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Load Capacity of Partially Embedded Root Piles in Rock Mass (Rhyolite)

Capacidade de Carga de Estacas Raiz Parcialmente Embutidas em Maciço Rochoso (Riolito)

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Abstract: Geotechnical investigation must be understood as the first foundation engineering activity to be developed in a project. In the case of the large projects developed in the Governador Eraldo Gueiros Port Industrial Complex/PE (SUAPE/PE Port Industrial Complex), many of them had to modify the original foundation design due to the diversity of soil types found in the area. In this location, expansive, collapsible and soft soils were found, as well as the presence of rocks. Regarding the rock mass of Petrochemicals Suape, after geotechnical and geological studies, it was understood that it is basically composed of three (03) types of geomechanical materials. The first material consists of a rhyolite matrix, the second comprises an intermediate material between rhyolite and trachyte and the third material consists only of basalt. To build part of the petrochemical infrastructure, it was decided to develop a deep foundation project using a root pile partially embedded in the rock mass. Since this is a foundation engineering solution that is rarely used in construction (since deep foundations were developed to be used in soils with low bearing capacity and not in rocky materials) and because it involves geological and geotechnical conditions that are peculiar to the region, there was interest in analyzing the results obtained from estimating the load capacity of the root pile using empirical, semi-empirical and load test methods. After analyzing the results of the data obtained and calculated, it was found that some methods used in this work resulted in conservative values and others did not provide reasonable estimates for the conditions obtained.

Keywords: Load Capacity; Root Pile; Rock Mass; Empirical and Semi-Empirical Methods; Load Test.

Resumo: A investigação geotécnica deve ser obrigatoriamente entendida como a primeira atividade da engenharia de fundações a ser desenvolvida numa obra. No caso das grandes obras desenvolvidas no Complexo Industrial Portuário Governador Eraldo Gueiros/PE (Complexo Industrial Portuário SUAPE/PE), muitas delas tiveram que modificar o projeto de fundações original devido a diversidade dos tipos de solos encontrados na referida área. Nesta localidade, foram encontrados solos dos tipos expansivos, colapsíveis, moles e presença de rochas. No que se refere ao maciço rochoso da Petroquímica Suape, após estudos geotécnicos e geológicos desenvolvidos, ficou compreendido que o mesmo é composto basicamente por três (03) tipos de materiais geomecânicos. O primeiro material consiste numa matriz de riolito, o segundo compreende um material intermediário entre o riolito e o traquito e o terceiro material consta apenas de basalto. Para construir parte da infraestrutura da petroquímica, optou-se por desenvolver um projeto de fundação profunda por meio do uso de estaca raiz parcialmente embutida no maciço rochoso. Por se tratar de uma solução da engenharia de fundações de pouca aplicação em obras (pois fundações do tipo profunda foram desenvolvidas para serem aplicadas em solos de baixa capacidade de suporte e não em materiais rochosos) e, por envolver condições geológicas-geotécnicas peculiares da região, desenvolveu-se, portanto, o interesse em analisar os resultados obtidos da estimativa da capacidade de carga na estaca raiz pelos métodos empíricos, semiempíricos e de prova de carga. Após analisar os resultados dos dados obtidos e calculados, verificou-se que alguns métodos utilizados neste trabalho resultaram em valores conservadores e outros não forneceram estimativas razoáveis para as condições obtidas.

Palavras-chave: Capacidade de Carga; Estaca Raiz; Maciço Rochoso; Métodos Empíricos e Semiempíricos; Prova de Carga.

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1. Introduction

É It is generally agreed that proper recognition of subsoil conditions is essential to guide the assessment, construction and instrumentation phases of a geotechnical project. The way in which the site is investigated must be adapted to the size of the project, the loads imposed and the complexity of the terrain.

Due to the variability of the subsoil or the level of loading imposed by the structure, many geotechnical works need to be excavated or supported on rock masses, including dams, tunnels, mines, slopes and foundations.

With their complex nature, rock masses show great variability in terms of the distribution of discontinuities and index and mechanical properties, so construction on rock substrates becomes an additional challenge for the professionals involved.

In the absence of a complete investigation, the parameters used in design are obtained from the values proposed in the literature, however, they usually apply to intact rock, which in most cases does not reflect the real condition of the material.

According to Silva (2013), deep foundation projects in rock substrate occur when the massif is at shallower depths and the surface layers of soil do not have satisfactory thickness and/or geotechnical conditions. In these cases, the pile can be positioned in relation to the rock mass in four ways, as shown in Figure 1.

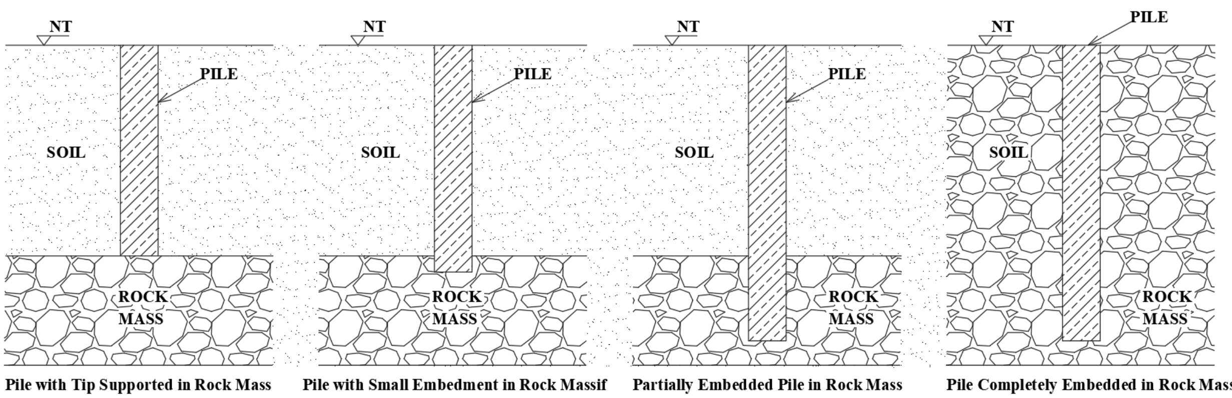


Figure 1 – Positioning the pile in relation to the rock mass.
Source: Authors (2025).

How the load is transferred between the top and bottom of a pile embedded in a rock mass is a subject that generates a lot of discussion in technical circles. According to Rowe & Armitage (1987), the load applied to a pile embedded in a rock mass is largely absorbed by the lateral resistance of the embedment.

For the authors, the resistance of the pile tip is only mobilized when there is considerable slippage at the pile-rock interface. In addition, the difficulty of guaranteeing effective cleaning of the base of the hole leads many designers to disregard the contribution of the pile tip resistance.

Similarly, the contribution of the soil section is usually disregarded when determining the pile's load capacity.

Thus, methods for estimating the load capacity of piles partially embedded in rock masses have focused on determining the lateral resistance of the embedment. Thus, accordingto Juvêncio (2015), the Project adopts excessive embedment, which consequently increases the costof the foundations.

According to Carter & Kulhawy (1988), in piles completely embedded in rock and subjected to axial compression, the tip resistance is only really mobilized at stress levels capable of causing some significant movement between the shaft-rock interfaces along the entire length of the pile.

In view of this, the majority of studies on load transfer in piles embedded in rock masses have in fact focused on evaluating the mobilization of the lateral resistance of the embedment and its influencing factors, such as the roughness and geometry of the pile, the relationship between the modulus of elasticity of the rock and the concrete of the pile, the way the pile is made, the degree of fracturing of the rock mass, the uniaxial compressive strength of the rock, among others.

With regard to roughness, according to Juvêncio (2015), lateral resistance generally has a bonding component as a physical result of cementation between the concrete and the rock or through the roughness produced mechanically along the embedment.

The typical load transfer mechanism, depending on the geometry of the roughness, can be summarized in three stages, according to Nunes (2002):

- Resistance guaranteed by pile-rock adhesion;
- Resistance due to friction and/or imbrication;
- Shear strength.

Johnston and Lan (1989) observed that when an axial load is applied to the pile, shear stress develops and vertical displacements occur with the consequent rupture of the adhesion between the pile and the rock, as shown in Figure 2 a) and b).

These displacements are accompanied by an increase in the diameter of the embedded section. Figures 2 c) and d) make it possible to analyze this mechanism because, according to Costa (2005), the concrete-rock interface can be interpreted as a discontinuity or joint in the system. In this way, the behavior of a pile embedded in rock can be studied in specimens using the direct shear test with constant stiffness. Figure 2 d) shows that in order to overcome the imbrication between the concrete and the roughness, volumetric expansion is required during shear.

Thus, the greater the roughness, the greater the dilatancy of the section, the normal stress to the pile shaft increases with the increase in dilatancy, thus increasing the lateral resistance. Withregardto pile geometry, which in turnis defined by the ratio between the embedment length (L) and the diameter (D), its importance in the load transfer of foundation scan be seen.

