BORED PILES INSTALLED BY SLURRY DISPLACEMENT

PIEUX FORES INSTALLES PAR DEPLACEMENT DE LA BOUE DE FORAGE

УСТРОЙСТВО БУРОВЫХ СВАЙ ПОД ГЛИНИСТЫМ РАСТВОРОМ

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SYNOPSIS-Three instrumented bored piles, .75-.90 m in diameter with penetrations varying from 13 - 22 m, were cast using the slurry displacement method. The soils at the test sites consisted of stiff clay near the ground surface underlain by a stratum of water-bearing sand.

The piles were loaded axially to failure in compression about one month after casting. The data which were obtained were analyzed to obtain curves showing side load transfer and tip resistance as a function of downward movement of the pile.

The maximum unit side load transfer was compared to the in situ shear strength of the soil and load transfer factors for both clay and sand were obtained. These factors compared favorably with those obtained for bored piles in the same area installed by using the dry technique.

Following the load tests, the three piles were extracted and examined carefully to ascertain effects of the construction technique on the concrete. A careful examination of the concrete at the soil interface revealed the existence of a stabilized coating of sand between the concrete and the soil.

In addition to presenting data on load transfer, the paper presents a discussion of the method of construction based on field observations, and gives recommendations for the improvement of the concreting operation.

INTRODUCTION

A recent phase of a comprehensive research program on bored piles was concerned with the evaluation of "the slurry displacement method" of construction. In this method the hole is stabilized with bentonite-soil slurry during drilling. The concrete is poured by use of a tremie, with concreting starting at the bottom of the hole. This method holds considerable promise for future application, since it ideally permits bored piles to be installed in virtually any type of soil. Previous research (Chadeisson, R., 1961; Burland, J. B., 1963; Farmer, I. W., et al., 1970) has left some unanswered questions as to the effect of the bentonite slurry and the construction techniques on transfer of load from the pile to the supporting soil.
This paper presents the findings of research conducted on three full-sized instrumented bored piles constructed at three test sites in Houston, Texas, U.S.A., by this method and gives an evaluation of the method.

DESCRIPTION OF WORK

In addition to preliminary borings made at the test sites, three NX core borings were made at each of the three test locations. Undisturbed samples for triaxial tests were recovered from one boring, and standard penetration tests and dynamic cone penetrometer tests were run in the other two. Each site consisted of strata of stiff fissured clays and silts overlaying a layer of water-bearing sand in which eadh pile was terminated.

The instrumentation of the piles consisted of sensitive electrical resistance cells (Barker, W.R. and L.C. Reese, 1969), placed in pairs inside the cage of reinforcing steel on two diametrically opposite generatrices. Soil profiles, pile geometry, and placement of instruments at each site are shown in Fig. 1.

The procedure which was followed in the installation of the piles was as follows:

1. A hole was augered without the use of slurry until a water-bearing, caving layer was encountered.
2. At that point a premixed bentonite slurry was immediately introduced and kept at a level within one to two meters of the ground surface, while

drilling with an auger was continued under the mud.

3. The drilling of the hole was finished with the auger or with a special drilling bucket, designed to clean the bottom of the hole, immediately before the reinforcing steel cage was set in place.
4. A tremie, ten inches in diameter, was seated in the hole. The tremie was temporarily sealed at the bottom.
5. The tremie was filled with concrete, the temporary seal was broken, and concreting was completed under gravity flow while keeping the tremie tip well embedded in the concrete.
6. The first portion of the concrete to reach the surface was wasted because this concrete was usually contaminated with slurry. The finished pile extended about 0.6 meters above the ground surface.

