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A case History on Evolution of Shaft Resistance in Long Piles Placed in Layered Soils

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Abstract. The ultimate theoretical design by load capacity for bored piles takes into account the sum of the individual contribution of the maximum unit shear stress of the pile in each layer and tip resistance, calculated using field or laboratory tests. Results of the measurements of a load test on an instrumented pile carried out in interlayered deposits in Mexico city and of the numeric models that are presented in this paper shows that, in long piles constructed in deposits where there are soils with contrasting rigidity, the development of the unit shear stress of the pile evolves in a different way in each layer and its peak value is not reached at the same time in all layers, since it evolves according to the nature and rigidity of the soil and of the construction process of the pile. By the time the pile reaches the ultimate shaft capacity, some layers have surpassed their peak contribution and are functioning in residual conditions; on the contrary, other layers have not reached their peak resistance. This condition can cause theoretical over-estimations for the shaft load capacity.

Keywords. Bored piles, side shear resistance, load test and numerical modeling.

1. Introduction

Recently, in Mexico City (CDMX), tall buildings with circular or rectangular piles in their deeply set (between 55 and 80 meters) foundations have been built. This has forced local engineers to develop and optimize the processes for construction piles without diminishing their load capabilities, as well as to understand the behavior of very long foundations faced with static and dynamic action in interlayered soils with contrasting rigidities.

Ibarra *et al.* [1, 2] presented results and an interpretation of static axial load tests on a instrumented pile with 1m in diameter and 55m length, located in the CDMX Lake area, where the transmission mechanisms of load-unload cycles of the pile studies were defined. In this paper we return to these experimental results to widen them using an axisymmetric finite element model (A-FEM). Based on the experimental and numerical results, a design strategy for long piles for CDMX soils is presented, where there are soft clay soils interlayered with hard and rigid layers with high.

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2. Stratigraphic profile

It was determined using standard and piezocone penetration tests (*SPT & CPT$_u$*); the mechanical properties of each soil layer, according Masing type constitutive model, were evaluated based on field tests as phicometer (*PHI*), Menard pressuremeter (*PMT*) and shear vane test (*VST*) along with laboratory testing and empirical correlations. The stratigraphy profile obtained is typical for the Lake zone of Mexico City (Table 1), in other words: the surface crust (*CS*); the upper silty-clay formation (*FAS*); the hard layer (*CD*), and the lower silty-clay formation (*FAI*). Finally, after 32.5m and up to the depth explored of 70m, the deep deposits (*DP*), made up of a volcanic tuff, were found, with a hard clay layer (FAP).

Table 1 Geotechnical model (see Figure 1)

Stratum	Depth (m) From	Depth (m) to	q_{T_mean} Mpa	N_{SPT}	γ kN/m^3	K_o	ν	R_{inter}	E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	E_{ur}^{ref} (kPa)	m	$c_{u\,ref}$ or c'_{ref} (kPa)	ϕ' (°)	$\gamma_{0.7}$	G_0^{ref} (kPa)
CS	0.0	4.6	0.8	-	15	0.5	0.35	0.18	8,077	8,077	24,231	0.5	50	0	1E-04	61,000
FAS1	4.6	20.2	1.73	-	12.3	0.5	0.45	0.18	7,524	7,524	22,572	1	61	0	5E-05	19,000
FAS2	20.2	24.1	1.73	-	14.2	0.5	0.45	0.18	24,282	24,282	72,846	1	90	0	5E-05	28,200
CD	24.1	27.5	-	52	16	0.64	0.3	0.18	37,643	37,643	112,929	0.5	122	18.2	5E-05	60,000
FAI	27.5	32.5	2	-	14.9	0.63	0.3	0.14	18,279	18,279	54,837	1	135	0	5E-05	22,000
DP1	32.5	43.3	-	46	16	0.53	0.4	0.14	53,826	53,826	161,478	0.5	10	32.4	1E-04	68,000
DP2	43.3	49.9	-	49	16	0.66	0.4	0.14	50,254	50,254	150,762	1	327	0	5E-05	63,000
DP3	49.9	60.0	-	65	16	0.54	0.3	0.18	72,668	72,668	218,004	0.5	216	25.8	1E-04	104,000
FAP	60.0	63.6	-	24	15.6	0.31	0.4	0.18	24,315	24,315	72,945	0.5	198	20.7	1E-04	54,000
DP4	63.6	70.5	-	-	17	0.44	0.35	0.14	33,418	33,418	100,254	0.5	10	37.5	1E-04	58,000
Determinated from:					Lab test	PMT & Emp Corr	Emp corr	Load test	Lab test & PMT			Load test	Lab test & VST		Lab text, PMT & Emp Corr	

