AXIAL LOAD TESTS IN DRILLED SHAFTS AND PILES IN A REFINERY: COMPARISON BETWEEN DESIGN AND EXPERIMENTAL RESULTS

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Drilled shafts and precast concrete piles were installed for the foundation of several structures in a reconfiguration project of a refinery in Minatitlan, Veracruz, in the Gulf of Mexico. The soils are alluvial deposits of flat land from the Coatzacoalcos River, which are embedded with layers of sand, soft clay, and peat. As part of a QC/QA program, load tests, both static and dynamic, axial and lateral, were performed. In this paper results of 32 axial tests are presented, in which typical load-displacement curves were measured at the top of the foundation elements; from the interpretation of these plots, data were recorded for ultimate side shear and in some cases for end bearing capacities, which are compared with the theoretical results of the final stage of the foundation design. The comparison shows that under allowable loads, side shear determines the bearing capacity of piles, even when the tip is embedded in a hard layer. Underestimation of the side shear is evident; therefore design criteria, as well as the methods used for obtaining soil parameters, should be reviewed.

INTRODUCTION

Project information

PEMEX developed the reconfiguration of a refinery located in Minatitlán, Veracruz (Fig. 1).

Fig 1. Project location

The main purpose of this project is to increase the production of processed Maya oil, for which the facilities were to be expanded to 72 Ha.

The structures for which the deep foundations were designed were: great-diameter vertical tanks, cooling towers, burners, pipe racks and turbo-generators, among others.

The main foundation solution for the heavier structures –or structures sensitive to differential settlements– was of precast concrete piles or drilled shafts cast in place, placing the tip into a hard stratum. Diameters of drilled shafts vary from 0.8 to 1.0 m, with lengths between 14 and 45 m; precast concrete piles were built with a square section of 0.4 to 0.5 m, with length varying between 27 and 48 m.

Soil conditions

The work site is on the left margin of the Coatzacoalcos River (near the Gulf of Mexico) and is located beside the old refinery. The soil is formed with embedding of clay and sand deposits, in a marginal lagoon, where recent granular fills were placed, to raise new platforms. The area was extensively studied in different phases; general conditions are as follows:

Unit 1. Recent granular fill (0.0 to 2.0m)
Unit 2. Old granular fill (2.0 to 5.2m)
Units 3,4 and 5. Alluvial cohesive soils (5.2 to 12.6m)
Unit 6. Alluvial granular soils (12.6 a 31.6m)
Unit 7. Deep granular deposits (31.6 to > 60m)

The water table was found at an average depth of 2m. Figures 2 and 3 show a plan view and a stratigraphic profile of the entire working area. Although this was the typical stratigraphy, notable variations of depth were found at Unit 7. Figure 2 also shows the location of the 32 load tests. Table 1 presents a synthesis of soil properties in each test site.

