

Bearing capacity of embedded piles in rocks considering their contribution to friction

Capacidad portante de pilotes empotrados en rocas considerando su aporte a fricción

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Abstract

This article has the objective to show a study of different existing theories that consider the friction contribution of embedded piles in rock. It initially summarizes the adopted criteria for such theories and the coefficient ranges considered for the application of their basic expressions. The results of these theoretical analyses were applied in the solution of the foundations of a hotel, and its marine, built on the beach in the city of Varadero, Cuba. The study area presented a high geologic and engineering complexity, with highly variable stratification characteristics together with distinct soil and rock properties, where the presence of a calcareous stratum stands out in the deposit. The calcareous layer is found in a variable depth within the strata and has a quality ranging from very poor to good, where all the pile tips were founded. In order to obtain the bearing capacities from the designed piles, it was necessary to take into account their friction contribution within the rock, which was done by validating the existing theories through the execution of in situ pile load tests, combined with the use of theoretical models. This exercise allowed the establishment of practical coefficient values that were required by the theories in such particular site conditions. It was finally possible to yield design solutions for the deep foundations of this case history, which comprised over 2000 driven piles..

Keywords: Piles in rock, bearing capacities, friction contribution, in situ pile load tests

Resumen

Este artículo tiene como objetivo mostrar el estudio realizado de diferentes teorías existentes para considerar el aporte a fricción de pilotes empotrado en roca. Para ello se resume inicialmente los criterios considerados en dichas teorías y los intervalos de los coeficientes que las mismas consideran para la aplicación de sus expresiones básicas. Los resultados de este análisis teórico fueron aplicados en la solución de las cimentaciones sobre pilote de un hotel y su marina construido en la playa de Varadero, Cuba. La zona de estudio presentaba una alta complejidad ingeniero geológica, con una estratificación muy variables y propiedades de los suelos y las rocas de la misma igualmente variable, destacándose la presencia de un estrato de calcarenita, que aparece a una profundidad variable y con una calidad desde buena hasta muy mala, donde siempre se empotraron los pilotes. Para la obtención de la capacidad portante de muchos de los pilotes, fue necesario tomar en cuenta el aporte a fricción en la roca, validándose las teorías existente a partir de la realización de pruebas de carga in-situ, combinadas con el empleo de los modelos teóricos, llegando a establecer los valores de los coeficientes que las mismas utilizan para las condiciones del caso de estudio. A partir de todo ello fue que se pudo dar solución a las cimentaciones sobre pilotes del caso de estudio, la comprendía más de 2000 pilotes hincados.

Palabras clave: Pilotes en rocas, capacidad resistente, aporte a fricción, pruebas de carga

1. Introduction

As known classical theories of Soil and Foundation Mechanics state, when a pile manages to become embedded in rock at least one time its diameter, that pile will work with support only at the tip and its friction contribution will not be taken into account, on the basis that the displacements at the tip of the pile will be negligible and thus pile-rock friction will not be generated. The above statement is true when the piles rest on high quality rocks. However, for cases where the piles are supported by low quality rocks, where they can be embedded in rock several times their diameter, the above statement is not true, and it is necessary to take the friction contribution within the rock into account when determining the ultimate bearing capacity of the pile.

In international literature, there are several works studying this issue, including among the most recent: Serrano (2008), Perez Carballo (2010) and Olmo (2011). All these studies agree that there are two groups of theories to establish ultimate resistance to friction in piles in rock τ_{ult}

- Theories that consider the τ_{ult} as a linear function of the compressive strength of rock R_c
- Theories that consider the τ_{ult} as a quadratic function of the compressive strength of rock R_c

In this research, all these theories were studied, establishing basic expressions and intervals of the coefficients proposed therein. This study was necessary to apply to the solution of the foundations over piles in a hotel on the beach in Varadero, Cuba. The engineering geological conditions of the area were very complex and variable, together with the existing soil and rock properties. This can be simplified in a stratification consisting of a series of soft soils, with variable thickness of fill, peat, silt and other very loose soils, none of which contribute to the bearing capacity of the piles, and a stratum of calcarenite, which appears at different depths and with very variable quality, from good to very poor. There, the piles achieve embedded length (EL) from only 1.5 times its diameter to more than 14 times its diameter, depending on the quality of the rock.

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ENGLISH VERSION.....

To determine the bearing capacity of piles, determinations for existing models, including the analyzed theories that consider the friction contribution within the rock where necessary, were combined with *in situ* pile load tests, which served to calibrate the models and set the interval at which coefficients taken into account in the conditions for the case study could vary. A very good engineering calibration of the models was achieved, and from it the resistant capabilities of the piles, for various conditions of support, were established. This allowed for a significant reduction in the length of piles, resulting in cost savings for construction.

2. Characterization of the engineering geology of the study area

In order to characterize the engineering geological conditions of the study area, which was necessary for the determination of the bearing capacity of the piles, several *in situ* studies were carried out by the companies GeoCuba (2008) and ENIA (2010, 2011). This research included surveys, description of strata, geophysical studies, and

laboratory studies in order to obtain the main physical and mechanical properties of soils and rocks.

