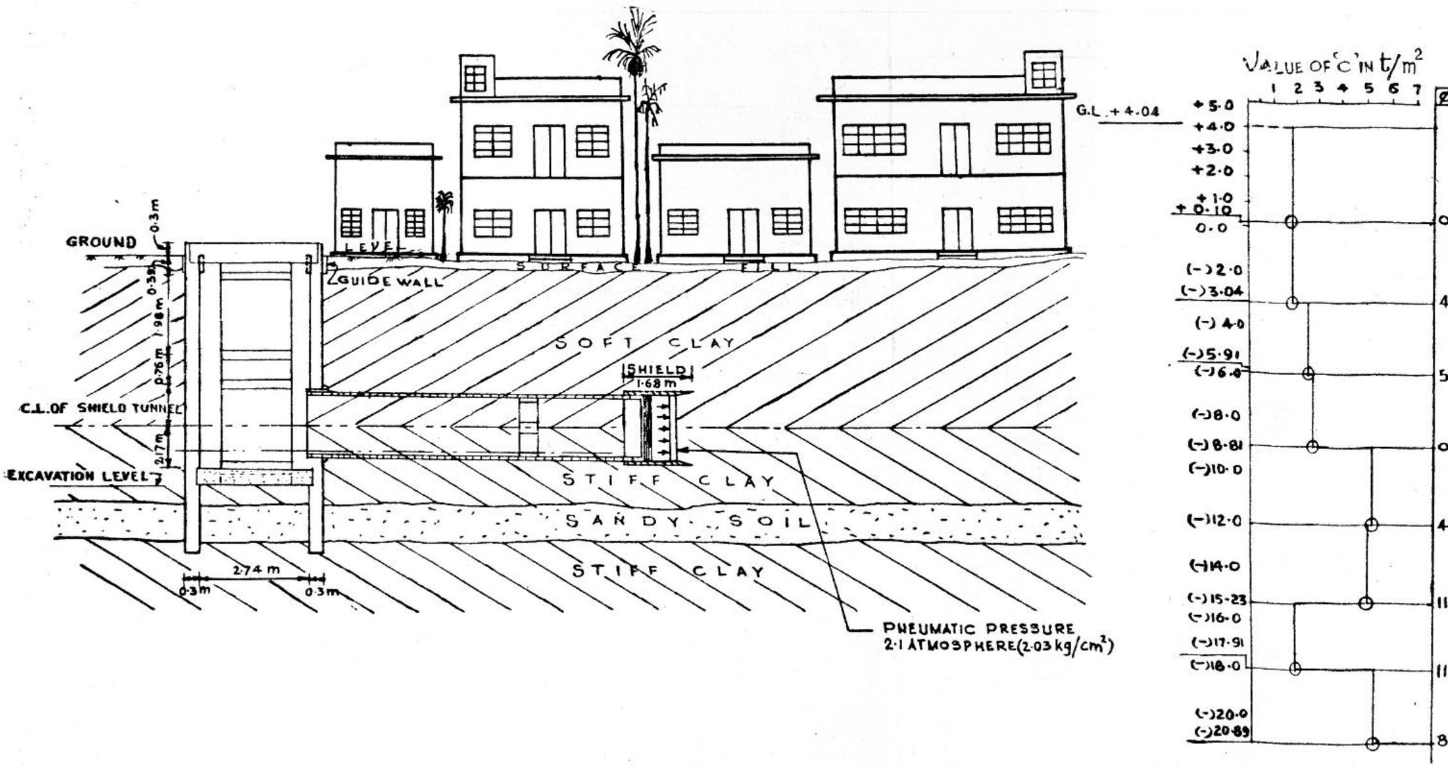


FIGURE : Shield tunnelling with compressed air.

thick layer of very stiff clay overlying sand, conglomerate or rock below. The shear strength of the soft clay ranges between approximately 0·07 kg/cm² near the top to 0·25 kg/cm² at lower levels. The permeability is about 10-8 cm/sec. and cm/cr is less than 3. The various known principles of soil mechanics are being extensively used here for the design and construction of dry docks, wharves, building foundations, etc. For the foundation of buildings piles are being used extensively.

The most important aspect of piles is the generation of heavy negative friction from the consolidation of soft clay underlying the fill. It has been estimated that the fill alone will cause settlement of more than 0·5 m over a very long period. It has been reported that some foundations on piles have settled more than 20 cm within a few years of construction. The piles were driven cast-in situ type. The bore holes indicated that they were resting on the stiff clay overlying conglomerate. Estimated negative friction based on current theories work out to 81.3 tonnes (80 tons) on 45.7 cm (18 in.) diameter piles and for safely carrying 30.5 to 40.6 tonnes (30 to 40 tons) load on 66 cm (26 in.) diameter piles are considered necessary. Negative friction causes maximum stress in the pile shaft at lower levels. It is imperative that the pile toe is clean and founded properly on hardstratum (Figure 8).

Detrimental effect of negative friction has also been observed in Calcutta and this aspect should be duly considered in foundation engineering.

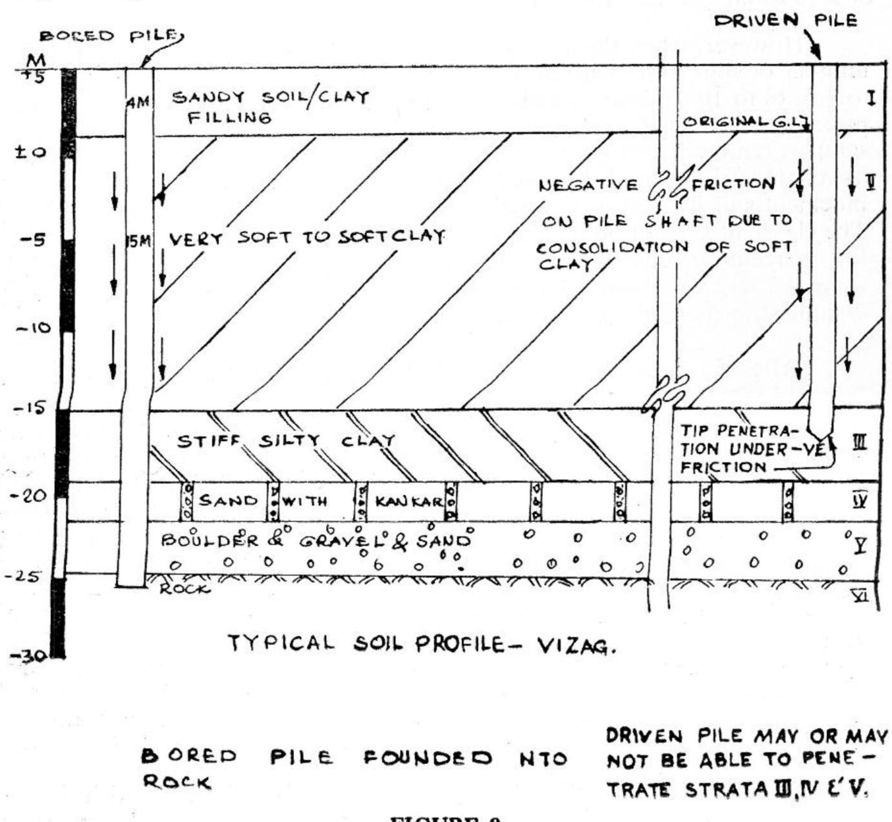


FIGURE 8.

EMISSION OF GAS FROM PERVIOUS STRATUM

At a site near Naharkatia in Assam, sub-soil consisted of 3 to 5 m of stiff clay overlying 20 to 25 m of soft to firm clay. This was underlain by a layer of sand. At all locations as soon as the bore-hole reached the sand layer, gas started to come out with great force. It did not subside even after a few months. Gas used to come out through the bore holes even after filling these with soil.

The problem was how to determine the effect of the emission of gas from the sand layer on the settlement of soft clay overlying it. No satisfactory answer could be found at that time. It was apprehended that the reduction in the pressure of gas may induce heavy settlement of soft clay and may cause trouble to the sophisticated plant to be constructed above. The site was abandoned and the plant was constructed on a better site nearby.

SLICKEN-SIDED CLAY AT DURGAPUR(12)

At Durgapur steel plant site a bluish grey clay of considerable thickness extends over a third of the site. Sub-soil investigation was carried out by normal shell and auger equipment and foundations were designed on the basis of laboratory tests on tube samples obtained by conventional open drive sampling technique. It showed an allowable bearing capacity of 27.3 tonnes/m^2 (2·5 ton/ft^2).

However, when the excavations for footings were opened up, a large number of slips took place along smooth shining circular planes in pits 2.4 to 3 m (8 to 10 ft) deep. Blocks of soil removed from the pit crumbled into pieces by slight jerking showing innumerable smooth surfaces. Small block of samples remoulded or even when squeezed in the palm completely changed its characteristics and behaved like very stiff clay. Tests on unconfined blocks of soil showed substantial increase in shear strength on remoulding. The clay had L.L. of 50-65 and P.L. of 20-25 and natural moisture only a few percentage above P.L. Close examination of disturbance during sampling technique showed that the soil loses its secondary structure and assumes the characteristics of an intact clay.

