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SYNOPSIS Large scale failures of pile foundations driven in liquefiable soil deposits have been reported during the 1964 Alaska and Nighaata Earthquakes. Where the subsoil did not liquefy, the piles did not fail. These conclusions were the basis of selection of a proper pile type for the Bongaigaon Complex wherein it was intended that the pile installation procedure shall also adequately densify the loose saturated sandy and silty subsoils prone to liquefaction during expected earthquakes at site. Studies were carried out to assess soil characteristics before and after piling and the resulting improvement in the subsoil. It is concluded that the subsoil densification achieved after piling is adequate to guard against possibility of liquefaction occurring within pile groups and thus ensure the safety of the piles.

INTRODUCTION

Downstream of the existing one million tonne Bongaigaon Refinery, a Petrochemical Complex is being set up. The Complex comprises of a 45000 TPA DMT Plant, a 6000 TPA Ortho-Xylene Plant and a 29,000 TPA Para-Xylene Unit. Future expansion includes a 30,000 TPA Polyester Fibre Plant. Onsite facilities for the plants consist of the main process units and buildings while offsite facilities comprise of storage tanks, ware houses, cooling towers,water and waste treatment plants, etc.

Bongaigaon is located in a highly seismic zone and seismotechnic studies carried out indicate that earthquakes of magnitude 7.0 on the Richter's scale with a return period of 79 years could be expected at site (Arya et al., 1973). More recent seismological observations in the region have predicted severe tremors to occur in the area in the early part of 1980-90.

The subsoil at site consists of loose to moderately dense sandy and silty deposits upto a depth of about 10 metres. These are underlain by dense sands, gravels and cobbles. Prevailing ground water conditions are high and within one to two metres of the ground level. Liquefaction studies carried out by laboratory shake table tests indicated that the subsoil at site has a strong tendency to liquefy to a depth of about 10 metres under design earthquake conditions (Arya et al. 1974). Therefore, in order to ensure safety of foundations during expected earthquakes, critical unit structures and equipments are mostly founded on load bearing piles.

PILE BEHAVIOUR DURING EARTHQUAKES IN LIQUEFIABLE DEPOSITS AND DESIGN APPROACH

Pile behaviour during earthquakes and case histories of large scale failures on account of liquefaction of surrounding subsoil has been reported by Ross et al. (1969) and Margasan (1975). The studies are based on observed pile behaviour during the 1964 Alaska and Nighaata earthquakes. Some of the important conclusions presented are :

- If soil supporting pile fails by liquefaction, the pile will also fail. If soil does not liquefy, the pile will not fail.
- Piles driven through liquefiable deposits into denser deposits fare no better than those terminated within loose to moderately dense strata without reaching denser strata.

The above conclusions are based on behaviour of piles driven in loose to moderately dense sands and silts which liquefied during the earthquakes and their comparison with piles driven in strata which did not liquefy. Piles in gravelly strata, regardless of standard penetration N values, behaved relatively well. It would thus appear from available studies that, if pile stability is to be ensured during earthquakes, liquefaction of the surrounding subsoil should be prevented at all costs.

Meyerhoff (1959, 1976) and Mitchell (1970) have reported that when piles are driven into sand, the soil near the pile is compacted to a distance of a few pile diameters. Thus, for liquefiable soil deposits, if the pile type, spacing and installation procedure is properly selected, the surrounding subsoil can be densified to a degree adequate to ensure the stability of the subsoil and hence the piles against liquefaction. In particular, this can be considered applicable to medium to large pile groups.

PHILOSOPHY ADOPTED FOR BONGAIGAON PILES

Liquefaction studies have indicated that from an initial relative density of 60% the final relative density achieved, when the soil is vibrated at the expected ground acceleration of 0.4 g, is of the order of 80%. This amounts to a volume reduction to the extent of 4 to 6% which was confirmed by experimental observations. Thus, to prevent liquefaction occurring during design earthquakes, the subsoil at Bongaigaon requires to be densified to 80% relative density (Arya et. al, 1974). Translated in terms of standard penetration 'N' values (Gibbs & Holtz, 1967) this represents an 'N' value of 10 blows/30 cm at the surface and 25 blows/30 cm at a depth of 10 metres below ground with linear variation in between. Choice of pile type is dictated by the necessity of increasing the relative density of the subsoil to the above values during the pile installation process. In order to achieve the desired densification, the piles should displace a volume of subsoil equivalent to 4 to 6% of the volume of soil.

The pile selected is a 40 cm diameter driven cast in situ RCC pile. This is a positive displacement type of pile. Only vertical piles have been employed, since reponse of vertical pile to earthquakes is superior as compared to batter piles owing to its greater flexibility (Margason, 1975). Based on analysis of the superstructures, pile spacing adopted for pile groups was generally of the order of 3 to 3.5 times the pile diameter. This corresponds to a volumetric displacement of soil to the extent of 8 to 11%. Pile depths ranged between 13 to 16 m.