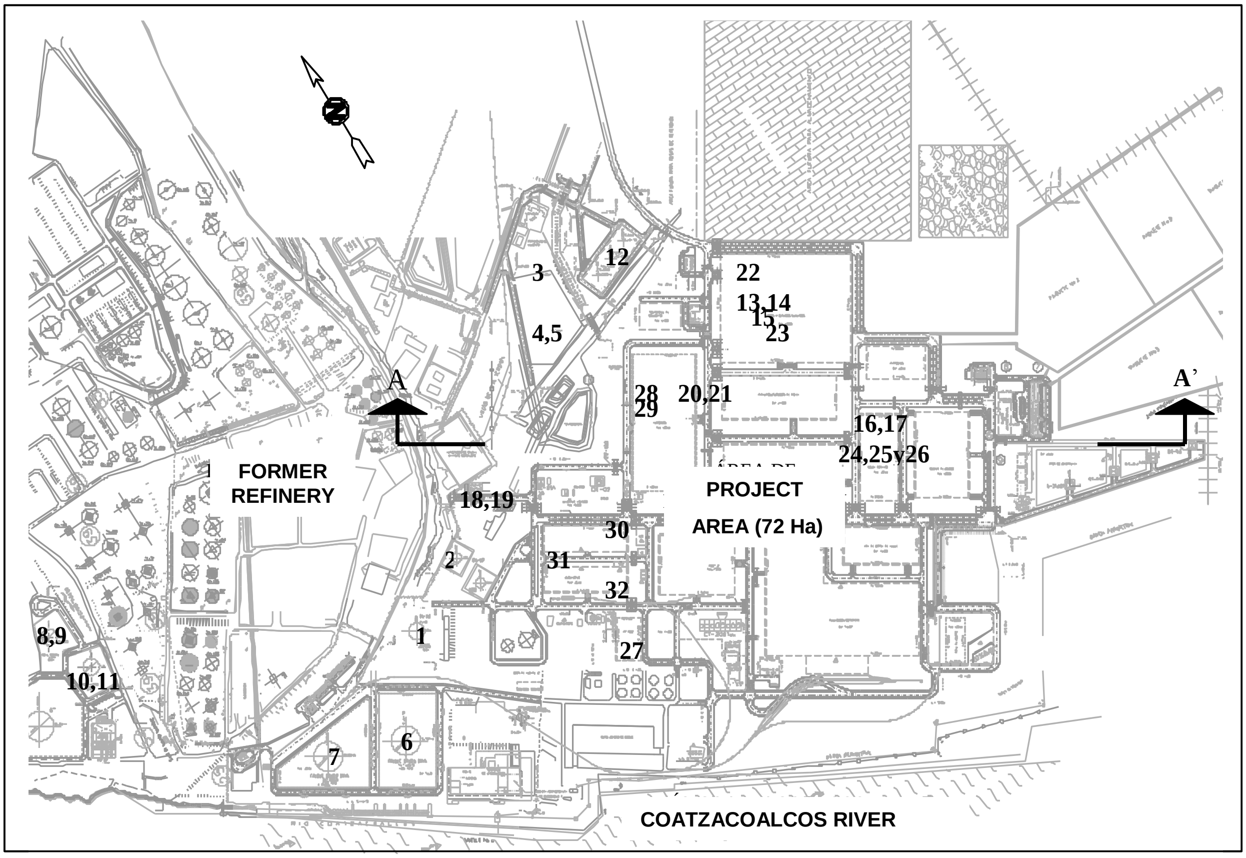


Fig 2. Plan of general view, with locations of test sites

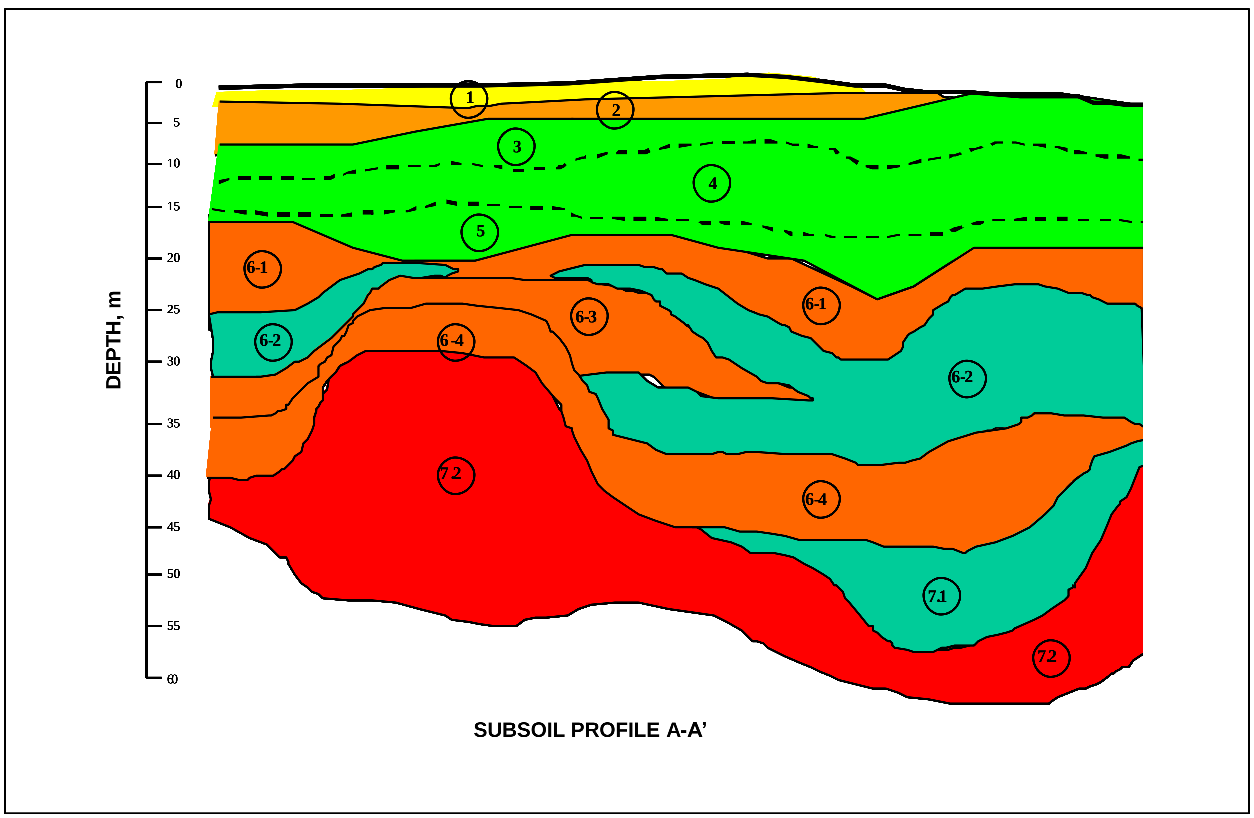


Fig 3. General stratigraphic profile

Table 1 Soil properties in test sites

UNIT	PROP.	SITES LOCATIONS																				
		1	**2**	**3**	**4,5**	**6**	**7**	**8, 9**	**10, 11**	**12**	**13,14,15**	**16,17**	**18,19**	**20,21**	**22**	**23**	**24,25,26**	**27**	**28, 29**	**30**	**31**	**32**
1 (SP,SP-SC)	dh (m)	0.5	3.0	1.6	1.5	1.6	2.6	1.1	0.0	2.0	1.4	4.0	0.0	2.4	2.0	2.0	4.0	1.2	1.5	0.0	0.0	0.0
	N	22	11	20	20	28	28	20	-	26	20-50	20-50	-	12-30	25-43	10-50	12-20	27	20	-	-	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	17.7	20.4	-	-	14.7	-	20.4	-	-	-	-	-
	w (%)	20	15	20	26	15	5	14	-	14	16	23	-	50	18	20	23	18	18	-	-	-
	γ (kN/m3)	18.0	17.1	17.5	17.5	18.0	17.9	18.3	-	17.3	17.2	17.7	-	17.3	17.2	17.2	16.9	17.3	18.1	-	-	-
	Cu (kPa)	0.0	0.0	0.0	0.0	0.0	0.0	9.8	-	0.0	0.0	9.8	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-
	ϕ (°)	36.0	31.5	25.0	30.0	36.0	36.0	25.0	-	34.0	30.0	36.0	-	27.0	30.0	30.0	28.0	37.0	30.0	-	-	-
2 (SP,SP-SC)	dh (m)	2.3	2.0	0.0	0.5	6.4	4.0	3.3	2.2	1.3	3.7	1.9	2.9	3.2	5.5	6.0	3.2	0.0	0.0	3.0	5.0	4.0
	N	14-18	6	-	10	20-25	39	10-18	20	15	16	20-30	12-20	12-15	5-24	4-17	8-10	-	-	9	6	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	13.9	5.9	2.5	-	-	13.9	-	-	-	-	3.9
	w (%)	23	15	-	41	17	8	22	14	14	24	26	18	20	22	30	26	-	0	20	15	-
	γ(kN/m3)	17.7	16.5	-	16.5	17.9	18.2	19.3	19.0	16.6	17.7	16.7	17.5	16.8	17.7	17.7	16.7	-	-	17.4	16.5	17.5
	Cu (kPa)	17.2	0.0	-	0.0	0.0	0.0	1.8	1.5	0.0	0.0	9.8	0.0	0.0	0.0	0.0	19.6	-	-	0.0	0.0	0.0
	ϕ (°)	20.0	28.5	-	28.0	35.0	38.5	30.5	33.0	30.0	29.0	26.0	30.0	24.5	29.0	29.0	21.0	-	-	29.5	28.5	29.0
3 (CH, MH)	dh (m)	0	2.8	1.4	1.3	2.6	0	5.2	7.0	3	0.0	0.0	1.6	1.2	0.0	0.0	0.0	3.2	4	3.0	2.8	1.5
	N	-	7	4	7	22-38	-	5-35	3-10	2	-	-	7	8	-	-	-	6	4-10	5	7	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	-	2.5	1.5	-	0.8	-	-	-	-	-	0.8
	w (%)	-	30	-	110	34	-	18	28	50	-	-	20	54	-	-	-	48	48	59	30	-
	γ(kN/m3)	-	17.4	16.4	17.3	17.5	-	19.1	19.6	15.7	-	-	17.3	17.0	-	-	-	17.8	16.8	17.2	17.4	17.3
	Cu (kPa)	-	10.3	41.7	19.6	9.8	-	8.8	16.9	14.7	-	-	19.6	19.6	-	-	-	29.4	12.8	14.7	10.3	14.7
	ϕ (°)	-	28.0	19.0	23.0	28.5	-	20.0	22.0	23.5	-	-	23.0	23.0	-	-	-	24.5	18.5	17.0	28.0	17.0
4 (CH, OH)	dh (m)	2.3	1.6	4.0	3.5	0.0	4.4	3.2	4.8	3.0	4.8	14.0	1.5	4.2	4.0	5.0	5.0	3.8	2	2.0	1.6	5.5
	N	6	10	1	3-7	-	2	9-11	4-11	2	5-9	2-9	6	6-9	4-10	2-10	5-8	2	3	6	10	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	1.3018	1.5	0.5	-	0.5	1.301787	-	-	-	-	0.4
	w (%)	41	160	117	202	-	347	109	95	202	175	42	40	291	100	100	42	137	137	50	160	-
	γ(kN/m3)	15.3	15.1	13.5	16.4	-	14.0	17.7	18.7	13.5	15.9	17.2	16.2	16.4	18.7	18.7	16.1	15.7	16.5	16.0	15.1	16.7
	Cu (kPa)	22.1	13.7	11.0	13.7	-	24.5	17.2	18.6	0.0	19.6	39.2	17.2	19.6	18.6	18.6	24.5	17.2	58.9	4.9	13.7	9.8
	ϕ (°)	18.5	26.0	4.8	20.0	-	22.0	23.5	27.0	26.5	20.5	2.5	21.0	22.0	27.0	27.0	21.0	18.0	12.0	15.0	26.0	20.0
5 (CH, MH)	dh (m)	1.5	3.4	1.3	1.2	0.0	9.0	6.2	4.4	19.3	3.0	3.2	5.0	2.2	5.0	4.0	0.0	5.6	8.0	4.0	3.4	11.5