Symbols:

$q_{T,mean}$: mean value of the CPTu tip resistance; **N_{SPT}**: Standard penetration number; γ: Unit weight

K_o: lateral earth pressure coefficient at rest; ν: Poisson ratio; **R_{int}**: Strength ration between soil and interface

E_{50}^{ref} & **E_{eod}^{ref}**: Plastic straining due to primary deviatoric loading and compression

E_{ur}^{ref}: Elastic modulus at unloading/reloading; **m**: Stress dependent stiffness factor according to a power law

$c_{u\,ref}$ or **c'_{ref}** : Drain/undrain Cohesion; ϕ': Friction angle; $\gamma_{0.7}$: Shear strain level at which G_s=0.7G_o; **G_o^{ref}**: Initial shear modulus

3. Load test

Static axial load was applied with load-unload cycles to an instrumented bored and cast-in-place concrete pile with circular section (1m diameter and 55m length), using a bentonitic slurry during construction in the Lake area of Mexico City.

Instrumentation. The pile was instrumented in 10 sections named N_1 (at 1m depth) to N_{10} (at 54m depth, close to the tip) selected by the stratigraphy of the site (Figure 1). In each section two sets of vibrating wire gauges, sister bar type, were attached to the pile rebar cage. Load on the head was applied using three hydraulic jacks of 10000kN each and was recorded with three load cells with a capacity of 8000kN each placed on the head assembly. Movements in two positions of the head assembly were measured, using two displacement transducers, two dial gauges and two scales graduated with thread and mirror. A detailed description of the instrumentation is presented in Ibarra *et al.* [1, 2].

Testing procedure. Two load-unload cycles were applied. In the first, a load was applied that was increasingly monotonically up to a load of 1,180t, and then pile was unloaded. The second cycle was applied in four increases to achieve 1,180t reached in the 1st cycle, and then with load increases of approximately 50t, up to 1,458t, showing a clear penetration of the pile in the soil (>15%D); and lastly the pile was unloaded using six decreases.

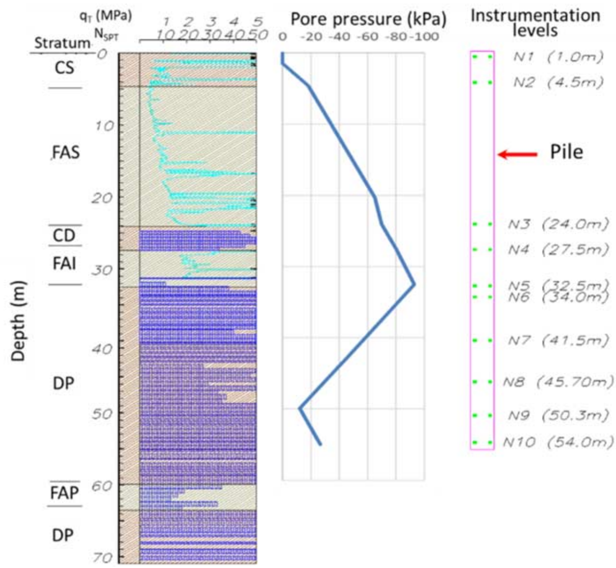

Figure 1 Geotechnical model and instrumented sections.

