Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)

Pile foundation design comparison with load testing

Comparaison de la conception des pieux forés avec les essais de chargement

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ABSTRACT: The case of the review of the design of drilled shafts (bored piles) for an industrial facility in the South East of Mexico is presented. The geotechnical information of the site is included, and from CPTU and SPT tests a geotechnical model is presented, from which a preliminary design is made. Likewise, the results of twelve load tests carried out at the site are reported, with diameters between 60 and 100 cm, and around 30 m depth; two of these tests were instrumented along the shafts. Based on the test results, design is reviewed, and pertinent conclusions are issued.

RÉSUMÉ: Le cas de l'examen de la conception des pieux forés pour une installation industrielle dans le sud-est du Mexique est présenté. L´information géotechnique du site est incluse, et à partir des tests CPTU et SPT un modèle géotechnique est présenté, à partir duquel une conception préliminaire est faite. De même, les résultats de douze essais de chargement réalisés sur le site sont rapportés, avec des diamètres compris entre 60 et 100 cm, et une profondeur d'environ 30 m; deux de ces tests ont été instrumentés. Sur la base des résultats des tests, la conception est revue et des conclusions pertinentes sont émises.

KEYWORDS: drilled shafts, bored piles, static load testing

1 INTRODUCTION

An industrial facility is built in the Southeast of Mexico, in an area of 700 Ha, approximately. The site is in a coast plain of the Gulf of Mexico, in an area with low and scarce hills. The soils are mainly deltaic deposits of loose sand in the upper part, above dense sand.

The area is in a high risk of marine and river flooding, and in very high risk of flooding by storm tide and high energy waves, therefore a 4.5 m fill was made in the whole zone. Afterwards, a soil improving was made, using dynamic compaction technique, for the loose fine sand found in the upper 10-12 m, to reduce the liquefaction risk, due to the high seismicity involved.

Most structures were founded with around 25,000 drilled shafts (bored piles), although in some structures rigid inclusions and a concrete slab were used. As part of the QA/QC program, static load tests were performed, in compression, tension and lateral load. In this paper twelve test results from compression tests are presented, comparing the outcome with the analytical design.

2 PROJECT DESCRIPTION

2.1 *Geotechnical conditions at site*

An intense exploration campaign was conducted to know the geotechnical conditions at site, including SPT, CPTU borings, as well as vibrating wire piezometers. Borings were performed before and after the fill placement, that was used as working platform, and was completed with the CPTU carried out for the dynamic compaction control. Figure 1 shows a representative profile of a CPT boring.

Table 1 presents the description of the different geotechnical units identified after the exploration campaign, as well as the geotechnical model developed after the field and laboratory tests. Selected parameters for the analytical design are in undrained conditions.

Table 1. Geotechnical model of the site

GU	Description	Depth (m)	SUCS	N_{SPT}	w %	% Fines	γ (kN/m^3)	c (kPa)	ϕ	Su (kPa)	E (MPa)
1	Soft and medium brown clay, low to high plasticity, with interbedding of dark brown clayed sand	0.00 a 0.55	CH, CL, SC	9	31	59	16.8	-	-	80	7.0
2	Poor graded dark gray fine sand Arena, loose and medium compacity, with shells and microfossils remains and a variable content of fine contents	0.55 a 6.20	SP, SP-SC, SC	13	22	10	16.7	0	39	-	16.5
3	Soft and very soft dark clay, low and high plasticity	6.20 a 14.00	CL, CH	4	40	88	16.8	-	-	40	6.4
4	Dark gray and yellowish brown clayed sand, with low to high compacity, with small shells and a variable content of fines	14.00 a 23.40	SC, SP-SC, CL	10	25	20	19.6	55	11	-	19.3
5	Greenish-gray and yellowish-brown clayed sand, with low to high compacity	23.40 a 27.90	SC, SP-SC	34	25	24	19.3	10	15	-	53.0
6	Olive and dark gray clay, with low to high plasticity and medium to high consistency.	27.90 a 45.70	CL, CH, SC	40	28	75	18.7	-	-	281	36.7

2.2 *Preliminary design of ultimate capacity of piles*

For the analytical design of the piles, LCPC-CPT method was used (Bustamante y Gianeselli, 1982), which estimates the point and skin friction ultimate capacity with the following equations:

$$q_p = k_c\, q_{ca} \tag{1}$$

$$f_p = q_c / \alpha_{LCPC} \quad < J \tag{2}$$

where q_p is the ultimate point capacity, k_c is a bearing capacity coefficient (empirical), q_{ca} is the cone point resistance (average of values 1.5 d up and down the pile tip), f_p skin friction of pile, q_c is the cone point resistance in the length of interest, α_{LCPC} is a friction coefficient (empirical), and J is the limit value of f_p.

