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# Estimation of Driven Piles Capacity in Texcoco Clay, Mexico City

## Estimation de la capacité portante de pieux battus dans l'argile de Texcoco, à Mexico

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**ABSTRACT:** The Basin of Mexico is formed by deep lacustrine deposits of exceptional properties. Piled-enhanced raft foundations, using driven concrete piles, are common practice in these deposits. In particular, at the former Texcoco Lake area the top 30m of soil column is comprised by very soft diatomaceous clay with shear strength ranging in the order of 7kPa + 1(kPa/m)z, where z is depth in meters. This paper evaluates the suitability of using available local and international pile design methods to estimate pile capacity in Texcoco Clay. Data from static pile tests carried out in former Texcoco Lake is used for the analyses.

**RÉSUMÉ:** Le bassin de Mexico est formé de dépôts lacustres profonds aux propriétés exceptionnelles. Les fondations mixtes de radiers sur pieux en béton armé sont une pratique courante pour ces dépôts. En particulier, dans l'ancienne région du lac de Texcoco, les 30 premiers mètres des terrains sont composés d'argile à diatomées très molle, dont la résistance au cisaillement est de l'ordre de 7 kPa + 1 (kPa / m) z, où z est la profondeur en mètres. Cet article évalue l'opportunité d'utiliser les méthodes de calcul usuellement pratiquées, au Mexique et à l'étranger, pour estimer la capacité portante de pieux dans cette argile. Les données provenant d'essais statiques sur pieux réalisés dans l'ancien lac de Texcoco sont utilisées pour les calculs.

**Keywords:** friction piles; validation, structured soils, Mexico City

## 1 INTRODUCTION

The New International Airport at Mexico City (NAIM, *Nuevo Aeropuerto Internacional de México*) site at the former Texcoco lake is an extensive undeveloped area which have had little coverage after the intense investigation work for Proyecto Texcoco (Comité Técnico Proyecto Texcoco, 1969) when the area was under consideration for a power station and desalination works. At the time of writing, as a result of political intervention, NAIM was cancelled at almost 35% completion. Piles and raft foundations were in place for the major landside NAIM structures.

The soft ground conditions coupled with the continuation of regional subsidence due to groundwater abstraction from the deep aquifer system present a considerable challenge for the foundation design in the Mexico City lacustrine zone (Gobierno de la Ciudad de México, CDMX, 2017). The NAIM design required a large scale ground investigation not only involving routine investigation but also trial areas as per Proyecto Texcoco spirit. As part of those trials, static pile tests were undertaken to assess the ultimate capacity of driven piles.

Raft foundations enhanced with friction piles are used for heavier structures which box shallow foundation type cannot support and reduce differential settlement. The friction piles reduce the soil deformation and transmit stresses to the deeper strata of lower compressibility (Zeevaert, 1983). The piles prevent exceeding the yield stress taking advantage of the comparatively high stiffness in unload-reload Mexico clay behaviour (Ovando-Shelley, 2011; Díaz-Rodríguez, 2003). To ensure that the foundation settles at the same rate as the regional subsidence, the friction piles are designed to operate at their ultimate load capacity minimizing the development of negative skin friction. If negative friction develops on the upper part of the pile, a gap might form between the underside of the raft and the soil surface. The gap formation poses a risk especially for seismic events: the loads are carried only by the piles which then plunge until there is sufficient bearing capacity at the raft-soil interface, as described by O'Riordan *et al.* (2018). Hence, the estimation of the pile ultimate capacity is paramount to achieve a safe and cost-effective design.

The NAIM Passenger Terminal Building (PTB), the Ground Transportation Center (GTC) and the Air Traffic Control Tower (ATCT) were designed to be founded on rafts enhanced with friction piles.

This paper describes the estimation of the ultimate load of driven piles at the NAIM site and compares the theoretical results with a pile trial test program commissioned by GACM (*Grupo Aeroportuario de la Ciudad de México*, public Mexican entity managing the NAIM construction) to CFE (*Comisión Federal de Electricidad*). To that end, section 2 provides a summary of the Mexico Basin stratigraphy together with a more detailed description of the engineering properties of the uppermost Upper Clayey Formation in which the piles are driven. Section 3 describes the pile capacity estimation using the Gobierno de la Ciudad de México (2017) NTC (*Normas Técnicas de la Construcción*) total stress $\alpha$ method and the Imperial College pile effective stress $\beta$ method (ICP). Section 4 presents and comments results from the CFE trial tests.