Experimental results. Including the load-displacement curves measured on the pile head assemble, load distribution curves and mean shear stress curves *versus* displacements between instrumented sections. Figure 2 shows the load *vs* vertical movement graph for the head assembly of the pile for two load-unload cycles, in which we can see clearly an initial tendency defined in the 1st cycle, an initial rigid behavior, up to slightly less than 800t and 19mm of movement (Point A). This behavior is mainly associated with the work done by friction in the shaft (Q_{fu}). At this point, the shortening of the pile measured with a tell tale placed at the point, was 9.5 mm. After Point A, we can clearly see a second tendency toward less rigidity that lasts through the end of the load branch of the 1st cycle that is related to the work mainly performed by the tip, with no change in the tip maximum resistance. The unloading branch of the 1st cycle shows a rigidity similar to the load branch (stretch B-C). For the 2nd cycle, there is a rigid initial stretch in reloading up to approximately 1200t (stretch C-D), and later a stretch with less rigidity (stretch D-E) and similar unloading rigidity to that of reload (stretch E-F).

Figure 3 shows load-displacement curves for the 1st and 2nd cycles, along with the curves corresponding to the tip and shaft work. We can see that, initially, the load put on the head assemble is taken only by the shaft (red curve), while the tip is not appreciably loaded in the first load phases (blue curve). For the 1st cycle, the ultimate shaft capacity of (Q_{fu1}=600t) is achieved. After this point, the shaft contribution decreases slightly and then holds a "residual" tendency, meaning that the shaft is no longer able to take on more load even when more load is placed on the head assembly. When the pile is unloaded for the first time, the shaft is not totally unloaded, instead the direction of the work done by friction is reversed, in such a way that by the end of unloading it is clear that extraction work continues at 400t of residual force, while the point remains "preloaded" at 460t. Summarizing, in the load branch, friction works upwards (opposite to the compression), while, in unloading, friction is opposite to pile extraction. This condition generates an important amount of remnant force in the shaft (400t) that must be reversed during the load branch of the 2nd cycle, so that, for the 2nd cycle, the shaft may reach a capacity of Q_{fu2}≥1000t, as a result of the ΔQ_{f1}=400t at extraction and the Q_{fu1}=600t at compression.

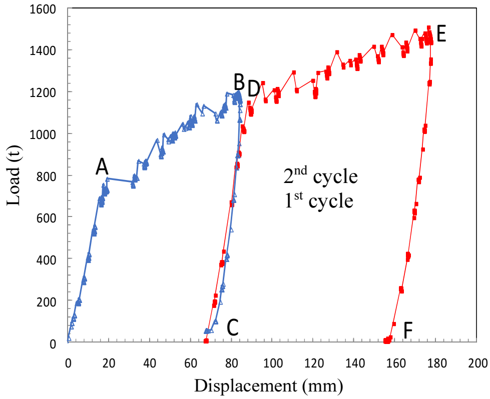

Figure 2. Load-movement graph for pile head assembly, 1st and 2nd load and unload cycles [2]

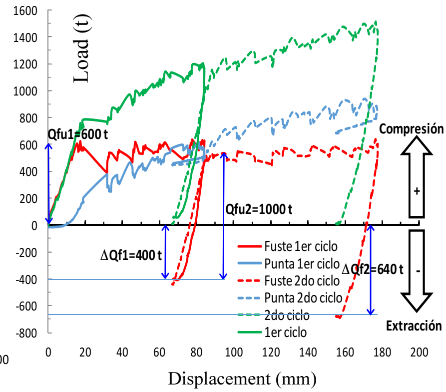

Figure 3. Load in head assembly, in tip and shaft *versus* head displacement [2]

Development of the unit shear stress in the shaft. Figure 4 shows the pile load distribution with depth, while Figure 5 shows the f_{s_mean} curves between instrumented sections, that are associated with certain layers, with relation to the vertical movement of the head assembly of the pile. The continuous vertical line in each graph shows, as a reference, 19mm of head assembly displacement, associated with the development of the ultimate shaft capacity (Q_{fu1}=600t) of the 1st load cycle.