For the soil in the site, the values considered for bearing capacity and friction are shown in Table 2, for drilled shafts with casing (A) and without casing (B, I).

Table 2. Values of k_c y α_{LCPC} used in ultimate pile capacity calculation

Type of soil (consistence / density)	q_c (MPa)	k_c	α_{LCPC}		J
		I	A	B	MPa
Soft clay	<1	0.4	30	30	0.015
Medium clay	1 a 5	0.35	40	80	0.035
Firm silt and clay	>5	0.45	60	120	0.035
Silt and loose sand	< 5	0.4	60	150	0.035
Medium dense sand and gravel	5 a 12	0.4	100	200	0.035 0.08
Very dense sand and gravel	>12	0.3	150	300	0.08 0.12

Estimate of skin friction capacity was made with and without limit of f_p; however, in the comparison with load tests results, the value of f_p was not limited.

In Tables 3 and 4 load bearing capacities are shown, while in Figure 11 load transfer curves both estimated and measured are drawn for Site 1, in 100 cm diameter piles.

Table 3. Ultimate load capacities calculated with unlimited f_p

Site	Ultimate load capacity, kN					
	d=0.6 m		d= 0.8 m		d=1.0	
	q_p	q_s	q_p	q_s	q_p	q_s
1_{CCD}	991	4611	1756	6484	2747	7809
1_{SCD}	991	4405	1756	5866	2747	7338
2	912	4856	1628	6475	2551	8093
3	952	4316	1697	5886	2649	7514

Note:
CCD after dynamic compaction
SCD before dynamic compaction

Table 4. Ultimate load capacities with $f_p < J$

Site	Ultimate load capacity, kN					
	d=0.6 m		d= 0.8 m		d=1.0	
	q_p	q_s	q_p	q_s	q_p	q_s
1_{CCD}	991	1746	1756	2325	2747	2884
2	912	1825	1628	2423	2551	3031
3	991	1746	1756	2325	2747	2884

Note:
d pile diameter
q_p point bearing capacity of pile
q_s skin friction bearing capacity of pile

It can be observed that to consider the restriction fp< J seems too conservative.

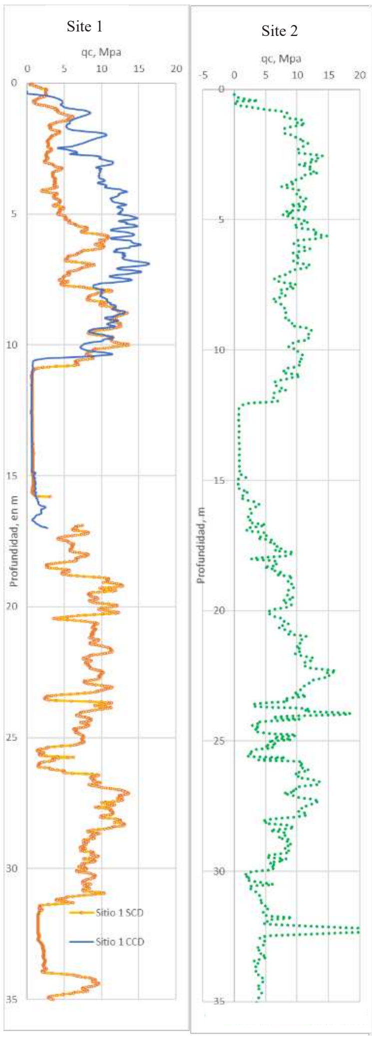

Figure 1. CPTU borings. For Sites 1 and 3 same information was used from 16 to 35 m depth.