The following types of soil and rocks, of varying thickness, positions, and engineering properties, were found in the general zone:

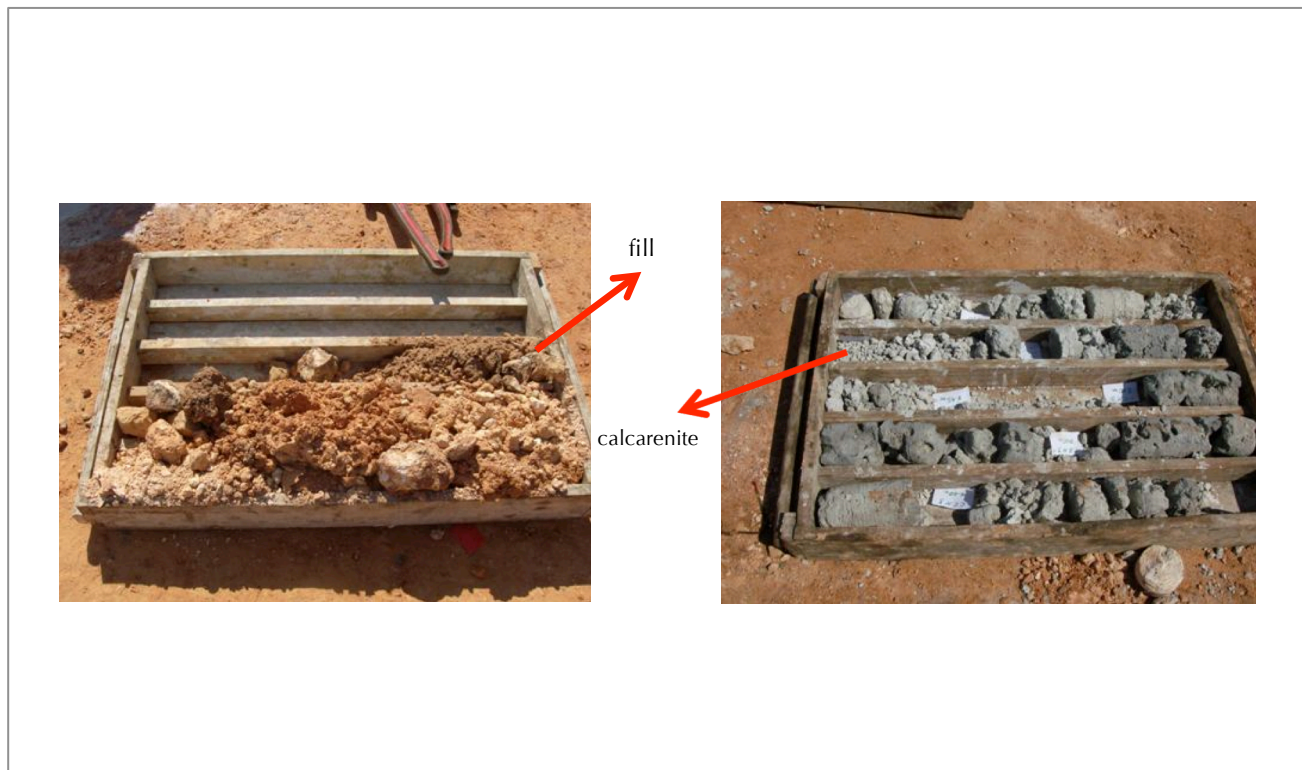
- Fill stratum, always on the surface but with different capacities
- Peat stratum, with low thickness
- Silt stratum, with low thickness

Stratum composed of a combination of various loose soils, which may or may not be present in different

- areas of the building
- Calcarenite stratum, which is present at very variable depths and qualities, from good from very poor

Figure 1 shows two photos of these soils during sample collection in the field, where the fill characteristics and especially the different qualities of calcarenite, with variable RQD values, can be observed.

Figure 1. Photos of the fill samples and calcarenite collected during exploration



For the purposes of this study, stratification can be simplified to two zones, the first of which is composed of fill, peat, silt, and other loose soils that may or may not be present, which is formed by soft soils that do not contribute to the bearing capacity of piles. The second zone is composed of calcarenite, with different thickness and quality, which is what actually contributes to the tip and friction, accordingly.

As previously mentioned, the quality of calcarenite is very variable, and can be grouped, based on the study of all the geological engineering research done, into 3 groups:

- Good quality calcarenite, with an average value of $R_c=9000$ kPa
- Poor quality calcarenite, with an average value of $R_c=1650$ kPa
- Very poor quality calcarenite, with a value of $R_c=1000$ kPa

The tested piles were always embedded, with different EL embedded lengths, in calcarenite, with one of the above qualities. The result was when it was embedded in poor or very poor quality calcarenite, it was necessary to consider the contribution to friction, whereas when it was embedded in good quality calcarenite, only the contribution in the tip was taken into consideration.