	N	35	6	4	12	-	2	37-50	28-44	3-8	7	3-18	3-6	4-11	10-38	5-32	-	3	2-4	3-7	6	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	2.1	0.5	1.5	-	0.8	-	-	-	-	-	0.5
	w (%)	36	110	28	54	-	32	48	30	33	48	43	15	50	35	44	-	44	44	40	110	-
	γ(kN/m3)	18.1	16.8	16.6	17.2	-	16.5	17.0	18.3	16.2	16.7	17.8	16.7	17.1	16.7	16.7	-	15.6	15.7	15.7	16.8	17.7
	Cu (kPa)	41.2	21.4	46.6	31.9	-	30.4	43.3	51.5	9.8	22.6	33.4	15.7	26.5	22.6	22.6	-	12.3	54.0	9.8	21.4	9.8
	ϕ (°)	26.0	23.5	8.0	24.5	-	26.4	30.5	25.0	26.0	26.5	2.5	23.5	24.0	26.5	26.5	-	24.5	14.0	19.5	23.5	18.5
6.1 (SC)	dh (m)	11.0	5.3	7.0	4.2	2.4	3.0	2.6	0.6	0.0	15.7	1.8	0.0	7.8	3.0	5.8	9.0	6	0.0	3.0	5.3	5.1
	N	40-65	28-45	12-38	11-16	48	31	43	50	-	15-29	8-37	-	20-38	14-42	10-23	14-18	35	-	13	28-45	22-42
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	10.0	-	3.9	-	4.9	10.0	-	-	-	-	-
	w (%)	20	27	21	19	25	17	23	20	-	26	31	-	12	28	30	31	20	-	20	27	20
	γ(kN/m3)	19.1	18.2	17.1	16.3	18.4	18.2	18.4	18.5	-	16.8	18.6	-	17.4	16.8	16.8	17.8	16.9	-	17.3	18.2	17.3
	Cu (kPa)	0.0	0.0	0.0	0.0	0.0	0.0	0.2	9.8	-	0.0	0.0	-	0.0	0.0	0.0	17.7	0.0	-	0.0	0.0	0.0
	ϕ (°)	35.6	34.5	28.4	28.0	37.0	36.8	37.5	37.0	-	32.0	31.0	-	32.0	32.0	32.0	27.0	35.5	-	28.0	34.5	28.0
6.2/6.3 (SC, CL)	dh (m)	5.2	3.7	6.8	4.3	10.6	3.0	6.3	4.6	2.0	6.0	5.3	0.0	9.0	13.0	7.5	8.0	5.9	15.5	17.0	3.7	0.0
	N	45-49	48	4-8	14-28	34-65	7	62-77	32-41	15	13-31	21	-	10-60	10-50	10-44	4-14	10	6-30	12-40	48	-
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	5.4	-	2.9	-	-	5.4	-	-	-	-	-
	w (%)	35	35	33	16	29	41	30	26	27	42	43	-	20	33	22	43	18	18	25	35	-
	γ(kN/m3)	18.3	18.4	17.0	17.3	18.6	17.1	19.4	18.9	17.5	17.7	17.5	-	18.4	17.7	17.7	17.5	16.8	17.2	18.0	18.4	-
	Cu (kPa)	47.1	47.8	13.5	0.0	24.9	12.07	78.5	54.0	5.9	17.2	88.3	-	2.5	17.2	17.2	34.3	34.3	29.4	9.8	47.8	-
	ϕ (°)	27.0	27.5	20.0	30	27.3	27.3	30.0	27.0	32.0	28.0	4.5	-	34	28.0	28.0	22.5	26	25.0	25.0	27.5	-
6.4 (SC, SM)	dh (m)	-	1.0	6.5	4.6	3.4	4.0	0.0	1.0	0.0	1.2	2.1	5.0	2.8	3.5	12.0	13.8	7.8	2.5	7.0	1.0	4.8
	N	-	56-68	45-81	54-61	80-95	38-45	-	-	-	45	>50	38-46	73-78	54-69	20-50	30-45	40	38	40-46	56-68	51-76
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	-	-	14.7	-	-	-	-	-	-	-	-
	w (%)	-	30	24	19	23	22	-	-	-	24	30	16	40	22	20	30	18	18	18	30	18
	γ(kN/m3)	-	18.9	19.0	18.8	19.1	18.5	-	-	-	18.8	18.6	18.7	19.4	18.8	18.8	18.6	19.1	18.0	18.3	18.9	18.3
	Cu (kPa)	-	5.9	0.0	0.0	8.3	27.5	-	-	-	0.0	9.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.9	0.0
	ϕ (°)	-	35.5	37.3	36.0	35.8	37.5	-	-	-	36.0	34.3	34	35	36.0	36.0	31.5	36.5	33.0	34.5	35.5	34.5
7.1/7.2 (SC, SM)	dh (m)	-	5.0	-	7.3	8.0	6.0	2.1	6.4	7.0	4.4	8.9	4.0	6.2	4.0	7.3	6.2	6.5	6.5	6.0	5.0	6.0
	N	-	80	-	42-75	54-100	72-96	58	71-91	22-66	61-100	50-56	58-67	66	51-59	50-73	26-60	68	45-64	58-80	80	53-68
	qc (MPa)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	w (%)	-	18	-	23	20	21	28	24	18	22	18	11	25	24	15	18	14	14	15	18	-
	γ(kN/m3)	-	19.0	-	19.2	19.4	19.0	19.0	19.5	19.3	19.4	19.6	19.4	19.3	19.4	19.4	19.1	19.4	18.9	18.8	19.0	18.7
	Cu (kPa)	-	68.7	-	29.6	0.0	0.0	73.6	112.8	107.9	0.0	9.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	68.7	0.0
	ϕ (°)	-	28.5	-	36.8	38.5	38.0	27.5	31.5	29.5	38.5	36.0	36	35	38.5	38.5	35.5	38	35	35.5	28.5	35.5

