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Power Line Foundation Design Using the Pressuremeter

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SYNOPSIS During the design phase of a 2000-towers electric power transmission line, a load test program was undertaken to evaluate the accuracy of new design methods for uplift and lateral capacity of drilled shafts. Seven uplift tests and six lateral load tests were performed in three different soil deposits: a medium clay, a very hard clay and a sand. The shafts were 2 ft in diameter and either 10 or 15 ft long. The pressuremeter test results are used together with existing methods to predict the behavior of the shafts.

INTRODUCTION

Foundations for free standing lattice type transmission towers generally consist of 4 drilled piers, one under each leg. Transmission towers must resist high overturning moments which are mainly due to high wind loads or broken wire conditions. Consequently, uplift loads generally control the foundation design of lattice type towers. However, lateral loads generally control the foundation design of two legged towers and pole structures.

This article addresses load tests which were utilized in the design of foundations for a transmission line which consisted of over 2000 free standing lattice type towers. The transmission line stretches over 500 miles from Utah to California. Load tests were performed at three representative sites in order to evaluate the precision of the pressuremeter methods for predicting the behavior of drilled piers subjected to vertical and lateral loads. A total of 6 lateral loads and 7 uplift tests were performed.

In the first part of this article, the soil properties including pressuremeter (PMT) and cone penetrometer (CPT) tests results are given for each site. In a second part, the behavior predictions based on PMT design methods are shown. In a third part, the load test program and results are presented. Finally, predicted and measured behavior are compared.

THE SOILS AND THE SITES

The primary zone of interest for this project is from the ground surface down to a depth of 15 ft. The water table was not encountered within that depth at any of the load test sites.

Delta Site: The soil at this site is a silty clay classified as CL. The average soil properties are as follows: Undrained shear strength 1.9 t/ft^2 calculated as 1/20th of the cone penetrometer point resistance, dry density 98 lb/ft^3, water content 24.5%, liquid limit 34.8%, plastic limit 17.8%.

Caliente Site: The soil at this site is a silty sand classified as SM-SP. The average soil properties are as follows: dry density 111 lb/ft^3, water content 4%. The average internal friction angle was 44° as obtained from the cone penetrometer point resistance by the Schmertmann method (18,9) and 48° as obtained from direct shear tests on recovered samples tested at in situ moisture conditions.

Alamo Site: The soil at this site is a silty to sandy clay classified as CL. The average soil properties are as follows: undrained shear strength 12.7 t/ft^2 calculated as 1/20th of the cone penetrometer point resistance, dry density 87.3 lb/ft^3, water content 15.5%, liquid limit 35.5%, plastic limit 13.5%A.

THE CONE PENETROMETER TESTS RESULTS

The cone penetrometer tests were performed by the Earth Technology Corporation using a standard electric cone pushed at 2 cm/sec using a 20-ton reaction truck. Both friction and point resistance profiles were recorded continuously and are shown on Figure 1 for the three sites.

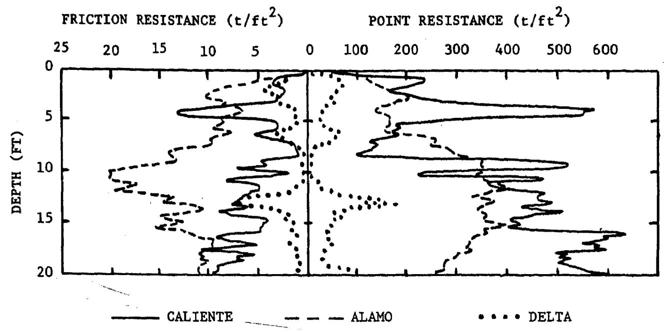

FIG. 1. Cone Penetrometer Data

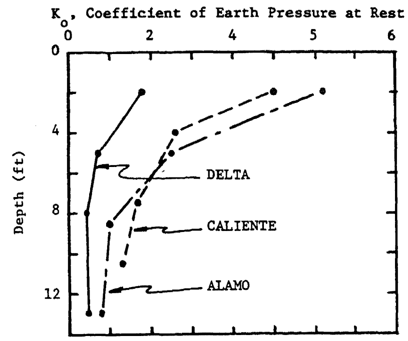

FIG. 3. Coefficient K_o Obtained from PMT Tests

THE PRESSUREMETER TESTS RESULTS

The TEXAM pressuremeter (4,9) allows to perform preboring and selfboring pressuremeter tests. It was used in preboring option at the three sites. The hole was prepared according to the tentative ASTM guidelines (5); at the Delta site it was done by hand augering in the dry; at the Caliente site it was done by rotary drilling with injection of foam and then of drilling mud; at the Alamo site it was done by rotary drilling with injection of air. For each test an unload-reload cycle was performed to measure a reload modulus E_R in addition to the conventional first load modulus E and net limit pressure p_L^* (Fig. 2).