After the excavations were studied, it was realised that normal sampling and laboratory testing techniques will not lead to a correct assessment of the properties of this soil and that direct load tests will be the only means for this purpose. It was at this time that a series of plate load tests followed by 2 Nos. footing [1.4 m $\times$ 1.5 m (5 ft $\times$ 5 ft)] load tests were conducted. Whereas laboratory tests indicated the failure load to vary from 7 to 10 kg/cm^2, all the load tests showed failure load between 2·5 to 3·1 kg/cm^2. The serious consequences that could have followed if designs were based on laboratory tests became quite apparent. Settlement characteristics observed on footing tests were also peculiar. At every load increment, 30 to 80 percent of the total settlement took place immediately on application of load, followed by typical time settlement curves for saturated clays. The load tests were conducted after complete settlement took place at each stage of loading. The settlement predicted by laboratory tests were also substantially more than that actually observed on the footings (Figure 9).

The various observations clearly indicated that the clay with intense secondary structures and slicken-sides cannot be investigated by sampling

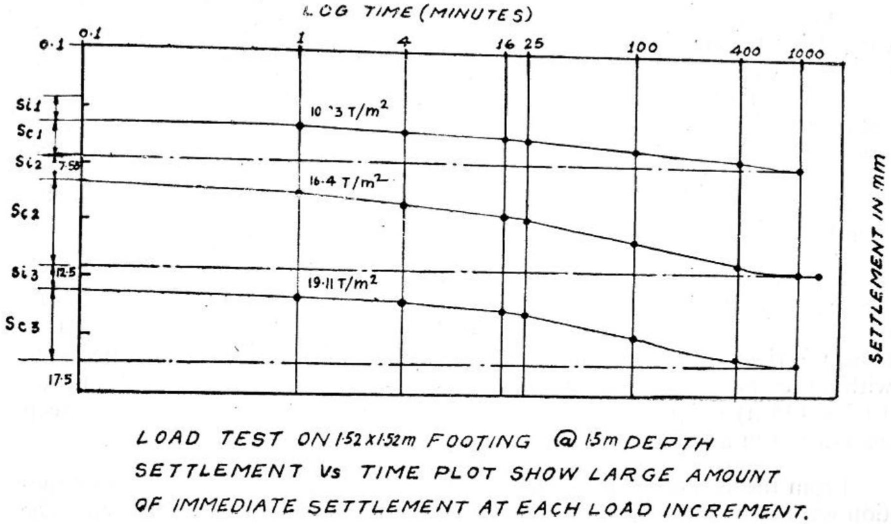


PLATE SIZE (m)	TEST DEPTH (m)	SAMPLE DEPTH BELOW PLATE (m)	L. L.	P. L.	N. M. C.	COHESION FROM TUBE SAMPLE	ULTIMATE FAILURE LOAD TONNES/m² LABORATORY	ACTUAL
0·3x0·3	1·52	0·15–3	56	19	23	1·05 kg/cm²	77·6	30·6
0·61x0·61	1·83	0·15–3	52	21	21	1·19 kg/cm²	90·72	28·42
1·52x1·52	1·52	0·15–3	57	20	22	0·84 kg/cm²	85·25	33·9

FIGURE 9 : Experience with slicken-sided clay at Durgapur.

and testing procedures in normal use which are essentially applicable for intact clays. It is desirable that some research schemes are undertaken to determine the best methods to identify such soils and also evolve in situ and laboratory tests to determine the characteristics.

HALDIA DOCK

This is one of the few places in India where sub-soil exploration results and Soil Mechanics have been used both for the design of this giant structure and for selecting the construction procedures. The sub-soil over most of the construction area consists of 6 m (20 ft) thick layer of silty clay at the top followed by 5 to 6 m (15 to 20 ft) thick layer of silty fine sand. This is underlain by 12 m (40 ft) of firm clay overlying firm sandy clay and sand below. The elevation of the sand layer varies considerably across the site.

The construction sequence required open dry excavation to a depth of 22 m (70 ft) from the surface and within 1·5 to 6 m (5 to 20 ft) from the sand layer. This required lowering the water-table by about 18 m (60 ft). The permeability of the sand stratum varied from 10^{-2} to 10^{-4} and the thickness from 3 to 12 m (10 to 40 ft). Calculations were made to determine the probable spacing and capacity of the deep well pumps and these agreed very well with that actually required at site to do the job. These deep well pumps having capacity varying from 300 to 3000 gph were installed at a spacing of 6 to 9 m (20 to 30 ft) around the entire site.

One interesting problem was observed. The discharge of many wells was getting progressively reduced with time and within 3 to 6 months, many of these used to get defunct. Every possible mechanical procedures for reviving the wells failed. At this stage when a replacement well at a distance of 3 m (10 ft) from a defunct well produced no water, but another one 6 m (20 ft) away produced water in sufficient quantity, the possibility of the sand stratum around the wells getting chocked with some chemical salts became very apparent and subsequently all wells were regenerated by chemical means.

The excavation was 22.9 m (75 ft) deep from the surface at river end of the structure. At the highest tide the water-level in the river was 0.6 to 0.9 m (2 to 3 ft) from the top surface. The excavation was about 61 m (200 ft) from the river. It was estimated that a safe open excavation cannot be made within the space available and subsequently the work was carried out by 10.7 m (35 ft) deep open excavation followed by a 12.2 m (40 ft) deep excavation in a braced double wall sheet pile coffer dam (Figure 10).

From the various difficulties it emerged that the problems of construction were not thoroughly examined at the time the design was finalised. The function of the Soil Engineers cannot be over just by determining bearing capacity and settlement characteristics, but we should study and guide each and every aspect of design and construction of such structures.

ACTION OF SULPHATE ON CONCRETE

Only a few years back, at a piling site in Bombay, load tests were carried out on 36 piles, 30 of these failed, most of them below the design load. Some of the piles were then pulled out and it was observed that the concrete in the shaft was very soft and could be removed by hand even in piles constructed 5 to 6 months earlier.

Following this, tests were made to determine the sulphate and other chemicals in soil and ground water when it was detected that sulphate in the ground water was more than 7000 ppm. It was considered that such high sulphate content severely attacked the green concrete leading to the failure of the piles. There were also other arguments for the failure of these piles, as near the site there are many buildings satisfactorily standing on similar bored piles constructed only a few years back.

In another site in Bombay trial bored piles constructed by two different agencies failed in testing. The reason for this was not properly investigated. However, the ground in this area is also known to contain high sulphate.

In many parts of India the sub-soil is known to contain more than 3000 ppm. sulphate content. In many areas close to chemical plants, the sulphate is known to be increasing with years. It is, however, not known whether along with sulphate, soil and water also contain other chemicals which will retard or prevent sulphate attack, particularly in areas close to the sea. Considerable controversy has started on this issue and various piling solutions are being offered by different agencies. According to the known codes of practice, in such high sulphate bearing soil bored cast-in situ piles with normal Portland cement should not be used but some are advocating that this can be used if the bore-hole is stabilised by bentonite instead of with water and casing. So far no reference to such a solution has been found. According to the published information, the best solution is