It is observed that the peak f_s values in the shaft (Figure 5) do not occur for the 19mm head displacement; for soft soils peak fs value is reached before the pile reaches the ultimate shaft capacity, which occurs at 19mm of vertical displacement measured at the head assembly and later a residual shearing resistance is exhibited. In turn, in mainly sandy soils with greater rigidity, the value of f_s grows with the movement of the pile, in such a way that the peak values of the shear resistance in the shaft are shown for large movements of the pile head, beyond the point at which the pile jointly reaches its Q_{fu}; including the fact that in some sandy levels the peak resistance for the end of the load branch of the 1st cycle is not developed,. Even though the more rigid sandy layers show a clear tendency to increase their f_s value, the total contribution of the pile resistance per shaft jointly does not increase, but instead reaches a maximum Q_{fu} value for a vertical movement of 19mm and stays that way until the end of the test.

4. Numerical model

The load test was numerically modeled using A-FEM. Calibration of this model was carried out based on the load test. The stratigraphy used was previously described, and the behavior of the soil is governed by a constitutive model of the Masing type with hardening and rigidity depend on small deformation, in order to describe behavior at load, unload and reload induced in the soil. In order to adequately represent the constructive process of the pile, the values of the interfacing element between the pile and the soil were determined taking into account the degradation in soil resistance during exposure to bentonitic slurry [3] and the experimental data. In Figure 6 are the comparisons of the load-displacement curves in the head assembly and the base of the pile, both numerical and experimental, in which we can see that the answer given by the model is similar to that observed in the field. In other words, the numerical model using Masing-type properties reproduces, with good approximation, the experimental results of the load test.

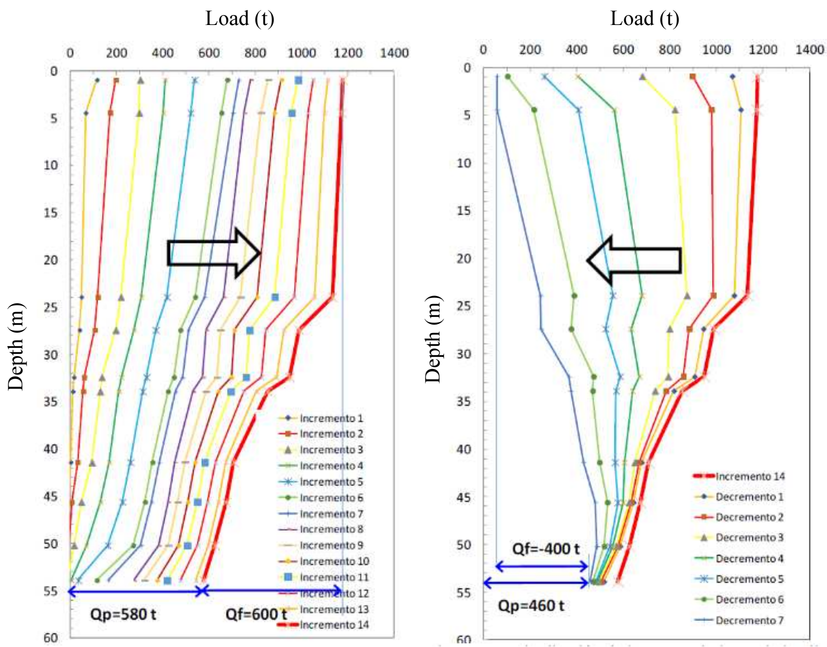

Figure 4. Distribution of load over the length of the pile for the 1st load and unload cycle (left & right).