Juvêncio (2015), based on studies carried out by Osterberg & Gill (1973), observes that as the (L/D) ratio increases, part of the applied load is progressively transmitted to the side walls. In Figure 3, it can be seen, for the condition that the modulus of elasticity of the rock is greater than that of the concrete, that in a geometry with a ratio of L=4D almost all of the applied load is transferred to the walls of the embedment. On the other hand, when looking at the L=D ratio, it can be seen that only 50% of the applied load is transmitted to the walls of the embedment.

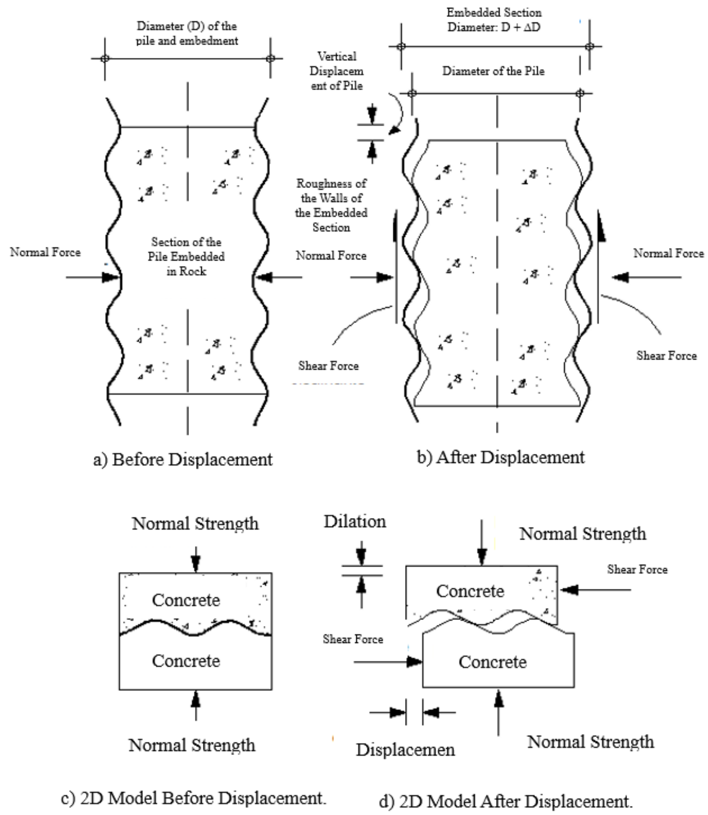


Figure 2 – Concrete/rock interface under axial loading.
Source: JUVÊNCIO (2015).

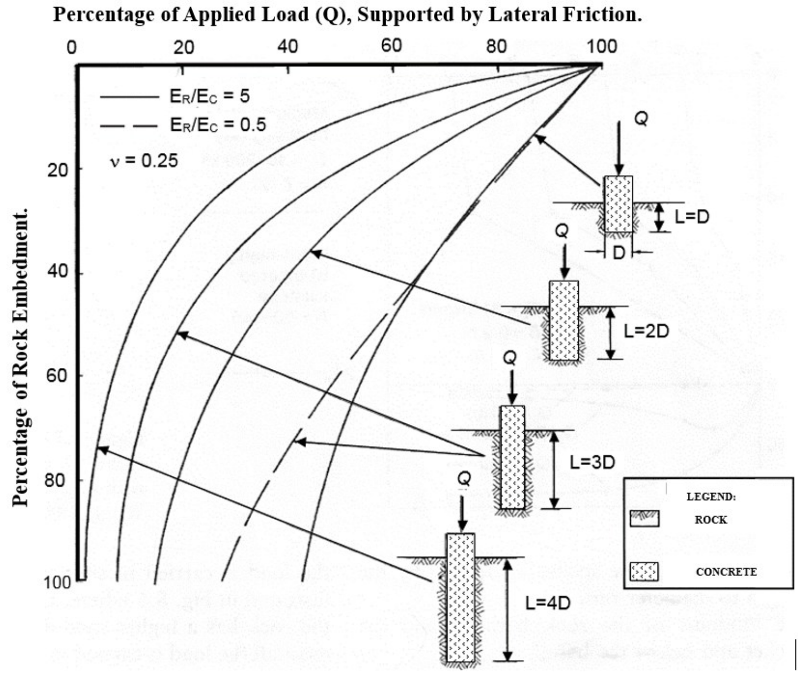


Figure 3 – Distribution of shear stress in the side wall in relation to the length of the inlay.
Source: JUVÊNCIO (2015).

Another factor influencing the lateral resistance of piles is the construction process. After the hole has been drilled, the debris or bentonite must be removed by injecting water. However, when the cleaning operation does not guarantee that the debris is completely removed, it can form a thin layer on the wall of the hole which can reduce the adhesion between the concrete and the rock, thus interfering with the shear stress at the pile-rock interface.

Despite the existence of several studies on the factors that influence the behavior of piles embedded in rock, the methods for estimating the load capacity do not cover all of these factors, so, given the difficulty in quantifying some calculation parameters, it is necessary to make great simplifications that may not reflect the complexity of the real conditions of the foundation.

Generally speaking, this paper aims to present the results of load capacity estimates for foundations embedded in the rock masses of the SUAPE/PE Industrial Port Complex, by analyzing the results obtained from three static load tests on root piles and the empirical and semi-empirical load capacity estimation methods found in the technical literature.

2. Point Resistance of Piles Embedded in Rock and Influencing Factors

The Federal Highway Association (2010) proposes some recommendations that should be taken into account when considering the tip resistance in the design of foundations embedded in the rock mass:

a) The tip of the pile must be supported on solid rock, with no joints or compressible cavities and at a depth of at least one diameter below the base of the pile;

b) There must be no significant cavities or voids under the rock at the base of the pile;

c) There must be certification of the cleanliness of the base of the hole..

The presence of joints or cavities in the rock mass can cause the backfill material to escape during pile driving, which can compromise the integrity of the design geometry of the foundation element. In addition, if these discontinuities are compressible, the pile may deform due to the movement of the joints after the load is applied.

Cabral & Antunes (2010) also comment on the subject. For them, the tip capacity of a pile is directly proportional to the efficiency of the drilling cleaning.

Therefore, when total cleaning of the bottom of the hole is not guaranteed, the portion relating to tip resistance must be carefully analyzed, especially in the case of piles embedded in low-quality massifs, i.e. extremely weathered and/or with a very discontinuous internal structure.

In addition, as already mentioned, the tip resistance of piles embedded in rock is only mobilized when the shaft shows significant displacements.

More recently, by analyzing 30 dynamic load tests, Juvêncio (2015) found that the ratio between the load reaching the tip and the load applied to the top of the pile is 9%. This value is also consistent with the results found by Carter & Kulhawy (1988) for the same ratio, where the load mobilized by the tip is around 10 to 20% of the total load.

In his work, Ladanyi (1977) considers that the pile tip supports between 5 and 25% of the total load, for embedment ratios (ratio between pile length and diameter) between 2 and 4.

However, many designers usually disregard the portion of the pile tip resistance, assuming that the entire load is borne by the stem resistance.

3. Estimating the Load Capacity of Piles in Soil and Partially Embedded in a Rock Mass

The axial load applied to the top of a pile partially embedded in rock is transferred to the ground by means of the shear stress in the pile section in the soil, the lateral embedment in the rock mass and the normal compressive stress between the pile tip and the rock. The general scheme of load transfer in piles partially embedded in rock mass is shown in Figure 4.

When a pile passes through heterogeneous layers, the deformations required to mobilize resistance in each section are different, due to the stiffness characteristics of the materials it passes through. As a result, load transfer along piles partially embedded in a rock mass becomes complex and there are few definitive conclusions. Therefore, due to the degree of uncertainty associated with the use of deep foundations partially embedded in rock, projects tend to be very conservative.

Juvêncio(2015) points out that most pile foundation projects, with partial or total embedment in rock, consider only the lateral resistance to support the entire load, rather than the combination of lateral and tip resistance. In addition, Musarra (2014) states that in piles embedded partially in rock, the contribution of the soil section is usually disregarded. However, through analysis of dynamic load tests, Juvêncio (2015) observed that in the piles tested, where there was a profile with gradual weathering of the rock, the portion of resistance supported by the soil typically reached 40% of the ultimate load applied.

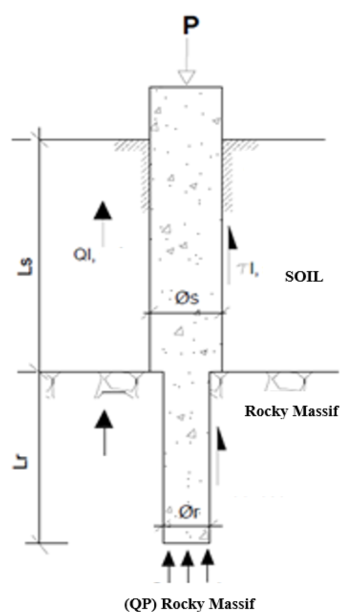


Figure 4 – General diagram of load transfer in piles partially embedded in a rock mass.
Source: Adapted from JUVÊNCIO (2015).