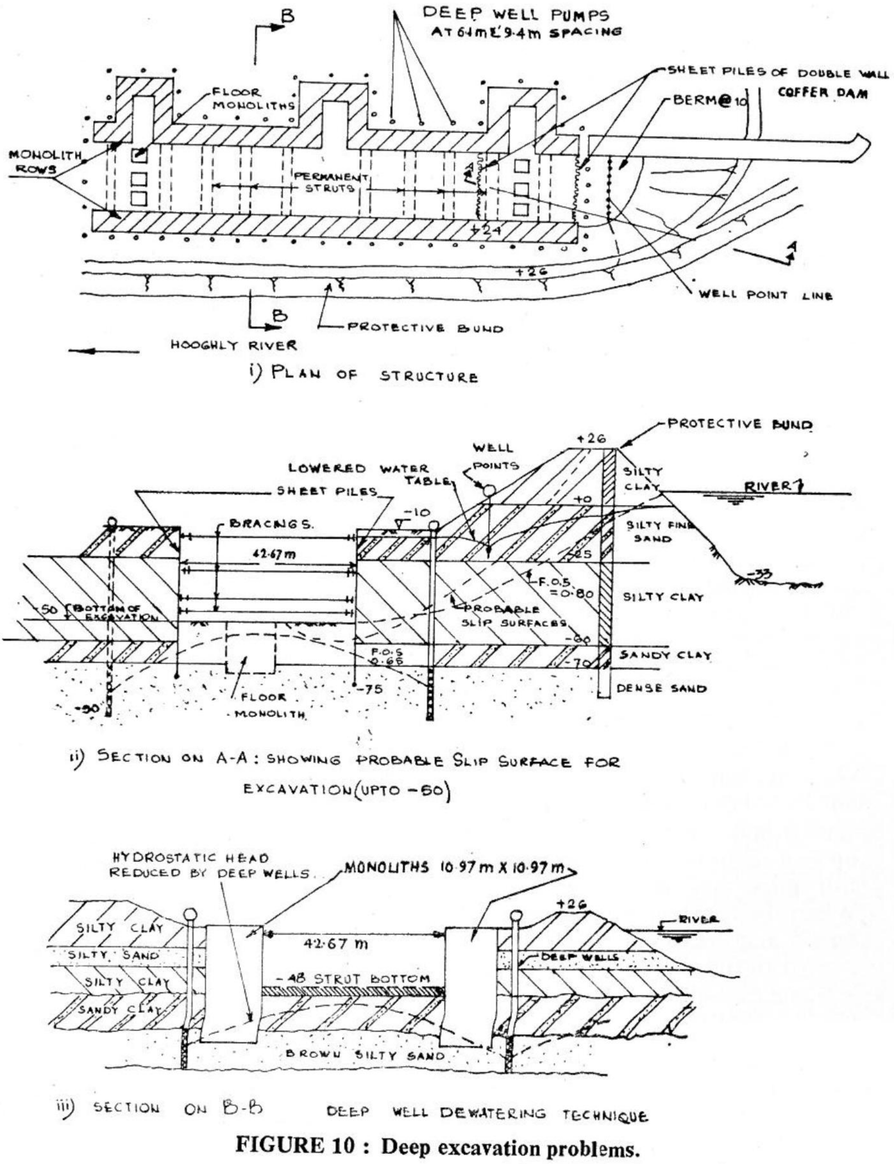


FIGURE 10 : Deep excavation problems.

to use precast piles with high grade concrete and suitably coated with bitumastic or fibre glass compound or the use of sulphate resistant cement, not normally available in India. This will more than double the foundation costs as compared with normal bored piles with normal Portland cement.

It is getting very imperative that some detailed investigation is done in this matter and some proper guide lines are issued to the foundation engineers.

DEWATERING FOR CONSTRUCTING HYDRAULIC AND OTHER STRUCTURES

It has been my privilege to prepare schemes for dewatering for three major structures such as docks and barrages and four minor structures for small canal and pumphouse structures. Most of these dewatering schemes had to be made only after contracts were signed. According to the contract the contractors are to be paid at a fixed unit rate for pumping per horse-power hour, subject to a ceiling of the dewatering cost.

In all the projects I had to examine, sub-soil information were utterly inadequate and except in one case no in situ permeability tests were conducted. Again when the problems are referred for solution, the construction is about to start. This does not allow time to make any field study and proposals are formulated on a lot of assumptions and guesswork.

Control of ground water often constitutes more than 20 percent of the project cost. Failure to assess the problem properly and mobilise suitable and adequate quantity of equipment has led to considerable delay on many projects. Neither the contractors nor the clients benefit from this situation. It is almost impossible to estimate the exact quantum of water to be pumped, but thanks to the work of Mansur, Kaufmann and others, it is now possible to prepare rational dewatering schemes and predict the quantum of water to be pumped provided satisfactory sub-soil information and permeability of strata are known. Technical and financial assessment of dewatering may also indicate that it may be better to adopt different types of sub-structure or control the water by cement and chemical grouting or provide positive cut-off by slurry trench or diaphragm wall, etc. As soils and foundation engineers, we should be more effective to select suitable sub-structure and guide and assist the construction in this field.

PREVENTING EXCESSIVE SETTLEMENT OF A WAREHOUSE[13]

A few years back a five-storey warehouse 61 m (200 ft) wide and 152.4 m (500 ft) long was constructed in Calcutta. Investigation was carried out by wash boring method and a layer of sand at 9 m (30 ft) depth was found. The bore holes were terminated when sand was encountered up to 18.3 m (60 ft) depth. The building was supported on driven cast-in situ piles and the structure was designed on columns and slabs. When the construction went up to 4 storey level, some cracks were noted on 1st and 2nd floor slabs. In spite of this the construction went on almost to full height when levels were taken which showed a crater type settlement with a maximum of 19 cm ($7\frac{1}{2}$ in.) at the centre and 3.8 cm ($1\frac{1}{2}$ in.) at the edges.

Further borings, with shell and auger technique were sunk up to 46 m (150 ft) depth, when it revealed that underlying the sand existing from 9 m (30 ft) to about 18 m (60 ft) depth there was a 7.6 to 9 m (25 to 30 ft) layer of firm clay. Underlying this firm clay at a depth of 27.4 m (90 ft), stiff to hard clay of normal Calcutta deposit existed (Figure 11).

The computed settlement agreed very closely with the observed values. It also indicated that if the structure is fully loaded, further settlement of about 20.3 cm (8 in.) will take place at the centre. The structural analysis indicated that it will fail unless further settlement is arrested.

A bold attempt was then made to provide a stepped basement in the middle 2/3rd area of the building. The deepest level was about 3 m (10 ft) below the original ground level.

With the commencement of excavation, further settlement stopped. The basement was completed and the building is now in operation for the last 6 years. Observation indicated no further settlement of the structure.

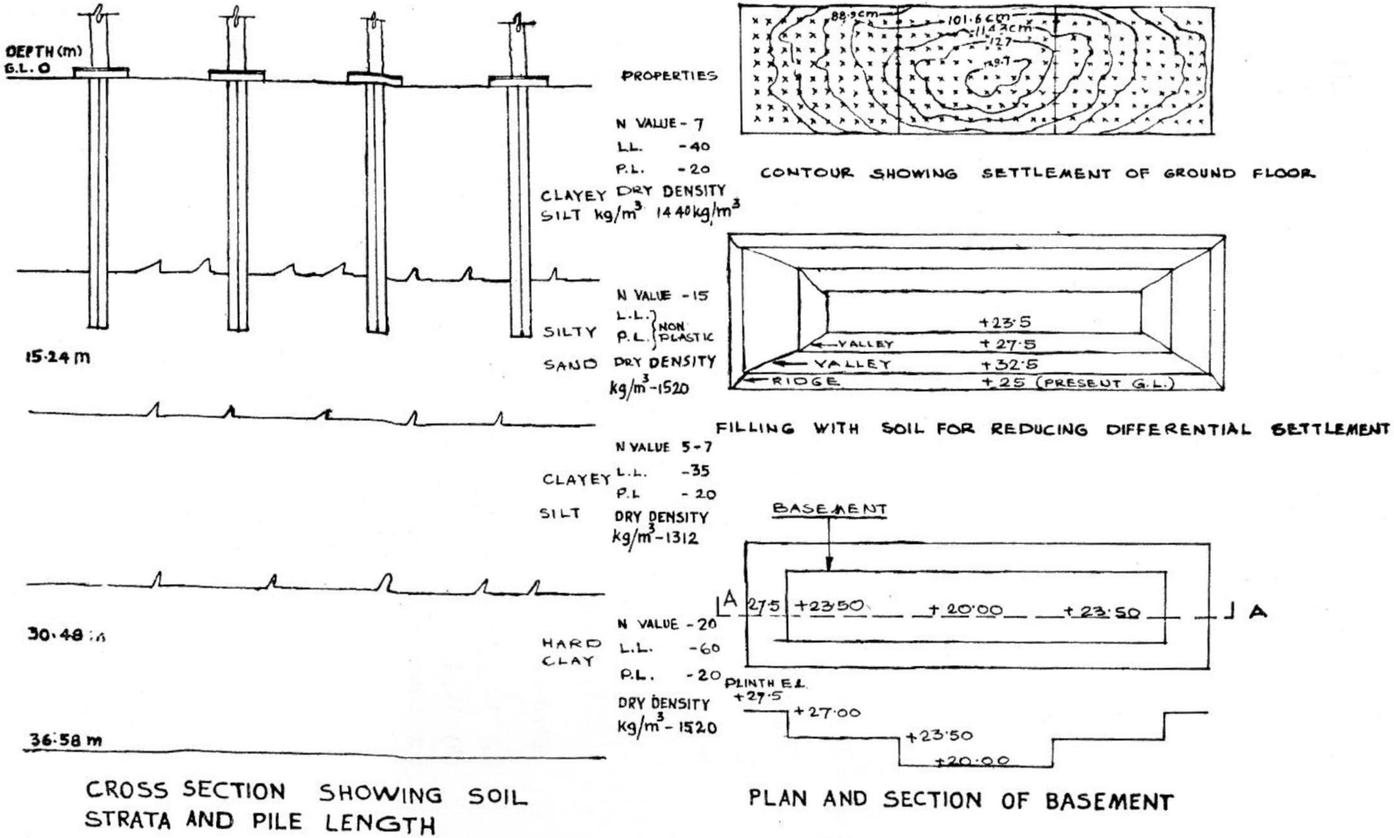


FIGURE 11 : Countering excessive settlement of a warehouse founded on piles.