## 2 MEXICO BASIN BACKGROUND

### *2.1 Mexico Basin background*

The Basin is a predominately flat lacustrine plain with a typical elevation of about 2,250 m MSL. The stratigraphy in the lacustrine zone comprises typically an upper made ground and a desiccated crust, the Upper Clayey Formation (Formación Arcillosa Superior or FAS) which can be 30m thick, a 0 to 5m thickness of dense sands/volcanic glass known as the Hard Layer (Capa Dura or CD) followed by the Lower Clayey Formation (Formación Arcillosa Inferior or FAI) which can reach 50m deep. Underlying the FAI a series of alluvial sand and gravels, cemented with clay and calcium carbonate called Deep Deposits (Depósitos Profundos or DP) up to 110m deep. The FAS is an extremely soft and weak soil. Further details on FAS properties can be found on Díaz-Rodríguez (2003) and O'Riordan *et al.* (2017).

Below the DP, alluvial fans and debris flows (lahars) interbedded with volcanic pumice and ash known as the Tarango Formation develop. The fresh water for the Mexico City area population is abstracted from this formation and it is designated as the aquifer in this paper.

From a hydrogeological perspective, the system comprising the FAS, CD, FAI and DP can be regarded as an aquitard from which brine was pumped at certain locations (Rudolph *et al.*, 1989). Refer to O'Riordan et al. (2018) for further detail on the NAIM site pore water pressure evolution.

### 2.2 *FAS geotechnical properties*

The FAS unit is characterized as silty clays with thin interbedded sand lenses. The following index properties at the airport site were obtained from testing by Ingenieria Experimental (IE) and Fugro reported by Arup (2017),

- Pore water salinity: typically 45 g/L;
- Clay fraction: typically 50 to 60%;
- Uncorrected water content: 150 to 350%;
- Liquid limit: 300% reducing to 150% with depth;
- Plastic limit: 40% to 100%;
- Unit weight: 12 kN/m$^3$;
- Specific particle density: 2.7;
- Void ratio: 4 to 8.

Nearly half of the clay fraction is smectite mineral. The presence of siliceous diatoms promotes an open structure giving unusual characteristics like high void ratio and ability to sustain high moisture contents. The pore water has a flocculating effect. Due to the high diatoms and salt content, the FAS soil results in an open structure with extraordinary properties.

Results of a large number of oedometer tests indicate that the virgin line compression index of FAS ranges between 4 and 10 with an average of 5.6. Recompression index, $C_r$, varies mostly between 0.1 and 0.4. $C_r/C_c$ ratio is generally greater than 10. The high recompression stiffness is also considered an attribute of the high diatom content because, for non-microstructured clays, the ratio $C_r/C_c$ is typically between 4 and 6 (Leroueil and Hight, 2003).

The yield stress ratio (*YSR*) for natural clays is defined as:

$$YSR = \frac{\sigma'_{vy}}{\sigma'_{vo}} \tag{1}$$

Where $\sigma'_{vy}$ is the effective vertical yield stress and $\sigma'_{v0}$ is the in situ effective vertical stress.

The oedometer test results indicate an average *YSR* equal to 1.8, decreasing to 1.0 about 20mbgl due to the effects of underdrainage from Capa Dura.

Figure 1 presents the oedementer tests plotted with the intrinsic compression line (ICL) reported by Ovando-Shelley (2011).