Notes: dh unit thickness; N blow count, SPT; qc point resistance, CPT; w water content; γ volumetric weight; Cu non-drained shear resistance; ϕ effective angle of internal friction

GEOTECHNICAL DESIGN

To determine the bearing capacity of the deep foundations in the final design stage, Zeevaert (1982) theory was used –following Terzaghi criteria–, assuming that the elements will be working in a combination of point bearing and side shear. The foundation depth was appointed up to the hard stratum, which in some cases met the upper boundary of Unit 7; although, in other cases, was in a substratum of Unit 6, adequate as long as it had enough thickness and resistance.

Piles were square, reinforced concrete precast elements with sides of 0.4 and 0.5 m, driven with diesel and hydraulic hammers, with pre-boring; drilled shafts were built with diameters of 0.60, 0.80 and 1.00 m; in both cases, the concrete used was of f'c = 35 Mpa.

Soil parameters for bearing capacity were obtained from unconfined compression tests, non-drained unconsolidated tri-axial tests (UU), as well as consolidated non-drained tri-axial tests (CU) with measurement of pore pressure, and consolidated drained tri-axial tests (CD). For some granular soils, SPT correlations were used.

For the determination of side shear, down-drag forces were considered, since an artificial fill was built (Unit 1). A preload was placed to reduce the negative side shear effect, but a residual settlement is expected, due to secondary compression. Definition of neutral plane, as well as calculation of positive and negative side shear were done using the procedure proposed by Zeevaert (1982).

The vertical bearing capacity was determined using a safety factor of 3 for the point bearing capacity; once the positive and negative side shear were defined, up to the maximum pile depth, a safety factor of 2 was used for the positive side shear.

Bearing capacity for accidental loads was obtained using a 1.3 factor for the above mentioned value.

Considering that, in certain events, deep foundations may have to resist uplift loads, tension capacity was calculated using similar criteria as in the calculation of positive side shear.

CONSTRUCTION PROCEDURES

Drilled shafts

Drilled shafts were cast in place, bored with a conventional system, using flight augers and buckets; concreting was made with the tremie procedure. In all cases, bentonite mud was used to stabilize the holes, plus a casing of 6 m length.

Fig 4. Drilled shaft construction

Piles

Piles were driven with single action diesel hammers of up to 149,160N-m (110,000 lb-ft) of energy in some areas, and using hydraulic hammers of up to 82,387N-m (60,757 lb-ft) in others. Prior to the driving, pre-boring was carried out, extending it up to 1 or 2 m before the pile tip, with diameters between the side and the diagonal of the pile. For long piles, splices were used, with plate welded unions.

Fig 5. Pile driving

LOAD TESTS

The owner's requirements for the QC/QA program for all driven/drilled shafts were: one static load test per 500 piles; one dynamic test per 200 piles; one PIT test per 50 piles. In this paper, results of the first two types of tests will be discussed.