3. Analysis of different theories to take into account the contribution to friction of embedded piles in rock

As was outlined in the introduction of the article, in international literature, there are several works studying this issue, including among the most recent: Serrano (2008), Perez Carballo (2010) and Olmo (2011). All these studies agree that there are two groups of theories to establish ultimate resistance to friction in piles in rock τ_{ult} .

- Theories that consider the τ_{ult} as a linear function of the compressive strength of rock R_c .

The general expression posed by this theory to obtain the ultimate friction resistance of pile in rock is defined by Formula 1.

$$\tau_{ult} = \alpha \cdot \sigma_c \quad (1)$$

According to Pérez Carballo (2010), the expressions proposed by the main authors are those that are shown in the following formulas:

- Thorne $\tau_{ult} = (0.05\sim 0.1) \cdot \sigma_c$ (2)
- Poulos and Davis $\tau_{ult} = 0.15 \cdot \sigma_c$ (3)
- Australian Code $\tau_{ult} = 0.15 \cdot \sigma_c$ (4)
- Hooley and Lefroy $\tau_{ult} = 0.3 \cdot \sigma_c$ (5)
- Spanish Technical Code $\tau_{ult} = (0.1\sim 0.4) \cdot \sigma_c$ (6)
- Theories that consider the τ_{ult} as a quadratic function of the compressive strength of rock R_c .

The general expression posed by this theory to obtain the ultimate friction resistance of pile in rock is defined by Formula 7.

$$\tau_{ult} = \alpha \cdot \sigma_c^k \quad (7)$$

All the authors of these studies agree that $k=0.5$, so what varies in each one is the range of values of α that were considered. According to Pérez Carballo (2010), the expressions proposed by the main authors are those that are shown in the following formulas:

- Rosenberg and Journaux $\tau_{ult} = 0.4 \cdot \sigma_c^{0.5}$ (8)
- Hovarth and Hovarth $\tau_{ult} = (0.2\sim 0.3) \cdot \sigma_c^{0.5}$ (9)
- Fleming $\tau_{ult} = 0.45 \cdot \sigma_c^{0.5}$ (10)
- Hooley and Lefroy $\tau_{ult} = 0.15 \cdot \sigma_c^{0.5}$ (11)
- AASHTO Manual $\tau_{ult} = (0.21\sim 0.26) \cdot \sigma_c^{0.5}$ (12)
- Carubba $\tau_{ult} = (0.10\sim 0.25) \cdot \sigma_c^{0.5}$ (13)

All previous theories can be expressed by Formula 7, considering the following variation intervals for variables k and α .

If $K=1$ then α takes the values of $\alpha = 0.05\sim 0.4$

If $K=0.5$ then α takes the values of $\alpha = 0.10\sim 0.45$

4. Determination of bearing capacity of piles.

The procedure for determining the bearing capacity of piles, given the complexity of the study area, is set forth in Figure 2.