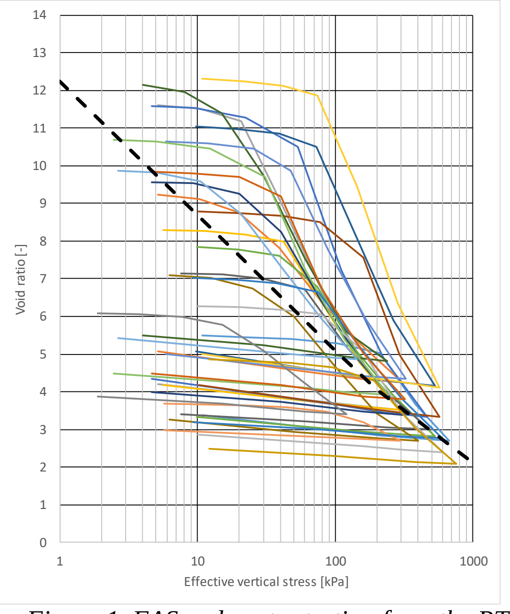


*Figure 1. FAS oedometer testing form the PTB and ATC NAIM ground investigation, Arup (2017). Intrinsic Consolidation line (Ovando-Shelley, 2011) dash line*

Saldivar & Jardine (2005) show that the sensitivity ($S_t$) can be estimated from the ratio between the yield stress ($\sigma'_{vy}$) and the reconstituted sample at the same void ratio ($\sigma'_{vy}$*) using the intrinsic properties as described by Burland (1990).

$$log(S_t) = \left(e_y - e_y^*\right)/C_c^* \tag{2}$$

Saldivar & Jardine (2005) proposed the following expressions to evaluate the intrinsic properties at a 100 kPa reference vertical effective stress of the Mexico City soil based on correlations with the void ratios at the clay liquid limits ($e_{LL}$).

$$e_{100}^* = 0.6534e_{LL} \tag{3}$$

$$C_c^* = 0.2343e_{LL} \tag{4}$$

Diatom content has a significant effect on the strength and stiffness properties of these clays. The measured friction angle is comparable in magnitude to those in sands and it can be explained by the fact that FAS contains glass particles and diatom shell fragments generating friction angles well in excess of 40 degrees (Leroueil and Hight, 2003).

Ovando (2011) establishes, for cone penetration testing (CPT) carried out at 20 mm/s, the following cone resistance ($q_c$) and undrained shear strength ($S_u$) relationship:

$$S_u = q_c/13.2 \tag{5}$$

A lower bound undrained shear strength of the upper FAS can be conveniently described with the following relationship:

$$S_u = 7 + 1.05z\ kPa \tag{6}$$

where z is depth in metres to 20m depth in July 2015.

The influence of diatoms on strength is very difficult to establish, due to their microscopic scale (typically 2 to 50 μm), however Díaz-Rodríguez (2011 and 2012) shows that at a *YSR* of 2, a normalized undrained compressive strength ratio, $S_u/\sigma_v'$, of kaolin of 0.45 rises to about 1.05 with the addition of 60% diatom by weight. Díaz-Rodríguez & Santamarina (2001) indicate that:

$$\frac{C_u}{\sigma'_v} = 0.85 \cdot YSR^{0.75} \tag{7}$$

As presented in Figure 2, the latter expression is well above expression (8) considering a friction angle $\varphi' = 34°$ and a constant $\Lambda = 0.9$ for sensitive and structured soils (Kulhawy & Mayne 1990):

$$\frac{C_u}{\sigma'_v} = \frac{1}{2} sin(\varphi') YSR^{\Lambda} \tag{8}$$

Expression (7) can be regarded as an upper bound related to a high diatom content when compared with the triaxial testing undertaken in the NAIM ground investigation (Arup, 2017), Giraldo (1996) research and the results of some mixtures of bentonite and diatoms undertaken by the authors (see Figure 2).

# 3 PILE CAPACITY ESTIMATION

## 3.1 *Total stress formulation*

The formulation recommended to assess the driven pile capacity by the Mexican code CDMX (2017) (NTC henceforth) is a total stress formulation ($\alpha$ method) based on the American Petroleum Institute (2010) recommendations. The shaft resistance ($\tau_s$) is estimated by:

$$\tau_s = 0.5\sqrt{\frac{\sigma'_v}{S_u}} \cdot S_u \tag{9}$$

If before driving the pile, preboring is undertaken, the above shaft resistance should be reduced by the following factor:

$$f_r = 1 - 0.4\, D_{perf}/D \tag{10}$$

Where $D_{perf}$ and $D$ are the preboring and the pile diameter/ side respectively.