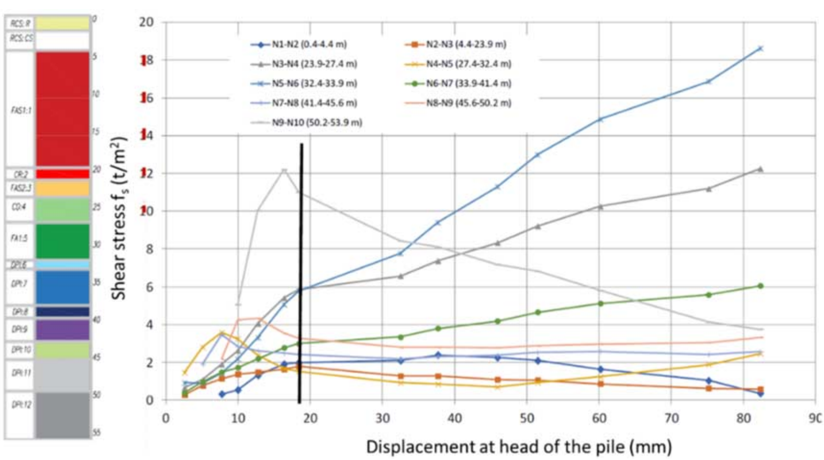

Figure 5. Variation of the shear stress (f_s) with respect to head assembly displacement, 1st cycle [2].

In Figures 7 and 8 we can see the shear stress in the interface and plastic areas for different conditions: load, 1st unload, reload and 2nd unload. In the load condition, the shear stress reaches their limits along the length of the shaft and, therefore, the entire interface of the shaft is in plastic condition; as well, the tip of the pile begins to work incipiently, producing few plastic areas at the base of the pile. Later, in the 1st unload, the shear stress of the interface is inverted and reaches its limit only in some areas, reducing the plastic area at the base. In the case of reloading, plastic points in the entire shaft is seen again along with the increase in plastic points at the base, without causing the total failure of the pile at this point, even though we can see important vertical displacements. Finally, in the 2nd unload, the shear stress of the interface is again inverted and the plastic areas are reduced, in a lesser degree with relation to the 1st unload.

The utility of the numerical models applied to the interpretation of load tests is: extend the experimental results, study pile behavior in other work conditions or geometries (*i.e.*, when the piles are exposed to an initial unload, as is common in top-down constructive processes) and help to evaluate the effectiveness of the constructive process (*ie*, detecting areas of the pile where it loses part of the tip or friction work for later corrections in the constructive process, or, when applicable, to make soil improvements around the pile).

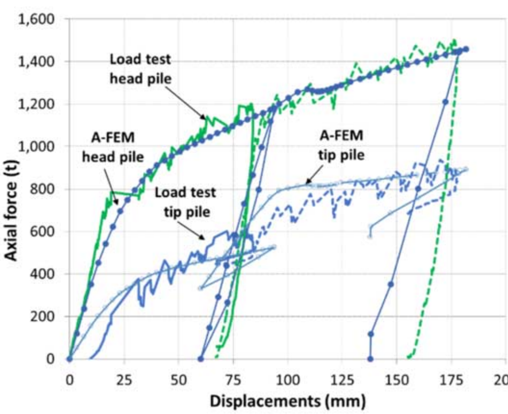

Figure 6. Experimental and numerical load-displacement graphs.

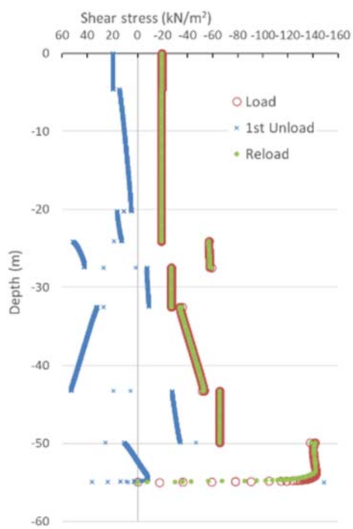

Figure 7. Shear stress in the soil-pile interface (A-FEM).

