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Review Paper

Soil-Structure Interaction and Pile Response Under Axial and Lateral Loads: A Review

Cheikh Ibrahima TINE[1], Oustasse Abdoulaye SALL[1]

[1]Department of Civil Engineering, UFR SI- Iba Der THIAM University of Thies, Thiès, SENEGAL

Corresponding Author: Cheikh Ibrahima Tine

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ABSTRACT

This article explores in depth study of the behavior of piles used as deep foundations and highlights the importance of considering soil-pile interaction when the pile is subjected to axial or lateral loads. This literature review provides a summary of knowledge on pile characterization, with an emphasis on the phenomenon of soil-pile interaction. It emphasizes the importance of accurate modeling of the soil-pile interface to predict displacements, settlements, and deflections, which are essential to the stability and durability of structures. The methods presented include classical theoretical approaches, in situ loading tests, and advanced numerical techniques, including finite element modeling. Reference standards and models, such as NF P 94-262, the Frank and Zhao (1982) model for axial settlement, and the Winkler (1867) method for lateral behavior, are discussed. The methodologies presented for calculating pile displacement and the results obtained demonstrate the necessity and importance of accurately characterizing soils through in situ tests, which are more representative than laboratory tests. The article highlights the crucial role of soil type and stiffness, pile type, and the presence of loads (axial or lateral) on pile response. Finally, it proposes perspectives such as the use of numerical calculation for a more in-depth study simultaneously integrating soil and pile parameters in order to optimize the dimensioning and design of deep foundations.

Keywords: Soil-pile interaction, Deep foundations, Pile behavior, Settlement, Deflection, Axial and Lateral loads, Numerical modeling, Finite elements.

INTRODUCTION

The study of the interaction between soil and foundations, particularly in the case of deep foundations, is crucial in geotechnical engineering. A thorough understanding of this phenomenon is essential in order to predict the mechanical behavior and deformation of piles under various load conditions. In order to study the behavior of a pile foundation, it is essential to take into account the characterization of from soil-structure interaction, it is essential to consider the characterization of the soil-pile interface is necessary for precise knowledge of the modules that characterize its deformability and stress paths. This knowledge should facilitate the optimization of structural and geotechnical design. The work of Selvadurai (1979) and Bouafia (2018) provides theoretical and practical foundations for analyzing this relatively complex phenomenon and constitutes the basic references for this article. According to Selvadurai, modeling soil-pile interaction is fundamental to determining the displacement of a pile in a soil mass. He provides a detailed analysis of the soil-foundation interaction problem, outlining the various approaches that have been proposed. Traditionally, it has been assumed that pile deformations are negligible because they transmit loads to a resistant soil. However, it is important to emphasize the need for a comprehensive analysis of soil/pile interaction, including the calculation of deformations such as settlement and deflection under loads and moments, which is essential to ensure the reliability, stability, and durability of structures on deep foundations. Numerical methods allow for accurate calculations, but approximations are often used to analyze displacements separately. In addition to theoretical modeling, deformations can be measured by full-scale loading tests, although their high cost limits their practical use. This article presents the methods commonly used to estimate pile settlement and deflection under vertical, horizontal, and/or bending moments.

1. GENERAL INFORMATION ON SOIL-FOUNDATION INTERACTION

The interaction between media in contact is a major topic in engineering because of its essential role in many applications. Historically focused on elastic analysis, research has gradually expanded to include nonlinear, inelastic, and time-dependent behaviors. A comprehensive study of an interaction problem requires the determination of stresses, strains, and the distribution of displacements in the contact areas. There are three main categories of interaction.

- The first concerns contact between elastic bodies: surfaces may be smooth or rough, and the contact area may change or remain fixed, which complicates the solution.
- The second category deals with the interaction between an elastic medium and a rigid body, where the geometry of the contact area is known and constant. When the contact varies, these problems fall into the first category.
- The third category concerns the interaction between elastic bodies and structural elements, considered as a special case of contact between elastic media. In this type of problem, one of the media is modeled using a structural element such as a beam, plate, or shell. This work focuses on this class, which deals in particular with interactions between beams or plates resting on linear elastic media. These models are particularly important in soil mechanics and foundation engineering, as they are used to analyze and design various foundation systems. The solutions derived from these analyses provide an essential basis for modeling and optimizing the behavior of structures resting on deformable soil.

