Field Tests of Drilled Shafts in Clay-Shales

Essais de Puits Forés dans les Schistes Argileux

R.P.AURORA Senior Engineer, Marathon Oil Company,
L.C.REESE Professor of Civil Engineering, University of Texas, U.S.A.

SYNOPSIS Four instrumented drilled shafts 0.75 to 0.91 m in diameter with penetrations of about 7 m were installed into clay-shale by three different construction procedures. The tips of the shafts penetrated 1.0 to 2.4 m into the clay-shale layer. Each shaft was axially loaded to failure using a reaction system in which all the tension steel could be recovered after testing. Instrumentation along the length of the shafts allowed curves to be obtained showing load as a function of depth. Field data were analyzed to find load transfer in side resistance and base resistance. Shear strength of the soils was determined from laboratory studies and from in situ techniques and load transfer factors were computed for each method of construction.

INTRODUCTION

The use of drilled shafts (also called drilled piers, bored and cast-in-place piles, and caissons) has increased in the United States because in many instances there are economic and other advantages to this kind of deep foundation. Until recently, drilled shafts were designed as spread footings and thus were usually under-reamed. The studies described herein are a part of a decade-long project during which 19 instrumented drilled shafts have been loaded in the field to failure (L. C. Reese, et al, 1969; W. R. Barker, and L. C. Reese, 1969; L. C. Reese, et al, 1973; D. E. Engeling and L. C. Reese, 1974).

The principal aim of the project and of these studies has been to gain information on the manner in which axial load is transferred to the supporting soil and to translate such information into design recommendations.

SOIL PROFILE, INSTRUMENTATION, AND SOIL STRENGTH DATA

Two test sites, about 320 km apart, were selected in the state of Texas, USA. One test site was in Montopolis (near Austin) and the other was in Dallas. The clay-shale layer was found at a depth of about 6 m from the existing ground surface at both test sites. In this study the soil referred to as clay-shale met the classification criteria suggested by N. R. Morgenstern and K. D. Eigenbrod (1974). The soil profile at the Montopolis site is shown in Figs. 1 and 2 and consisted of the following layers, starting at the ground surface: a layer of medium to stiff clay about 3 m thick, a layer of water-bearing sand and gravel about 1.3 m thick, a layer of stiff to hard clay about 1.7 m thick, and a thick layer of clay-shale. The soil profile at the Dallas site is shown in Fig. 3 and consisted of the following layers: an upper layer of sandy clay approximately 4.0 m thick, a layer of sand and gravel about 1.8 m thick, and a thick layer of clay-