4. Estimating the Load Capacity of Piles Embedded in Rock

Regarding the estimation of the bearing capacity of piles embedded in rock, many authors have focused their studies on finding correlations between the lateral frictional resistance of the embedment and the tip resistance of the pile with the uniaxial compressive strength of the rock.

In order to estimate the load capacity of piles embedded in rock masses, some authors have developed methods for determining the portion relating to lateral resistance and/or tip resistance. In an attempt to establish formulations to determine the load capacity of piles embedded in rock masses, different correlations with rock mass parameters have been established.

Table 1 details some of these methods according to the parameter used to determine the pile's load capacity.

Table 1 – Methods for Estimating the Load Capacity of Piles Embedded in Rock Masses.

Methods:	**Rock Mass Parameter:**
Rosenberg & Journeaux (1976)	Compressive Strength of the Rock (σ_u).
Meigh & Wolski (1979)	
Paulos & Davis (1980)	
Rowe & Armitage (1984)	
Zhang & Einsten (1998)	
Cabral & Antunes (2000)	Simple Compressive Strength of the Rock and Degree of Alteration and Presence of Fractures.
Compressive Strength of the Rock and Degree of Alteration and Presence of Fractures.	Simple Compressive Strength of the Rock, Type of Rock, Degree of Alteration, RQD and Spacing of Discontinuities.

Source: Authors (2025).

5. General Characteristics of the Study Area

The study area is part of the Suape Industrial Complex, in the municipality of Ipojuca - PE, Figure 5, and is currently home to the Petrochemical Company of Pernambuco (Petroquímica Suape). Located to the south of the city of Recife, Ipojuca is bordered to the north by the municipality of Cabo de Santo Agostinho, to the south by the municipality of Sirinhaém, to the east by the Atlantic Ocean and to the west by the municipality of Escada.

Figure 5 – Location of Petroquímica Suape.
Source: http://especiais.jconline.ne10.uol.com.br/documento-suape-2015/ (accessed on 13/05/24).

5.1 Local Geology

According to Lima Filho (1996), the geological units that occur in the municipality of Ipojuca are basically made up of the Gneissic-Migmatitic Complex, Granitoid Rocks, the Pernambuco Group and Quaternary Coverings.

The site of the project is located in the Pernambuco Group, through the Ipojuca Formation, which according to Pfaltzgraff (1999), is made up of volcanic rocks of Cretaceous age, whose main petrographic types are: andesites, basalts, rhyolites, trachytes and volcanic agglomerates, occurring in the form of effusions, dykes and sills. According to Assis (1990), of the extrusive volcanic rocks, basalts are the oldest, while rhyolites are the youngest.

5.2 Development Information

Petrochemical Company of Pernambuco, Petrochemical Suape, togetherwith Integrated Textile Company of Pernambuco, CITEP, formthe Chemical-Textile Industrial Complex (PQS). Strategically located in the Suape Industrial Port Complex, the Industrial Complex is part of the Federal Government's Growth Acceleration Program - PAC, with Petrobras as its shareholder, Figure 6.

In an area of 70 hectares, PQS has three integrated industrial units, Figure 6:

- PTA unit;
- Polymer and polyester filament unit;
- PET Unit.

In addition, to meet stringent international environmental standards, the PQS company has a shared water and effluent treatment utilities unit.

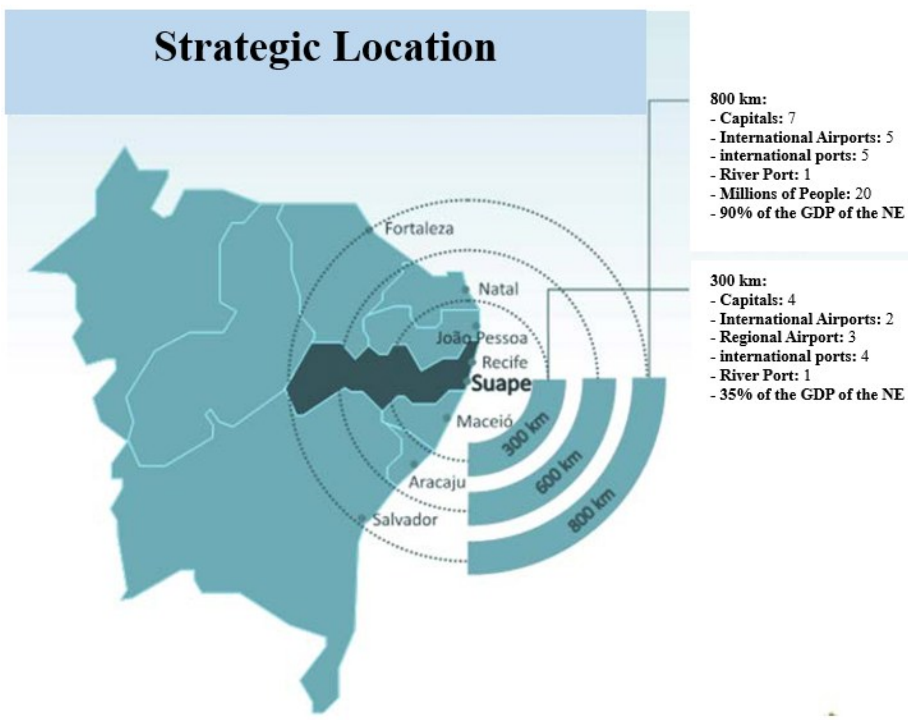


Figure 6 – PQS component units.
Source: http://www.pqspe.com.br/a-empresa/apresentacao (accessed 13/05/24).

Petrochemicals Suape produces purified teraphthalic acid, known as PTA. This product is the raw material for the production of textile polyester, PET resins, packaging, as well as industrial fibers used in the manufacture of tires, materials and equipment for the electrical and automotive sectors and the oil industry.

Inaugurated at the beginning of 2013, the PTA unit has the capacity to produce 700,000 tons of teraphthalic acid. To do this, the PTA plant has large processing units and machinery, which include state-of-the-art technologies.

5.3 Information About the Building Foundations

An extensive geological-geotechnical reconnaissance campaign was carried out at the site to prepare the foundations for the PTA plant, as described in the next chapter.

The results showed that the terrain before the earthmoving work had a variable and complex profile, made up of layers of residual soil with a thickness of between 1 and 26 meters, followed by rock masses with a thickness of between 2 and 15 meters in certain sections.

According to Coutinho (2008), in order to overcome the unevenness of the area, it was necessary to carry out extensive earthworks through cuts and embankments (Figure 7).

When defining the type of foundation used at the PTA plant, the technical team needed to design a foundation that would withstand the high loads from the petrochemical plant's structures and allow its large machinery to function properly, Figure 8. This is because the design of machine foundations is a very complex task, given the various static loads, as well as the dynamic loads they will be subjected to.

Figure 7 – Aerial view of the site after earthworks.
Source: https:// pedevelopment.com/2009/10/21/petrochemicals-suape-will-generate-5-300-vacancies/ (accessed on 13/05/2024).

a) b)

Figure 8 – Transportation and installation of one of petrochemicals Suape's towers.
Source: https://www.petronoticias.com.br/archives/tag/petroquimica-suape (accessed on 13/05/2024).

Therefore, given the heterogeneity of the terrain and the characteristics of the structure to be built, the technical team proposed the following foundation solutions:

- Soil-supported footings: In the early stages of the project, the idea of rock-supported footings was considered for the sections where, after earthworks, the competent rock mass was close to the ground surface and the soil had reasonable support conditions. However, after feasibility studies, this solution was discarded. The final solution consisted of soil-supported footings.

- Metal piles: The use of three types of metal piles was defined which, depending on the subsoil and loading conditions, can have their length entirely in soil, or with at least one meter in rock.

- Continuous auger piles: continuous auger piles with a diameter of 600 and 300 mm and a length of more than 4 m were allowed.

- Root piles partially embedded in rock: root piles partially embedded in rock and piles driven completely into the ground were basically proposed. The effective length initially established for embedding the piles in the rock mass was 4 to 5 meters.

5.4 Field Research

According to Coutinho (2008), due to the size of the site and the distribution of the structures to be built, the plan of the development was divided into two areas, ISBL and OSBL, as shown in Figure 9.

