Geotechnical Engineering in the XXI Century: Lessons learned and future challenges
N.P. López-Acosta et al. (Eds.)

doi:10.3233/STAL190167

Deep Foundation in Deposits of Alluvial-Lacustrine Origin

Alberto CUEVAS[a,1], Lorena VEGA[a] and Ernesto PATIÑO[a]
[a] *Diseño geotécnico, Ingenieros Cuevas Asociados, México*

Abstract. In the southern zone of Mexico City, a foundation with deep piles was designed shortly before the earthquake of September 19, 2017; on account of this, regulation was updated two months later. This paper presents both review criteria the bearing capacity of foundation piles.

Keywords. Foundation, drilled shafts, bearing capacity, technical standards Mexico City.

1. Introduction

To the south of Mexico City, a 34-storey tower and eight basements laid at a depth of -29.5 is built, in an area that is geo-technically characterized for being on the boundaries of the lake zone and transition zone [1]. This zone is influenced by the old Churubusco riverbed, and as a result, the upper clay series is interspersed with silty sandy strata of alluvial origin that were deposited during the regressions of the old lake, forming a complex stratigraphy. Also, the deep volcanic soils are interspersed with alluvial deposits. It should be noted that the design stage for this project was completed shortly before the Puebla earthquake of September 19, 2017; this means it complies with the guidelines of the complementary technical standards, NTC-2004 [2]. Later, when the NTC-2017 [3] were published, a revision was made in order to compare both sets of regulations. In this case, the results for the foundation are presented with 30 m drilled shaft piles measured from the bottom of the excavation.

2. Geotechnical conditions

The stratigraphic interpretation was based on four samplings at depths of 70 m. In each one, the electrical cone and the standard penetration test, a selective sampling survey, and seven in situ tests with a phicometer were employed, the water pressure was determined with a piezometric station. The location of soil exploration is shown in Figure 1.

The stratigraphic model consists of three main deposits: superficial layer (from 0 m to 5 m), alluvial-lacustrine series (from 5 to 22 m) of medium to rigid consistency; and alluvial-volcanic series (from 22 m) formed by compact deposits of fine, medium and

[1] Corresponding Author, Alberto Cuevas Rivas, Ingenieros Cuevas Asociados, Hidalgo No 77, Col. San Lucas Tepetlacalco, Tlalnepantla, Edo. Méx., México; E-mail: alberto.cuevas@ingenieroscuevas.com.

coarse sand, with gravel. The water table is associated with a decrease in pressure. The water table begins at 6.0 m and has a maximum pressure of 88 kPa at a depth of 14.8 m and pressure is zero at depth of 22 m. Table 1 shows the mechanical properties of the soil used for the geotechnical design.

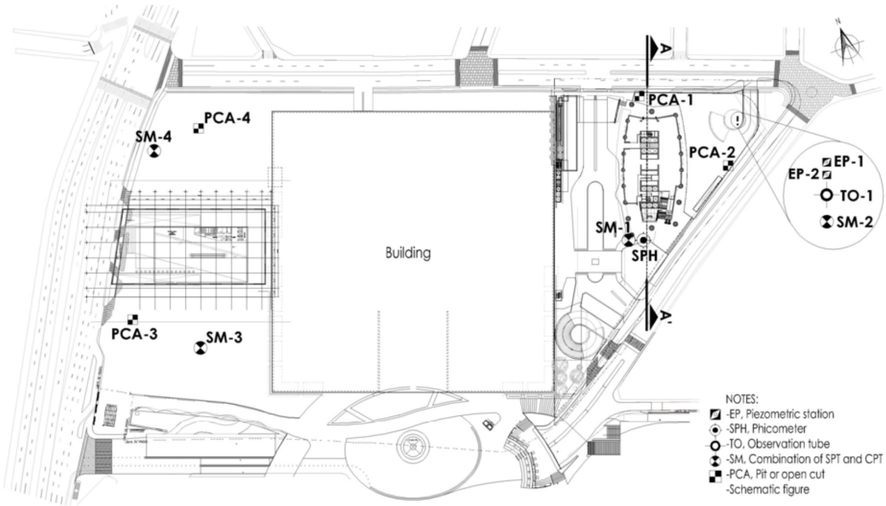

Figure 1. Soil exploration position.