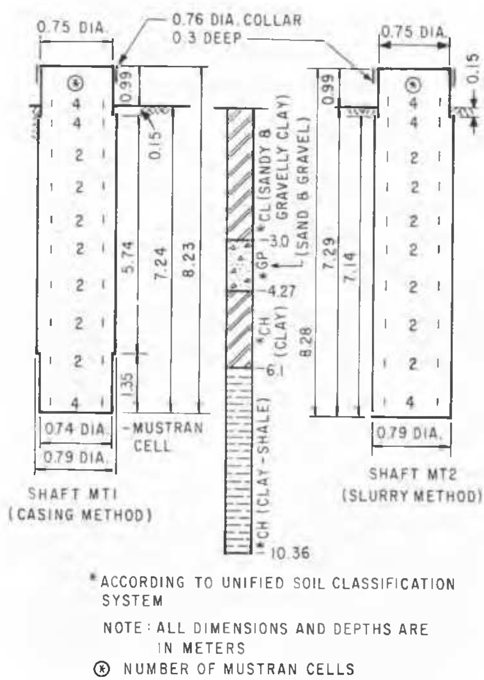

Fig. 1 Details of Shafts MT1 and MT2 at Montopolis

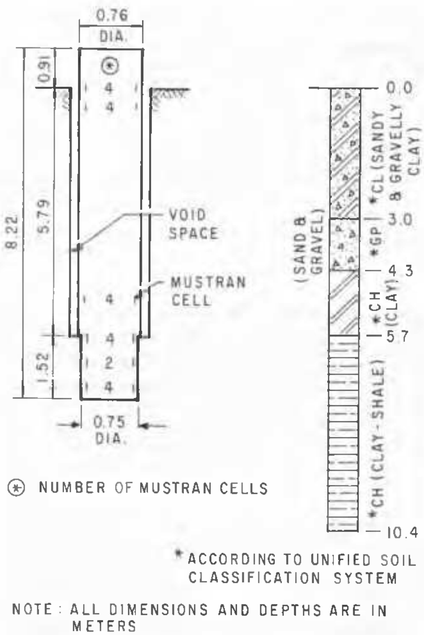

Fig. 2 Details of Shaft MT3 at Montopolis

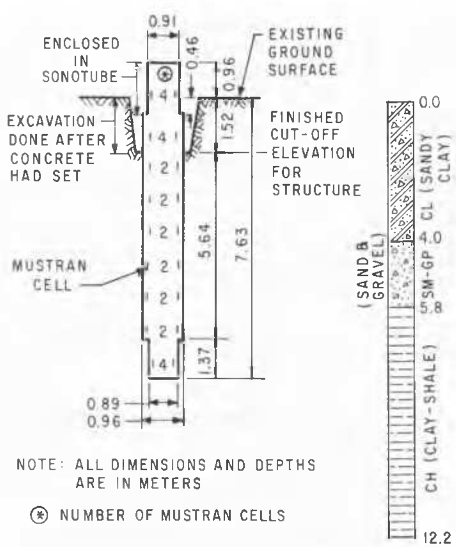

Fig. 3 Details of Shaft DT1 at Dallas

shale. Three shafts were tested at the Montopolis site and one at the Dallas site.

As shown in Figs. 1, 2, and 3, "Mustran" cells that employ electrical resistance gages for measuring strain were embedded at various depths within each shaft. Readings from the Mustran cells allow for a determination of the distribution of axial load along the length of the shaft (W. R. Barker, and L. C. Reese, 1969).

Measurement of the in situ shear strength of soils was made by both field and laboratory techniques. The field methods employed were the dynamic cone penetrometer test, the standard penetration test, and the static penetrometer test. The static penetrometer consisted of a cone at the end of a 2.54 cm diameter steel rod. The cone had a base diameter of 3.57 cm and a height of 2.45 cm. The dynamic cone had a base diameter of 7.62 cm and a height of 6.35 cm. The dynamic cone was driven by a 77-kg hammer with a free fall of 76 cm. Static cone tests were conducted in the clay-shale by drilling a large diameter hole to the top of the formation. Technicians worked in the hole and the cone was forced into the clay-shale, using the kelly of the drilling rig as the reaction. Standard penetration tests were done only at Montopolis up to the top of clay-shale. Core samples were taken only at the Montopolis site using a double-barrel coring tool.

Unconsolidated-undrained triaxial tests were run on core samples obtained at the Montopolis site and the undrained shear strength, c_u, was estimated mostly from these tests. Indirect methods for obtaining shear strength from results of dynamic penetration test (R. B. Peck, et al, 1974; D. E. Engeling, and L. C. Reese, 1974) were used for comparison. The results of the static cone tests conducted at the Montopolis site were used to estimate the bearing capacity factor for the cone, $N_{c, cone}$, and it was found to be approximately equal to 19. This value of $N_{c, cone}$ was used to obtain the approximate shear strength of clay-shale at the Dallas site from results of in-hole static cone tests. The shear strength of clay-shale at the Dallas site was estimated to be about 310 kN/m^2, while the clay-shale at the Montopolis site had a shear strength of about 710 kN/m^2.

CONSTRUCTION DETAILS

Three shafts MT1, MT2, and MT3 were constructed in Montopolis using the following scheme. Three anchor shafts with enlarged bases were built such that their locations, in plan, were at the vertices of an equilateral triangle whose sides were about 5.5 m each. One test shaft was located at the midpoint of each side of the triangle. The scheme used for the reaction system is shown schematically in Fig. 4. This arrangement permitted each anchor shaft to be used twice during the three load tests. Besides, all the high strength bars could be recovered after completion of load tests. High strength bars were also used for the load test on the test shaft DT1 at the Dallas site.