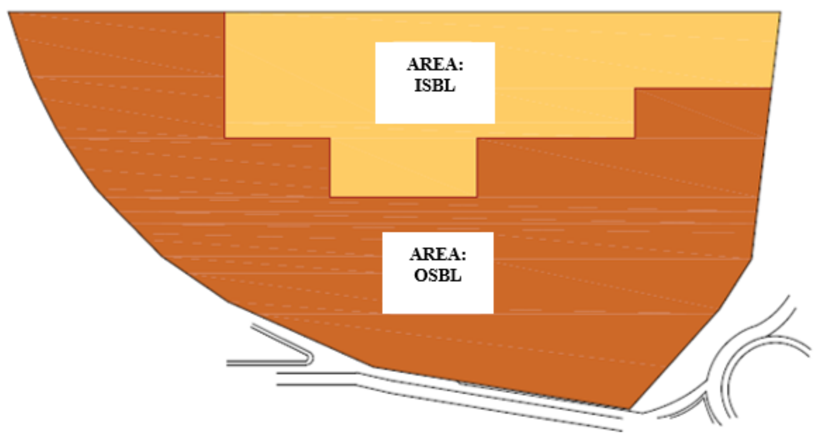


Figure 9 – Divisions of the Petrochemicals Suape Area.
Source: COUTINHO (2008).

The areas were investigated by a first campaign, an additional geotechnical investigation campaign and a complementary campaign, using percussion (SPT) and mixed (SP and SM) boreholes, CPTU, seismic CPTU, resistivity measurements, piezometers and the collection of undeformed and deformed soil samples and rock cores. The approximate number of points investigated can be seen in Table 2.

Table 2 – Field Research Program.

Research Methods:	Quantity:
SPT	107
SM	125
CPTU	36
Seismic CPTU	9
Piezometer	2
Shelby Type Samples	7
Blocks and Deformed Samples	8

Source: Authors (2025).

5.5 Geophysical Tests

Initially, the use of geoelectric tests in the PTA area was aimed at better assessing the morphology of the rock mass and analyzing the properties of the topsoil in order to select a site for electrical grounding.

With regard to determining the morphology of the rock layers, the ST and SM boreholes carried out did not allow a complete analysis of the position and geometry of the top rock, since the geology of the area is complex and is characterized by a vertical succession of alteration soil followed by layers of rock with different conditions, distributed throughout the terrain.

Given this situation, the geophysical investigation sought to help map the main structural features in the area, as well as estimate the depth of the alteration mantle and the top rock. To this end, two geophysical techniques were used to measure electroresistivity: vertical electrical sounding (SEV) and electrical walking (CE).

According to Coutinho (2008), the apparent resistivities were measured using the Schlumberger arrangement, which consists of a symmetrical linear array of four electrodes. The equipment used consisted of a GEOTRADE model GTR-3 meter and accessories such as a power source, cables, reels, electrodes, portable radios and others, as shown in Figure 10.

a)

b)

Figure 10 – Resistance meter, source and other equipment.
Source: COUTINHO (2008).

The distribution of the investigation points can be seen in Figure 11. Eight vertical electrical surveys were carried out with a maximum spacing, AB, of 600m. With regard to the electrical walking technique, as presented by Coutinho (2008), three resistivity profiles were carried out, totaling 1650 meters of AC. The lengths of AB and NM122 were 140 meters and 20 meters respectively. The distance between two successive measurements along the profile was 30 meters.

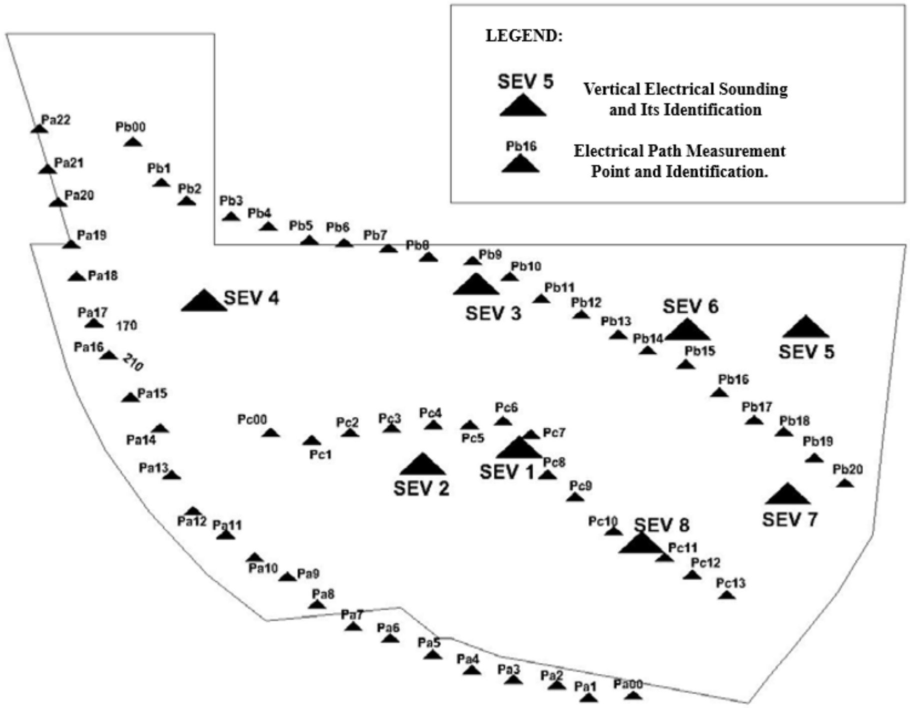


Figure 11 – Resistance meter, source and other equipment.
Source: COUTINHO (2008).

5.6 Geological and Geotechnical Description of the Rock Massif

After jointly analyzing the surveys carried out, the technical team chose a mixed survey to represent each sector and some auxiliary surveys (see Figure 12).

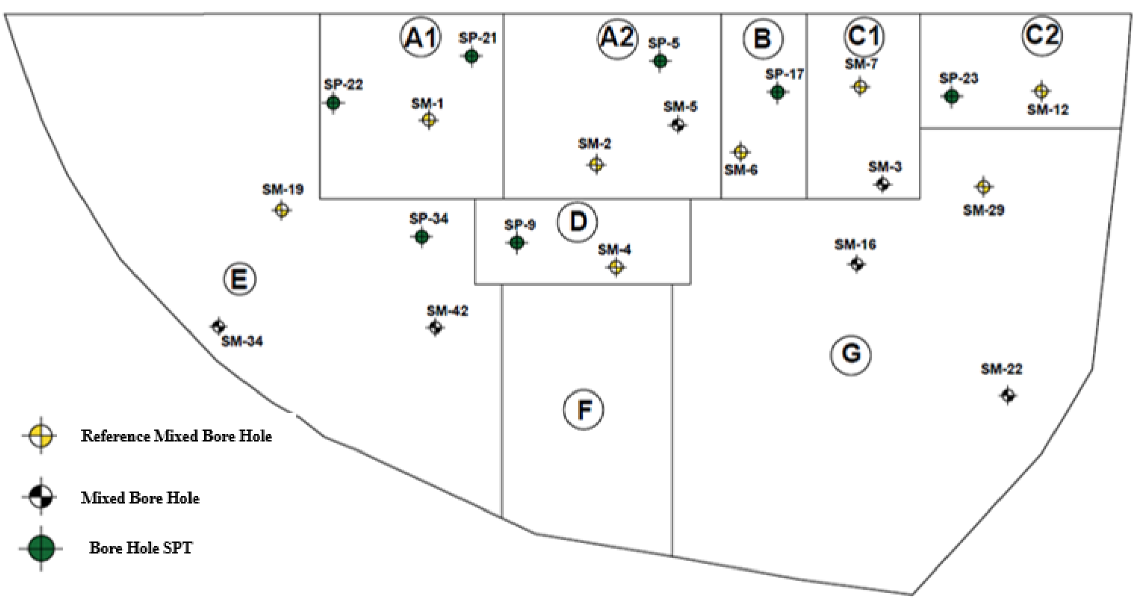


Figure 12 – Location of the mixed survey representing each petrochemical sector.
Source: Adapted from COUTINHO (2008).

Table 3 summarizes information on the classification of the rock mass in the Petroquímica Suape sectors.

Table 3 – Main Characteristics of the Rock Matrices of the Suape Petrochemical Land in Each Sector.