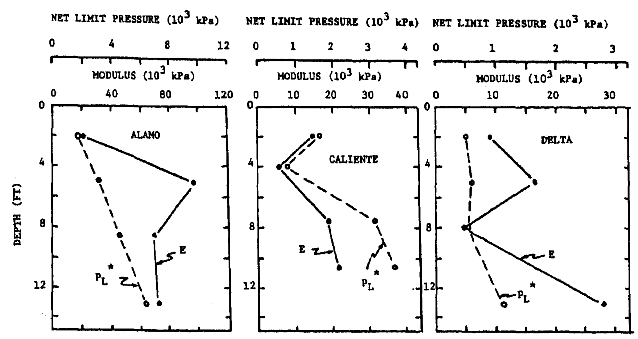

FIG. 2. Pressuremeter Data

The detailed pressuremeter curves can be found in Reference 3. The coefficient of earth pressure at rest K_o was evaluated from the beginning of the pressuremeter curve using a new method. The accuracy and reliability of the K_o values obtained from a preboring pressuremeter test by using the existing method (2) is a controversial matter. With this existing method the more disturbed the soil the higher the K_o value. The new method (4) is based on the analogy between the determination of the preconsolidation pressure from a consolidation test curve and the determination of the horizontal pressure at rest from a preboring pressuremeter test curve. This new method led to much more feasible K_o values (Fig. 3) than the existing method in those three soil deposits overconsolidated by dessication.

Undrained shear strength S_U values were obtai ed from the PMT results at Delta and Alamo usi four different methods (3). The highest value of S_U were the ones which corresponded best wi 1/20th of the cone point resistance; they were obtained by the method

$$S_u = p_y - p_{OH}$$

where p_y is the yield pressure in the PMT test (2) and p_{OH} is the total stress horizontal pre sure at rest.

THE LOAD TEST PROGRAM AND RESULTS

Four drilled shafts were constructed at each site, using a bucket auger in the dry. Shafts 1, 2 and 4 were approximately 10 ft long and shaft 3 was approximately 15 ft long. At Delt and Alamo the average diameter was 26 in. and 25.5 in., respectively. In the sand at Caliente the diameter varied from 26 in. to 36 in. averaging 29 in. The true diameters are repor ed in detail in Pacal and Shively (16).

Uplift tests and lateral load tests were performed. The shafts were loaded to failure or the maximum jack capacity of 200 kips. The lo was applied in equal increments, each incremen lasting 15 minutes on the average. Some shaft were loaded in uplift, some laterally, some bo in uplift and laterally. For the shafts which were loaded in uplift and laterally, the uplif test was performed first. The results of the seven uplift tests and the six lateral load tests are shown on Figs. 7 to 12 and Figs. 4 to 6, respectively.