Shaft MT1 was built by the casing method. Drilling was done to about 2.5 m from ground surface without the aid of bentonite slurry. Thereafter slurry was used to drill the hole up to the top of clay-shale,

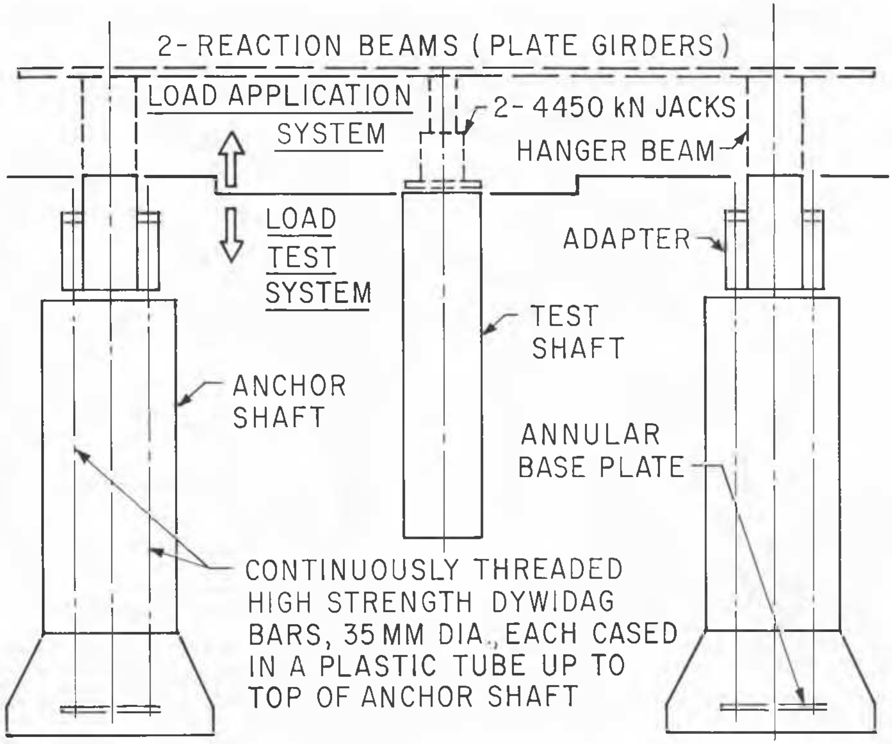

Fig. 4 Schematic Details of Reaction System

keeping the level of slurry within 0.75 m from ground surface. Chunks of soil brought out by the auger were continuously examined to mark strata changes. Drilling was stopped temporarily and a steel casing was "screwed" into the drilled hole until the casing penetrated about 0.10 m into the shale. All drilling mud was cleaned from within the casing and another auger was lowered through the casing to drill a dry hole into the clay-shale below the bottom of the casing without the aid of slurry. The steel cage, with Mustran cells, was lowered into the finished cased hole and concrete was placed with the aid of a tremie. The casing was gradually pulled out of the ground by means of a crane as the level of concrete rose inside the augered hole. Shaft DT1 was also constructed by casing method, with the exception that slurry was not used at any stage. Shaft MT2 was constructed by slurry displacement method using the procedure described by L. C. Reese, et al (1973). Shaft MT3 was drilled without any slurry. A cardboard tube (Sonotube) was used as a form for the upper portion of the shaft (see Fig. 2). An air void was kept between the outer steel casing and the Sonotube all the way from ground surface up to the top of clay-shale. Thus, shafts MT1 and DT1 were constructed by the casing method, MT2 by the slurry displacement method, and MT3 by the dry method.

TEST RESULTS AND INTERPRETATION

Typically, load was applied to the top of a test shaft by two hydraulic jacks which were pressurized by means of a hydraulic pump. Each load increment was applied in about 3 minutes and Mustran cell readings were taken 30 seconds after the load increment had been applied. Mustran cells located above the ground surface were utilized to obtain calibration curves for the outer Mustran cells within the test shaft, because of readings of Mustran cells above the ground surface could be correlated to the known load applied by the calibrated hydraulic jacks. The settlement near the top of the shaft was measured by dial gages.