Sector:	Rocky Matrix	Degree of Change	RQD (%)	Continuity Spacing	Roughness	Filling Discontinuities:	RMR Classification:				
							RMR	Class	Description	Cohesion (MPa)	Friction Angle 0
A1	Rhyolite	AII	90 - 100	> 100 cm	Rough	Sound Walls, No Filling..	42	III	Regular	200 - 300	$20^0 - 30^0$
A2	Rhyolite - Quartz / Trachyte	III a IV	14 - 69	16 à 100 cm	Slightly Lisa to Lisa	Walls with incipient alteration, with or without signs of water percolation. Absent filling..	32	IV	Poor	100 - 200	$10^0 - 20^0$
B	Rhyolite - Quartz / Trachyte	III a IV	26 - 50	20 à 100 cm	Lisa		22	IV	Poor	100 - 200	$10^0 - 20^0$
C1	Basalt	I	91 - 100	< 60 mm	Slightly Smooth	Sound Walls, No Filling.	33	IV	Poor	100 - 200	$10^0 - 20^0$
C2	Rhyolite - Quartz / Trachyte	III a IV	0 - 46	< 60 mm	Lisa	Walls with incipient alteration, with or without signs of water percolation. Absent filling.	29	IV	Poor	100 - 200	$10^0 - 20^0$
D	Rhyolite - Quartz / Trachyte	IV - V	16 - 93	20 a 100 cm	Rough to Slightly Smooth	Altered Walls, Missing Fill, Open Fracture.	43	III	Regular	200 - 300	$20^0 - 30^0$
E	Rhyolite - Quartz / Trachyte	I a II	60 - 100	20 a 100 cm	Slightly Smooth	Without Filling to Hard Stone Filling..	56	III	Regular	200 - 300	$20^0 - 30^0$
G	Basalt	II a IV	16 - 62	16 a 100 cm	Slightly Smooth		36	IV	Poor	100 - 200	$10^0 - 20^0$

Source: Authors (2025).

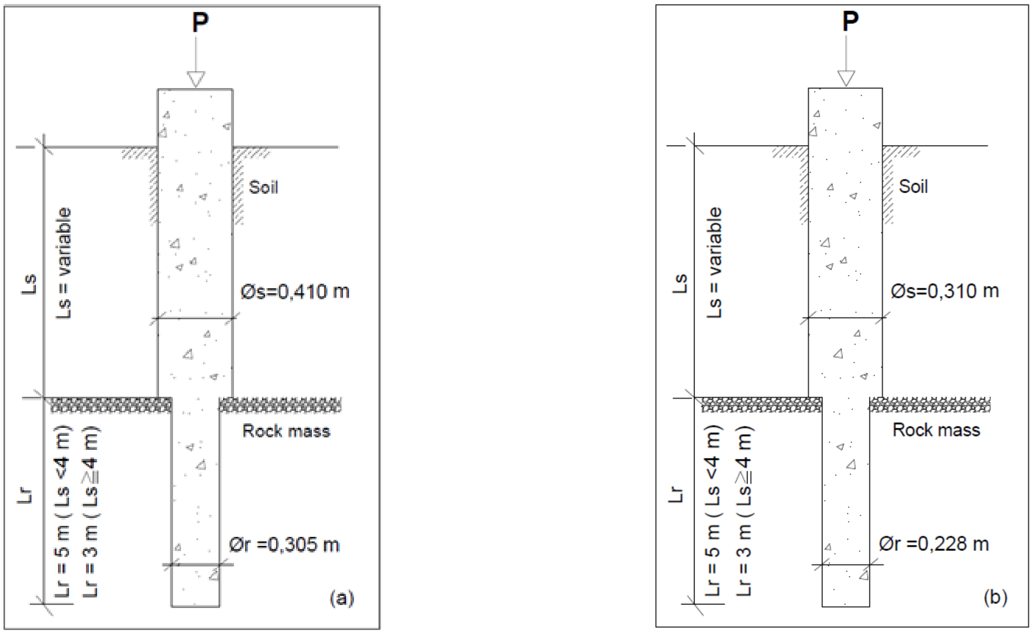
6. Pile Construction Characteristics

The root piles, executed at Petroquímica Suape, were concreted with CA-50 steel reinforcement with a diameter of 200 mm and 25 mm for the longitudinal bars, and CA-25 for the transverse stirrups, with a diameter of 8 mm. The concrete strength was specified as at least Fck = 20 MPa.

In the design phase, based on the characteristics of the subsoil and the level of loading imposed by the structure, two types of root piles partially embedded in the rock mass were proposed, R1 and R2 (see Figure 13).

The length embedded in the rock mass for the two types of pile was set at between 3 and 5 meters. In some situations, after a more detailed analysis of the terrain or due to some executive difficulties, the embedded length was reduced.

To analyze the load-bearing capacity, we used data from three root piles driven into the rock mass of Petroquímica Suape. Table 4 shows the names used to identify the piles analyzed, their geometric characteristics and the working load for which they were designed.



a) b)

Figure 13 – (a) General diagram of type R1 root cuttings (b) General diagram of type R2 root cuttings. Source: Adapted from COUTINHO (2008).

Table 4 – Pile Design Characteristics..

Piles	**ϕ_S – (mm)**	**ϕ_R – (mm)**	**L_S – (mm)**	**L_R – (mm)**	**Workload – (kN)**
E25	410	305	6,70	3,30	1000
E100	410	305	6,70	1,30	1000
E110	310	228	6,70	3,30	750

Source: Authors (2025).

Where:
ϕ_S : diameter of stem section in soil;
ϕ_R : diameter of the rock shaft section;
L_S : stem length in soil;
L_R : stem length in rock.

As shown in Table 4, there is a reduction in the diameter of the pile in the rock mass section. This variation is due to the drilling process, since in the soil section the hole is drilled with the aid of a metal jacket so that debris does not invade the hole. Once the rocky top is reached, the rock drilling equipment must be introduced into the hole, so the diameter in the rock mass will consequently be smaller.

Piles E25 and E100 were drilled in sector A2, while pile E110 was drilled in sector B (see Figure 12).

The exact location of the piles in sectors A2 and B could not be determined, however, the geotechnical profiles in which the piles were driven are known.

Figures 14, 15 and 16 below show the representative geotechnical profiles of piles E25, E100 and E110.

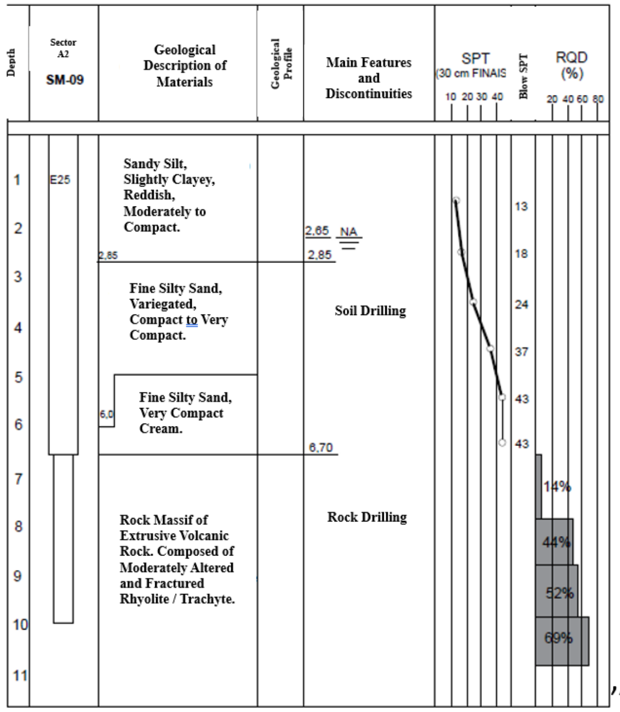


Figure 14 – Geotechnical profile of the execution point of pile E25 in sector A2.
Source: Adapted from COUTINHO (2008).

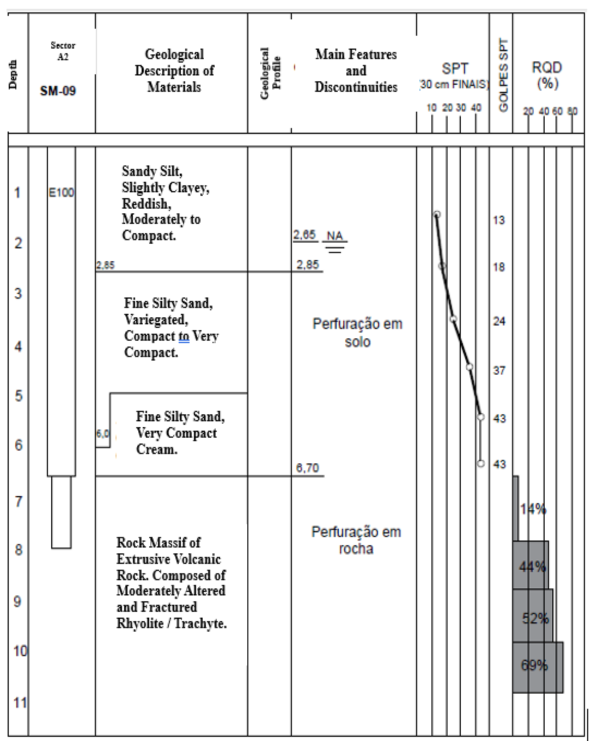


Figure 15 – Geotechnical profile of the execution point of station E100 in sector A2.
Source: Adapted from COUTINHO (2008).