The end bearing ($\sigma_b$) is estimated using the classical closed-form solution:

$$\sigma_b = 9C_u \tag{11}$$

## 3.2 *Effective stress formulation*

Saldivar & Jardine (2005) assessed the applicability of the Imperial College pile (ICP) design method to concrete piles driven in FAS. Those authors found that the ICP method was more reliable than the total stress capacity method that was recommended by the NTC in 2005.

The shaft resistance is evaluated using the following expressions:

$$\tau_s = \sigma'_{rf} tan(\delta_f) \tag{12}$$

$$\sigma'_{rf} = 0.8\sigma'_c \tag{13}$$

$$\sigma'_{rc} = K_c \sigma'_{vo} \tag{14}$$

$$K_c = (2.2 + 0.016YSR - 0.87\, log\, S_t) YSR^{0.42} (h/R^*)^{-0.20} \tag{15}$$

With solid cylindrical piles, $R^*$ is the pile's outer radius, h is the distance to the pile tip and $\delta'_f$ is the interface friction angle. A $\delta'_f$ of 36 degrees has been considered for this site. Refer to Jardine *et al.* (2005), Saldivar and Jardine (2005) and Saldivar (2002) for further detail. For the end bearing, those authors propose to take $0.7q_c$ from the local CPTu test. A similar relationship is obtained combining expressions (5) and (11).

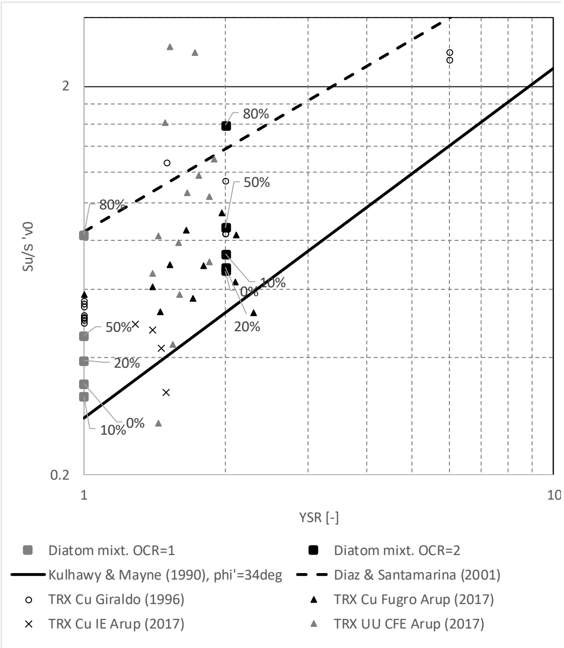


*Figure 2. Undrained shear strength Normalized with Respect to Effective Confining Stress. The pointers show the diatom content in the bentonite-diatom mixtures.*

## 4 PILE TEST PROGRAM

The projected PTB was a 1500m long building with the GTC located between the south limbs (see Figure 3). The PTB and GTC raft was enhanced with 500 mm square piles 9 m spaced and structurally disconnected to the foundation raft. The pile length ranged between 15 and 17m. The piles were installed before the excavation from a 1m thick tezontle (local light pumice aggregate) platform on the natural ground using a follower to drive the piles to their final depth. The test piles were made long enough to stick above the working platform to allow being dynamically tested before the excavation proceeded. A 75% of the pile side diameter preboring on the 70% of the pile length was performed. After the pile driving, an excavation as deep as 6.5m was undertaken to achieve the required compensation. The pore water pressure used to assess the effective stress is the pre-excavation pore water profile presented by O'Riordan *et al*. (2018) minus the excavated total stress ($\sigma_{exc}$). Figure 4 plots $S_u/\sigma'_v$ vs effective vertical stress, post-excavation. Based on the *YSR* data collected during the NAIM ground investigation (Arup, 2017), the following $S_u/\sigma'_v$ to *YSR* relationship was adopted:

$$\frac{S_u}{\sigma'_v} = 0.55 \cdot YSR^{0.75} \tag{16}$$

Figure 4 shows a reduction of $S_u/\sigma'_v$ as effective stress exceeds 60 kPa. It is considered that the soil underwent a destructuration process reducing its undrained shear strength due to the underdrainage in the lower part of the FAS.