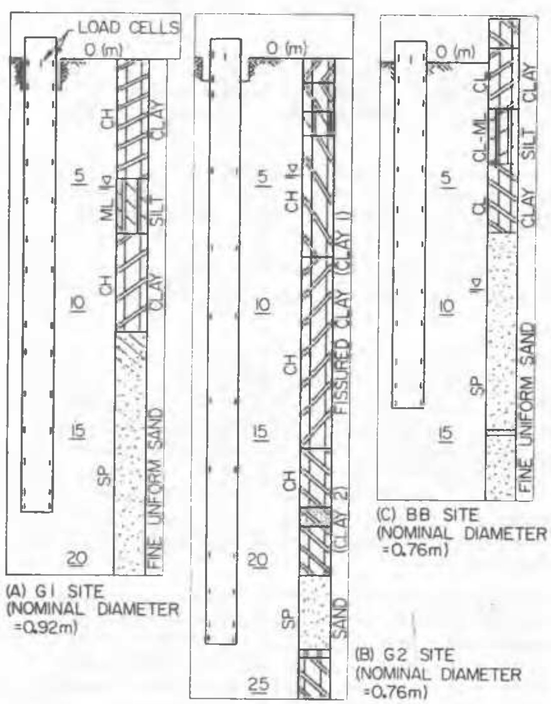

Fig. 1 Soil Profiles and Sketches of Location of Shaft Instrumentation

The piles were load tested about a month after casting except for pile BB, which was tested at an early age of 16 days. The load tests were performed using the quick-test procedure (Fuller, F.M. and H.E. Hoy, 1970). A load increment equal to about one-thirtieth of the estimated ultimate load was applied every two and one-half minutes. For each increment of load application, the load-measuring cells were read by use of a high-speed digital data acquisition system.

The three piles were extracted after load testing. When a pile was removed from the soil, its overall length and its diameter at each of the instruments levels were measured. The soil-concrete interface was carefully examined by studying soil still adhering to the pile surface, and the concrete was examined for possible slurry contamination.

RESULTS OF TEST

The difficulties in evaluating the shear strength of stiff fissured clays are well known. It was established earlier that a large number of samples is needed to allow a consideration of a statistical average of the shear strength of stiff clay where slickensides and fissures may cause considerable scatter in the results (O'Neill, M.W. and L.C. Reese, 1970).

A comprehensive program of soil testing was conducted with the following tests being performed: a dynamic cone penetrometer test, the standard penetration test, the pocket penetrometer test on intact chunks of the specimens, the quick triaxial test, and a multi-stage triaxial test (performed by personnel of the Texas Highway Department). As might have been expected, the scatter in the results from these different techniques was extreme. The data was studied and the undrained shear strength of the clay was estimated on the basis of all available information (see dashed curves in Figs. 2, 3, and 4). Also shown in Figs. 2, 3, and 4, are results from undrained triaxial tests on specimens trimmed to 35.5 mm in diameter, with confining pressure equal to the overburden pressure, and results from the standard penetration test. The following equation was employed to convert the N-value to shear strength (Terzaghi, K. and R. Peck, 1967):

$$s \text{ (Tons/square foot)} = \frac{N}{15} \qquad (1)$$

Where sands were encountered in the borings, an estimate was made of the drained shear strength at the overburden pressure by use of the following relationships:

$$s = \bar{p} \tan \bar{\phi} \qquad (2)$$

where

$\bar{p}$ = effective overburden pressure

$\bar{\phi}$ = friction angle, estimated from the N value, using U.S. Bureau of Reclamation penetrometer correlation charts (Gibbs, H.S. and W.G. Holtz, 1957).

This definition given to the shear strength of sand is only a convenient one. It relates to the in situ strength of the sand prior to the installation of the pile.

Readings from load cells were recorded with a high-speed data acquisition system for each of the stations along a pile and for each load increment. Readings at the station near the groundline allowed the development of a calibration curve for load in the pile at that point. The applied load was measured by reading the pressure with a pressure transducer in a calibrated hydraulic ram. The ram was specially built to minimize friction. The calibration curve obtained from the top station, adjusted as necessary to reflect variations in pile diameter, was used for computing the load for all other stations. The stiffness of the pile under axial load was obtained analytically using the modulus of elasticity of the concrete obtained from independent tests.