2.3 *Construction procedure of piles*

Drilled shaft construction was performed with the aid of a steel casing, installed with a vibrodriver (see Figure 2), with a length between 12 and 15 m; afterwards, boring inside the casing was made, using bentonite mud, until the project depth was reached. Then, steel cage was placed, and concrete poured with tremie procedure (Figure 3). After placing concrete, casing was retrieved using the vibrodriver. Pile diameters were 60, 80 and 100 cm. A fraction of the piles was performed with the continuous flight auger procedure (CFA). Pile integrity tests were carried out, randomly, in a representative amount of piles.

Figure 2. Steel casing placement with vibrodriver.

Figure 3. Overview of drilled shafts construction.

3 COMPRESSION STATIC LOAD TESTS

Static load testing was performed in three different sites, named Site 1 to 3. In Table 5 general characteristics of piles are presented, for each site. Piles were built solely for testing purposes. As reaction system, similar piles were used, placed around the test pile. This was complemented with a steel frame, formed with three beams, Figure 4.

Between the load-reaction system, concrete cubes were built, for transmission of load from the steel frame to the reaction piles, using high-strength bars. In the test pile a concrete header was built for load application.

For load application, a set of 200 t hydraulic jacks system was used. Load was measured using electronic cells in the head of the pile. For deformation measurements dial indicators were placed at 120º between them, plus a graduated scale in one of the faces of the test pile header.

Load testing program, as well as increment load duration were made after ASTM-D1143 standard. In two cases, the body of the pile was instrumented using steel and concrete strain gages, plus tell tales.

Figure 4. General arrangement for compression static load test.

4 TESTS RESULTS

4.1 *Interpretation of load tests*

Figures 5, 6 and 7 show the load-displacement plots, for the different test pile diameters, regardless of the site. Site 1 tests are noted, since a set of tests were made in a field with dynamic compaction (CCD), and the other without dynamic compaction (SCD). Between Site 2 and 3 there is a distance of 400 m approximately, and between this two and Site 1, around 600 m.

In Figures 8 and 9 load transfer curves are presented, for 100 cm diameter piles, instrumented in Site 1.

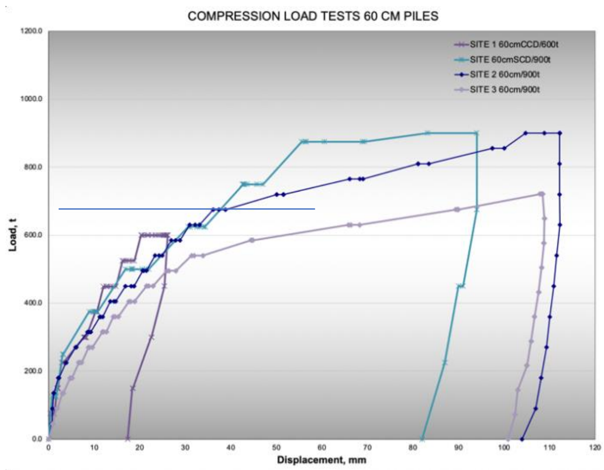

Figure 5. Compression test plots for 60 cm de diameter piles.

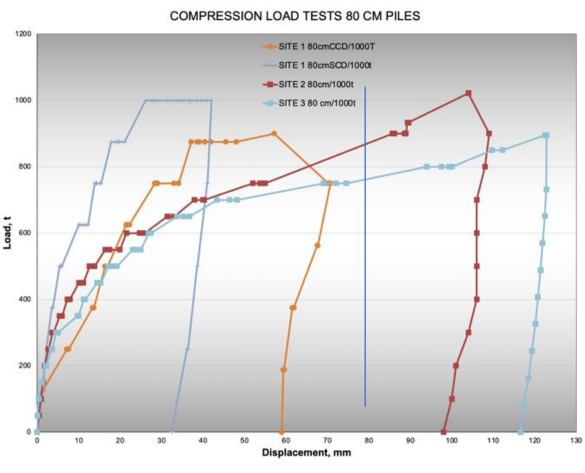

Figure 6. Compression test plots for 80 cm de diameter piles.