GACM commissioned CFE on 2018 to enact a static pile test program about one year after the PTB and GTC piles were installed as an additional control measure. At the time of the pile testing, the PTB foundation raft was cast and the GTC raft was completed except a central abeyance zone as shown in Figure 3.


*Figure 3. Pile test locations. The PTB sits on the X shaped raft and the GTC on the round raft.*

The test program comprised seven piles and was carried out by CFE when a large excavation around the pile had been performed, unloading the ground. As the natural ground sloped northwards, the northern elevation was 2226 m MSL and the southern was 2228 m MSL, resulting in less excavation on the north. The low permeability and the high salt content of the groundwater inhibited pore water pressure recovery after unloading as the installed vibrating wire piezometers showed. For that reason, it was assumed that despite the change of total stresses the effective stress remains unchanged after pile installation. CFE made a CPTu testing up to Capa Dura near each pile test location as shown in Figure 5. Those CPTu's were consistent with the ground investigation reported at Arup (2017).

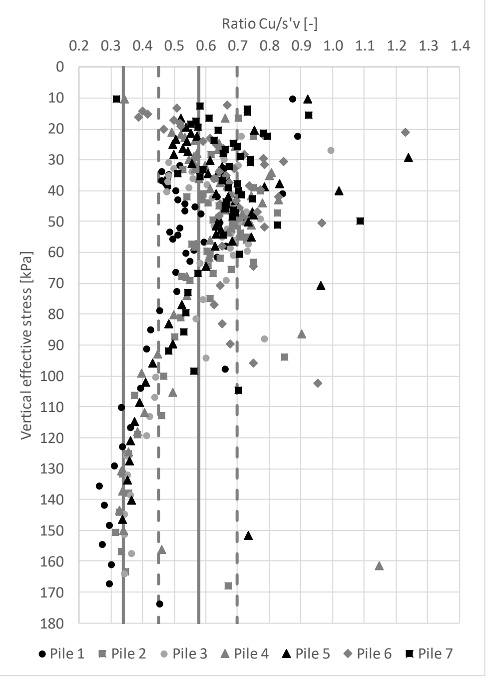


*Figure 4. Su/σ'v vs effective vertical stress*

Figure 6 presents the normalized load-displacement curves for the seven pile tests. The shape of the curves is virtually identical up to the maximum load. The displacement required to mobilize maximum load is 2.5% of the pile side length ($L$=500 mm). The soil behaves in a quasi-elastic fashion up to the maximum load. The post-peak behaviour suggests that the initial soil structure collapses but the load only drops between 60% and 78% of the maximum load at a 10% of side length ($L$) displacement due to the high friction angle. This agrees well with ring shear testing on FAS reported by Saldivar (2002).

Table 1 summarises the test results together with the predictions from ICP and NTC (CDMX, 2017) methods. The pile capacity methods yield comparable results. The ICP method overestimates the pile capacity in the piles in which the $S_u/\sigma'_v$ ratio is lower (piles 4,5 and 6).

In order to match the pile test capacities, a slight increase of the pore water pressure (around 10 kPa) is required. A modification of the $Su/\sigma'_v$ to $YSR$ relationship -increasing the factor 0.55 in expression (16)- is also required to apply the ICP method.