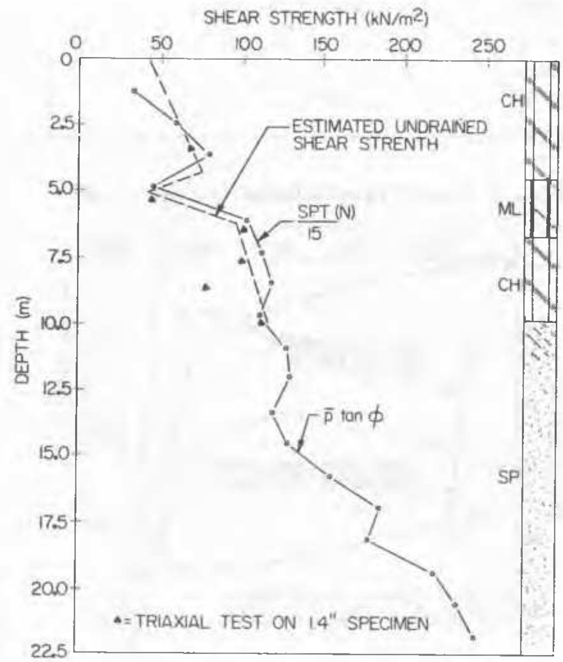

Fig. 2 Shear Strength Profile - G1

Load-distribution curves, Figs. 5, 6, and 7, were obtained by plotting the load measured by the load cells versus the depth of the pile.

The load-settlement curve for the top of a pile was obtained by direct measurement. For the tip of a pile the load was obtained from load-measuring cells at the tip or by extrapolation of the load-distribution curves (see Figs. 5, 6, and 7), and the settlement was computed by subtracting the computed elastic compression from the observed settlement of the top of the pile. The load-settlement curves for the top and tip of each pile are shown in Figs. 8, 9, and 10.

Load-transfer curves, Figs. 11, 12, and 13, were obtained at different depth in the pile by plotting the shear stress developed at a point, obtained by a differentiation of load-distribution curves, versus the displacement of that particular point with respect to its original position. The displacement at a point is computed from know-

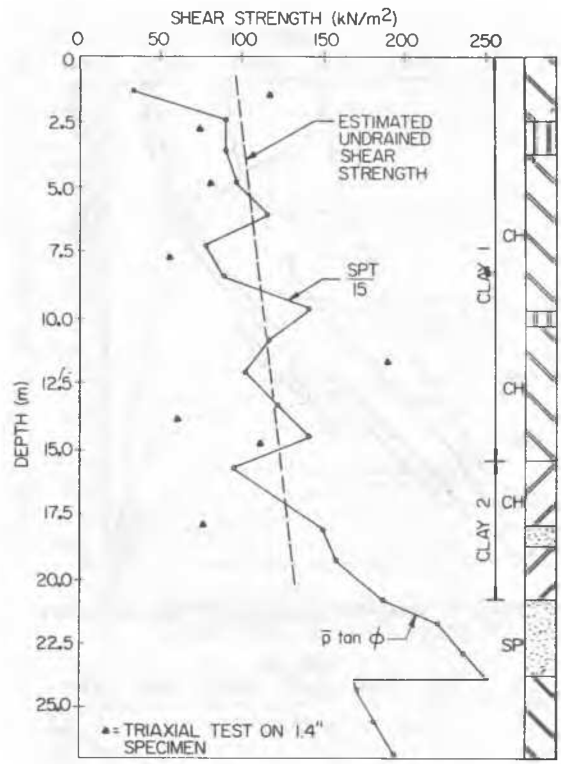

Fig. 3 Shear Strength Profile - G2

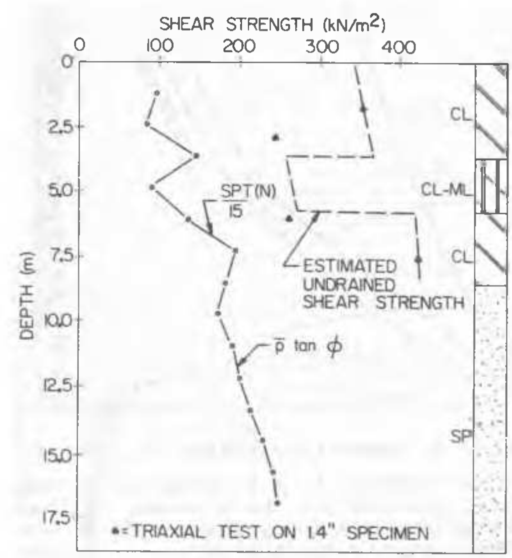

Fig. 4 Shear Strength Profile - BB

ledge of the butt settlement, the load-distribution curve, and the elastic properties of the pile.