Static tests

Several static load tests were performed: both compression and tension (lateral tests were also performed, but are not included in this paper). The reaction system for all of them was an arrangement of four reaction drilled shafts or piles, built close to the load test. A steel frame was used, formed of a main beam and two secondary beams (Figs. 6 and 7). Connection between the reaction piles and secondary beams was achieved using high strength threaded bars, acting against a concrete cube, which was cast with the reinforcement bars of the reaction piles. Distance between reaction piles and test piles was between 4 to 5 diameters.

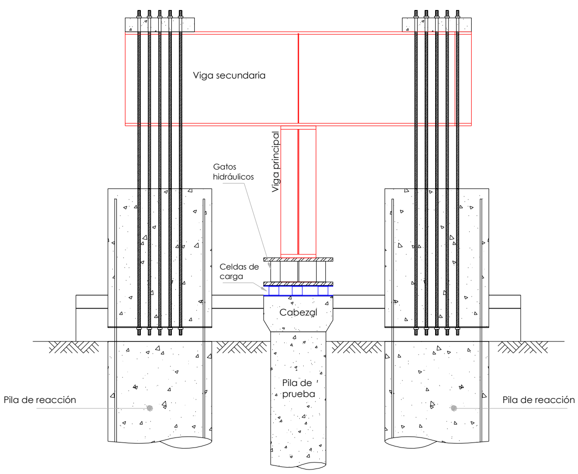


Fig 6. Reaction system scheme

The reference system for the deformation measurements included two steel channels placed beside the test pile. They were fixed to the ground at a distance of at least 3 diameters of the test pile.

One end of the channel was fixed, while the other was free, allowing the free deformation of the reference system caused by temperature changes, without affecting measurement gauges.

Fig 7. Reaction system

Vertical displacements were measured at the top of the piles using three dial gages, placed at 120°, plus the typical wire and mirror arrangement. The dial gages had a precision of 0.01 mm with a maximum travel of 50 mm; each one was fixed to the reference system using magnetic bases.

Load was applied using one or several hydraulic jacks and a manual pump. All the elements (dial gages, jacks, pumps) were calibrated. With these items, a maximum compression load of 5 MN (~500 t) was applied.

Dynamic tests

To perform dynamic tests in piles, the same hammer used for driving the piles was used. To test the drilled shafts, a special device was designed and built for this purpose, consisting in a free fall hammer, with enough weight to mobilize the bearing capacity of the soil, up to twice the allowable load.

To protect the top of the drilled shafts, a concrete extension was built, within a 2 m casing. Additional cushioning was introduced with a wood bed and steel plates.

Present experience in dynamic testing suggests a falling weight of between 1% to 2% of the resistance of the soil intended to be mobilized. Thus, to test 0.8m diameter drilled shafts, a weight of 156 kN (16 t) was used. This weight was lifted with a crane, and was let fall freely from different heights on top of the pile (Fig. 8). To assure an axial load, a steel structure was used as a fixed guide. With this system, loads of up to 6.8 MN (~680 t) were applied.

Fig 8. Dynamic tests for drilled shafts

Testing methods

For static load tests in compression, the ASTM D-1143 standard was followed, applying 8 load increments of 25% of the allowable load, with each increment applied when a displacement rate of less than 0.25 mm/hr was obtained, limiting each stage to 2 hr. Once the proper test load was reached, it was kept for 12 hr, and then it was unloaded in four decrements, each within one hour. For tension tests the ASTM D-3689 standard was used, with criteria equivalent to those in the compression tests.

Interpretation

Static tests

Load-displacement curves were analyzed for drilled shafts, with fifteen compression tests and four tension tests. For piles, five tests were analyzed (four in compression, one in tension; 24 tests in all). In general, registered curves followed the typical C shape described by Hirany and Kulhawy (1989), except in two tests, where type B curves were observed, due to the proximity to failure (Fig. 9).

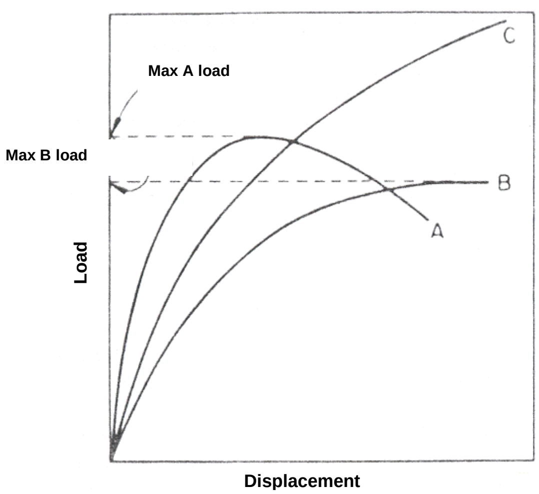


Fig 9. Typical load-displacement curve shapes (Hirany and Kulhawy, 1989)

To estimate the ultimate load of type C curves, Chin's (1970) method was used, where the load-displacement curve is extrapolated to an asymptotic behavior, assuming the maximum applied load was close to the failure point.

Fig. 10 shows a typical shape of a load-displacement curve after Chin´s criteria; ultimate load is calculated with the reciprocal of the slope in the final portion. Similar analyses were made for the 24 static load tests presented in this paper.

Ten tests presented a ratio of maximum applied load vs. maximum extrapolated load between 70% and 90%; nine tests registered ratios between 50% to 70%, and five had ratios of less than 50%. For the last ones, it was not possible to extrapolate clearly the ultimate load, and some assumptions had to be made.