RESIDUAL SOILS

Except the coastal tracts, most of the areas in peninsular India are covered by deep formation of residual soil—(1) Black cotton soil, and (2) Lateritic formation. Swelling characteristics of black cotton soil has been most extensively studied and satisfactory solution to foundation problems have been evolved. All these are well-known to the engineering profession. However, extensive studies on laterites in India have not been made so far. Mostly problems in this area are related to stability slopes. It is suggested that research institutes should take up some studies on this soil.

Some Aspects of Piling in India

Our discussions of foundation problems in India will be incomplete unless we discuss some aspects of piling work in India.

Prior to the Second World War, precast concrete and timber piles were the most extensively used piling systems. Since 1950, along with the increased tempo of construction activity, various piling systems have been introduced in India on a wide scale. Precast piles are now used on a limited scale for special purposes. The following systems are the most extensively used in India today :

(1) Displacement type

- (*a*) Timber piles mostly 3 to 6 m (10 to 20 ft)
- (*b*) Driven cast-in situ piles
  - (*i*) Simplex
  - (*ii*) Vibro
  - (*iii*) Franky.

(2) Bored cast-in situ

- (*a*) With bailer and lined with casing
- (*b*) With percussive tools stabilised with bentonite
- (*c*) Under reamed piles
- (*d*) Bored compaction piles.

The techniques of forming piles by the above systems are well-known to the engineering profession. There are many efficient and technical piling companies in India. By and large, structures on concrete piles are behaving well. However, some aspects of piling work have recently been examined by CBRI and others. It will be worthwhile to discuss a few of these findings here.

TIMBER PILES IN CALCUTTA

There is a notion that these piles compact the ground. Experiments carried out to check this notion revealed that far from compacting the ground, the soil actually becomes weaker. Figure 11 shows that after driving, the soil adjacent to the piles offered almost zero static cone resistance up to about 2 m depth in a ground which had offered substantial resistance before pile driving. There was also no evidence of compaction below this level.

The cause is simple. These piles are generally driven from the bottom of pits 1.5 to 1.8 m (5 to 6 ft) deep. Usually some water collects in these pits. On such a ground these 5.5 to 6 m (18 to 20 ft) long piles are placed vertically and the top is simply held by a man standing on a 2-legged frame, on the top of which rests a pulley used for dropping a manually operated hammer. The jerks from the dropped hammer try to shift the pile and the man holding the top continuously moves it back to its original position. In the process the ground surrounding the pile gets heavily disturbed and with water in the pit it becomes a soft mass. Only after the pile penetrates a couple of metres that it probably develops some fixity and penetrates the ground without lateral movement.

There is a notion that the carrying capacity of piles will increase with the pile cap. To check this a four pile group was tested with and without the pile cap resting on the ground. Figure 12 shows that ground within the pile group started to share some load only after the load on the piles exceeded 70 percent of their ultimate failure load. This series of tests were conducted on piles driven with proper guide and in dry pits. In actual field conditions the ground would not possibly have shared any appreciable load until the failure of the piles([14]).

It will be seen that in both these aspects, the prevailing notion is contrary to the actual facts, but still the wrong practice goes on.

DRIVEN CAST-IN SITU CONCRETE PILES

The purpose of piling is to transfer the load to satisfactory strata through sound concrete shafts. In India we are only using unlined concrete shafts. In many countries unlined cast-in situ concrete shafts are not permitted. This is because of the damage to green concrete when the casing is being driven at the adjacent location.

In an effort to examine this, some observations by CBRI did show 12.7 to 19 mm ($\frac{1}{2}$ to $\frac{3}{4}$ in.) lateral displacement and about 6.3 mm ($\frac{1}{4}$ in.) vertical movement of the adjacent piles. However, some piles extracted to a depth of 9 m (30 ft) in Calcutta did not show any crack or deformity on the shaft. Load test results also did not show any structural defects.

However, in a project near Durgapur a large number of driven cast-in situ piles were examined to 4.6 to 6 m (15 to 20 ft) depth. It was observed that except the last pile in a group, horizontal thin hair cracks have developed in all the other piles to a varying intensity, down to the full depth investigated. Another finding was that although the 40.6 cm (16 in.) diameter casing was driven with a 44.5 cm ($17\frac{1}{2}$ in.) diameter shoe, the shaft diameter throughout the depth investigated, varied between 35.6 cm and 39.4 cm (14 in. and $15\frac{1}{2}$ in.) only. Evidently the reinforcement cage with about 35.6 cm (14 in.) outer diameter did not allow the ground displacement to squeeze the shaft less than this size. Otherwise the problem could have been worse.

In another project in West Bengal, driven cast-in situ piles were exposed to a depth of 9 to 12 m (30 to 40 ft). Although the pile diameter was supposed to be more than 40.6 cm (16 in.), the shaft diameter was mostly below 38 cm (15 in.). In addition to this, segregated and weak concrete was found at many levels on many of the pile shafts.

As a point of interest it may be mentioned that in both these two cases, load tests showed the piles to be perfectly satisfactory. There are a

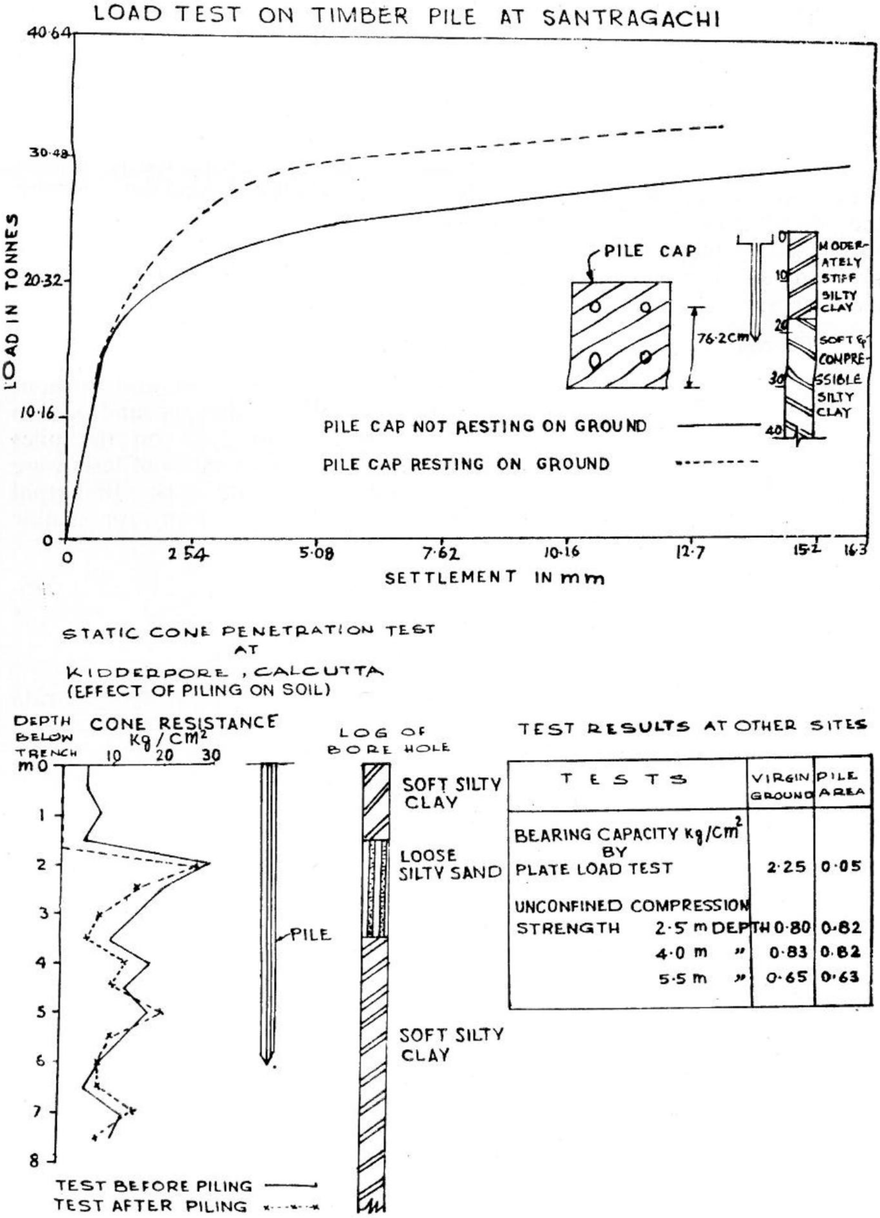


FIGURE 12 : Study on Salballah timber pile in Calcutta region.

number of problems for constructing such piles satisfactorily. The most important is the ingress of water often with silts and sand, through the joint between the casing and the expendable shoe. This generally happens when the piles are long and driven into a pervious stratum of sand. Even when there is no ingress of water and if the casing is not filled with concrete up to an appropriate height before lifting the casing from the shoe, the unbalanced hydrostatic head at the bottom may severely affect the green concrete. Piling foremen and operators are generally aware of the inflow of water and soil through the joints, but many of them are not aware of the consequences of unbalanced hydrostatic pressure when they pull out the casing initially. It would be of interest to all foundation engineers if some such piles can be examined right up to the toe. It may, however, be mentioned that except in extreme cases, routine pile tests will be unable to detect such structural defects of long piles at their lower levels.