Previous research has shown that in clay soils the side load transfer can be related to the shear strength of the soil by a parameter conventionally called the α factor. In this study, the average α factor (α_{avg}) shall be defined as the ratio of the average

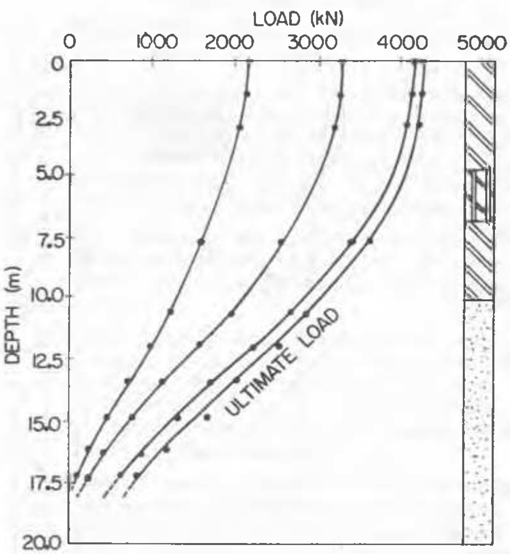

Fig. 5 Load-Distribution Curves - G1

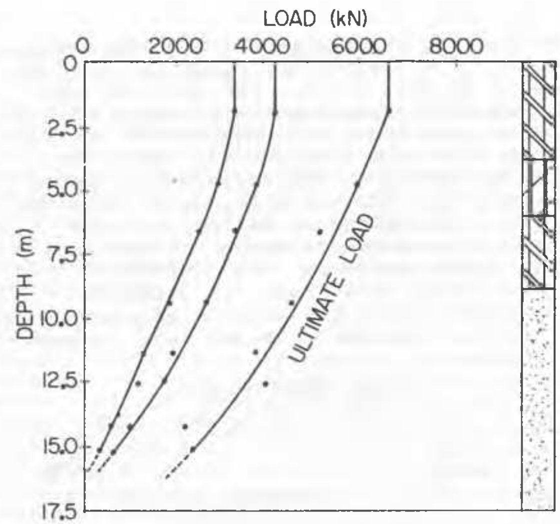

Fig. 7 Load-Distribution Curves - BB

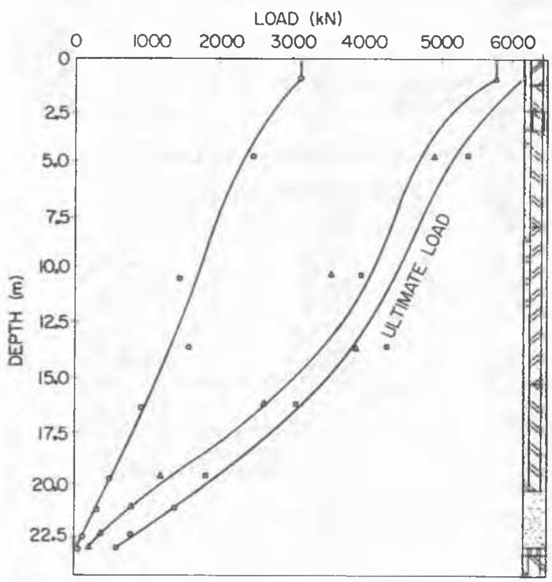

Fig. 6 Load-Distribution Curves - G2

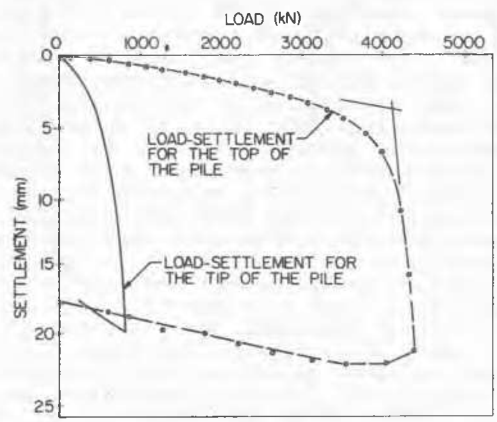

Fig. 8 Load Settlement Curves - G1

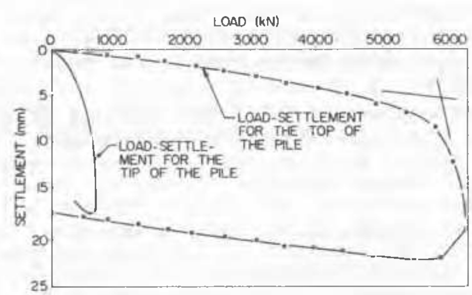

Fig. 9 Load-Settlement Curves - G2

unit load transfer in a stratum to the average shear strength of the stratum. It has been found convenient in this study to extend the α factor concept to sands to serve as a means for describing the fraction of the in-situ indicated drained strength of sands actually mobilized in skin friction. The effects of the unknown earth pressure coefficient and the adhesion factor between concrete and sand are lumped into the α factor. Table I summarizes the results of α_{avg} factors obtained from the three tests. This table also