1.1. Idealized soil behavior

The mechanical response of natural soils can be influenced by various factors. These include the shape, size, and mechanical properties of soil particles, the configuration of the soil structure, intergranular stresses, stress history, and the presence of moisture in the soil, the degree of saturation, and the permeability of the soil (Selvadurai, 1979).

These factors generally contribute to stress-strain phenomena that exhibit distinctly nonlinear, irreversible, and time-dependent characteristics, and to soil masses that display anisotropic and non-homogeneous material properties. Thus, any attempt to solve a soil-foundation interaction problem while taking all these material characteristics into account is clearly a daunting task.

To obtain meaningful and reliable information for practical soil-foundation interaction problems, it becomes necessary to idealize the behavior of the soil by taking into account certain specific aspects of its behavior. The simplest type of idealized soil response assumes linear elastic behavior of the soil support medium. It is recognized that the assumptions of linearity and reversibility of deformations implicit in linear elastic behavior are not strictly valid in practice (Selvadurai, 1979). However, considering this ideal response of linearly deformable soil greatly simplifies the analytical treatment of

the problem and provides useful solutions of considerable practical importance (Selvadurai, 1979).
Winkler's original method (1867) is probably the simplest idealization of soil behavior. In this method, the soil's response to the applied normal load is represented in terms of independent spring elements (Figure 1). The springs are capable of resisting only compressive forces, and there is no mechanical continuity between adjacent springs. The solution to the contact problem for Winkler's infinite plate is obtained using static equilibrium equations for a differential unit of the plate. Winkler's simplified approach does not take into account the mechanical interactions between the spring elements. Nevertheless, it qualitatively captures certain aspects of the actual behavior of the soil and has been successfully applied to solve many practical engineering problems such as beam foundations on elastic bases (Hetenyi, 1946).
Attempts to describe the linear elastic response of soil in terms of a more rigorous and consistent theory of elasticity have been made by Reissner (1958) and Vlazov and Leont'ev (1966). In these developments, the soil is represented by a homogeneous and isotropic continuum whose stress-strain response is characterized by the linear elastic properties of the soil. Although the idealization as a linear elastic medium represents a considerable improvement over the Winkler model, it is mathematically complicated and generally does not allow a direct solution of the soil-foundation interaction problem without the introduction of appropriate simplifications. Analytical approximations that facilitate the mathematical treatment of soil-foundation interaction solutions based on this approach are discussed further in the following paragraphs.
An attempt to link the Winkler model to a soil model based on a linear elasticity approach, but offering a greater degree of mathematical sophistication, was developed by Filonenko-Borodich (1945) and Pasternak (1954). Developments based on this approach led to a two-parameter soil model represented by independent springs with elastic shear elements. The behavior of this type of soil model is determined by two linear elastic constants that can be evaluated in relation to the mechanical properties of the actual soil. Attempts to use a two-parameter formulation based on the principles of linear elasticity and apply it to soil-foundation interaction problems have been reported by Vlazov and Leont'ev (1966), Kerr (1964), Kogut (1972), and Meyerhof (1967, 1974).

1.2. Foundation behavior

Foundations or structural elements supported by soil can be approximated as consisting of idealized beams, plates, and shells resting on a linearly elastic deformable soil (Selvadurai, 1979). The behavior of beams is generally described by the classical beam theories of Bernoulli-Euler and Timoshenko, while that of plates is described by the classical plate theories of Poisson-Kirchhoff. For flexible foundations resting on soil, the effects of localized loads often require modifications to the classical theories. For example, Reissner's thick plate theories (1950, 1958) are used to analyze plate foundations subjected to localized loads or when the thickness-to-length ratio of the plate is high. Reissner-Mindlin thick shell theories are applied in situations where the thickness of the shell foundation cannot be neglected. Attempts to model the behavior of flexible foundations resting on soil using more sophisticated and realistic models of soil behavior have been reported by Reissner (1958), Vlazov and Leont'ev (1966), Kerr (1964), and Vesic (1961, 1963).