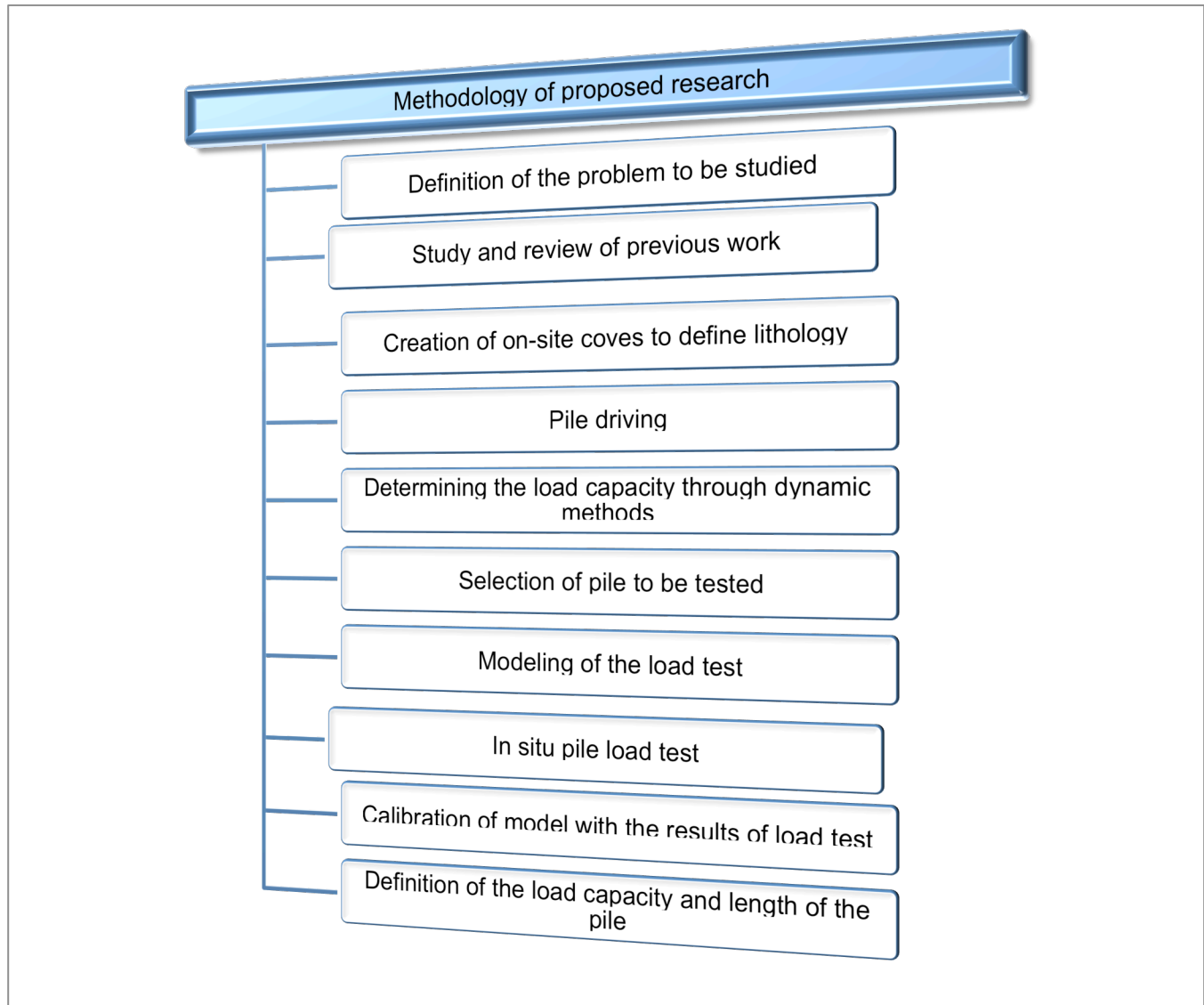


Figure 2. Proposed methodology to determine the bearing capacity of piles

From the above methodology, there has already been a very general overview of the first 3 steps in Sections 1 and 2 of this article. While we will not analyze those related to steps 4 and 5, due to the length of the article, we note that 36 pile driving tests were initially conducted, controlled for various conditions of pile length and stratification characteristics. From the results of these tests, it was possible to have an initial approximation of the bearing capacity of the piles, with the application of different formulas of dynamic methods. Together with the initial geological engineering characterization of the area, this made it possible to determine the piles that needed to be tested in situ

4.1 Conducting situ load tests on chosen piles

From the analysis of the results obtained from the first

5 steps of the methodology defined in Figure 2, it was determined that 8 in-situ load tests were needed for piles of different lengths and embedded with different EL embedded lengths, in calcarenite of different qualities. In this article we will discuss the results of 5 load tests: one of a pile embedded in very poor quality calcarenite, two of piles embedded in poor quality calcarenite, and two of a pile embedded in good quality calcarenite.

The load tests were performed on individual piles, applying three load cells of 1000 kN capacity each, and placing a reaction pile above them, always greater than the load applied to the pile. Figure 3 shows 2 photos of the characteristics of the load tests.



Figure 3. Photos of conducting load tests

Next we will show the results of two of the completed load tests, one with the pile embedded in poor quality calcarenite and the other embedded in good quality calcarenite.

The first of these load tests was carried out on Pile 026, 11 m long, 9 m of which were driven, in a stratification whose Zone 1, composed of soft ground, with a capacity of 4 m, embedding of 5 m in the poor quality calcarenite. Figure 4 shows the results of the load deformation curve obtained from the load test.

As shown in Figure 4, the pile failed with an ultimate load of 1305 kN and with a final deformation of 41 mm.

After unloading, a yield stress of 25.8 mm was maintained, corroborating the occurred failure.

The second load test was carried out on Pile 006, 9 m length, 7.2 m of which were driven, in a stratification whose Zone 1 had 6 m, embedding the pile 1.2 m in good quality rock. Figure 5 shows the results of the load deformation curve obtained from the load test.

As shown in Figure 5, the pile was able to carry a load of up to 1902 kN, but for that load, it had almost a linear behavior, with a final deformation of less than 10 mm. After unloading, it had a yield stress of only 2 mm, which indicates that it did not reach failure.