summarizes the results of previous load tests on bored piles cast in stiff fissured clay in dry holes using no slurry or casing. In this series of tests, the piles were also located in Houston, Texas, and were instrumented and tested in a manner similar to that followed in this study (O'Neill, M.W. and L.C. Reese, 1972).

DISCUSSION OF RESULTS

The load-settlement curves for piles G1 and G2 (Figs. 8 and 9) indicate a sharp drop at ultimate loads which is explained by a lack

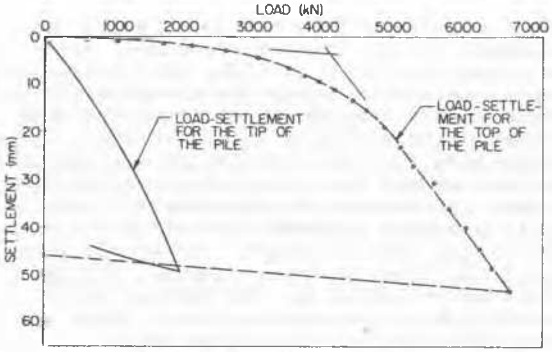

Fig. 10 Load-Settlement Curves - BB

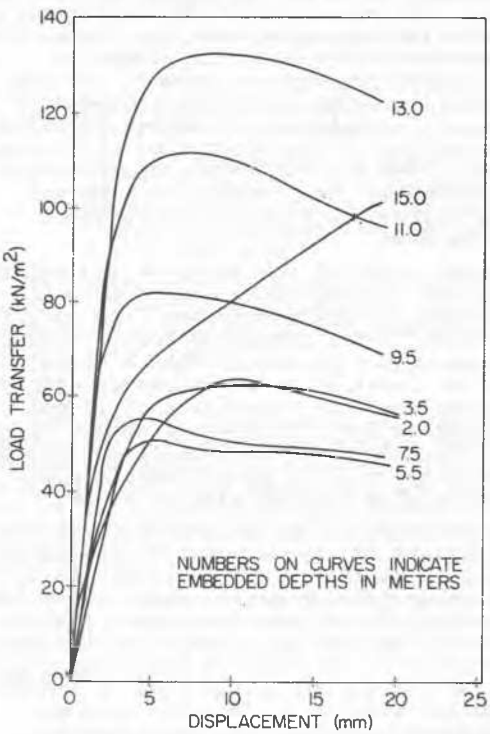

Fig. 11 Load-Transfer Curves - G1

of high tip resistance and by the sensitivity of the clay formation. For pile BB (Fig. 10) the load kept building up even at high displacement, which should be a characteristic of bored piles with tips in dense sand. Because of construction techniques, apparently there were less soft deposits at the bottom of pile BB and that pile developed a considerable tip resistance. In the other cases (piles G1 and G2) the bottom sediments contained soft material, and the tip resistances were considerably smaller than those computed from bearing capacity theories.