1.3. Interface behavior

Defining the conditions at soil-foundation interfaces is a complex aspect of the soil-foundation interaction problem. Interface behavior can vary from a completely smooth surface to a fully adherent surface with significant friction and adhesion. Experimental studies show that interface behavior depends on the type of soil, the type of foundation material, the loading conditions, and the deformation history (Selvadurai, 1979). The boundary conditions at the soil-foundation interface are therefore ideally determined from experimental studies specific to the problem under consideration (Selvadurai, 1979). The mathematical formulation of these boundary conditions is also complex, as it must take into account mechanical interactions and the distribution of stresses and deformations at the interface. Attempts to model the boundary conditions at the soil-foundation interface have been reported by Vesic (1961, 1963), Meyerhof (1967), Wriggers (1976), Herrmann (1978), de Pater and Kalker (1975), and many other researchers.

1.4. Analytical techniques

The solution to soil-foundation interaction problems generally relies on determining the distribution of contact stresses at the soil-foundation interface and evaluating the deformations, moments, and stresses in the foundations and supporting media. Several analytical and numerical approaches can be used to address these situations. Exact analytical solutions, derived from potential theory and integral transformations, are mainly applicable to simple geometries and serve as references for validating other methods. Superposition techniques offer approximate solutions by combining elementary solutions, while approximate analytical methods, such as asymptotic series and perturbations, can be used to address more complex configurations. Variational methods, based on minimizing potential energy, also provide useful solutions for non-trivial systems. Finally, numerical methods such as finite element methods (FEM), boundary element methods (BEM), and finite difference methods (FDM) are the most versatile for modeling realistic geometries and nonlinear behaviors. Together, these techniques form an essential basis for modern analysis of soil-foundation interactions in geotechnical engineering.

2. IDEALIZED SOIL RESPONSE MODELS FOR SOIL-FOUNDATION INTERACTION ANALYSIS

The models used to analyze soil-foundation interaction are mainly elastic or elastoplastic models, which may or may not take into account the effect of time (Selvadurai, 1979). In this review, we will focus on the three main models used to calculate foundation displacements. These models assume that the soil reaction is a linear function of the displacements of the interface layer between the soil and the foundation. The response of each model is reflected in the settlement of the soil surface under the effect of a system of external loads. This settlement generally corresponds to the displacement of the soil-foundation interface layer and is essential information for the analysis of soil-foundation interaction.

2.1. Winkler model

The soil modeling proposed by Winkler (1867) assumes that the soil reaction pressure q, at any point with coordinates (x, y) in the interface layer, is directly proportional to the settlement w of the soil at that point and is independent of settlements at other points (Figure 1.a):

$$q(x, y) = k_s . w(x, y) \quad [1]$$

where k_s represents the soil reaction coefficient. An equivalent quantity commonly used for piles is the lateral reaction modulus, or horizontal modulus, denoted E_s , such that:

$$E_s = k_s D \quad [2]$$

where D is the diameter of the pile or the width of the foundation.

This assumption was used in the work of Mandel (1936), Koronev (1960), Hetenyi (1966), etc. Physically, the Winkler model consists of treating the soil as a system of infinitely close elastic springs, independent of each other and having a constant stiffness k_s . Soil settlement occurs in the area below the loading surface, and outside these areas there is no settlement. Furthermore, the settlement of the loaded area in the case of a rigid foundation (Figure 1.c) remains the same in the case of a flexible foundation (Figure 1.d).

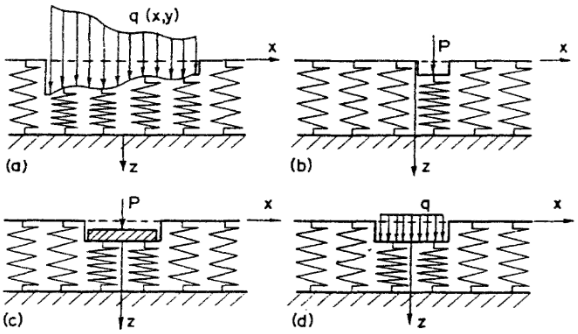


Figure 1. - Soil displacement for the Winkler model, (a) any load (b) concentrated load, (c) rigid foundation, (d) flexible foundation (Selvadurai, 1979)