The reduction in the diameter of piles is due to the stresses generated in the ground and movement of soil during driving of adjacent pile. The observations indicate that its effects could be severe on unreinforced part of the shaft. This aspect requires closer examination.

One important aspect of driven cast-in situ piles is the pre-determination of the safe carrying capacity by some formulae. In India Simplex and Hiley formulae are most widely used. A series of tests on more than 30 piles, 21.3 to 27.4 m (70 to 90 ft) long were conducted by CBRI. Tests were carried out by normal load test, by Housel, C.R.P. and other methods. Piles were tested to failure. Figure 13 shows the results of tests on a typical pile. The Table (on page 26) shows the comparative results by various methods. Some important findings were :

(1) The pile passes from elastic to plastic state when it settles more than 10.2 mm (0.4 in.).

(2) Housel tests indicate a very sharp break from elastic to plastic state at a settlement generally less than 5.1 mm (0.2 in.).

(3) Hiley formulae are very conservative and the capacity indicated by it corresponds more to the yield capacity than to the ultimate capacity. Consequently a factor of safety of less than 2 may be applied to determine the allowable safe capacity.

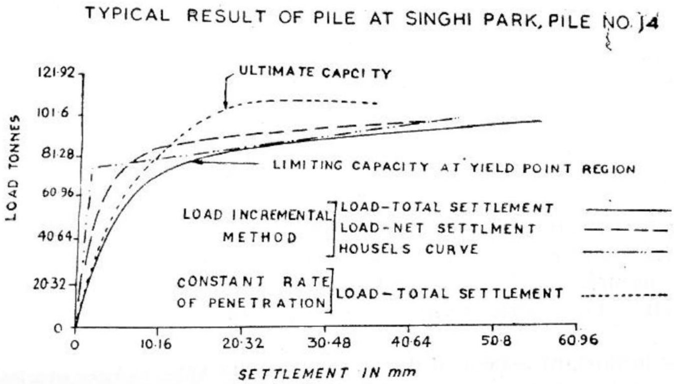
(4) Simplex formula overestimates the capacity, but the safe capacity determined with a factor of safety of 2·5 agrees very well with the test results.

Such findings create confidence in the pile driving formula, but so far no explanation could be found as to why the Hiley formulae consistently gave ultimate load lower than the actual test results (Figure 14).

BORED PILES

Except in special conditions, bored piles are used in cases where there is a hard, preferably rocky stratum within reasonable depth. For small diameter the most popular method is to use bailers and stabilising the sides of borings by casings. If the soil is firm and cohesive this method of boring proves to be satisfactory, but it has severe limitations if the soil consists of a thick layer of clean sand under water.

Bored piles founded on hard strata are considered to be end bearing piles. In recent years at a number of sites, in different parts of India, such piles were either pulled out or excavated right up to the toe. In practically every case so examined, 7.6 to 22.9 cm (3 to 9 in.) of bare reinforcement has been found to stick out at the bottom with soft and loose soil existing between the underside of the concrete shaft and the firm founding strata.



DIMENSIONS		DYNAMIC FORMULA		SIMPLEX FORMULA		LOAD TEST					
LENGTH m	DIA (cm)	ULTIMATE CAPACITY SIMPLEX	ULTIMATE CAPACITY HILEY	Fs	SAFE CAPACITY (T)	YIELD CAPACITY (T)	Fs.	SAFE CAPACITY (T)	ULTIMATE CAPACITY (T)	Fs	SAFE CAPACITY (T)
23·4	45·5	141	104	2·5	56	80	1·5	53	110	2·0	55

TABLE

A FEW TYPICAL LOAD TEST RESULT OUT OF MANY

DIMENSIONS		DYNAMIC FORMULA		SIMPLEX FORMULA		LOAD TEST					
LENGTH (M)	DIA (Cm)	ULTIMATE CAPACITY (T) SIPLEX	ULTIMATE CAPACITY (T) HILEY	Fs	SAFE CAPACITY (T)	YIELD CAPACITY (T)	Fs	SAFE CAPACITY (T)	ULTIMATE CAPACITY (T)	Fs	SAFE CAPACITY (T)
24·0	45·50	160	80	2·50	64	100	1·50	67	122	2·0	6·1
23·2	,,	147	84	,,	59	95	,,	63	125	,,	63
28·4	,,	140	87	,,	56	94	,,	62	111	,,	56
26·5	,,	118	87	,,	47	83	,,	56	95	,,	48
25·0	40·5	96	76	,,	38	60	,,	40	88	,,	44

FIGURE 13 : Pile-load tests in Calcutta region.

About 3 years back on a project in the U.K., toe of a four hundred bored piles resting on a rocky stratum were studied. There also practically without exception there was soft soil at the toe of the piles of as much as 0.45 m (1·5 ft) thick. Experiments led to the conclusion that the piles should be considered as friction piles and not end bearing piles.

This aspect has been examined recently and it has been found that one of the sources for this defect is the boring tool itself. The most popular boring tool is the bailer with a heavy flap about 7.6 cm (3 in.) above the cutting edge. With such tools it is not possible to clean the last 7.6 to 15.2 cm (3 to 6 in.) of the hole. In the borings which could be made reasonably dry, I have followed a simple procedure for cleaning the bottom. This consists of pouring a couple of buckets of concrete, vigorously sludging it with the bailer and removing the bottom material. In larger diameter holes, the procedure is generally repeated once more. In all the cases where I have followed this, excellent results were obtained. Tests showed complete recovery of the piles on removal of the load and were substantially better than the piles where such cleaning was not done.

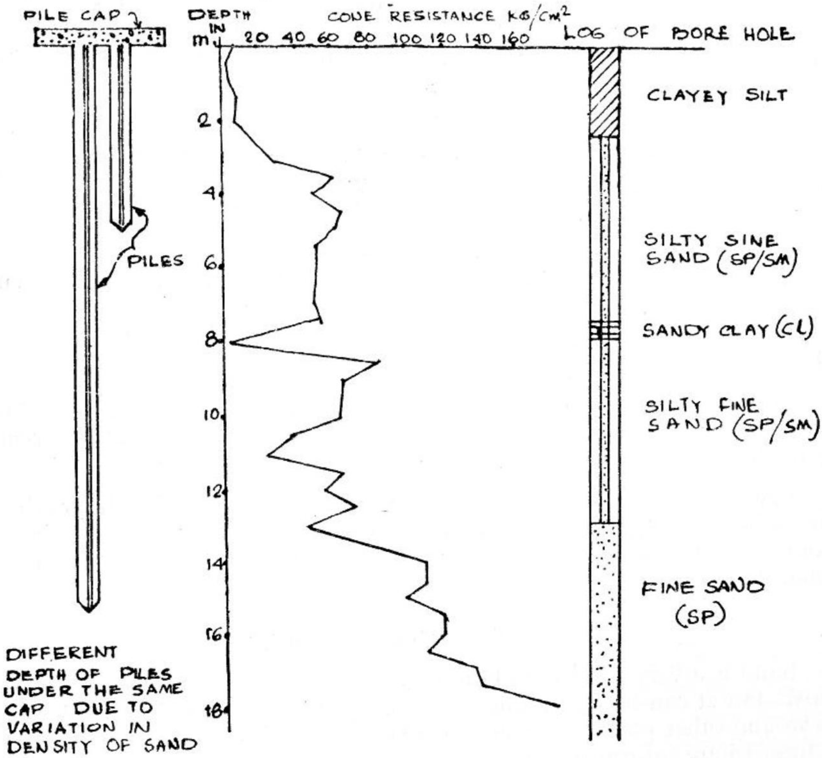


FIGURE 14 : River belt deposit of Calcutta and pile driving difficulties (Typical sounding test result where a pile could not be driven below 5 m depth).

However, it is felt that it would be premature to make any generalisation but the fact that the cleaning of the bottom of pile holes appears to be a difficult job and its importance should be duly considered, particularly if the holes are not dry.

Another serious problem for the bored piles is the construction of sound shaft, if the holes are wet.

The most simple procedure is to use a concrete placer. Extensive experiments have been carried out in the U.K. on this method of concreting. It has been observed that frequently concrete comes out of the placer only when it has travelled some distance up through the water. This and many other observations have led this method of concreting to be discontinued in the U.K. and is superceded by tremie method of concreting.

There are so many problems of cleaning and concreting that some authorities and consultants are now avoiding the use of bored piles in water bearing ground where the bore-holes cannot be made dry before concreting. Bored piling is a very simple technique, and it is felt that efforts should be directed to overcome the sources of weakness rather than any attempts to avoid the use of such piles.

The above discussions on driven and bored cast-in situ piles will lead one to appreciate that both these two widely used systems should be used with care, duly considering and overcoming the problems mentioned above, as well as many other problems which could not be discussed in such a short time and space.

Under-reamed short bored piles, which have been made popular by CBRI for excellent solution of foundation problems in black cotton soil area, have been examined by completely pulling out the piles. All of these were found to be perfectly satisfactory.