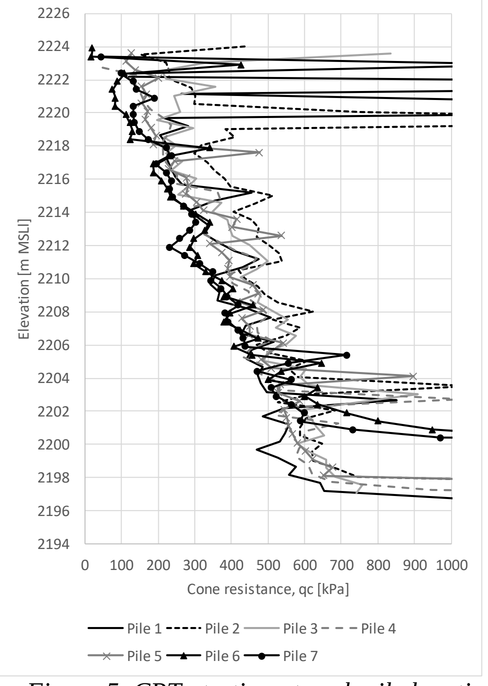


*Figure 5. CPTu testing at each pile location*

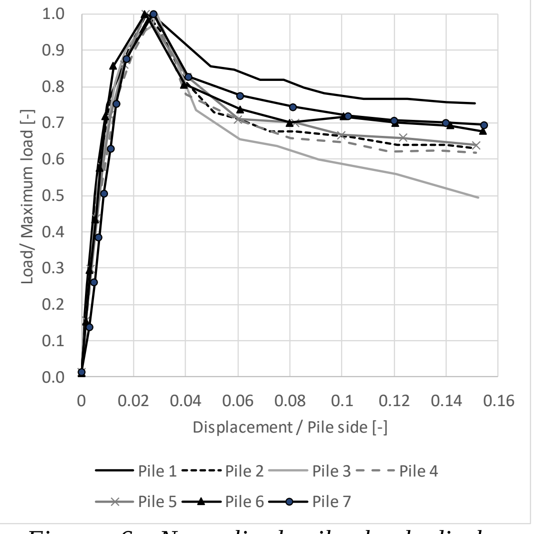


*Figure 6. Normalised pile load displacement curves*

# 5 CONCLUSIONS

The prediction of pile capacity for these piles of less than 20m length is very sensitive to the calculation of effective stress, and the design parameters that are associated with that. This is because groundwater is close to the ground surface and the unit weight of the soil is only about 12 kN/m$^3$ (at 10m depth the vertical effective stress is of the order of 20 kPa).

Comparable pile capacities are obtained from the total stress method recommended in the NTC (CDMX, 2017) and the ICP method. A good match between the pile test and the total stress method can be achieved by adjusting the effective stress profile, however little insight on other soil properties is given. Despite being more complex, the ICP method provides feedback on the other soil properties which can be used to assess the soundness of the prediction. In the CFE pile test program, the *YSR* is within the expected limits. However, the sensitivity ($S_t$) is greater than to be expected from the pile test load-displacement curves (peak/ residual relationship). This effect might be related to the variable diatom content. This experience demonstrates that a good pile capacity in clayey soils does need to go beyond the routine shear strength assessment and a proper effective stress profile and history/ state parameters are required.

*Table 1. Summary of results*

| Pile | Length [m] | *YSR* | $S_t$ | $S_u$ [kPa] | $\sigma'_v$ [kPa] | $S_u/\sigma'_v$ | $\sigma_{exc}$ [kPa] | $Q_{test}$ [kN] | $d_{Qmax}$ [mm] | $Q_{ICP}$ [kN] | $Q_{NTC}$ [kN] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 15.8 | 1.7 | 15.9 | 38 | 42 | 0.73 | 69 | **545** | 13 | 565 | 553 |
| 2 | 15.2 | 1.4 | 16.7 | 34 | 44 | 0.70 | 54 | **556** | 13 | 563 | 571 |
| 3 | 15.3 | 1.4 | 13.9 | 27 | 38 | 0.70 | 47 | **563** | 15 | 523 | 493 |
| 4 | 14.6 | 1.9 | 12.5 | 23 | 26 | 0.88 | 52 | **455** | 14 | 450 | 407 |
| 5 | 14.7 | 1.8 | 10.7 | 22 | 23 | 0.98 | 43 | **354** | 12 | 391 | 364 |
| 6 | 17.6 | 1.5 | 7.2 | 20 | 19 | 0.99 | 20 | **354** | 12 | 377 | 361 |
| 7 | 16.4 | 1.3 | 9.8 | 21 | 28 | 0.66 | 32 | **405** | 14 | 436 | 400 |

**Note:** $Q_{test}$, test capacity; $d_{Qmax}$, **settlement** at max load; $Q_{ICP}$ ICP capacity; $Q_{NTC}$, NTC (CDMX, 2017) capacity.

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