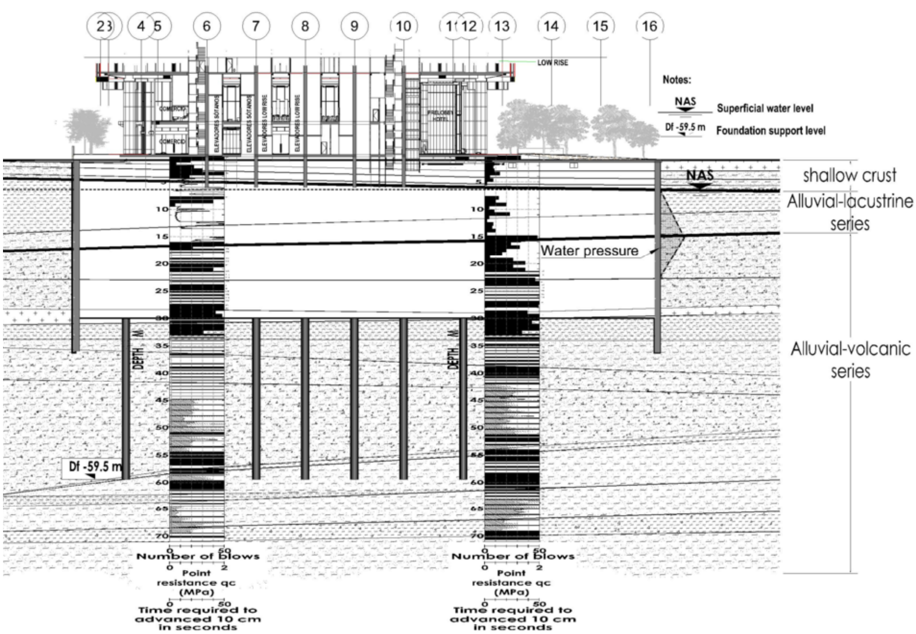

Figure 2. Stratigraphic profile.

Table 1. Geomechanical design model.

Stratigraphic unit	Depth m	γ kN/m^3	φ °	c kPa	E MPa
Superficial layer	0.0-5.0	15.7	8	63.7	18.1
Alluvial-lacustrine series	5.0-10.6	14.7	10	53.0	11.5
	10.6-14.8	12.7	5	55.9	9.0
Alluvial-volcanic series	14.8-22.0	15.7	34	63.8	15.9
	22.0-28.0	17.1	39	25.5	58.9
	28.0-29.2	14.7	25	24.5	14.7
	29.2-33.0	16.7	36	78.5	49.0
	33.0-54.0	18.6	39	78.5	145.7
	54.0-61.0	17.6	38	156.9	98.1
	61.0-66.0	18.6	37	78.5	245.3
	66.0-69.0	17.6	38	166.7	196.2

3. Geotechnical design of the foundation

The Project loads were grouped according to their magnitude: basement loads (15,000 kN); building loads, approximately 50,000 kN; and central core loads (70,000 kN). For this reason, the main foundation was resolved with 30 m long (effective length) drilled shafts in the alluvial-volcanic series, of variable diameters, from 1.3 to 2 m. For the parking lots, the solution was with a 3.5 m wide, shallow square foundation. Next, the revision for the piles' bearing capacity is presented, taking into account the guidelines of the NTC-2004 and the NTC-2017; Table 2 summarizes the most important aspects related to this project. (For a more comprehensive guide to the regulations and their changes, please refer to the article by Jaime in the References section [4]).

Table 2. Comparison between NTC-2004 and NTC-2017 for deep foundations in friction soils.

	NTC-2004	NTC-2017	Observaciones
Fr point	0.35	0.35	Frictional soil
Fr skin resistance	0.7	0.65	
Skin resistance	$Cf = AlfFr$	$Cf = P_p Fr \sum_{i=1}^{m} \overline{Pv}\beta_i L_i$	NTC-2004 does not indicate how to calculate the adhesion, Cf in friction soils.
Maximum adhesion value	$f \leq 0.3Pv$	$\overline{Pv}\beta_i \leq 200\ kPa$	Frictional soil
Point bearing capacity	$Cp = (\overline{Pv}Nq^*Fr + pv)Ap$	$Cp = (\overline{Pv}Nq^*Fr + pv)Ap$	Frictional soil
Scale factor	$F_{re} = \left(\frac{B+0.5}{2B}\right)^n$	Disappear	Applied in diameters greater than 0.5 m

Notes: Fr, reduction factor; Cf, skin resistance; Al, lateral area; P_p, perimeter; $\overline{Pv}$,effective vertical stress; β_i, factor β for each stratum; L_i,thickness of each stratum; f, skin friction; pv, vertical stress; Ap, base area of pile; B, diameter the pile; n, exponent that depends on the compactness of the soil.