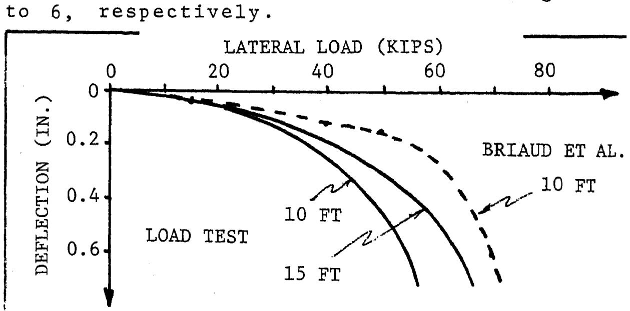

FIG. 4. Lateral Load Test: Delta Site

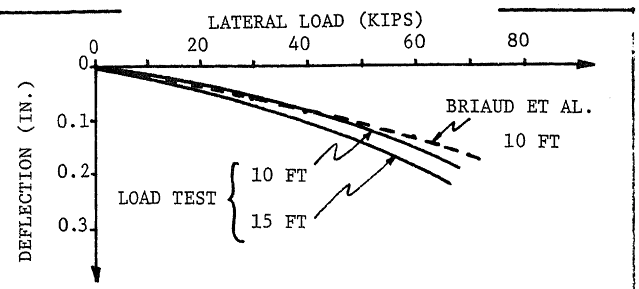

FIG. 5. Lateral Load Test: Caliente Site

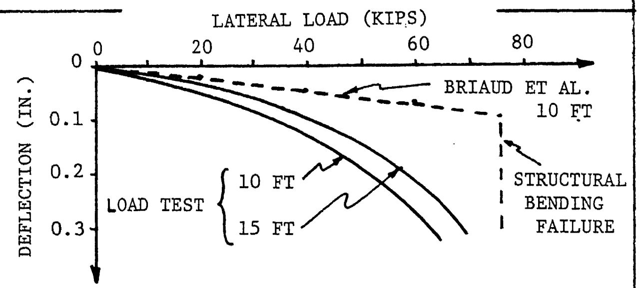

FIG. 6. Lateral Load Test: Alamo Site

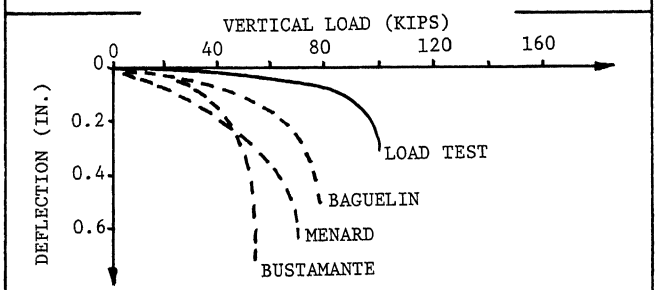

FIG. 7. Vertical Load Test: Delta Site - 10 Ft Shaft

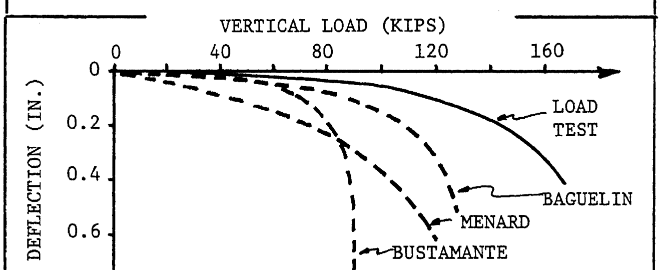

FIG. 8. Vertical Load Test: Delta Site - 15 Ft Shaft

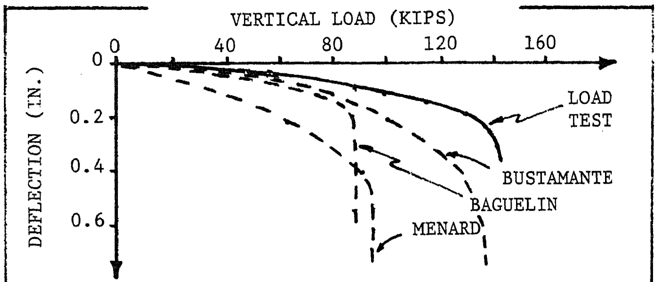

FIG. 9. Vertical Load Test: Caliente Site - 7 Ft Shaft

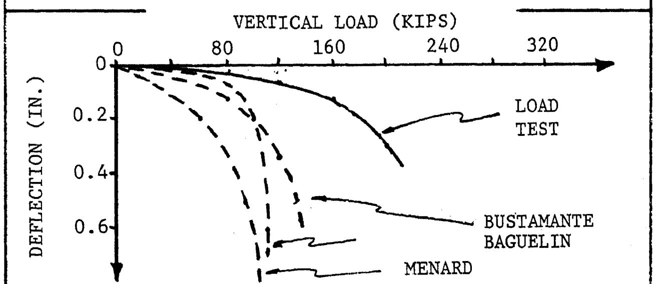

FIG. 10. Vertical Load Test: Caliente Site - 10 Ft Shaft

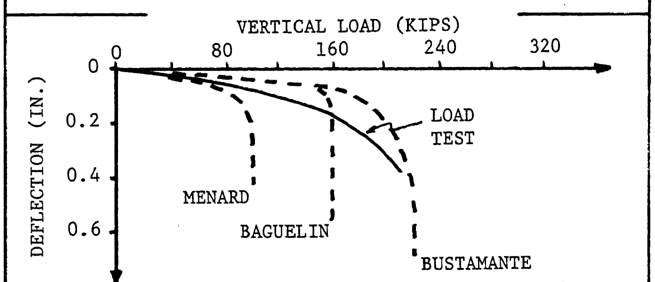

FIG. 11. Vertical Load Test: Alamo Site - 10 Ft Shaft

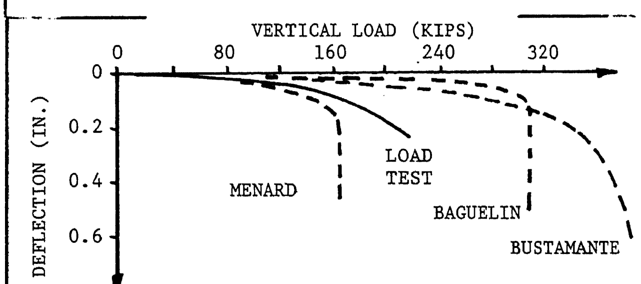

FIG. 12. Vertical Load Test: Alamo Site - 15 Ft Shaft

As can be seen on those figures, the "plunging" failure load can be estimated with reasonable accuracy for four of the seven tension tests. For the other three tests graphical extension of the load test curve to a deflection of 1/10th of the pile diameter was used; the interpretation necessary to extend the curve leads to a doubtful accuracy. The ultimate loads are listed in Table. 1.

LATERAL LOAD DESIGN METHOD AND PREDICTION

The method used is the one developed by Briaud, Smith, and Meyer (6,9,10,17). The method uses the finite difference approach to the solution of the governing differential equations and relies on soil reaction curves (p-y curves) obtained from the pressuremeter curve.

This PMT method uses two concepts which are different from current methods: 1. the depth D_c of the zone of reduced lateral resistance due to the lack of vertical confinement close to the ground surface is considered to be a function of the relative pile-soil stiffness instead of a function of the soil properties alone. 2. The soil lateral resistance is considered to be made