The load-settlement curves for the test shafts are

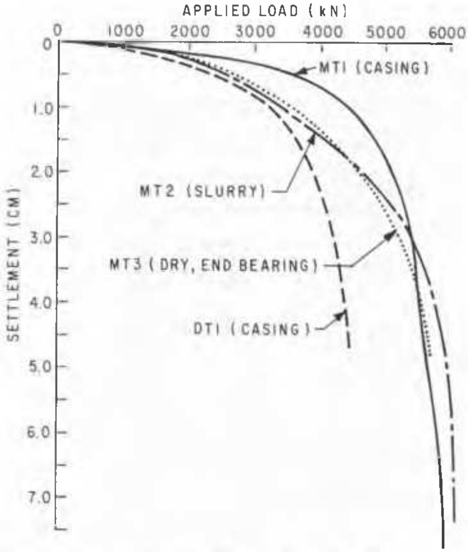

Fig. 5 Load-Settlement Curves of Test Shafts

shown in Fig. 5. These curves indicate that the shafts at the Montopolis site failed at applied loads of about 5000 kN, while the shaft at the Dallas site failed at applied loads of about 3500 kN. The settlement of the tops of the shafts at the failure loads was in the order of 4 cm.

Load distribution curves, similar to the one shown in Fig. 6, were obtained for all the test shafts. By

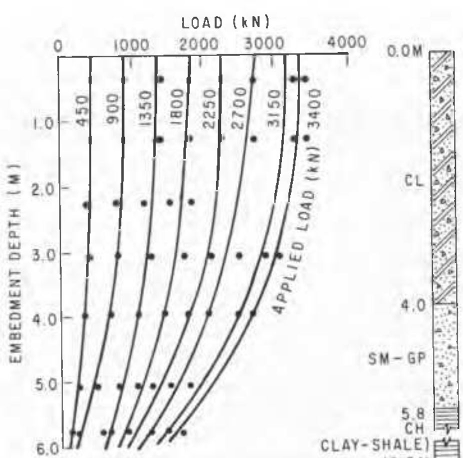

Fig. 6 Load Distribution Curves: Shaft DT1

differentiation and integration of the load distribution curves, curves can be obtained as a function of depth showing load transfer in side resistance versus relative movement between the shaft and the soil. In

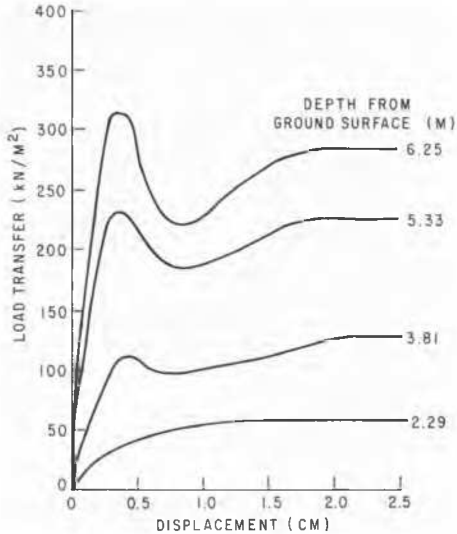

Fig. 7 Load Transfer Curves: Shaft DT1

performing the above computations it was necessary to have the top settlement and the shaft's geometry and elastic properties. An example of such curves is shown in Fig. 7 for shaft DT1. The upper two curves in this figure show an unusual drop in load transfer following the point of maximum load transfer. This is probably due to experimental error. Curves of load transfer in side resistance for shafts MT1 and MT2 could not be directly obtained due to malfunction of some Mustran cells, although the information on maximum load transfer in side resistance was obtained indirectly. The base resistance versus base movement curves for all the test shafts were obtained by analysis of field measurements and are presented in Fig. 8.

In March 1976, several papers were published as the Geotechnique Symposium in Print on Piles in Weak Rock, and some of these papers related to the behavior of axially loaded cast-in-place bored piles tested to failure or near failure (D. J. Mallard and J. L. Ballantyne, 1976; D. L. Webb, 1976; and L. C. Wilson, 1976). In writing the preface to the above noted symposium, M. J. Tomlinson (1976) presented an interesting summary of observed ultimate side resistance and base resistance of piles in weak rock. The upper portion of Table I is abstracted from Tomlinson's summary and the lower portion shows the observed results from the study reported herein.