2.2. Elastic continuum model

In the previous paragraph, it was stated that, according to the Winkler model, only the soil directly beneath the loading surface undergoes settlement. However, in reality, settlement is not limited to this area and also extends to the lateral areas. By modeling the soil as a semi-infinite, elastic, and continuous medium characterized by two parameters, Young's modulus E and Poisson's ratio ν**,** it is possible to take into account the continuity of settlements. The first work in this field was carried out by Boussinesq (1885), who studied the problem of a semi-infinite homogeneous isotropic linear elastic medium subjected to a concentrated vertical load P (Figure 2). In general, applying elastic continuum theory to solve the problem of soil-foundation interaction leads to complex mathematical problems. Solutions to these problems have been proposed in the work of Gorbunov-Pasadov (1941, 1949), Galin (1961), Lur'e (1964), Harr (1966), Popov (1971), and other researchers.

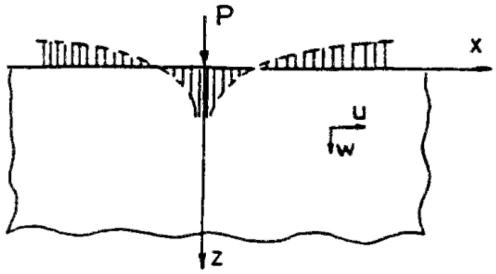


Figure 2. - Soil settlement due to a concentrated load for the

2.3. Two-parameter models

The Winkler model has limitations in its ability to describe the continuous behavior of real soil, while the elastic continuum model is mathematically complex. These challenges have led to the introduction of simplifying assumptions to develop other models. The term "two-parameter" means that the model is defined by two independent elastic constants. There are two main families of two-parameter models. The first family is based on the Winkler model, but corrects its discontinuous nature by incorporating mechanical interaction between the springs. These models were developed by Filonenko-Borodich (1940, 1945), Hetenyi (1946), and Pasternak (1954), where the interaction between the springs is provided by elastic membranes, elastic beams, or elastic layers capable of tangential deformations. The second family is based on the elastic continuum model (E**,** ν**)**, introducing constraints or simplifications concerning the distribution of displacements and stresses, as proposed by Reissner (1958) and Vlazov and Leontiev (1949, 1966).

3. VERTICAL DISPLACEMENT OF A PILE UNDER AXIAL LOADING

The vertical displacement or settlement of an isolated pile under a service load is used to evaluate the displacement of the entire pile foundation, taking into account any group effect (Bouafia, 2018). Depending on the pile/soil configurations, the settlement may be small and not decisive for the design of the foundation, but in other cases it is crucial to consider it. Furthermore, in this study, it is also interesting to be able to describe the evolution of settlement according to the depth of the pile. For a pile in compression, settlement is often greater at the head, and it is this value that is used in design to verify the permissible settlement of the pile. The interest in describing the evolution of settlement is based on the need for a better understanding of the behavior of the pile from the head to the tip, taking into account the type of soil around the pile. This will also facilitate the determination of stress, force, and deformation expressions along the pile.

The settlement of an isolated pile is calculated using different methods, which fall into four main categories:

- Empirical methods
- Elasticity theory methods
- Load transfer theory methods,
- Numerical methods.

Empirical and elasticity theory methods generally define procedures for calculating settlement at the pile head. As for load transfer theory methods and numerical methods, the settlement at any point of the foundation can be determined either analytically or numerically. In the following, we will discuss in greater detail the methods based on transfer theory, which allow us to follow an analytical solution and understand the parameters that contribute to the formation of settlement t pending on depth.

3.1. Empirical methods

These methods, which provide an approximate estimate of settlement at the pile head, are based on the compilation of a number of observations on piles. Correlations between settlement at the pile head$\mathbf{s_0}$ and pile diameter**D** are proposed by Meyerhof (1956) and Frank (1995). Vesic (1977) proposes in his formula to take into account the elastic shortening of the pile$\Delta\mathbf{L}$. The relationships proposed by these authors are shown in Table 1. These methods are of limited use in the preliminary phase of a foundation project and must be followed, at an advanced stage of the project, by a more rigorous analysis of pile displacement.