Site	Drilled Shaft Average Diameter (in.)	Drilled Shaft Length (ft)	Weight of Shaft (kips)	Maximum Load Applied (kips)	Estimated Ultimate Load (kips)	Baguelin Jezequel Shields (kips)	Bustamente Gianeselli (kips)	Menar Gambi (kips
Delta	26	9.4	5	102	110	78	54	78
	25	9.4	5	102	110	75	52	75
	26	14.4	8	158	170	126	84	126
Caliente	28.1	9.4	7	200	250*	108	166	128
	29.7	7.2	4	136	150	90	135	95
Alamo	25.5	8.9	5	200	250*	153	210	99
	25.5	13.9	7	200	350*	310	376	170

*these estimated ultimate loads are very doubtful because plunging failure was not reached and graphical extrapolation was necessary.

TABLE 1: Predicted and Measured Ultimate Capacities.

of two components: pile-soil friction and soil frontal reaction; these two components have been acknowledged for years for the vertical capacity of piles but have not yet been used for the lateral behavior of piles.

The results of the predictions using the above method are shown on Figures 4 to 6. These predictions were true predictions in the sense that the load test results were not known when the predictions were made. A similar true prediction was reported earlier (14).

VERTICAL LOAD DESIGN METHOD AND PREDICTIONS

Methods based on the pressuremeter were used to predict the ultimate uplift capacity and the load-displacement curve at the top of the shafts. Each one of the methods used was developed by more than one researcher; for convenience they will be referred to as the Menard method (13,15), the Baguelin method (1,2) and the Bustamante method (11,12). These methods are described in detail in reference 9. The results of predicted versus measured ultimate uplift capacity are shown in Table 1.

The predicted load-displacement curves are shown on Figs. 7 to 12 together with the load test results. All predictions were made prior to receiving the load test results.

ACKNOWLEDGEMENTS

The cone penetrometer tests and the laboratory tests were performed by the Earth Technology Corporation.

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