3.1. Point bearing capacity for frictional soils

The point bearing capacity is a function of the effective stress of the soil at the level of load and the bearing capacity factor Nq, which depends on the properties of the soil, Figure 3 shows the Nq values proposed by different authors [5,6,7], the Nqmin and

Nqmax are included, which are comparable to the Meyerhof values for surface foundations and for piles, respectively. It is also observed that Berezantzev's Nq is an average value, in relation to the upper and lower limits established by the aforementioned criteria.

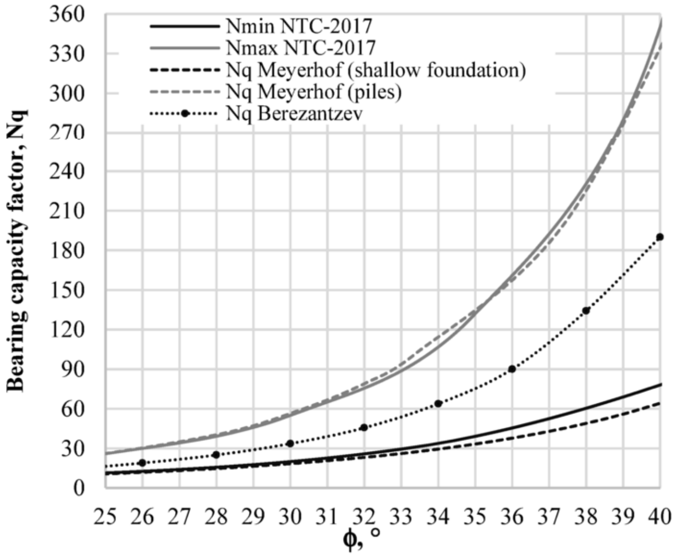

Figure 3. Comparison between Nq factors.

Figure 4 presents the revision criteria for point bearing capacity according to the NTC-2004, and the NTC-2017; in addition, a curve associated with the structural strength of concrete (f′c=35 MPa) is displayed.

In this case, the design value was determined to be the lowest value between the soil capacity and the structural strength is 19.6 MPa. Likewise, Figure 4 shows the effect of the scale factor considered in the NTC-2004, in this project was almost equivalent to the established limit for structural resistance.

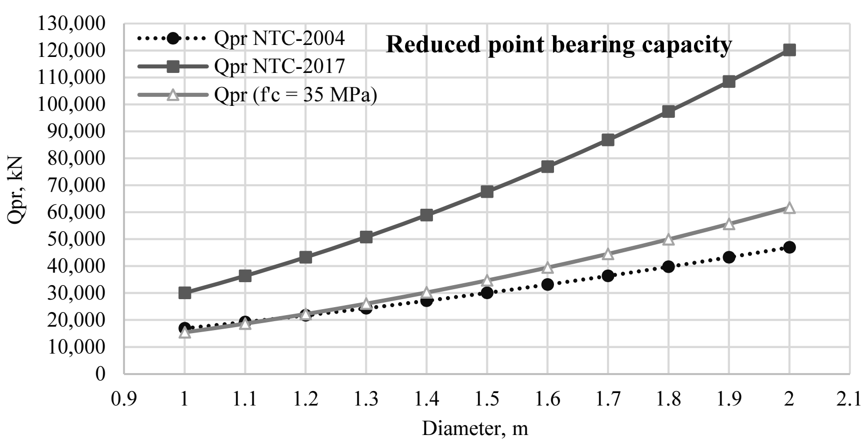

Figure 4. Reduced bearing capacity at the point, Qpr.

3.2. Bearing capacity in the shaft for frictional soils

Figure 5 demonstrates the comparison of the revised criteria to define skin friction capacity, according to Table 2, the maximum skin resistance must be limited.