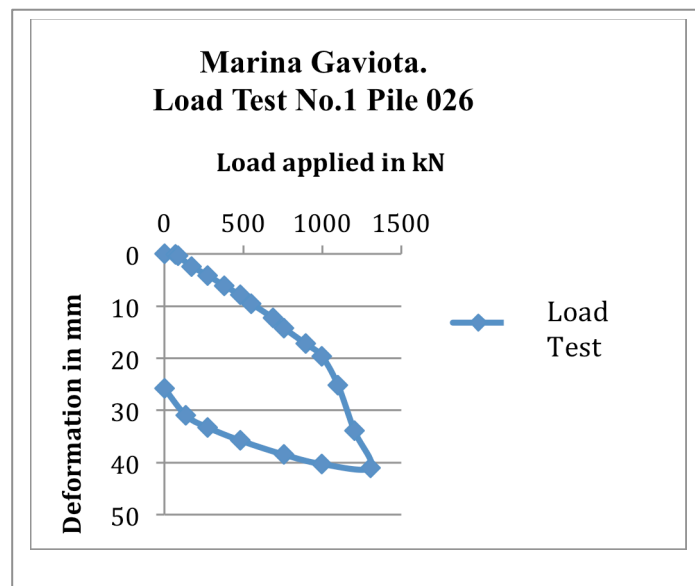


Figure 4. Results of load test on Pile 026



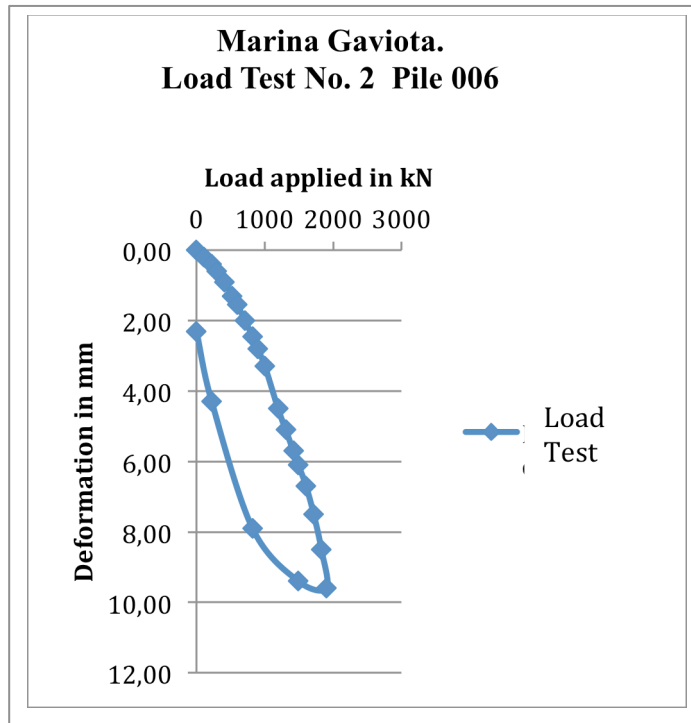


Figure 5. Results of load test on Pile 006

To obtain the ultimate load reaching failure on Pile 006, it was necessary to apply the graphic analytical method defined in the Proposed International Standard for the design

of pile foundations (Ibañez, Quevedo 2011). The result was an estimated ultimate load capacity of 2300 kN, as shown in Figure 6.