FIELD STUDIES

Field studies were carried out to ascertain the improvement in the subsoil characteristics on account of pile driving operations. Tests were conducted before and after piling in pile group locations.

Following programme of testing was carried out :

(a) Conducting standard and cone penetration tests before piling. These were followed by cone penetration tests after piling.

(b) Conducting lateral loading tests on single piles driven in isolation. These were followed by tests on single piles driven within a group.

Standard Penetration Tests (hereinafter designated as SPT) were conducted in 150 mm diameter boreholes. Cone penetration test (designated as CPT) were conducted using a cone of 65 mm base diameter and apex angle of 60^{o}. A bentonite slurry of 6% concentration was continuously circulated to eliminate the friction between the soil and rods. The hammer weight and fall is the same as for SPT tests. For interpretation of results of CPT, a correlation was set up between SPT and CPT blow counts.

Lateral pile load tests were carried out by jacking against adjacent pile. Loads were applied in suitable increments at the pile top which corresponded to pile cut off level.

DISCUSSION OF TEST RESULTS

SPT and CPT tests before and after piling:

Fig. 1 presents a typical subsoil profile

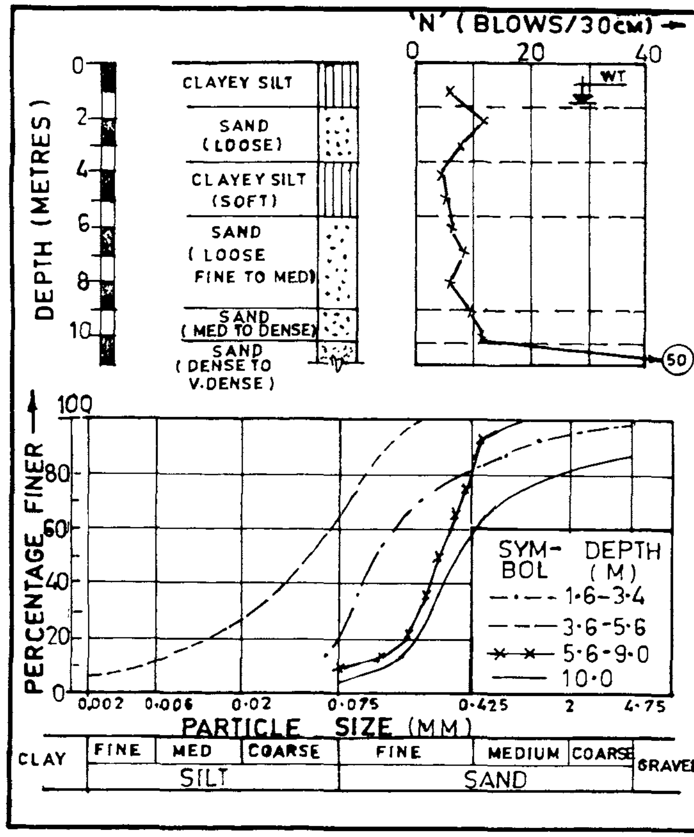

Fig. 1 : Typical Subsoil Characteristics

existing at site. In the upper loose sand layer (from 1.6 m to 3.6 m depth) the 'N' values range from 8 to 12 blows/30 cm while in the lower loose sand layer (5.6 m to 10.1 m) these are in the range of 6 to 12 blows/30 cm. In the silt layer (3.6 to 5.6 m) 'N' values of 4 to 6 blows/30 cm were recorded. At some locations the silt layer has a thickness of upto 4.0 m and with silt content of upto 65%. In the dense sand layer (below 10.1 m depth) high 'N' values of about 50 blows/30 cm were recorded.

TABLE I. Correlation between SPT (N) and CPT (Nc) Values

	N/Nc Ratio	
Depth (m)	Range	Average
0 - 1.6	0.8 - 1.2	1.0
1.6 - 5.0	0.3 - 0.8	0.5
5.0 and below	0.4 - 0.8	0.6

Table I gives a correlation between SPT (N) and CPT values (Nc) for the site. These are based on a large number of tests conducted at site. Generally, upto a depth of 1.6 m, N/Nc ratio is about 1.0 while in the lower strata it is 0.5 to 0.6.

SPT/CPT tests were conducted for six different pile groups before and after piling. For discussion purposes, however, only two are considered, namely, pile groups for 23-C-001 and for 25-C-001 since these are representative of the general trend.