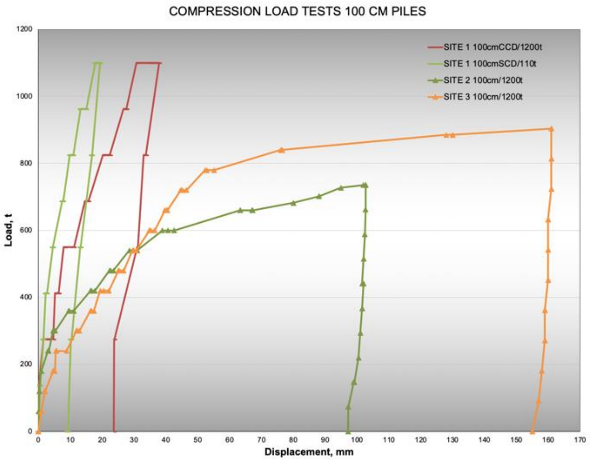

Figure 7. Compression test plots for 100 cm de diameter piles.

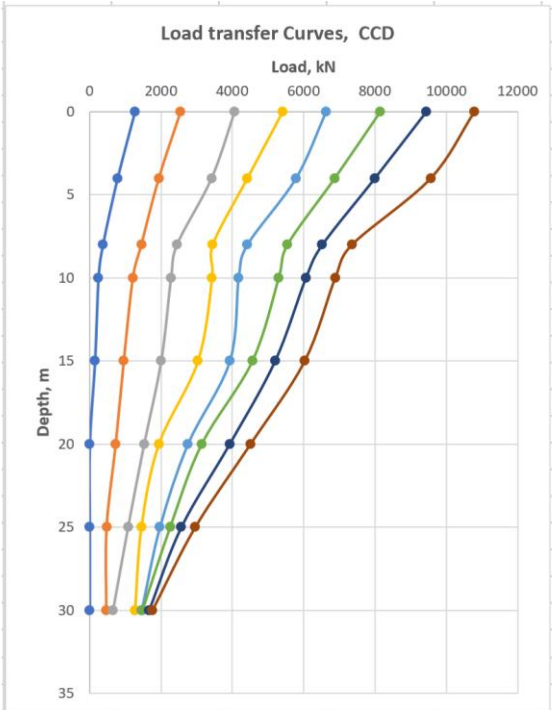

Figure 8. Load transference curves for 100 cm diameter pile in Site 1, with dynamic compaction.

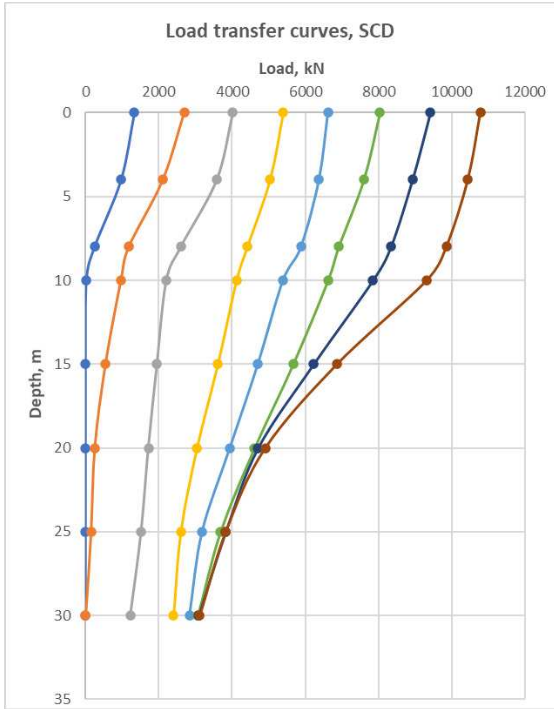

Figure 9. Load transference curves for 100 cm diameter pile in Site 1, without dynamic compaction.

4.2 *Analysis of results of load tests*

Table 6 shows main results and interpretation data. In Figure 10 a comparison of the bearing load capacities is presented, after the different procedures appointed in Table 5.

Table 5. Main results from load tests.