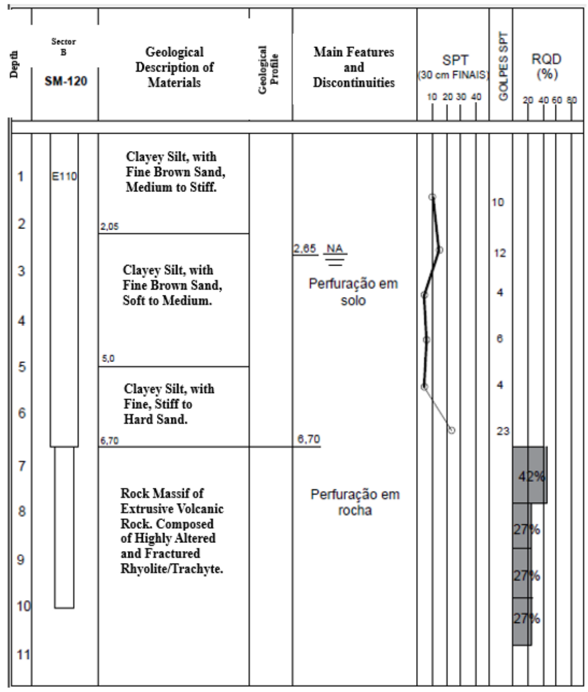


Figure 16 – Geotechnical profile of the execution point of station E110 in sector B.
Source: Adapted from COUTINHO (2008).

7. Estimating the Load Capacity of Piles Using Empirical and Semi-empirical Methods

The load capacity of the piles is estimated using empirical and semi-empirical methods, as detailed in Table 5.

The methods applied to the soil section were chosen because of their applicability to root piles and because they make use of SPT test data.

With regard to the selection of load capacity estimation methods developed for embedment in rock masses, the best-known methods in the technical field were chosen, as well as those using different methodologies.

Table 5 – Methods for Estimating the Bearing Capacity for the Piles Analyzed.

Excerpt from the pile:	Methods for Estimating Pile Bearing Capacity:
Embedded in the Ground	Décourt&Quaresma (1978)
	Aoki&Velloso (1975)
	Cabral (1986)
Embedded in the Rocky Massif	Rosenberg & Journeaux (1976)
	Zhang & Einsten (1998)
	Meigh & Wolski (1979)
	Cabral & Antunes (2000)
	Rowe & Armitage (1987)
	Poulos & Davis (1980)
	Horvath (1978)
	España (2011)

Source: Authors (2025).

7.1. Estimation of the Lateral Resistance of the Ground Section (Q_{LS})

According to the profiles shown in Figures 14, 15 and 16, the lateral resistance values from the contribution of the soil section were estimated for each pile (see Table 6), according to the methods described in Table 5.

Table 6 – Lateral Resistance of the Pile Shaft in Soil.

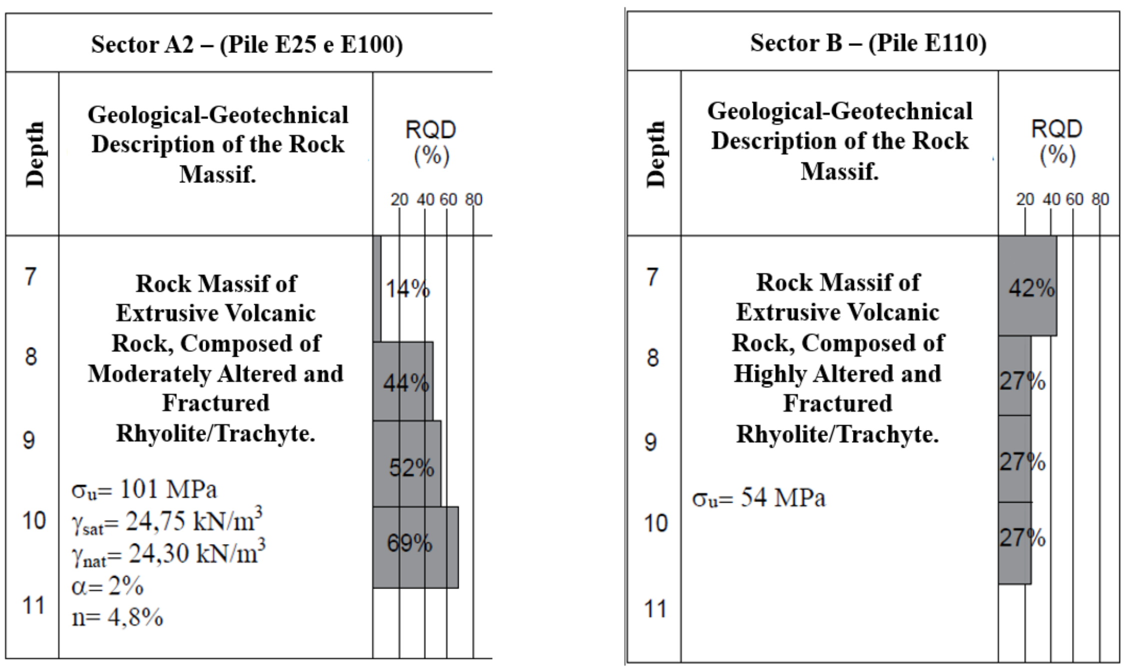
Method:	Pile E25	Pile E100	Pile E110
	Q_{LS} - (kN)	**Q_{LS} - (kN)**	**Q_{LS} - (kN)**
Aoki & Velloso (1975)	818,25	818,25	100,52
Décourt&Quaresma (1978;1996)	1100,73	1100,73	282,97
Método de Cabral (1986):	1170,71	1170,71	170,74

Source: Authors (2025).

7.2. Estimating the Load Capacity of a Section Embedded in a Rock Massif

With regard to the length embedded in the rock mass, the load capacity was determined using methods that take into account certain parameters of the rock mass, such as the compressive strength of the rock matrix, RQD, degree of alteration, fractures, etc.

Thus, based on the results obtained in the investigation campaign carried out in sectors A2 and B, it was possible to present a summary of the characteristics of the rock mass (see Figures 17 a) and b)) in which the E25, E100 and E110 piles were driven.



a) b)

Figure 17 – Summary of the characteristics of the rock mass: a) Sector A2 and b) Sector B.
Source: Adapted from COUTINHO (2008).

In the case of the empirical methods that use Equation 1 to determine the unit lateral resistance ($\tau_{l,r}$), Table 7 contains the coefficients μ and S, applied to piles E25, E100 and E110.

Table 7 – Coefficients for Determining Unitary Lateral Resistance, Using Empirical Methods.

Methods:	Coefficients:	
	μ	**S**
Rosenberg & Journeaux (1976)	0,37	0,51
Zhang & Einsten (1997;1998)	0,40	0,50
Meigh & Wolski (1979)	0,22	0,60

Rowe & Armitage (1987)	0,45	0,50
Poulos & Davis (1980)	0,05	1,00
Horvath (1978)	0,20	0,50

Source: Authors (2025).

From the determination of $\tau_{l,r}$, the lateral resistance of the rock fill ($Q_{l,r}$) is obtained. For the methods of Rosenberg & Journeaux (1976), Hovarth (1978) and Meigh & Wolski (1979), the pile's load capacity is composed only of the lateral contribution of the rock mass embedment.

The Cabral & Antunes (2000) method considers that $\tau_{l,r}$ is equivalent to 3.5% of the pile's unit point resistance ($q_{p,r}$).

With regard to the empirical methods that refer to the pile tip resistance, and which use Equation 2, the coefficients η and ρ in Table 8 were used.

$$Q_{p,r} = n.\sigma_u^{P} \qquad \text{Equation 1}$$

Table 8 – Coefficients Used in Empirical Methods for Determining Unitary Tip Resistance.

Methods:	**Coefficients:**	
	η	**ρ**
Poulos& Davis (1980)	0,20	1,00
Rowe & Armitage (1987)	1,00	1,00
Zhang & Einsten (1998)	3,00	0,50
Cabral & Antunes (2000)	0,30*	1,00

Source: Authors (2025).

Note: *For pile E110 the value of η was adopted as 0.1, due to the condition of the rock mass.

In order to use the empirical methods presented, some considerations were made about the uniaxial compressive strength of the rock, based on the limitations imposed by each method and the range of values for which they were devised.

In the methods of Horvath (1978), Meigh & Wolski (1979) and Zhang & Einsten (1998), lateral resistance is conditioned by the lowest value between σ_u / P_{atm} and Fck / P_{atm}. Therefore, as in piles E25, E100 and E110 there is σ_u > fck, it was established that the correlations proposed by each author would use the strength of the concrete and not the σu values of the rock matrix.

The correlation proposed by Rowe & Armitage (1987) was established for soft rock with $\sigma_u \leq 30$ MPa. An analysis will therefore be made of the applicability of the method based on the characteristics found for the rock mass of Petroquímica Suape. For this purpose, the value of the compressive strength of the rock matrix for piles E25, E100 and E110 will be adopted as $\sigma_u = 30$ MPa.

The same analysis will be carried out using the Rosenberg & Journeaux (1976) method, but the uniaxial compressive strength will be limited according to the application range of the method.