5. Geotechnical design

The results obtained from the instrumentation of the load test pile and its numerical model have repercussions in the following aspects of the geotechnical design: *i)* Shear stress developed in the pile shaft (f_s); *ii)* load path in real conditions for pile work; and *iii)* constitutive equation for soil model.

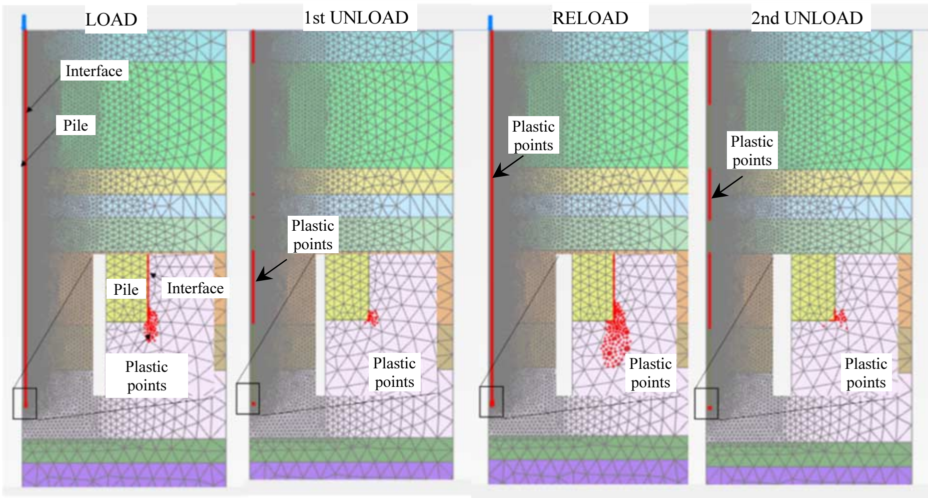

Figure 8. Plastic zones determined using the A-FEM for different stages of numerical analysis.

Shaft resistance (Q_f). The value of the shaft resistance of the pile for layered deposits is found using next expression:

$$Q_f = \pi D \sum_{i=1}^{n} \alpha_i f_{s,i} h_i \qquad (1)$$

where D is the diameter of the pile, α_i is the factor of correction for the constructive process of the pile, $f_{s,i}$ is the resistance to the shear stress of layer i in the shaft, h_i is the thickness of layer i; and n is the number of layers along the length of the pile.

One of the most difficult aspects to evaluate is factor α_i, given that it depends on the constructive process of the pile, in other words, in the magnitude in which the construction of the pile alters the mechanical properties of the soil surrounding it. There are various aspects of the constructive process that are of interest: the type of support used during the excavation and pouring of the pile (metal casing, bentonitic mud, polymers, etc.), the deforming or alteration induced during the excavation (bottom clean-out bucket, hollow auger, etc.), the time the soil is exposed to support fluids, the soil improvement of the previous to or after pile construction (*ie,* injections), and constructive errors (loosening of the walls fragments of the excavation, hydraulic fracturing, excessive remolding during excavation, etc.). In ideal conditions in the constructive process, this factor is α_i=1, but if the constructive process is inadequate, some low values can be obtained, *ie* α=0.05. In Figure 9 we compare the values of αf_s for the case studied, taking into account the conditions with no soil alteration (α=1) and the condition observed, where the average construction factor was α=0.27. In the first case there is a friction load capacity of $Q_{f,\alpha=!}$=3,084t, but taking into account the real constructive process the friction capacity is reduced to $Q_{f\_a\sim0.27}$=766t, which is 24.8% of the nominal capacity. In relation, the results of the load test indicate a value of Q_{fu1}=600t for the first load cycle (Figure 3).