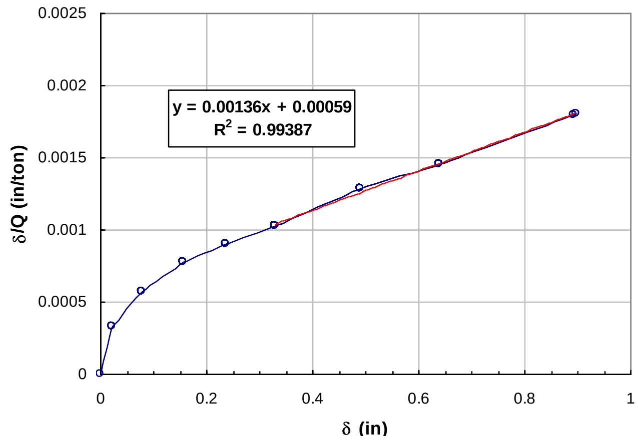


Fig 10. Chin´s method to estimate ultimate load

Load and displacement were measured only at the top of the piles, so there are no records of a direct measurement of the side shear, as in a full instrumented test; however, side shear was estimated in accordance with the shape of the curves. Fig. 11 shows typical load-displacement curves for point bearing, side shear and total load (Kulwahy, 1991). It is shown that the side shear follows an elastic-plastic behavior, and its peak is reached with relatively small displacements, generally within a few millimeters; meanwhile, the point bearing curve follows an increasingly quasi-linear trend, and the ultimate resistance mobilization requires bigger displacements, up to 10% to 15% of the pile's diameter. Both behaviors combined are measured at the top of the piles, and the side shear can be located approximately between points A and B of each record.

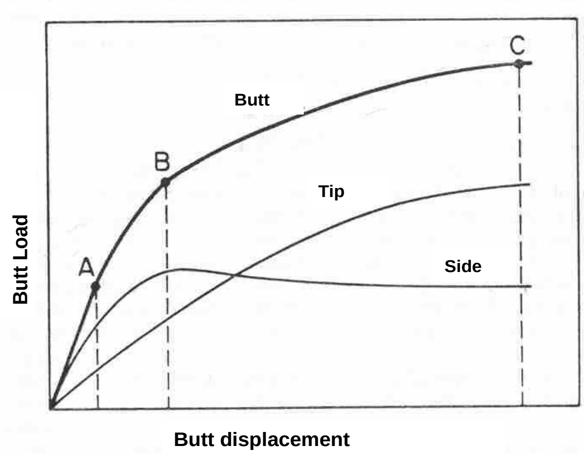


Fig 11. Load-displacement curves in drilled shafts (Kulhawy, 1991)

Tamez (2003) suggests a simple graphic method to determine side shear load between points A and B (Fig. 12); a secant straight line is extended through B and C points of Fig. 11; the value at the intersection with the vertical axis will be an approximation of the side shear. This methodology was used to estimate the side shear in the tests.

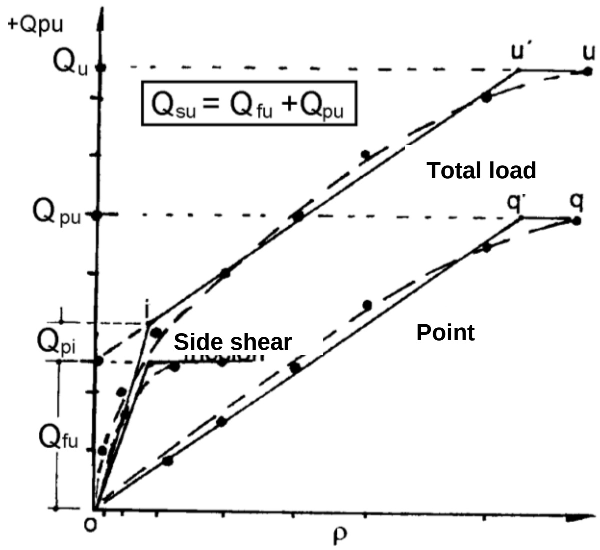


Fig 12. Graphic method to estimate side shear (Tamez, 2003)

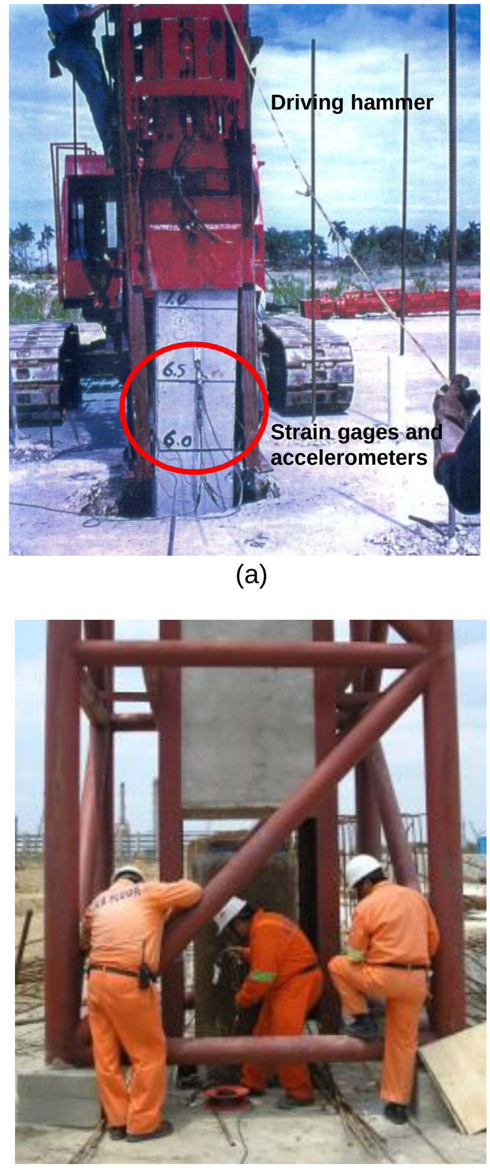
Dynamic tests

In this case, an impact load is generated, either with a driving hammer or with a drop hammer, and measurements of the pile's top force and velocity are taken during the impact loading with four strain sensors and four accelerometers connected to a Pile Driving Analyzer (PDA); afterwards, an analysis is conducted that reduces the dynamic force-motion measurements to a static load-set curve. The general arrangement is shown in Fig 13.

The analysis was performed using the commonly used CAPWAP software which, based on an elastic pile model and a static, elastic-plastic and viscous dynamic soil model, matches computed signals with measured signals in a trial and error type of signal matching procedure.

Summary of results

Table 2 shows a summary of the 32 static and dynamic tests. General pile and test data are presented, as well as calculated side shear and point bearing loads. Experimental results are included: maximum applied load, and pile head displacement in static tests.