Table 1 . Empirical formulas for the settlement of an isolated pile according to various authors

Authors	Relationships
Meyerhof (1956)	$s_0 = D/(30F_s)$ F_s where the safety factor is generally taken to be equal to 3
Frank (1995)	$s_0/D = 0,6$ % for bored piles $s_0/D = 0,9$ % for driven piles
Vesic (1977)	$s_0 = D/100 + \Delta L$ $\Delta L = N_0 L/E_p A_p$

3.2. Methods of elasticity theory

In this model, the pile/soil system is assumed to have isotropic elastic behavior (Figure 3). Authors such as Poulos (1968), Randolph (1978), and Banerjee and Butterfield (1978) have presented their methods for calculating pile head settlement using this method, based on Mindlin's (1936) fundamental solution to the problem of a vertical force buried in a semi-infinite elastic massif. The settlement at the pile head s_0 under the action of a concentrated force at the headN_0 is generally given by:

$$s_0 = \frac{N_0 . I_v}{E(L).D} \quad [3]$$

I_v, called the settlement factor, depends on the relative compressibility of the pile/soil, i.e.

$K = E_p/E$, the slenderness ratio L/D, and Poisson's ratioν .

Randolph and Wroth (1978) presented an analytical formulation of the settlement factor, valid both for homogeneous soil (**E** constant with depth) and for Gibson soil (linear variation of modulus **E** with depth):

$$I_v = 4(1+\nu)\frac{1+\frac{8}{\pi\xi\lambda(1-\nu)}\frac{L}{D}\frac{tanh(\mu L)}{\mu L}}{\frac{4}{(1-\nu)\xi}+\frac{4\pi\beta}{\alpha}\frac{L}{D}\frac{tanh(\mu L)}{\mu L}} \quad [4]$$

where:

$$\beta = \frac{E(L/2)}{E(L)} \quad [5]$$

$$\lambda = 2(1+\nu)\frac{E_p}{E(L)} \quad [6]$$

$$\alpha = ln\left[2\frac{L}{D}(0{,}25 + (2{,}5\beta(1-\nu) - 0{,}25)\xi)\right] \quad [7]$$

$$\mu L = \frac{2\sqrt{2}\frac{L}{D}}{\sqrt{\alpha\lambda}} \quad [8]$$

In the case of a semi-infinite soil (h infinite), we **take $\xi = 1$**, and in the presence of an elastic substrate, characterized by a modulus $\mathbf{E_b}$, the factor ξ is calculated as follows:

$$\xi = \frac{E(L)}{E_b} \quad [9]$$

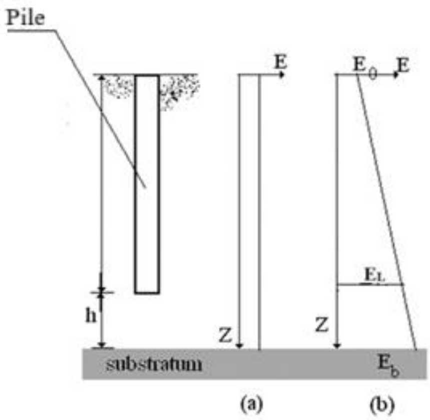


Figure 3. - Diagram of the elastic model of the soil/pile system (*a: Homogeneous soil; b: Gibson soil*) (Selvadurai, 1979)

3.3. Methods using t-z and q-z curves or load transfer theory

- **Principle of transfer curves**

The **t-z** method, also known as the transfer curve method, was first proposed by Coyle and Reese in 1966. Its purpose is to calculate the vertical displacement of a pile subjected to axial loading. This method is based on the definition of curves that relate the shear stress on the lateral surface of the pile (**τ**) to the vertical displacement of a pile section (s) at different depths. These curves are also called **t-z** curves or transfer curves. The construction of these **t-z** curves is based on data collected during in-situ instrumented pile loading tests, laboratory tests on pile models, or in-situ tests such as CPT or PMT. The first **t-z** curves were developed by Coyle and Reese in 1966. A typical **t-z** curve is shown in Figure 4.