Thus, for $5 \leq \sigma_u / P_{atm} \leq 340$ MPa and $P_{atm} = 0.1$ MPa, the compressive strength of piles E25, E100 and E110 will be adopted as $\sigma_u = 34$ MPa..

With regard to the Cabral & Antunes (2000) and Poulos & Davis (1980) methods, the uniaxial compressive strength values defined for each sector were maintained. The σu and Fck values used in the application of each method are shown in Table 9.

Table 9 – Values Used for Uniaxial Compression of Rock According to Each Method.

Methods:	**σ_u / Fck (MPa)**		
	E25	**E100**	**E110**
Rosenberg & Journeaux (1976)	34	34	34
Horvath (1978)	20	20	20
Meigh & Wolski (1979)	20	20	20
Poulos& Davis (1980)	101,3	101,3	54
Rowe & Armitage (1987)	30	30	30

Zhang & Einsten (1998)	20	20	20
Cabral & Antunes (2000)	101,3	101,3	54

Source: Authors (2025).

The España (2011) method uses coefficients relating to the type of rock, degree of alteration and spacing of discontinuities to determine the admissible pressure (Padm). The coefficients adopted, which best represent the conditions of the rock mass, are shown in Table 10.

Table 10 – Coefficients Used in the España Method (2011) for Determining Allowable Pressure.

Coefficients:	**E25**	**E100**	**E110**
Po (MPa)	1,00	1,00	1,00
α_1	0,80	0,80	0,80
α_2	0,50	0,50	0,50
α_3	0,64	0,64	0,59

Source: Authors (2025).

As shown, the España (2011) method considers that τl,r is equivalent to 10% of qp,r. Based on the considerations adopted and the use of empirical and semi-empirical methodologies, the load capacity of the section embedded in the rock mass was obtained for piles E25, E10 and E110, as shown in Table 11.

Table 11 – Pile Load Capacity.

Method:	**Pile E25**			**Pile E100**			**Pile E110**		
	$Q_{l.r}$ (kN)	**$Q_{p.r}$ (kN)**	**Q_{ult} (kN)**	**$Q_{l.r}$ (kN)**	**$Q_{p.r}$ (kN)**	**Q_{ult} (kN)**	**$Q_{l.r}$ (kN)**	**$Q_{p.r}$ (kN)**	**Q_{ult} (kN)**
Rosenberg & Journeaux(1976)	7063,17	-	**7063,17**	2782,46	-	**2782,46**	5280,01	-	**5280,01**
Horvath (1978)	2826,76	-	**2826,76**	1113,57	-	**1113,57**	2113,12	-	**2113,12**
Meigh & Wolski (1979)	4195,50	-	**4195,50**	1652,77	-	**1652,77**	3136,31	-	**3136,31**
Poulos & Davis (1980)	4203,35	1479,48	**5682,82**	1655,86	1479,48	**3135,34**	3142,17	350,94	**3493,12**
Rowe & Armitage (1987)	7789,63	2190,74	**9980,36**	3068,64	2190,74	**5259,38**	5823,06	1224,22	**7047,29**
Zhang & Einsten (1997;1998)	5653,51	979,73	**6633,24**	2227,14	979,73	**3206,87**	4226,23	547,49	**4773,72**
Cabral & Antunes (2000)	884,91	584,20	**1469,11**	348,60	584,20	**932,80**	446,52	220,36	**666,88**
Espana (2011)	1636,35	756,19	**2392,54**	644,62	756,19	**1400,81**	815,78	281,82	**1097,60**

Source: Authors (2025).

Based on the results shown in Table 11, there is considerable variation between the load capacity estimates obtained by applying each method. The methods of Rosenberg and Journeaux (1976), Rowe & Armitage (1987) and Zhang & Einsten (1998) presented very high estimates in relation to the other methods, for all piles.

Juvêncio (2015) compared the average lateral resistance of the embedment, mobilized in 30 field tests, with the estimate proposed by some of these methods. As a result, he found that most of the methods used in his work presented overestimated values for the unit lateral resistance (τl,r), in relation to the measured values.

Analyzing the data together, it can be seen that pile E25 has the highest estimates for the lateral resistance of the embedment, which is due to its lateral area and the reasonable condition of the rock mass. In the case of pile E100, although it is inserted in the same rock mass as pile E25, it has a 60% smaller lateral area.

With regard to pile E110, its lateral area is only 26% smaller than pile E25, but it is inserted in a massif with less favorable conditions. Therefore, as the Cabral & Antunes (2000) and España (2011) methods take into account not only the rock's compressive strength, but also other parameters of the rock mass, the estimates for the load capacity of pile E110 were much lower than the other methods applied.

The Cabral & Antunes (2000) method provided the most conservative estimates, while the Rowe and Armitage (1987) method once again showed much higher values than the other methods.

According to Williams & Pells (1981) and Carter & Kulhawy (1988) apud Juvêncio (2015), the ratio between the tip load and the load applied to the top of the pile is 10% to 20%.

However, the pile tip resistance obtained by applying the Cabral & Antunes (2000) method represents between 33% and 62% of the load capacity.

8. Presentation and Estimation of Load Capacity by Static Load Tests

As highlighted in this work, the results of three slow static load tests carried out on root piles partially embedded in the rock mass of Petroquímica Suape, E25, E100 and E110, are analyzed.

The static compressive load tests were carried out in accordance with the specifications of NBR-12131/2020 for slow loading, with the test load reaching twice the working load of the piles.

According to Coutinho (2009), a hydraulic jack with a maximum capacity of 320 tons was used to apply the vertical loads, reacting through a system consisting of piles anchored in the ground and a metal beam, as shown in Figure 18. The settlements were measured using duly calibrated dial gauges with a sensitivity of 0.01mm, installed diametrically opposite each other on the pile head. Figure 18 shows the structure of a load test carried out on the petrochemical site.

Figure 18 – Structure assembled to perform the load test at Petroquímica Suape.
Source: Adapted from COUTINHO (2008).

The piles were loaded in 10 stages, each corresponding to 20% of the pile's working load. From the readings taken in the tests, the load-relief curves (see Figures 19, 20 and 21) shown below were obtained.

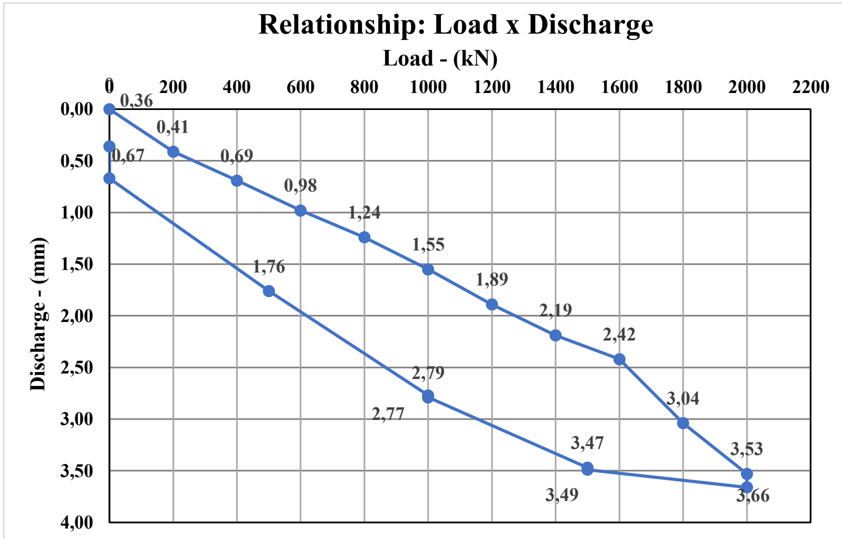


Figure 19 – Load-settlement curve of pile E25.
Source: Adapted from COUTINHO (2008).

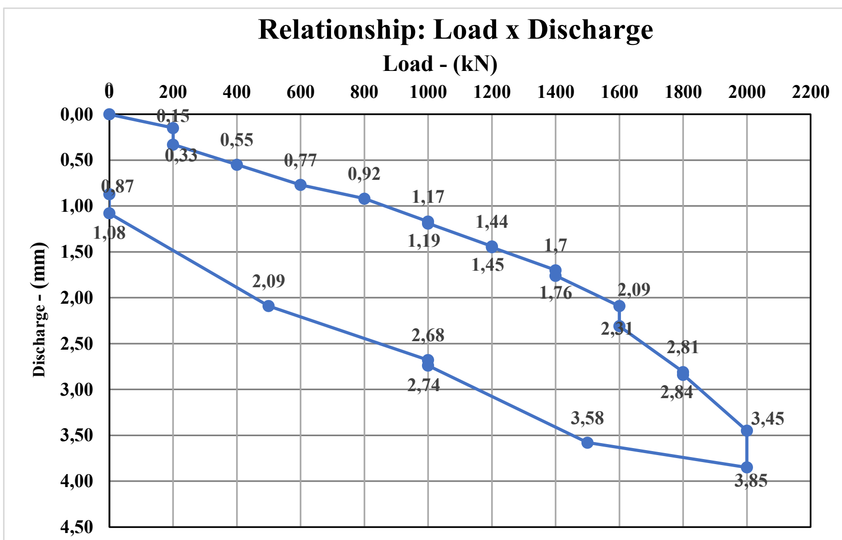


Figure 20 – Load-settlement curve of pile E100.
Source: Adapted from COUTINHO (2008).