The factor α is defined as the ratio of the maximum unit load transfer in side resistance to the shear strength of clay-shale. The values of α for the clay-shale were found to be 0.58, 0.52, 0.97, and 0.91 for MT1, MT2, MT3, and DT1, respectively. The

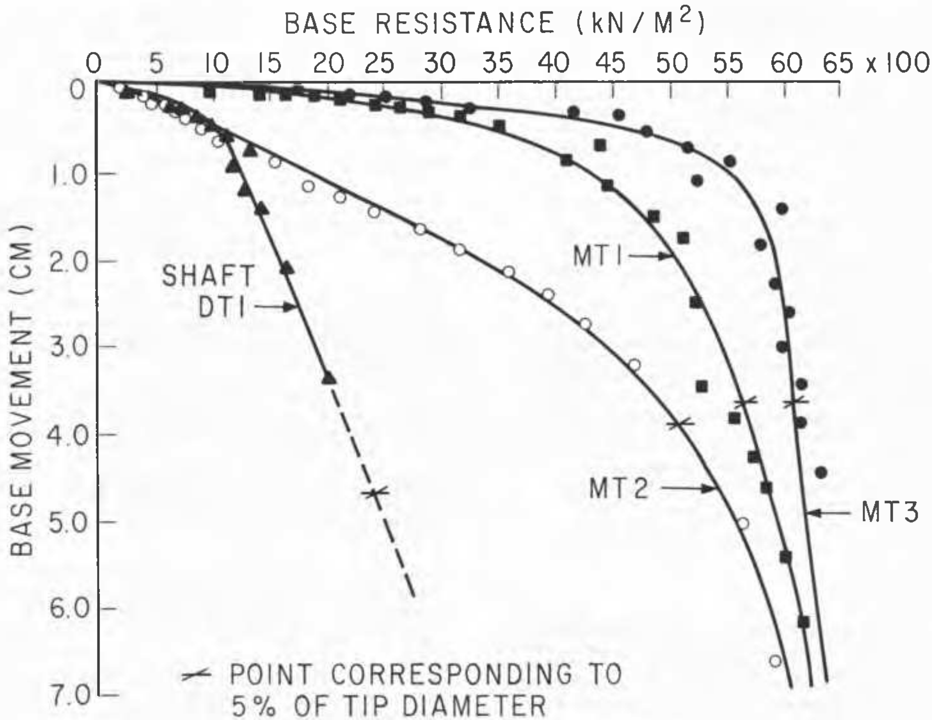

Fig. 8 Base Resistance Versus Base Movement Curves

Table I Ultimate Side Resistance and Base Resistance of Drilled Shafts in Stiff Soils and Rocks

Type of Soil or Rock	Ultimate Side Resistance kN/m^2	Ultimate Base Resistance kN/m^2	Diameter of Drilled Shaft mm	Reference
Chalk	500	-	1050	M. J. Tomlinson (1976)
Diabase (weak, clayey)	122	2650	615	
Mudstone (weak, clayey, cretaceous)	120 to 184	6800	670	
Shale (fragmented)	250	-	900	
Clay-Shale (at Dallas)	278	2443	889*	This Study
Clay-Shale (at Montopolis)	366 to 690	5125 to 6131	737* to 787*	
			* Base Diameter	

ultimate base resistance is defined as that corresponding to a tip movement of 5% of the tip diameter. The values of the bearing capacity factor N_c were found to be 8.0, 7.2, 8.6, and 8.5 for MT1, MT2, MT3 and DT1, respectively.

Generally, less than 0.5 cm movement was required to mobilize maximum load transfer in side resistance of clay-shale. Study of curves presented in Fig. 8 revealed that base movements of about 4 cm were required to mobilize ultimate base resistance.