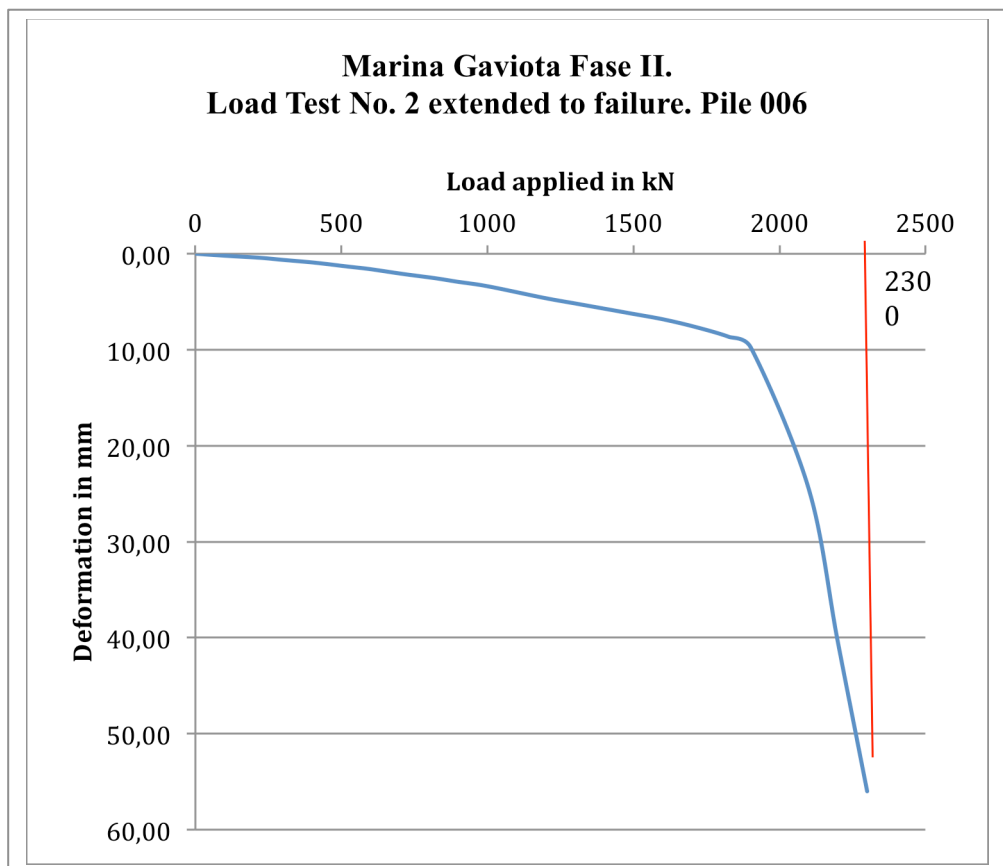


Figure 6. Results of the load test on Pile 006, extended to failure

4.2 Determination of the bearing capacity of piles applying theoretical models

To determine the bearing capacity of the pile, expressions defined in the Proposed International Standard for the design of pile foundations (Ibañez, Quevedo 2011) are utilized. As what is being determined is the ultimate vertical bearing capacity of the pile, Q_{vult} , safety factors equal to the unit are used in all cases. Therefore, the ultimate bearing capacity of the pile is defined by Formula 14, shown below:

$$Q_{vult} = Q_{pvult} + Q_{fvult} \tag{14}$$

where:

Q_{pvult} : Ultimate vertical load capacity at tip

Q_{fvult} : Ultimate vertical load capacity for friction

In the case of contribution at tip, Formula 15 is applied, considering the contribution only when the pile is embedded in good quality rock. In the case that it is embedded in poor quality or very poor quality rock, the friction contribution within the rock will be considered.

$$Q_{pvult} = A_p \cdot R_c \cdot k_{sq} \cdot d_r \tag{15}$$

where:

A_p : Area of pile

R_c : Average simple compressive strength of the rock

k_{sq} : Factor that takes the quality of the rock into account, determined according to RQD

d_r : Factor that takes the EL embedded length of the pile in the rock into account, determined by Formula (16).

$$d_r = \left(1 + 0.4 \frac{LE}{D}\right) \leq 3.5 \tag{16}$$

where:

D: Diameter of pile

In the case of friction contribution in the rock (Formula 17), everything analyzed in Section 3 of this article will be included when obtaining the τ_{ult} .

$$Q_{fvult} = P_p \cdot \tau_{ult} \cdot LE \tag{17}$$

where:

P_p : Perimeter of pile

Applying these expressions and determining the deformations of the pile foundations for the classic expressions for the same intervals of the load test, the theoretical models can be calibrated with the values of the load test. Figures 7 and 8 show the results of such adjustments for the load tests on Pile 026 and Pile 006 respectively.