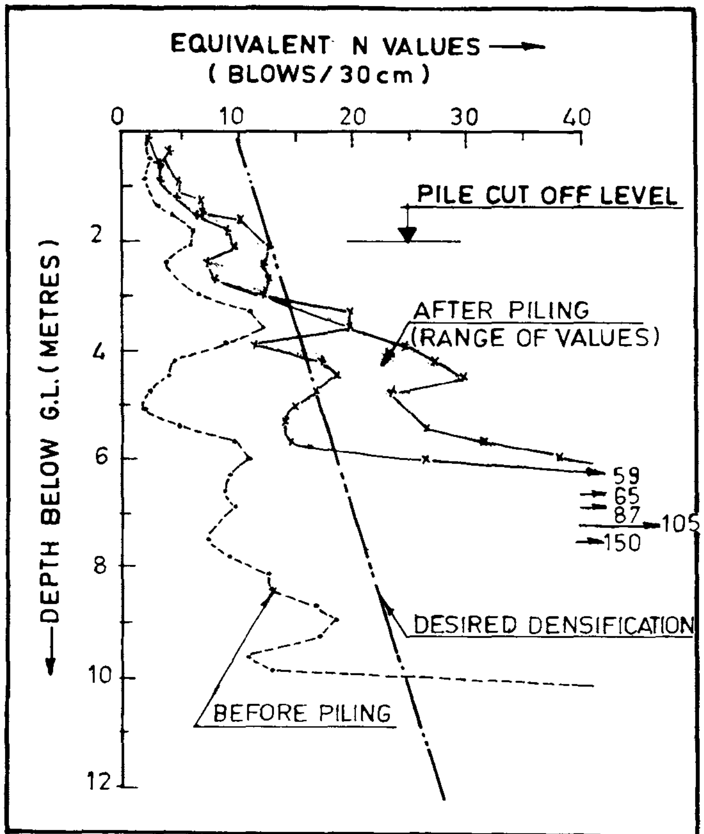

Fig. 2 : Pile Group 23-C-001

Fig. 2 shows the test results for 23-C-001. The CPT results are plotted in terms of equivalent 'N' values based on correlation developed. It is seen that upto 3.5 m depth, post piling 'N' values are 50 to 100% higher than pre-piling values. In the silt layer from 3.5 to 5.5 m, this increase is of the order of 300-400%. In the sand layer from 5.5 to 10 m the post piling 'N' values increase to 5 to 10 times the initial values before piling. It is also seen that below 3 metre depth the post piling 'N' values are always considerably above the desired densification line.

Fig. 3 above shows test results for 25-C-001. In this case the post piling 'N' values cross the desired densification line at a depth of 3.8 m and remain substantially above thereafter. The order of increase and the trend of 'N values after piling is more or less the same as for 23-C-001.

The above results indicate that the subsoil has been densified substantially above minimum requirements during the pile installations. This corroborates laboratory predictions considering the large volumetric displacements of the subsoil by piles. It is particularly interesting to note that even in the silty deposit between 3.5 to 5.5 metres an increase in the blow counts to the extent of 300-400% is recorded.

Lateral Load Tests

Fig. 4 presents the load deformation plots for some of the piles tested. TP2 and TP3 are initial test piles which were driven in isolation. TP6 and 885 were driven within the pile group for 22-E-001 and form part of the group. This group consists of a total of

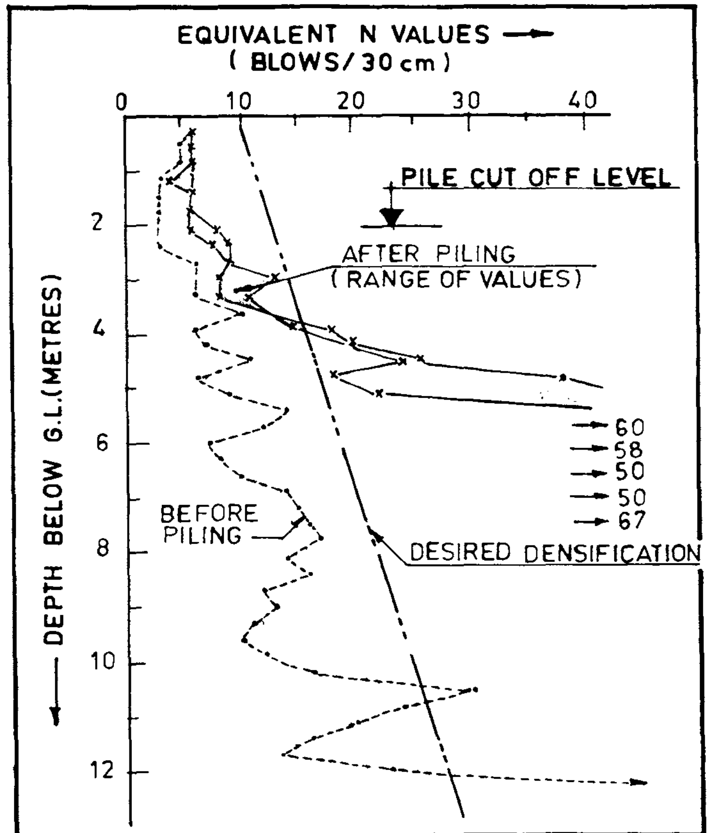

Fig. 3 : Pile Group 25-C-001

25 piles, driven in a 5 x 5 pattern at a spacing of 1.2 metres both ways.