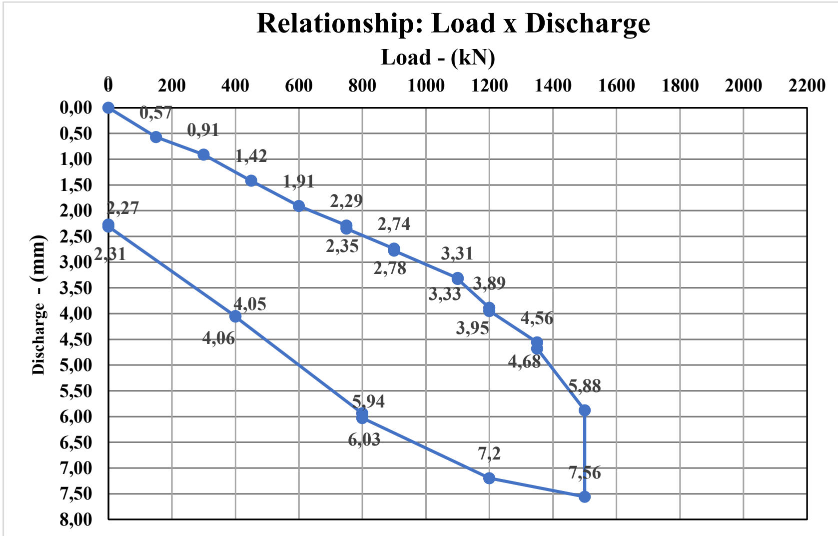


Figure 21 – Load-settlement curve of pile E110.
Source: Adapted from COUTINHO (2008).

Figure 19 shows the load-relief curve obtained for pile E25. In the static load test, pile E25 was subjected to a maximum load of 2,000 kN, reaching a maximum settlement of 3.66 mm. After unloading, a residual settlement of 0.36 mm was observed.

In the static load test carried out on pile E100, a test load of 2,000 kN was applied. The total settlement found was 3.85 mm, and after unloading the residual settlement was 0.87 mm (see Figure 20).

With regard to pile E110, the static load test reached a test load of 1,500 kN, with a total settlement of 7.56 mm and a residual settlement after unloading of 2.27 mm (Figure 21).

In order to estimate the breaking load based on the results of the static load tests, the methodologies established in the literature were used. The methodologies used and the results obtained for the breaking load for the three piles are detailed in Table 12 below.

Table 12 – Breaking Load Obtained by Load Test Interpretation Methods.

Methods:	**Piles:**		
	E25 (kN)	**E100 (kN)**	**E110 (kN)**
Terzaghi (1943)	3750	2300	1600
Van Der Veen (1953)	3750	2300	1600
Chin&Kondner (1970;1971)	3846	3125	1820
Davisson (1972)	3700	2300	1580
Décourt (1996)	**3880**	**3189**	**1830**
ABNT (2010)	3730	2300	1600

Source: Authors (2025).

Based on the estimates of the pile breaking load, using the results of the load tests, it can be seen that the methods converged to very close values.

As the load tests carried out did not reach failure, the Van Der Veen (1953) method was used to extrapolate the load-rupture curve. The method could be applied to all the load tests and showed lower breaking load values than the Chin & Kondner (1970;1971) and Décourt (1996) methods for all the piles.

With regard to the Terzaghi (1943), Davisson (1972) and ABNT (2010) methods, as they were applied to the curve extrapolated by Van Der Veen (1953), their estimates of the conventional breaking load ended up having very close values.

The estimates obtained by the Décourt (1996) method showed the highest breaking loads, the Chin & Kondner (1970; 1971) and Décourt (1996) methods reached very close values, which was to be expected, since both are based on linear adjustments based on the load/shock or settlement/load relationship.

Since stiffness is conceptually defined as the ratio between load and settlement, the graph obtained between this quantity and the settlements should correspond to a hyperbola with an asymptote at the point where the stiffness is zero.

However, because the load tests did not achieve significant displacements, the curves of the Décourt (1996) method did not show a shape where the stiffness converged to a minimum value. It was therefore not possible to use the method to separate the lateral and tip resistance.

Analyzing the results obtained by the methods of interpreting the load tests together, an average breaking load is obtained for each pile, using the methods of Van Der Veen (1953), Chin & Kondner (1970; 1971) and Décourt (1996), Table 13.

Table 13 – Average Breaking Load Obtained by Load Test Interpretation Methods.

Piles:	**Average Breaking Load - (kN):**
E25	3825
E100	2871
E110	1750

Source: Authors (2025).

9. Conclusions

Da Analysis of the load capacity for each segment of the pile partially embedded in the rock mass, using empirical and semi-empirical methods, shows that:

a) For the section of the pile in contact with the ground, the Aoki-Velloso (1975) method gave the lowest results for the lateral load capacity. On the other hand, the Cabral (1986) method provided the highest estimates, except for the pile driven into the most unfavorable subsoil, pile E110. In addition, the methods of Décourt-Quaresma (1978; 1996) and Cabral (1986) generally gave the closest results.

b) A joint analysis of the values obtained for the bearing capacity from the lateral section embedded in the rock mass shows that the methods used gave different estimates for the same pile. The methods of Rosemberg & Journeaux (1976) and Rowe & Armitage (1987) showed the most divergent results from the average found by the other methods.

c) With regard to the portion of the load capacity obtained by the tip of the pile, it can be seen that the values found were lower than the portion coming from the lateral section embedded in the rock mass. However, the pile tip resistance obtained by applying the Cabral and Antunes (2000) method represents between 33% and 62% of the load capacity. In addition, the Cabral and Antunes (2000) and España (2011) methods provided the most conservative estimates compared to the other methods.

The following conclusions were drawn from the analysis of the failure load based on the load-restraint curves:

a) The Van Der Veen (1953) method could be applied to all the load-restraint curves to extrapolate the failure load, despite the fact that the loads imposed in the load tests did not give considerable settlements.

b) The Terzaghi (1943), Davisson (1972) and ABNT (2010) methods could only be applied to the curves extrapolated from the Van Der Veen (1953) method, since the displacements found in the tests were minimal.

c) The Chin-Kondner (1970;1971) method and the Décourt (1996) method were also used to estimate the breaking load, although the curves obtained showed different behavior. Both provided higher estimates than the breaking load estimated using the Van Der Veen (1953) method, with a difference of between +3% and +30%.

d) For the loading imposed in the load test, the settlements obtained were governed mainly by the stiffness of the pile as a structural element and by the soil layer. It is therefore to be expected that the tip resistance was not developed.

With regard to the analysis of the best composition for determining the load capacity of the piles studied, it was observed that:

a) Among the methods used in this work for evaluating only the lateral resistance of the embedment in the rock mass, for calculating the load capacity of the piles, the methods of Horvath (1978) and Meigh and Wolski (1979) presented good estimates in relation to the breaking load of piles E25, E100. The other methods showed load capacity values much higher than the breaking load for all the piles.

b) The Hovarth (1978) method, when combined with the Décourt-Quaresma (1978;1996), Aoki-Velloso (1975) and Cabral (1986) methods, resulted in load capacity values reasonably close to the breaking load of the E25, E100 and E110 piles, with percentage differences of -32% to + 26%.

c) The load-bearing capacity obtained by applying the España (2011) method and taking into account the lateral resistance of the soil section, gave results with a percentage difference in relation to the breaking load of -7 to -16% for piles E25 and E100. With regard to pile E110, this composition resulted in more conservative values, with -31% to -21% in relation to the breaking load.

d) By associating the Cabral and Antunes (2000) method with the Décourt-Quaresma (1970;1971), Aoki-Velloso (1975) and Cabral (1986) methods, it was found that in all the piles, the load capacity values were much lower than the breaking load of the piles.

e) The methods of Cabral and Antunes (2000) and España (2011) showed the best results when the contribution of lateral resistance from the soil section is taken into account.

f) The application of methods that present estimates for the load capacity of piles based on correlations with the rock's compressive strength should be used with caution, since the conditions of alteration and fracturing of the rock mass have a considerable influence on the load borne by the piles. An example of this is pile E110, whose load capacity estimates were much higher than the breaking load resulting from the interpretation of the load test.

g) The Rosenberg and Journeaux (1976), Rowe and Armitage (1987) and Zhang and Einsten (1998) methods did not provide reasonable estimates for the piles studied, compared to the other methods. One possible explanation is that most of these methods were developed for sedimentary soft rock, which differs from the conditions of the rock mass present in this study.23.

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