(a)

(b)

Fig 13. General arrangement for dynamic load testing, a) in piles; b) in drilled shafts

Comparison between measured and calculated side shear

Fig. 14 shows the results of estimated side shear obtained from load tests versus the calculated one. Except for a few data, measured friction is up to 50% higher than the design friction, though values up to 100% and 200% higher were found, phenomenon that has been reported by other authors for drilled shafts in granular soils; Rollins *et al* (2005) present results where average side shear measured in drilled shafts in sands, with important presence of gravels, is 100% higher than the one calculated with classic theories, increasing with gravel content.

Table 2 Summary of results for design and test loads

Test No. and type		L (m)	D (m)	Qmax (kN)	δ (mm)	Allowable load (kN)			Measured load (kN)		
						Side shear	Point	Total	Side shear	Point	Total Chin
1	EC D	14.0	0.6	2207	13.5	684	4393	5077	1274	1793	3067
2	EC D	22.0	0.6	1776	7.7	1252	2183	3435	1294	1156	2450
3	EC D	27.0	0.6	4454	19.6	1118	6349	7467	2891	2744	5635
4	EC D	28.0	0.6	4591	10.6	1736	5951	7687	2499	5664	8163
5	D D	28.4	0.8	5592	-	2348	10579	12927	4292	-	-
6	EC D	25.6	0.6	3865	4.8	2034	7234	9267	2940	6860	9800
7	EC D	27.0	0.6	3090	25.2	1550	5680	7229	2009	1764	3773
8	EC D	24.8	0.6	2727	5.5	2111	2578	4689	2744	4018	6762
9	D D	24.0	0.8	6847	-	2723	4583	7307	5508	-	-
10	EC D	26.5	0.6	2663	1.6	1740	3664	5403	2009	1254	3263
11	D D	27.5	0.8	4159	-	2407	6513	8920	3156	-	-
12	EC D	25.7	0.6	3924	24.1	1757	6772	8529	2156	2509	4665
13	EC D	37.1	0.6	3806	8.3	3133	7770	10903	3136	5027	8163
14	ET D	37.1	0.6	2119	8.1	3133	7770	10903	3920	-	-
15	D D	35.7	0.8	5857	-	4020	13813	17833	4459	-	-
16	EC D	37.1	0.6	4880	22.8	1724	6227	7951	2450	5086	7536
17	D D	37.1	0.8	5886	-	2298	11071	13369	4410	-	-
18	EC D	13.0	0.6	2158	44.3	420	2748	3168	1421	1029	2450
19	ET D	13.0	0.6	814	13.4	420	2748	3168	1294	-	-
20	EC D	31.5	0.6	2168	3.8	2124	5434	7558	1637	3214	4851
21	ET D	31.5	0.6	932	4.4	1758	5434	7192	1637	-	-
22*	D P.	35.0	0.4	2668	-	5452	2412	7864	1588	-	-
23*	D P	34.0	0.5	2394	-	7819	2878	10698	1392	-	-
24*	EC P.	36.5	0.5	2449	7.1	2139	2590	4729	980	2940	3920
25*	ET P.	36.5	0.5	1165	17.9	2617	2587	5204	1637	-	-
26*	D P	36.5	0.5	2963	-	2139	2590	4729	2274	-	-
27	EC D	20.0	0.45	1776	8.4	615	983	1598	1029	1744	2773
28	EC D	35.0	0.8	3051	4.8	2799	14053	16852	3254	1205	4459
29	ET D	35.0	0.8	667	2.5	3355	14053	17408	3254	-	-
30*	EC P	18.0	0.4	746	4.0	715	826	1541	392	1098	1490
31*	EC P	13.0	0.4	1138	7.2	1853	1868	3720	588	1049	1637
32*	EC P	22.0	0.4	2207	7.3	768	777	1545	735	3038	3773

TEST NOTATION: EC D Static compression in drilled shaft; ET D Static tension in drilled shaft, EC P Static compression in pile ET P Static tension in pile; D D Dynamic in bored pile; D P Dynamic in pile

Although the drilled shafts that were tested were drilled through stratiphications of sands and clays (where sand content of the embedded pile length of Units 1,2, 6 and 7 varies between 38% and 79%), a similar trend is observed as in the one reported by Rollins for granular soils, but with more constrained differences.

Average side shear along the pile shaft varies from 32 to 62 kPa, with a mean value of 47.3 kPa (Fig. 15). These values consider the average of all strata involved, so unit side shear for granular soils is expected to be greater. Some authors have reported measurements of unit side shear in drilled shafts in sandy soils with gravel at about 10 times the ones reported herein (Harraz *et al*., 2004). It must be acknowledged that it was not possible to find a clear trend between sand content along the shaft and the increase of unit side shear (Fig. 16), maybe due to the lack of a more detailed analysis, where other properties may be taken into account, such as sand density and grain size distribution.

For driven piles, the contrary effect is observed; that is, in most of the tests, side shear loads measured were in accordance with the design calculations, or even up to 50% less (Fig. 14). This can be explained by the time that passed between pile driving and testing, in general 30 days, or, more importantly, the pre-boring effect, drilled for most of the shaft's length.