In order to compare the pile top movements at specific loadings, deformations at various lateral loads were read from Fig.4 and are presented in Table II. The improvement in performance for piles driven within group is highlighted in this Table. At the rated pile capacity of 2.0 tonnes, the deformations of TP6 and 885 are about 1/3 of the corresponding values for TP2 and TP3 and at the expected ultimate load of 4 to 5 tonnes, these are 1/3 to 1/4 the deformations of TP2. Also, significant are the rebound values on removal of test loads. While TP2 and TP3 have shown residual deformations of 9.3 and 2.4mm under final loads of 4.3 and 3.2 tonnes respectively, for TP6 and 885 the residual deformations are only 0.4 and 0.7 mm respectively for a peak load of 5.4 tonnes. Thus, the lateral pile movements have decreased significantly owing to densification of supporting subsoil. In other words, pile capacity to support horizontal loads has been enhanced.

CONCLUSIONS

The present study has enabled verification of the extent of improvement in the characteristics of liquefiable soil deposits existing at Bongaigaon Complex site on account of pile driving. It is considered that the densified deposit would not liquefy during expected earthquakes and stability of pile groups is ensured. This is particularly true for inner rows of piles.

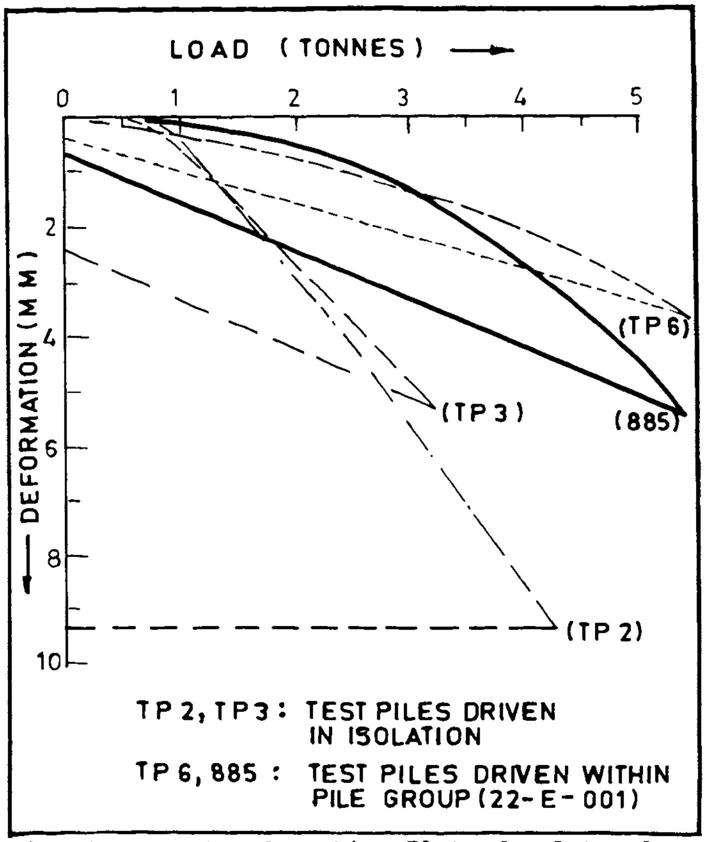

Fig. 4 : Load Deformation Plots for Lateral Pile Load Tests

Lateral pile load tests have indicated a substantial increase in pile capacity owing to soil densification. Thus, even if the outer row of piles in a group are partially affected due to soil liquefaction outside the group, the reserve capacity of the inner piles is adequate to still ensure safety of the structure.

TABLE II. Lateral Load Test on Piles: Load - Deformation Values

Load (Tonnes)	Deformation of Pile Head (mm)			
	Single piles in Isolation		Single Piles within Group	
	TP2	TP3	885	TP6
1.0	0.4	0.3	0.2	0.4
2.0	3.0	2.6	0.6	0.8
3.0	5.6	4.9	1.3	1.2
4.0	8.5	5.3 (@ 3.5T)	3.0	1.8
5.0	9.4 (@ 4.6T)		4.4	2.9
5.8	-	-	5.4	3.6
Rebound after release of loads	nil	2.9	4.7	3.3

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