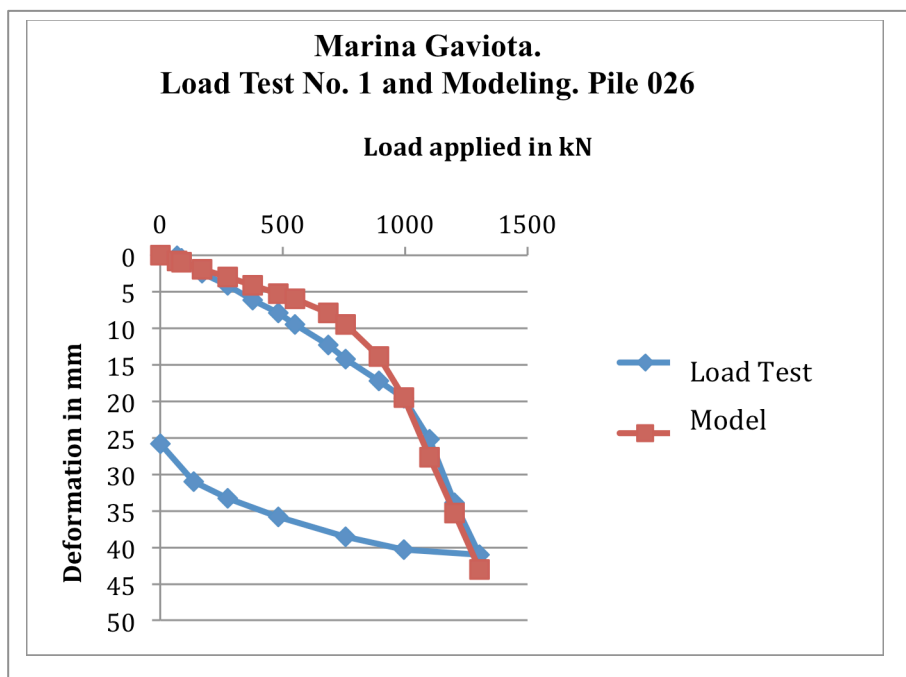


Figure 7. Results of the load tests and modeling of Pile 026



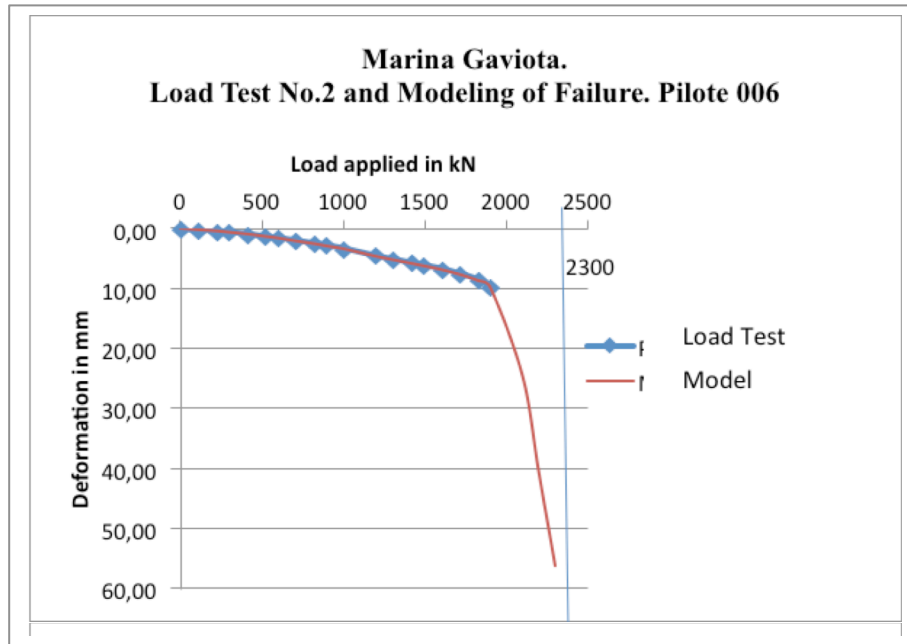


Figure 8. Results of load test and modeling of Pile 006

As shown in Figures 7 and 8, the adjustments obtained between theoretical models and load tests were very good from an engineering standpoint.

Table 1 shows the numerical results of adjusting the values of α for both methods considering the linear relationship with R_c and those that considered the quadratic relationship, for the determination of Q_{fvult} .

Table 2 shows the numerical results obtained in the determination of the contribution at the tip of all of the analyzed piles.

Table 3 shows the numerical results obtained in determining the Q_{vult} by theoretical models, considering the linear and quadratic methods when determining the Q_{fvult} , and the values of the ultimate bearing capacity obtained from the load test Q_{vultpc} .

Table 3 shows the good calibration achieved between the Q_{vult} values obtained from theoretical models and the Q_{vultpc} values obtained from load tests. In the case of piles 010, 026, and 037, where the friction contribution in the rock was considered, it was not possible to establish what method would be the best suited to the analyzed case: that which considers the linear function or that which considers the quadratic function. However, in general, both are very well suited to the cases.

Table 4 shows the intervals of values of α obtained in the adjustments made when determining the Q_{fvult} for both methods, as well as the intervals obtained from the analysis of international literature. As shown here, α intervals obtained in the investigation are among those mentioned in the literature, adjusting to their lower limits.

Table 1. Values of α and Q_{fvult} obtained for all the analyzed piles

Pile	LE (m)	Rock Quality	R_c (kPa)	Linear Formula Q_{fvult}		Quadratic Formula of Q_{fvult}	
				α	Q_f (kN)	α	Q_f (kN)
010	5.4	Very poor	1000.00	0.1	864.00	0.1	864.00
026	5.0	Mala	1650.00	0.09	1188.00	0.12	1233.00
037	5.7	Mala	1650.00	0.12	1806.00	0.15	1757.00

Table 2. Values of Q_{pvult} obtained for all the analyzed piles

Pile	LE (m)	Rock Quality	R_c (kPa)	Q_{pvult} (kN)
010	5.4	Very poor	1000.00	56.00
026	5.0	Poor	1650.00	92.40
037	5.7	Poor	1650.00	92.40
006	1.2	Good	9000.00	2376.00
007	0.6	Good	9000.00	1728.00



Table 3. Values of Q_{vult} and Q_{vultpc} for the analyzed piles

Pile	LE (m)	Rock Quality	Q_{pvult} (kN)	Q_{fvult} (kN)		Q_{vult} (kN)		Q_{vultpc} (kN)
				Linear Formula	Quadratic Formula	Linear Formula	Quadratic Formula	
010	5.4	Very poor	56.00	864.00	864.00	920.00	920.00	900.00
026	5.0	Poor	92.40	1188.00	1233.00	1280.40	1325.40	1305.00
037	5.7	Poor	92.40	1805.00	1757.00	1898.40	1850.40	1850.00
006	1.2	Good	2376.00	0.00	0.00	2376.00	2376.00	2300.00
007	0.6	Good	1728.00	0.00	0.00	1728.00	1728.00	1700.00

Table 4. Intervals of α values obtained in the investigation and proposed in the literature

Type of formula for obtaining the τ_{ult} according to the R_c	Formula	Values of α from the literature	Obtained values of α
Linear	$\tau_{ult} = \alpha \cdot R_c$	0.05~0.40	0.09~0.12
Quadratic	$\tau_{ult} = \alpha \cdot R_c^k$ (k=0.5)	0.10~0.40	0.10~0.15

4.3 Definition of the bearing capacity and pile length

Following the methodology presented in Figure 2 and performing similar analyses to those set forth in this article for all the conditionals of the piles in the building, combining the results of the application of the calibrated theoretical models, the dynamic formulas from the results of controlled pile driving tests, and the results obtained from all the load tests that were carried out, and applying the appropriate safety coefficients, it was possible to determine the bearing capacity of the piles for all the defined zones, the pile length, and the

number of blows in the last foot that must be secured during pile driving to ensure said bearing capacity

Figure 9 shows a plant of the building and the different zones defined as typical for piles from the geological engineering conditions thereof, the bearing capacity of Q_{vtrab} , and the length of the piles

Similarly, Table 5 summarizes the values of Q_{vtrab} , the length of the piles, and the number of blows in the last foot that must be secured during the pile driving of each of the established zones.