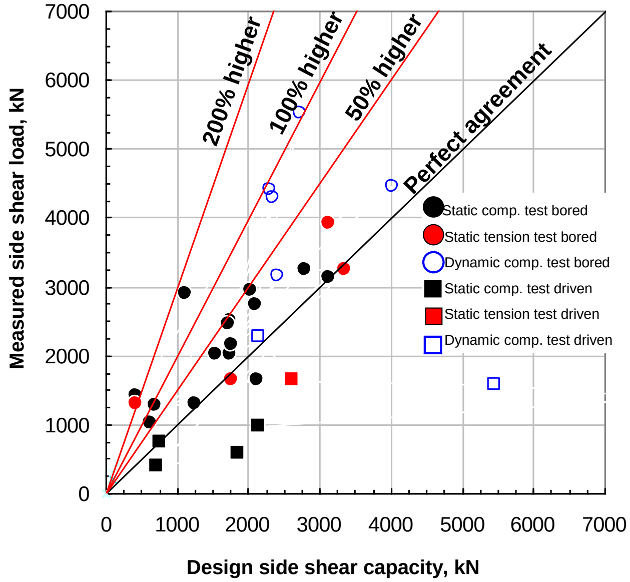


Fig 14. Comparison between measured versus design side shear

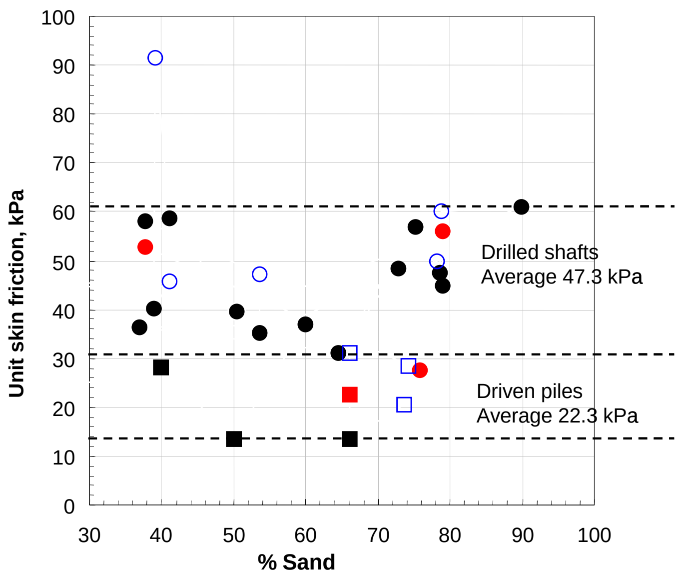


Fig 15. Average unit side shear versus sand content

It is noticed that, for driven piles, the average unit side shear is between 15 and 31 kPa, with a mean of 22 kPa; these values are below the ones registered for drilled shafts, in all cases. This is opposite to the reports of several authors (i.e. Meyerhof, 1976), who limit the unit side shear for drilled shafts up to 50%, compared with driven piles. This shows the trend to punish side shear in drilled shafts, perhaps due to the influence of the construction procedure (for instance, use of bentonite mud); however, other topics of drilled shaft behavior have been avoided, such as the dilating effect of dense granular soils and the roughness of the boring itself (Rollins *et al.*, 2005; Harraz *et al.*, 2005), and the increase of lateral pressure during concreting (Tamez, 2003). More often, for drilled shafts in granular soils, the increase of measured friction against the calculated friction is reported (Mendoza and Ibarra, 2006). The influence of each factor on the side shear is not known, and is the subject of future research.

Comparison between measured side shear and allowable load

Fig. 16 shows the results of the side shear obtained in the tests against designers' total allowable load capacity (friction and tip). It shows that for drilled shafts, measured capacity due to side shear constitutes at least 60% of the allowable load; in effect, for most piles this percentage is 75%, and in no too few cases it reaches 100% or more.

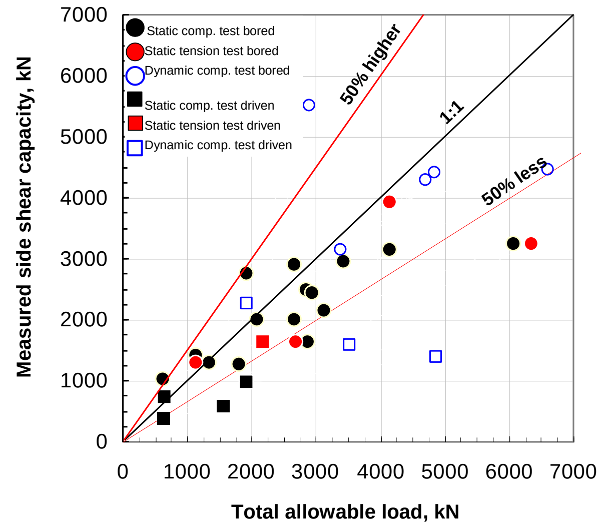


Fig 16. Measured side shear versus allowable design load

The above data ratify the idea that the work performed by the piles under allowable loads is mainly by side shear, contrary to the "bearing point pile" concept of the design stage; in fact, the side shear is developed first, with small displacements, even of a few millimeters. The high contribution of the tip, estimated during design, is kept as an important reserve to admit excess loads, such as those due to earthquakes, if large displacements are allowable.

CONCLUSIONS

It is observed that the general practice of calculating load capacity in deep foundations tends to trust most of the capacity on the point of those elements, underestimating load capacity by side shear, probably due to fear of the influence of the constructive procedure (for example with bentonite mud, or drilling prior to driving the piles in). Nonetheless, it is shown that load capacity due to measured side shear, in the case of piles, is very near the total load capacity allowable by design, keeping an important reserve of capacity at the point if large deformations are allowable. It is understood that the greater the length of the foundation elements, the larger the side shear's contribution to the allowable load capacity. It was observed in the tests that are the subject of this article that in piles with length from 13 m onwards, the contribution of load capacity by side shear constitutes more than 60% of the allowable design load capacity.

Taking into account the difference between the calculated load capacities and those measured in field, a probable underestimation of the soil's mechanical parameters is noticeable, due to the exploration methods and lab tests commonly used (SPT, UU). It is considered preferable to carry out field tests such as pressuremeter, electric cone, dilatometer, phicometer.

Based on the above points, we suggest carrying out a review of the design approaches for drilled shafts, especially in granular soils, with emphasis on the significant capacity increase due to friction of the piles embedded sandy and gravelly soils.

For driven piles, side shear resistance was surely affected due to preboring, done in mosto f the length of the pile.

The need to employ the results of load tests to bring the measured data closer to the calculated data, to get a better understanding of these foundation elements, is emphasized.

The benefits of using dynamic load tests, calibrating them with conventional static tests, is shown, making it possible to do them on both piles and foundation piles, as long as the adequate hammer is used in each case, so it can at least activate the resistance of the shaft's friction capacity and part of the tip's capacity.

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