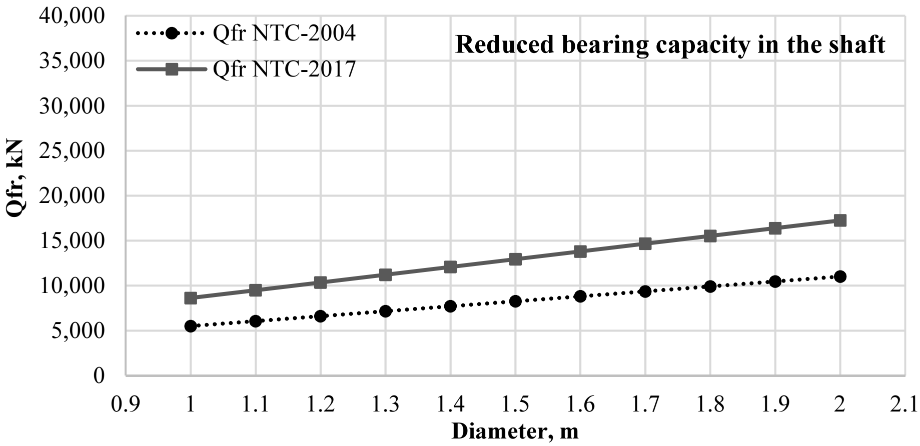

Figure 5. Reduced bearing capacity in the shaft.

In this case the design prior to the earthquake on September 19,2017 was calculated as a function of the initial stress of the soil, σ, the friction between the pile and the ground, $K_s tan\varphi$, the quality of the characteristics of the soil, the pile material, the procedure and the quality of the construction [8]:

$$fs = \sigma\, K_s tan\varphi \tag{1}$$

Where $K_s tan\varphi$ must be equal to or less than 0.3, to comply with the NTC-2004. Likewise:

$$fs = \sigma\, K_s tan\varphi = \sigma\, \beta \tag{2}$$

Therefore, in the criterion stated in the NTC-2017, it is understood that the beta, β, parameter is equivalent to:

$$\beta_i = \ K_s tan\varphi \tag{3}$$

The beta limit values (for sand) are defined by Reese [9]:

$$\beta_i = 1.5 - 0.24\sqrt{z_i}\,; \quad 0.25 \leq \beta_i \leq 1.2 \tag{4}$$

On the other hand, the Reese [9] criterion for sandy soils with gravel (Eq.5) was revised, in order to establish a reference frame for limit values for skin friction (Figure 6).

$$\beta_i = 2 - 0.15(z_i)^{0.75} \quad 0.25 \leq \beta_i \leq 1.8 \tag{5}$$

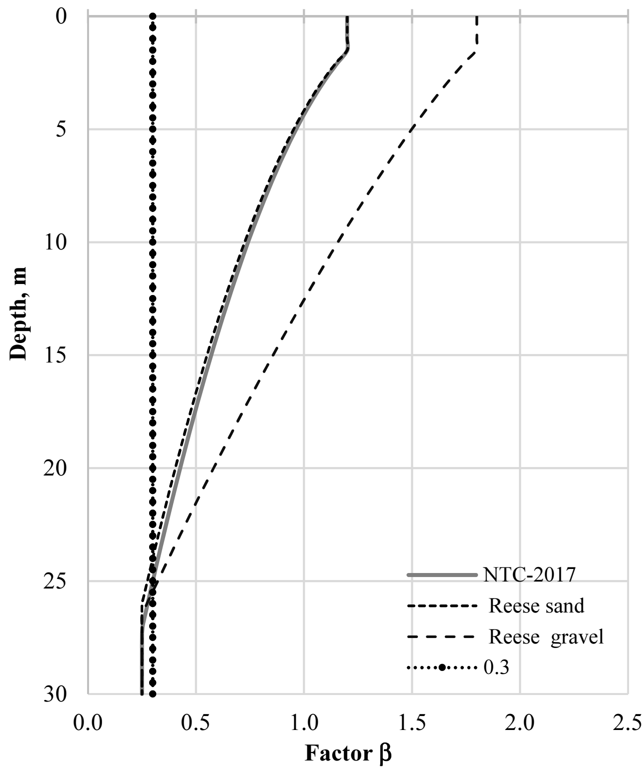

Figure 6. Limit values for beta, β.

In accordance with the previous graph, it is observed that there is a critical depth (26 m) from which β factor is constant (0.25); however, comparing the criteria of the NTC-2004 for frictional soils, if β is considered equivalent to 0.3, this explains the difference in the graph on Figure 5. In addition, one can observe that this limit is a very conservative value, if it pertains to a soil with gravel, like the materials of the alluvial-volcanic unit.