Load path. In Figure 3 we can see that upon unloading and reloading the pile, there is an increase in shaft resistance of Q_{fu1}=600t to Q_{fu2}=1000t. This is a result of, as previously commented, a remnant load in the shaft and tip pile generated in the unloading, in such a way that when the unload is done there is clearly extraction work still being performed at 400t of residual force, which the tip is "preloaded" at 460t. In this context, in order to consider this contribution of the geotechnical design of the piles, they would have to be preloaded in a 1st cycle (load and unload). Due to the fact that this preload condition is not a natural condition in production piles, it is important to remember that the 2nd load-unload cycles in axial compression tests in piles, will show an increase in load capacity per shaft that should not be taken into account in the geotechnical design of the elements of the foundations [4]. However, when a building has more than one basement and the constructive process uses a top-down system, where the piles are built previously and the first load put on the piles is the unload of land generated by the excavation of basements, it is important to study both the capability of the piles in work with tension and the increase in the friction component.

Constitutive equation. There are studies of the advantages of using advances constitutive models in geotechnical analysis of piles as compared to using traditional models, *ie* [6], in which the conclusion is that the selection of a constitutive model has no impact on the analysis of load capabilities per tip, but has an impact on the determination of the movements induced in the base and in the modeling of shaft behavior. In the case studied, where load-unload-reload conditions are presented for the pile, the Masing type model used was able to adequately represent the behavior observed in the field.

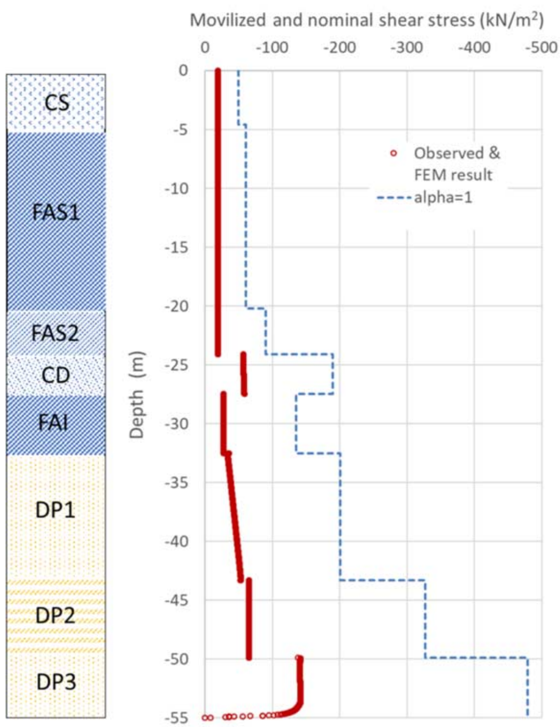

Figure 9 αf_s values for two conditions; α=1 (value at which the constructive process does not induce soil alterations) and the observations from load test α~0.27.

6. Conclusions

Depending on the characteristics of the construction procedures used in the long-bored piles, there is little or much alteration in the surrounding soil, having high impact on the value of deformation and the load capability. For processes in which bentonitic slurry is used for supporting the excavation, this is a critical aspect, since the time during which the soil is exposed to bentonitic slurry reduces exponentially its resistance to the shear stress. In the case studied, the reduction seen was around 27%. At the same time, the development of the unit shear stress, f_s, evolved differently in each layer, in agreement with the nature and rigidity of the soil and the building process of the pile, in such a way that at the moment the pile reaches Q_{fu} capacity, some layers, mainly the fines, have surpassed their peak shear stress contribution and are working in residual conditions, while the f_s resistance of the other layers, generally granular soils, are not. Even though the more rigid sandy layers show a clear tendency to increase their f_s level, the total contribution of shaft pile resistance jointly does not grow simultaneously but reaches a maximum value of Q_{fu} for a vertical displacement of 19mm and stays constant through the end of the test. This seems to be an effect of compensation.

In the light of the results seen in the load pile instrumented, it is evident that considering the contribution caused by friction in each level, taking the maximum corresponding f_s value, can cause theoretical overestimations of Q_{fu} in long piles built in layered soils with contrasting rigidities. To confirm this idea, it is recommended further monitoring to evaluate effects of construction process on α factor, and further evaluation of similar project data so that recommendation for estimating skin friction can be better estimated.

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