Bored compaction piles, recently introduced by CBRI, have also been found satisfactory so far in all respects.

Techniques for Improving Soils

In our discussions of soil mechanics problems we should mention some of the techniques for improving weak soils which can be of great help to the soil engineers.

Few techniques have become very popular in Europe for improving weak soils for supporting structures. Some of these have already been introduced in India and in some areas Indian engineers have made substantial new contributions.

DENSIFICATION OF SAND[16]

Sand is a very good foundation material provided it is dense, but in a loose state it can be dangerous for foundation. Experiments in Japan, Alaska and other places have shown that deposits of sand having SPT less than 15 are susceptible to liquifaction during earthquake or shock loads and silty sand with SPT of less than 10 should be considered unstable soil. Under the able guidance of Prof. Shamsher Prakash some very good work has been done at the School of Earthquake Engineering at Roorkee to determine the liquifaction potential of a sand mass.

Sand deposit liable to liquifaction should be densified. Densification also enables the sand mass to carry higher intensity of load and make considerable saving in the cost of the foundation. The techniques of densification of sand by driving pipes with expandable shoe or by simple compaction piles or by vibroflotation, etc., are known, but it is rather surprising that inspite of extensive deposit of sand mass in Northern and North-Eastern India susceptible to severe earthquake, densification of sand is not being used as a routine procedure. It will be worthwhile if the engineering profession utilises this technique more seriously.

IMPROVEMENT OF SOFT CLAY

Along the coast of India there are many places where there is thick deposit of soft clay. Mention may be made of a few important locations.

In Kandla there is a 10 to 12 m of thick layer of very soft clay underlying 2 to 4 m of recent fill or a desiccated crust. In Bombay around the Thana Creek Port, new Bombay city, proposed Nhava Sheba dock area, etc., there is extensive deposit of soft highly plastic marine clay, sometimes 15 to 20 m thick overlying hard rock. The upper part of this clay above the mean sea level has undergone some partial desiccation, but in many places particularly in Nhava Sheba area 10 to 15 m thick clay deposit is still undergoing consolidation under its own weight. In Cochin, 10 to 20 m thick layer of very soft clay underlying 2 to 3 m of surface fill makes it difficult to construct even a 2-storey building without heavy settlement and differential settlement. In Vizag Port area under a recent fill of 2 to 4 m there is a 15 to 20 m thick layer of highly plastic, almost homogeneous, soft clay. It is estimated that under the weight of the recent fill alone the clay will settle 0·5 to 1·0 m over long years. In Haldia, in Calcutta and in many other places of Bengal and Orissa, thick layers of soft clay have posed serious foundation and excavation problems. Whilst there are problems of foundation in other parts of India, but it is in these soft clay areas that I have encountered practically all foundation failures and more challenging problems as a Soil Engineer. Some of these have been described earlier.

One of the techniques of solving many foundation and excavation problems in soft clay is to improve the shear strength and consolidate the ground before putting structures on top. This is achieved by pre-loading the soil and accelerating the consolidation of the clay by installing vertical drainage channels. For a long time 4.9 to 6.1 m (16 to 20 ft) diameter sand drains used to be the only mode of forming the vertical channel until Kjellman invented the cardboard wick in late thirties. This is now a very popular method in Japan.

Following the theoretical studies by Barron, it was clear that the diameter of sand drains plays less important part than their spacings. Relationship between diameter and spacing showed that smaller diameter drains will have great economic advantages over larger diameter ones. Various efforts were made to reduce the diameter of sand drains but no satisfactory system was evolved to reduce the diameter less than 30.5 cm (12 in.). In 1967 when I was confronted with the foundation problems at Salt Lake in Calcutta, I invented a simple method of performing sand drain by filling sand in cylindrical jute bags and lowering the same in a whole formed either by boring or driving a tube with expandable shoe. The system proved satisfactory in every way and sandwick was born. This system has drastically reduced the cost of forming sand drain. In spite of larger number of 5.1 to 7.6 cm (2 to 3 in.) diameter sandwicks required at any site compared with 30.5 to 40.6 cm (12 to 16 in.) diameter sand draining, the total cost of forming and installing sandwick is one-third to one-fourth the cost of sand drains at any site. It is now being used as a standard system in various parts of the world. In India it has been most extensively used at Haldia Oil Refinery for oil storage foundations where it saved more than Rs. 3 crores over conventional foundations. It is now being used at Vizag to consolidate the soft clay in the ore stockyard area.

I have also been associated with CBRI for inventing and developing a technique called "Rope Drain" similar to sandwick. In this method strips of coir fabric is rolled into a pipe with or without leaving a hole at the centre. Experiments carried out even in very soft clay showed that clay does not penetrate much beyond 3.2 mm (1/8 in.) from the outer surface of the pipe and clear water from the consolidating clay comes out through the pipe. This is as efficient as the sandwick and is likely to prove cheaper in many parts of India.

The invention of sandwick and Rope Drains and their low price has made the solution of foundation problems in soft clay area very much in favour of the use of these techniques.

IMPROVING SOFT CLAY BY STONE COLUMN (FIGURE 15)

When the circumstances do not permit the use of pre-loading as a means of improving the soil and there is a firm stratum at shallow depth, full or partial replacement of the soft clay is often done to support structures on the ground directly. Stone column is basically a partial replacement of soft clay.

So far in India, it is only in Kandla for a fertiliser site and storage tanks, that an extensive number of 75 cm diameter stone columns were constructed in soft clay. The work was carried out by bored piling equipment. Casing was driven ahead of the boring and keeping the bore filled with water. After reaching the firm strata 0·5 m of sand was placed at the bottom and then 10 cm down stone chips mixed with sand was rammed down in 0·5 to 1·0 m layers. The ramming was done with a specially designed 2.03 tonnes (2 tons) rammer dropping 2 to 4 m. Casing was withdrawn as the filling and ramming progressed. This method of forming stone columns was developed by Indian engineers and has been found to be extremely satisfactory.

A series of load tests on single and group of stone columns were conducted and it was found that a 75 cm diameter stone column formed in this manner can carry 30.5 tonnes (30 tons) load. All settlement took place on application of load. Piezometer and settlement studies indicated no movement with time.

Stone columns technique is now being extensively used for supporting tanks, buildings, for increasing sliding resistances of embankments in soft clay area in Europe and other countries. There is every reason to believe that it will also find increasing use in India.

Time will not permit to discuss the many techniques that can help foundation engineers in solving difficult problems quite economically. I have found that our soils and foundation engineers do not consider it important to be up-to-date in this respect and heavily depend on contractors to offer some solutions which are sometimes contradictory with each other. It is also essential for a healthy growth of soil mechanics that we seriously endeavour to introduce and develop new techniques suitable for the conditions in our country.

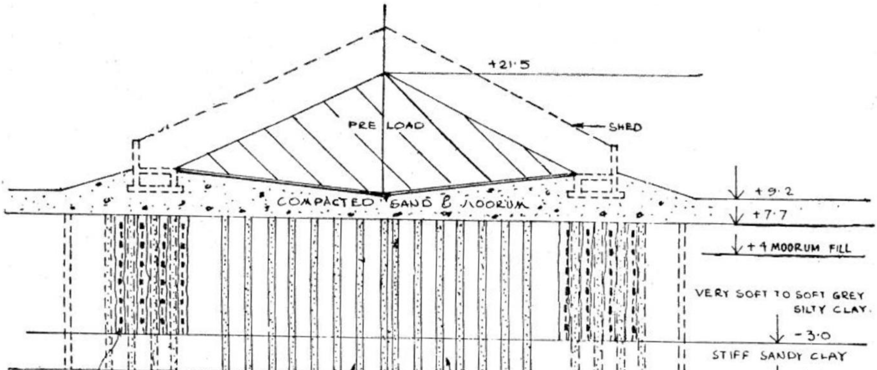


FIGURE : 15 Stone columns and sand drainage arrangement for a typical silo.

Comments on Sub-Soil Exploration

It is not possible to discuss the problems confronted by soil engineers without some reference to the sub-soil exploration being practised in India. It had been my pleasure to set up and develop the first commercial soil exploration unit in India and be associated with hundreds of investigation works. At many of the sites work was done earlier by other organisations. It has also been possible for me to be associated with CBRI's work in Calcutta and undertake some research work in the field with them to improve the existing techniques and develop new ones.

At many sites the investigation done by different agencies led to so diametrically different conclusions that it is appropriate for us to discuss some of our findings, which are repeatedly leading to inaccurate sub-soil exploration results and the means to overcome the same.

SINKING OF BORE-HOLE

The most extensively used methods of sinking bore-holes are wash boring and shell and auger boring. Both the techniques have their advantages and disadvantages, which are discussed below.

(*i*) *Wash Boring*

Although the technique is simple, yet it has some inherent drawbacks as listed below :

(*a*) Disturbance at the base of bore-hole due to downward action of jet water, if mechanical pumps are used.

(*b*) Difficulty in identification of sub-strata for want of samples.