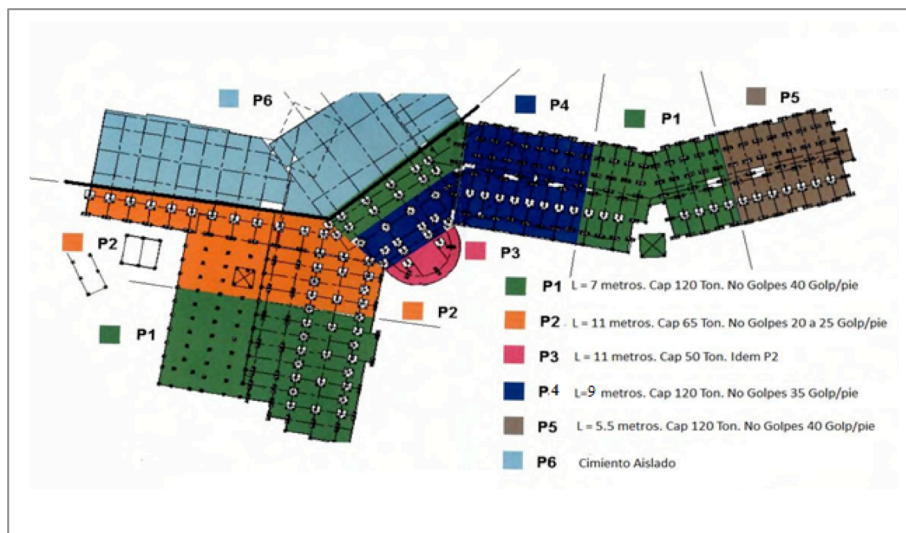


Figure 9. Zoning of the pile solutions, with their Q_{vtrab} and length



Table 5. Values of Q_{vtrab} , length of piles and the number of blows in the last foot that must be secured during the pile driving of each of the established zones.

No	Identifier	Length (m)	Q_{vtrab} (kN)	Number of blows/foot
1	P1	7	120	40
2	P2	11	65	20 a 25
3	P3	11	50	20 a 25
4	P4	9	120	35
5	P5	5.5	120	40

The above results brought about a rationality in the solution of pile foundations for the analyzed building, with consequent cost savings. This allowed verification during pile driving that the obtained results were valid, having a very low loss of piles (less than 3%), with a behavior in each zone as established in the investigation.

5. Conclusions

- A general methodology is established to address the solution of pile foundations in highly variable and complex geological engineering conditions, using a combination of correct characterization of the different existing stratifications, and the physical and mechanical properties of soils and rocks that compose them. Using theoretical methods of determining Q_{vult} , where those that consider friction contribution in rock are included as well as dynamic formulas from the pile driving test results and in situ load tests, Q_{vult} can be put as close to reality as possible.
- Through studying the literature about methods for taking friction contribution in piles embedded in rock into account, it was concluded that there are two groups of methods. The first considers a linear relationship between τ_{ult} and R_c from the α coefficient, finding that this value, according to the different authors consulted, takes values in the range of $0.05 \leq \alpha \leq 0.4$. The other considers a quadratic relationship between τ_{ult} and R_c from coefficient α , finding that the value, according to the different authors consulted, takes values in the range of $0.1 \leq \alpha \leq 0.4$.
- From the results of the load tests, it was possible to calibrate the theories and coefficients used to consider friction contribution in the rock, so that for the methods that consider the linear relationship between τ_{ult} and R_c , the interval of variation α was $0.09 \leq \alpha \leq 0.12$, while for the methods that consider the quadratic relationship, the interval was $0.1 \leq \alpha \leq 0.15$, both being within those values defined in the literature, always tending to their lower values.
- From determining Q_{fvult} , including therein the friction contribution in the rock by the analyzed methods, and the Q_{pvult} by classic methods, it was possible to obtain a satisfactory engineering adjustment between the load deformation curves obtained from load tests and those proposed by theoretical methods, demonstrating the validity of the latter.
- With the application of the general methodology proposed in the solution of pile foundations of the analyzed building, the following was achieved: rational results for the foundation, proposed zoning depending on geological engineering characteristics, allowing to define the value of Q_{vtrab} , the length of the piles, and the minimum number of blows needed to secure the last foot during pile driving for each of the zones. All of this was corroborated satisfactorily during pile driving.



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