The general observation can be made that the peak load transfer in stiff clays is reached for relative movement between the pile and the soil of 4 to 5 mm, while for sands this relative movement varied between 5 mm for pile G1 to 10 mm for pile BB (Fig. 11,

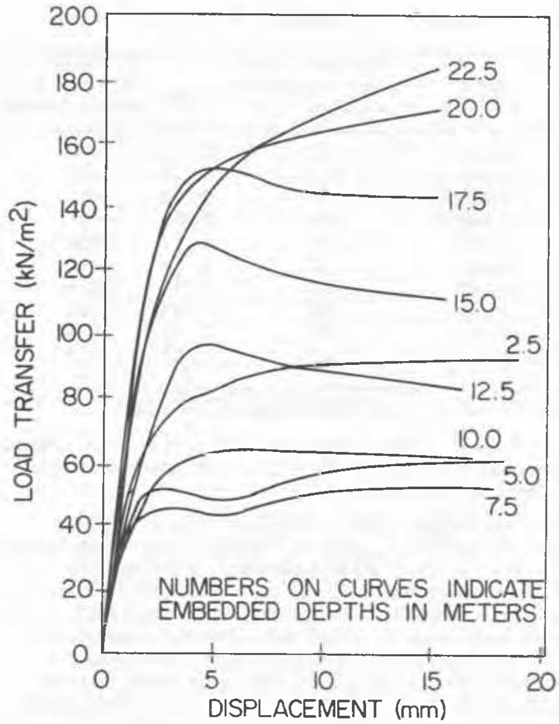

Fig. 12 Load-Transfer Curves - G2

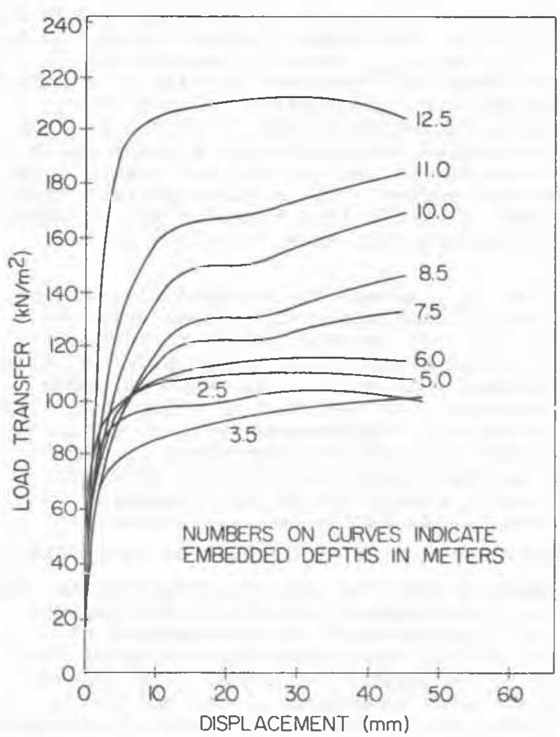

Fig. 13 Load-Transfer Curves - BB

Table I Summary of Results From Load Tests

Pile	Soil	Average Peak load transfer kN/m^2	α_{avg}	Ultimate Tip Resistance kN/m^2
G1	clay	52	0.65	---
	sand	103	0.70	1140
G2	clay 1	74	0.75	---
	clay 2	159	1.08	---
	sand	176	0.67	1450
BB	clay	92	0.25	---
	sand	150	0.75	4000
S1T1*	clay	49	0.44	1090
S2T1*	clay	59	0.53	970
S3T1*	clay	62	0.54	935

*O'Neill, M.W. and L.C. Reese, 1972.

12, and 13). The reasons for this wide range of displacements at failure in sands is not yet well understood.

As can be seen from studying the α values given in Table I, the α factor from the tests where the slurry displacement method was used are generally as high or higher than those from tests where the dry-casting method was used. However, due to the difficulties involved in measuring the shear strength of fissured clays, and due to the unreliability of some of the shear strength tests used, the authors are reluctant to recommend higher α values for piles cast by slurry displacement.