(*c*) Collapse of non-cohesive strata and failure to advance through gravelly layer.

In an instance, the base of the bore-hole was such disturbed in a silty layer that the recorded values were between 2 and 4 up to considerable depth, whereas in adjacent properly stabilised shell and auger bore-hole, the values ranged between 8 and 20 (Figure 16).

(*ii*) *Shell and Auger Boring*

This technique has become quite popular in this country as it can be used under all soil conditions and as undisturbed soil samples collected helps to identify the various soil strata. It has, however, one major drawback of heavy disturbance at the base of the bore-hole if the shell (bailer) is used defectively. When the diameter of the shell is close to the diameter of the casing, the sludging operation of the shell causes suction and induces boiling at the toe of the bore-hole. This disturbance generally makes the sand loose over considerable depth around the bore-hole.

In this technique casings are used to stabilise the bore-hole. These casings are in pieces of different lengths. It is often difficult to note the bottom of the casing in relation to the bottom of the bore-hole and if there is heavy boiling of sand due to defective sludging then it is very common for the sand to rise up the casing. In one instance SPTs were indicating very high N-values. On close examination it was noticed that the spoon sampler was getting jammed in the sand inside the casing and the entire length of casing was moving with the spoon sampler.

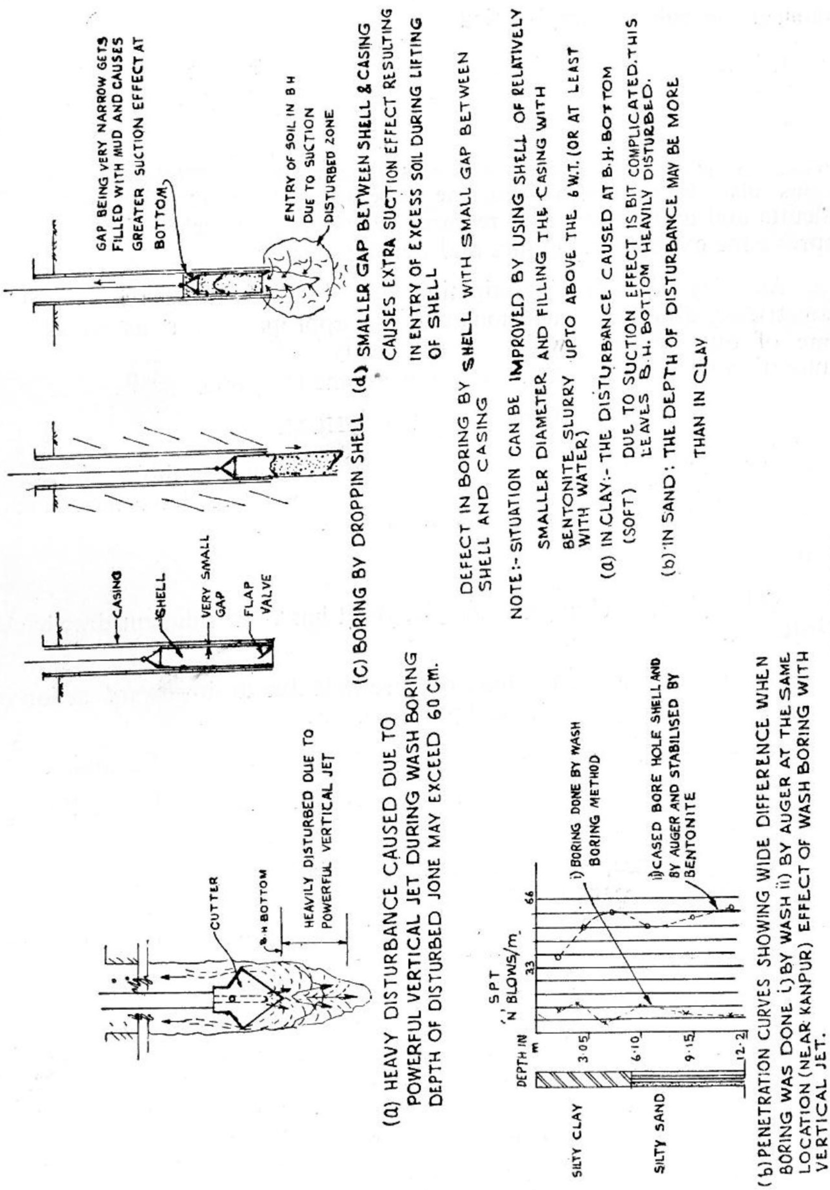


FIGURE 16.

Research by CBRI showed that most of these defects can be substantially overcome if the bore-hole is filled up with bentonite slurry instead of with water and a shell with a larger clearance with the casing is used.

In the Yamuna River bed at Kairana the *N*-values in a cased bore-hole full of water were 18 and 32 at 6 m and 10 m depth. On subsequent test with bentonite slurry the *N*-values at the same depth were 54 and 57. In Figure 16 the various *N*-values under different conditions of testing as observed at a site in Calcutta are shown. *N*-values obtained in a cased bore-hole full of water sunk by Shell and Auger boring were generally low.

In very soft clay, the shell acts almost as a piston in the casing and under its influence the clay squeezes in the hole formed below the casing. The clay surrounding the tip of the casing often gets heavily disturbed. Filling up the casing with water or bentonite improves the situation, but cannot absolutely eliminate the effect.

(*iii*) *Development of Bentonite Mud Drilling Techniques*

The necessity of development of a simple and cheap technique using indigenous equipment was felt and a study was taken up by the CBRI. Elimination of the sources of disturbances were also kept in view. As a result of the study Bentonite Mud Drilling method using simple hand operated equipment was evolved([17]).

By this method, the jet of bentonite slurry could be much controlled that the base of the bore-hole is not disturbed. Use of bentonite slurry prevents sand boiling, soft soil flowing, sedimentation and collapse of bore-hole due to thixotropy and hydrostatic balancing.

For identification of sub-strata, standard penetration test and boring can be done alternately such that there is no gap in between two tests. In addition to that, undisturbed soil sample can be collected wherever desired. The samples from the spoon samplers give a better picture of the structure of soil layers than disturbed soil samples obtained by shell. Besides, almost a continuous record of strength characteristics up to the explored depth is obtained, which is an additional advantage.

The efficiency of the method has been proved beyond doubt. At Patna in the Ganga River bed bore-holes up to a depth of about 100 m were sunk by this method, where other sophisticated equipment failed. Even thin gravelly layers at this site were penetrated by developing gravel traps. The gravel trap was in the form of a cup fitted just above the cutter. The trap got filled with dislodged gravels and the system was withdrawn from time to time for clearing the trap for further use.

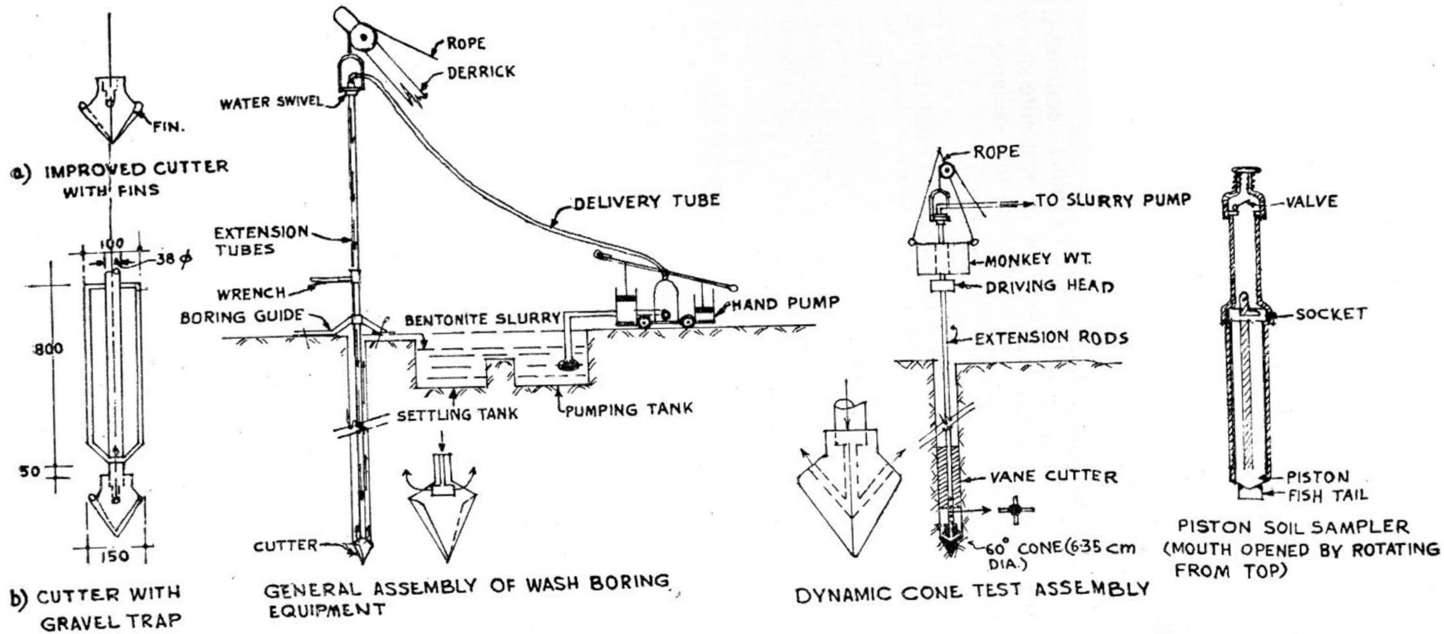
For extraction of undisturbed soil samples through such bore-holes, a special type of piston sampler was also designed. The piston closing the mouth of the sampler is provided with a fish-tail. On embedment into soil, the fish-tail holds the piston in position. On giving a rotation to the extension rods from the top, the piston gets unlocked through a connecting rod operating inside a socket. Sample will enter the sampler on pushing (Figure 17).