With regard to the effects of various construction techniques, it can be noted that the test shafts MT3 and DT1, which were constructed without any slurry, exhibited high α values, in excess of 0.90. It must be pointed out that the α values, reported herein, are based upon the shear strength of clay-shale determined by applying a confining pressure equal to the estimated effective overburden pressure. An α value of 0.5 may be a reasonable estimate for shafts constructed by slurry displacement method or casing method when slurry is used; while a conservative α value of 0.75 may be appropriate for shafts using dry method or casing method without any use of slurry. The N_c values determined from ultimate tip resistance data also suggest the beneficial effects of avoiding the use of slurry. N_c value of 8.5 for shafts constructed by dry methods, and a value of 7.5 for shafts constructed by use of slurry, seem appropriate for design purposes, based on this limited experience.

CONCLUDING REMARKS

1. The ultimate axial capacity of drilled shafts in clay-shales is affected by the construction method.
2. Drilled shafts installed without the aid of slurry depicted α values in excess of 0.9, in comparison to α values close to 0.5 for shafts installed with the aid of slurry.
3. The N_c values of drilled shafts installed without the aid of slurry were about 8.5, in comparison to N_c values as low as 7.0 for shafts installed with the aid of slurry.
4. Less than 0.5 cm movement was required to mobilize maximum load transfer in side resistance of clay-shales, while a base movement of about 4 cm was needed to mobilize ultimate base resistance. Therefore, for computing allowable axial loads on drilled shafts in clay-shales, lower factors of safety for skin friction may be used in comparison to the factor of safety for base resistance.
5. The magnitude of load transfer in side resistance, ranging from 366 to 690 kN/m^2 is rather significant. Many codes of practice ignore side resistance of drilled shafts. The results of this study suggest that these codes are overly conservative.
6. Rather high base resistance values ranging from 5125 to 6131 kN/m^2, measured in this study, suggest that clay-shales can be used effectively to design an economical foundation system using drilled shafts. These values are far in excess of commonly used design values.

ACKNOWLEDGEMENTS

The research reported herein was sponsored jointly by the Texas Highway Department and the Federal Highway Administration. Farmer Foundation Company of Houston and Martin & Martin Company of Dallas, made significant financial contributions to the research project. The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the opinion or policies of any of the sponsors. This report does not constitute a standard, specification, or regulation.

REFERENCES

Barker, W. R., and L. C. Reese, (1969), "Instrumentation for Measurement of Axial Load in Drilled Shafts" Research Report 89-6, Center for Highway Research, The University of Texas at Austin, 1969.

Engeling, D. E., and L. C. Reese, (1974), "Behavior of Three Instrumented Drilled Shafts Under Short Term Axial Loading", Research Report 176-3, Center for Highway Research, The University of Texas at Austin, 1974.

Jorden, E. E., and M. Dobie, (1976), "Tests on Piles in Keuper Marl for the Foundations of a Blast Furnace at Redcar", Géotechnique, Vol. 26, No. 2, pp 105-114.

Mallard, D. J., and J. L. Ballantyne, (1976), "The Behavior of Piles in Upper Chalk at Littlebrook D Power Station", Géotechnique, Vol. 26, No. 2, pp 115-132.

Morgenstern, N. R., and K. D. Eigenbrod, (1974), "Classification of Argillaceous Soils and Rocks", Journal of the Geotechnical Engineering Division, Proceedings of the ASCE, Vol. 100, No. GT10, Oct. 1974, pp 1137-1156.

Peck, R. B., W. E. Hanson, and T. H. Thornburn, (1974), Foundation Engineering, Second Edition, John Wiley & Sons, Inc., New York, 1974.

Reese, L. C., M. W. O'Neill, and F. T. Touma, (1973), "Bored Piles Installed by Slurry Displacement", Proceedings, Eighth International Conference on Soil Mechanics and Foundation Engineering, Moscow, 1973, pp 203-209.

Tomlinson, M. J., (1976), "Preface (to Géotechnique Symposium in Print on Piles in Weak Rock)", Géotechnique, Vol. 26, No. 2, pp 1-4.

Webb, E. E., (1976), "The Behavior of Bored Piles in Weathered Diabase", Géotechnique, Vol. 26, No. 2, pp 63-72.

Wilson, L. C., (1976), "Tests of Bored Piles and Driven Piles in Cretaceous Mudstone at Port Elizabeth, South Africa", Géotechnique, Vol. 26, No. 2, pp 5-12.