Site	Diám.	Qmax	Qult (1)	Qult (2)	Qult (3)	Qult (4)
	m	kN	kN	kN	kN	kN
1 CCD	60	5886	5886	-	5602	2737
	80	8829	7358	-	8240	4081
	100	10791	9437	-	10556	5631
1 SCD	60	8829	-	8584	5396	-
	80	9810	-	-	7622	-
	100	10791	-	-	10085	-
2	60	8829	6622	7358	5768	2737
	80	10026	5886	8633	8103	4052
	100	7210	4326	7161	10644	5582
3	60	7073	5297	6082	5268	2737
	80	8780	5396	7554	7583	4081
	100	8868	4120	8437	10163	5631

Ultimate load determined with:

(1) Offset Davisson (1972) criteria
(2) 10% of pile diameter displacement criteria
(3) Calculated in Table 3, with unlimited value of f_p
(4) Calculated in Table 4, with f_p < J

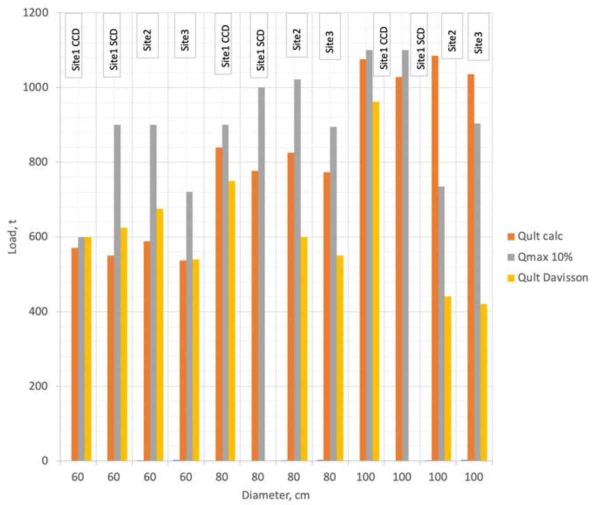

Figure 10. Ultimate load comparison, after different criteria.

It can be observed that, for 60 cm diameter piles, agreement between load determined with offset Davisson method and analytical calculation, is reasonably similar. For 80 and 100 cm diameter piles, the agreement comes closer with the criteria of failure at 10% of the diameter of the pile.

As for the load transfer curves, for 100 cm piles, in Figure 11 a comparison between the measured and calculated behavior is presented. It can be seen that final values of bearing capacity are very similar; however, both tests were carried out only with a displacement around 3% of the diameter, therefore ultimate tip capacity may not be fully developed. Besides, the load transfer curves measured reflect skin friction values relatively high in the soft clay and fine shallow deposits, and low values in the sandy clays of medium to high compacity. This could be provoked by residual stresses in the pile (Fellenius, 2015), due to settlements after the fill placement, generating consolidation in soft clay stratums between 10 and 15 m in the testing sites.

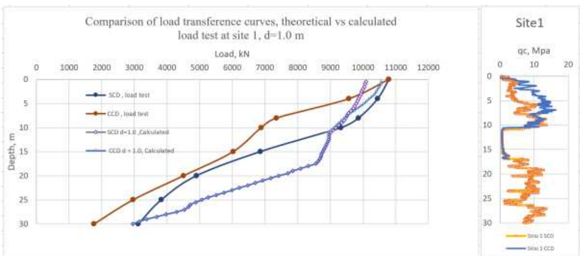

Figure 11. Load transfer curves comparison (calculated vs. measured), Site 1, 100 cm diameter pile

5 CONCLUSIONS

A comparison is presented, between analytical calculation of load bearing capacity of pile foundation and results from full-scale load tests, for piles with diameters of 60, 80 and 100 cm, with 29 and 30 m in length, for three sites relatively close from each other, during the construction of an industrial facility in the southeast of Mexico.

Comparing the results of measurements with analytical calculations, it is noted that the restriction of $f_p < J$ seems extremely conservative, as load testing results showed.

For load tests, interpretation of ultimate load with two different criteria does not show a clear convergence with analytical calculation, or even between them. Although some authors (Fellenius, 1980; Ng et al., 2004) suggest interpretation with at least four different procedures, uncertainty remains. For ultimate load determination, authors suggest the criteria of failure at a displacement equivalent of 10% of pile diameter, and the use of conservative safety factors, especially in sites where soil conditions may vary in short distances.

Instrumentation in the body of two 100 cm diameter piles revealed abnormal load transfer curves, with relatively high skin friction values in the upper portion of the piles, which may be due to residual stress caused by the settlements induced by the embankment built to rise the working platform.

After load testing and its comparison with calculated values from close CPTU, it is possible to apply a statistical approach with the results of CPTU in the rest of the site, to achieve a more rational design for the piles.

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