It may be claimed that this Bentonite Mud Drilling method is a superior means of bore-hole sinking. Where this method fails, Shell and Auger technique can be adopted with the precautions discussed earlier. For example, Shell and Auger boring will be more useful in gravelly layers or in salt water, in which bentonite coagulates.

VANE SHEAR TEST

In soft clays, the effect of sample disturbance could be severe. Therefore, estimation of property at the site by vane-shear test is important. In this respect, it can be pointed out that the vane should be embedded at the base of the bore-hole beyond the disturbed zone, which could be as deep as twice or thrice the bore-hole diameter. It has been noticed at many

SIMPLIFYING SOIL EXPLORATION THROUGH DRILLING MUD



CORRELATIONS AMONGST
N – VALUE (SPT), N_c – VALUE (DYNAMIC CONE), C_r – VALUE (STATIC)
FOR CALCUTTA, REGION

$N \equiv N_c$ FOR CLAYS
$\equiv \alpha N_c$ FOR SANDS
(α VARIES BETWEEN ·8 AND 1·0)

$C_r \equiv \beta N$ FOR CLAYS
$\gamma . N.$ FOR SANDS
β VARIES BETWEEN 1 AND 2, ASSUMED VALUE OF $\beta = 1{\cdot}5$
γ VARIES BETWEEN 3·0 AND 7·0, ASSUMED VALUE OF $\gamma = 4{\cdot}0$

FIGURE 17.

sites that the vane is pushed 20.3 to 25.4 cm (8 to 10 in.) below 15.2 to 20.3 cm (6 to 8 in.) diameter bore-hole in very soft clay. Such test results are liable to be very erroneous.

SAMPLING AND LABORATORY TESTING

Now-a-days quite a lot is stressed for the use of thin tube sampler at one end and highly sophisticated laboratory test at the other end. The intermediate stage is often overlooked and that is the procedure for preparation of test specimen. Often two or three 3.7 cm ($1\frac{1}{2}$ in.) diameter sample tubes are pushed simultaneously into the same sample of soil inside the main 10.2 cm (4 in.) diameter sample tube. Later, the test specimens are pushed out. It is doubtful if the soil can remain undisturbed in such a situation. Better method of preparation of test specimen would be to trim out the sample from block samples and it is highly essential that everywhere we follow this procedure.

DEVELOPMENT OF CONE PENETRATION TESTS

Whatever care may be exercised, the operational disturbances of bore-hole sinking cannot be altogether eliminated, which may be reflected in standard penetration tests or laboratory tests. Besides, a fair amount of guess work has to be done to assume the capacity of soil layers in between two consecutive tests.

All these lead to the importance of deep sounding using penetration tests. These tests assess the strength characteristics of soil strata in real undisturbed state and also provide the most important continuous record without missing any thin layer.

For this purpose, the performance of static cone penetration test has been universally accepted. The cost of the appliance and restricted depth of probing due to limited capacity of the machine are the drawbacks.

It was, therefore, considered necessary to develop an alternate method making use of simple indigenous equipment. A study was taken up by the CBRI and the dynamic cone penetration test with circulation of bentonite slurry may be termed as a significant contribution for soil exploration by Indian Engineers.

Only hand operated appliances are used in this technique. A 6.4 cm cone is given under impact from a 66 kg rammer falling freely from a height of 75 cm. Simultaneously, a 5 percent bentonite slurry is pumped into the drill rods, which comes out of the cone and through apertures in between the 4 vanes provided on the drill rod just above the cone. With driving of cone the drill rods are rotated and the vanes trim the bore-holes for free flow of the slurry around the drill rods. This eliminates the skin friction and intrinsic cone penetration resistance is obtained (Figure 17).

From a large number of studies, the correlation suggested by the CBRI between *N*-value (SPT), *Nc*-value (dynamic cone) and *Cr*-value (static cone) are shown in Figure 17. It may be pointed out that studies abroad and at CBRI recommend evaluation of *N*-value (SPT) for penetration of the spoon sampler from 30 to 60 cm and not from 15 to 45 cm[18].

From the above discussion it will be appreciated that there are many sources of error in sub-soil investigation work and why the results of two agencies of the same site can be widely different. Significant contributions have been made by Indian engineers in this field, but it is a pity that in spite of lower cost of investigation by these methods, they are not as widely used as it should be.

The most important point of sub-soil exploration work is the personnel consisting of site engineers, foremen and the winch operators. Many of us have found excellent improvement by properly explaining every boring problem and its significance to every field staff including winch operators and some labourers. I would recommend this training to every one engaged in sub-soil exploration work.

Role of Soil Engineers

Since the commencement of soil mechanics in India, considerable awareness has developed about the use of this subject. Our teaching and research institutions have played a vital role in its development. Today there is hardly any engineering institution where this subject is not taught in the undergraduate and post-graduate levels. There is perhaps more research work going on in this field than in any other branch in Civil Engineering and thanks to our professors, our treatment of this subject is as advanced as anywhere else in the world.

In the practical field, there is great awareness about the application of this subject. Today there is seldom any project where sub-soil is not being done. In some difficult areas, sub-soil exploration is being done even for small residential buildings. My observations over the years lead me to this conclusion that we should endeavour to do considerable improvement in this field. Boring, sampling and preparation of sample for testing leaves a lot to be desired. From the difficulties associated with improving these aspects of the work, many of us are of the firm conviction that emphasis should be laid on the use and development of in situ testing techniques. It must be mentioned that our research institutions, particularly CBRI, have contributed significantly in this field, but more requires to be done.

Perhaps our greatest weakness is that both in our training and in our work, we do not spend sufficient time to understand the sub-soil formations and live with it. The subject we are dealing with makes it imperative that we spend more time in the field than in the design office. We should start right from the sub-soil exploration to feel the soil, guide the in situ tests etc., for quite sometime before we may begin to understand the intracacies of sub-soil formations. I am informed that in some countries the local code requires a trained engineer to directly supervise at least 50 percent of the investigation work at any site.

Following field investigations and laboratory testing comes the first important aspect of soil mechanics in selecting the foundation types commensurate with the requirement of superstructure, and setting out the methods, with precautionary measures, to be adopted for the construction of foundations. If piles are to be used then the length and the type of pile suitable, should be recommended. Then comes the most important of all functions, that is to supervise work directly as in the case of piles, or to assist in the construction of safe and sound super-structure. I consider that soil engineers are the best to undertake this supervision work. From the sub-soil exploration results at a few locations they can formulate their thinking about the various assumptions made for the design of sub-structure and detect any variation in the ground which may require substantial change in the design and construction of foundation before they are completed. It had been my privilege to be the Resident Soil Engineer of a major steel project and to guide the construction work on many projects by frequent site visits. On many occasions, I detected substantial variations in the ground conditions from that revealed by earlier investigation and drastic change had to be made on many occasions.

I have a plea to my young friends—please do not consider it deregatory or a wastage of time to guide, observe and stand by the boring and investigation work. Trained in theoretical principles it is here that you will understand the problems as well as the beauty of foundation engineering. I would also request you to show interest to learn and make use of the various construction techniques in ground engineering including the piling systems. Then and then only it will be possible to efficiently function as soils engineer and earn the respect of the civil engineering profession.

Soil mechanics has made a good advance in the last 25 years and with the increased tempo of construction activity, and the zeal of our professors, scientists and engineers in this field, I have no doubt that in years to come, the importance and scope of this subject will grow immensely and make it the most respected and useful branch in Civil Engineering.

It is indeed a matter of gratification that this conference is being held in Kuruskhetra which occupies such an important place in the history of Indian civilisation and culture. It is here that the great holy war between the Pandavas and the Kauravas, as described in the Mahabharata, was fought and the lofty principles of Bhagawat Gita representing the quintessence of Hindu Religious Philosophy was enunciated by Lord Krishna. One of the great teachings of Gita is that we should work hard for the attainment of our cherished goals without attachment for personal gains.

As engineers it is, therefore, our duty to selflessly devote adequate time for proper development of our respective interest in civil engineeing and thereby contribute our might to achieve our cherished goal to make our country great and usher in an era of plenty and prosperity.

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