In the clay 2 layer at the G2 site, which is a sensitive, nonfissured organic clay, (O.C. R. ≃ 2), the α_{avg} value is considerably higher than that obtained in clay 1, a stiff fissured clay. Examination of the soil-concrete interface in clay 2 after the pile was extracted indicated that a shear failure occurred in the natural soil about 1 cm from the surface. The authors believe that in such clays the load transfer may be equal to the undisturbed shear strength of the clay.

The low α_{avg} values in the hard clay at the BB site is consistent with other observations that the fraction of shear strength that can be mobilized by bored piles in clays diminishes with increasing shear strength of the clay. In the case of piles installed by the slurry displacement method, a limiting value of load transfer equal in magnitude to the shear strength of a hardened coating of slurry at the soil-concrete interface (described below) is indicated.

OBSERVATIONS ON THE CONSTRUCTION PROCEDURE

It appears that the main disadvantage to the slurry displacement method of construction is the likelihood of the development of a smaller tip resistance than expected for bored piles tipping in sands. The deposition of loose sediments at the bottom of the hole can create a weak layer of material at the tip of a pile, resulting in a reduced bearing capacity of the tip. These sediments could be the result of the sloughing of the walls of the hole, the deposition of soils suspended in the mud, or the entrainment in the hole of loose soil from the ground surface. A rich, well-dispersed slurry, maintained near the ground surface, should reduce the amount of sloughing and sedimentation of soils in suspension. Special methods for the cleaning of the bottom of the hole should be given serious attention. In this connection, it was observed that a drilling bucket cleaned the bottom of the hole better than an auger. After the bottom of the hole has been cleaned, the hole should be protected at the surface by a short piece of casing to prevent loose surface soil from falling in the hole.

The first flow of concrete may be used advantageously to clean the bottom of the hole. However, the appearance of the tip of the extracted piles suggested that this first flow was ineffective for the test piles. A banded polyethylene sheet used in the construction, to temporarily seal the end of the tremie, probably prevented an effective washing action of the first flow of the wet concrete. The size of such a sheet should be minimized in the future, or a better sealing technique, such as a flap gate, should be used.

A vigorous scour of the walls of the drilled hole by the flowing wet concrete is desirable to prevent mud entrapment. An analysis of the flow is not attempted here. However, it can generally be stated that a clean tremie (at least 30 cm in diameter) and high-slump concrete (greater than 15 cm) are essential for a successful tremie operation. Furthermore, it is believed that good bearing piles cast by slurry displacement need to be of a minimum size of 60 cm.

It was observed in the BB and G1 piles that small amounts of slurry were trapped by the concrete. The slurry was observed mainly in the upper portion of the pile and on the instrumentation cables, the gages, and the reinforcing spirals which provided horizontal obstructions to the flow of the concrete. Slurry contamination was minimal in the pile G2, because the slurry for that pile was much lighter than that for the other two piles. The contamination of the concrete did not seem to impair significantly the structural quality of the piles. It is, however, recommended that the horizontal steel ties be kept to a minimum, and in situations where the mud becomes flocculated and heavily charged with sand (specific gravity > 1.35 - 1.4) the mud should be replaced by a lighter mud, before proceeding to the concreting operation.

A hard coating of mud and sand was observed at almost all levels on the surface of the extracted piles. Furthermore, field examination of the concrete-soil interface indicated a failure surface in the soil rather than at the interface in most soil strata, which suggests that this coating has a higher shear strength than does the surrounding soil. At the BB site, no failure surface was observed in the hard clay, and there was evidence of failure taking place at the interface. The measured shear transfer of about 92 kN/m^2 in that clay formation is believed to be an indication of the strength of the coating at that level, and may be considered as an approximation of the maximum dependable shear transfer in hard clays.

CONCLUSION

The main findings of this research are:

1. The load transfer of bored piles installed by slurry displacement is comparable in stiff clays to that of bored piles cast by the dry technique.
2. In the reported tests the load transfer in sand was found equal to about 0.7 times the integral on the periphery of the pile of the product $\bar{p}$ tan ϕ.
3. The tip resistance of bored piles installed by slurry displacement may be greatly reduced by the entrappement of soft soils at the tip of the pile.

The authors believe that the slurry displacement method can be safely used in the construction of bored piles designed to carry axial loads.

ACKNOWLEDGMENTS

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