3.3. Reduced total bearing capacity

The total bearing capacity reduced due to compression can be reviewed with the following expression; its result should be compared with structural resistance; the lower of these values will govern the final design.

$$Qtr = Cf + Cp - Wp \tag{6}$$

Finally, Figure 7 shows the comparison of the reviewed criteria defined by the NTC-2004 and the NTC-2017; for the norms prior to the earthquake, the influence of the scale factor applied to the point bearing capacity is observed, as well as the restriction for the shaft, which results in the 2017 standards setting a higher bearing capacity limit. However, in any project, the capacity of the soil versus the structural resistance should be compared, in order to limit the stress in the pile. It is also recommended to verify the design values with load tests on piles that are truly to scale.

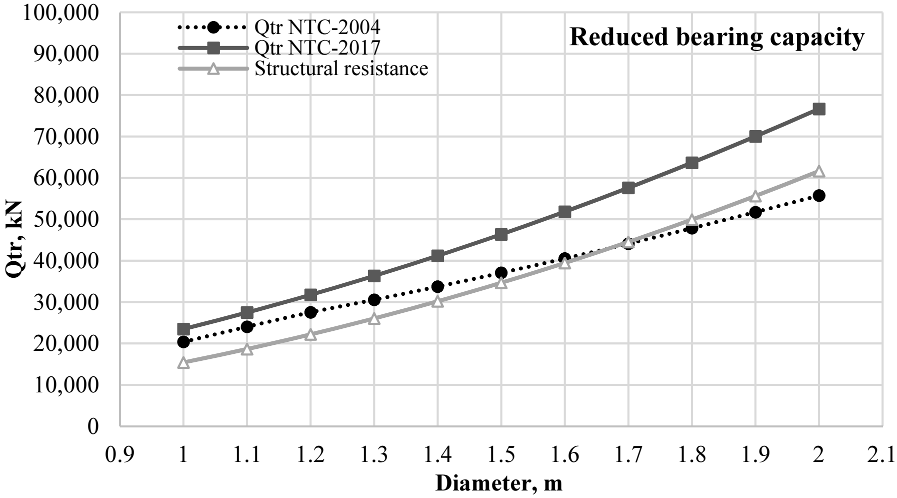

Figure 7. Total reduced bearing capacity.

4. Conclusions

The current standard in Mexico City establishes a revision criterion that "allows" higher bearing capacity values compared to the 2004 standards; since the scale factor that reduced point bearing capacity was eliminated and the values limiting the skin friction have changed. However, the NTC-2017 imposes a critical depth and maximum limits for soil resistance; and the final geotechnical design must also be compatible with the structural resistance of the element, so that the stress is within compliance and at acceptable levels. Finally, the importance of testing piles to ratify the soil parameters and the design bearing capacity is highlighted.

References

[1] Tamez, E., *et al*, *Manual de diseño Geotécnico*, COVITUR, México, 1987.
[2] Gobierno del Distrito Federal, *Normas Técnicas Complementarias para Diseño y Construcción de Cimentaciones*, Gaceta oficial del Distrito Federal Décima cuarta época, México, 2004.
[3] Gobierno de la Ciudad de México, *Normas Técnicas Complementarias para Diseño y Construcción de Cimentaciones*, Gaceta oficial de la Ciudad De México Vigésima época, México, 2017.
[4] Jaime, A., *et al*, Comparación de la Normas Técnicas Complementarias para el Diseño y Construcción de Cimentaciones 2004 y 2017 del Reglamento de Construcciones para el Distrito Federal, *XXIX Reunión Nacional de Ingeniería Geotécnica,* Sociedad Mexicana de Ingeniería Geotécnica (2018).
[5] Meyerhof, G.G, Bearing Capacity and Settlement of Pile Foundations, *Journal of the Geotechnical Engineering Division,* 102 (1976), 197-228.
[6] Meyerhof, G.G, The ultimate bearing capacity of foundations, *Geotechnique,* (1951),301-332
[7] Berezantzev V.C, *et al,* Load bearing capacity and deformation of piled foundation, *Proc. 5th Int. Conf. Soil Mech 2* (1961), 11-15
[8] Vesic´, A. S., *Design of pile foundations*, Transportation Research Board, Washington D.C., 1977
[9] Reese, L., C, *et al*., *Analysis and design of shallow and deep foundations,* John Wiley & Sons, New Jersey, 2006