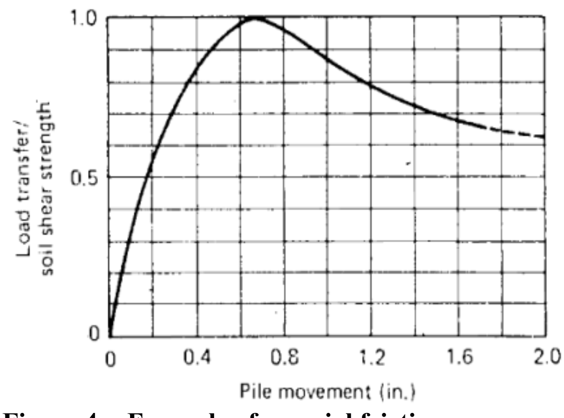


Figure 4. - Example of an axial friction mobilization curve (Coyle and Reese, 1966)

The **t-z** method can be implemented in different ways. For example, it can be based on discretizing the pile into a series of segments. Figure 5 shows that the pile has been discretized into three segments for simplicity. The method is based on solving the equilibrium equation (10) for a section of pile under compression.

$$E_P A_P \frac{d^2 s}{dz^2} - \pi D \tau(s) = 0 \qquad [10]$$

The main steps of a numerical solution method for the t-z method used by Poulos and Davis in 1980 can be summarized as follows:

1. The process begins by imposing a small tip displacement $\mathbf{s_p}$ on segment 3 (Figure 5). The tip resistance is then approximately calculated using, for example, Boussinesq's theory (1885), considering the base of the pile as a rigid, circular section.
2. At mid-height of segment 3, a displacement $\mathbf{s_3}$ is arbitrarily considered (for example, $\mathbf{s_3}$ is taken to be equal to $\mathbf{s_p}$). Based on the **t-z** curve corresponding to the depth of segment **3**, the shear stress $\boldsymbol{\tau_3}$ around the pile section is determined. The equilibrium of forces applied to element 3 is then used to calculate the head force N_3.
3. Assuming that the load within the small segment 3 varies linearly and taking into account the Young's modulus $\mathbf{E_p}$ of the pile, the elastic deformation $\mathbf{\Delta s_3'}$ at mid-height of the segment is calculated. The sum of $\mathbf{\Delta s_3'}$ and s_p gives the new displacement value s_3' at mid-height of segment 3.
4. The two values $\mathbf{s_3'}$ and s_3 are compared. If $\mathbf{s_3'}$ does not coincide with s_3 within a tolerance, a new value of $\mathbf{s_p}$ is imposed and the calculation is repeated. If the tolerance is met, the calculation moves on to the second pile segment until the head load N_0 and head displacement $s_{(0)}$ are obtained. This process is repeated for different peak displacement values in order to obtain a series of values for N_0 and s_0. These values are then used to plot the load/settlement curve.

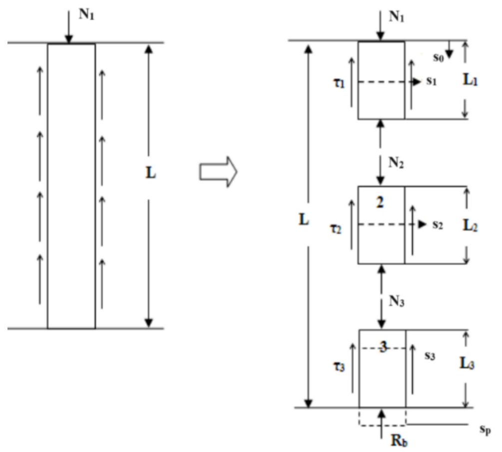


Figure 5. - Principle of the transfer curve method (Coyle & Reese, 1966)

- **Load transfer parameters R_0 and B_0**

The bearing capacity of a pile subjected to a vertical load is the sum of the tip resistance q_p and the lateral frictionτ at the soil/pile interface. For small settlements, the tip pressure and shear stress are defined by two parameters, R_0 and B_0,which are used in load transfer theory to evaluate pile settlements. It is assumed that the stress mobilized at the soil/pile interface at a given depth is proportional to the settlement at that same depth:

$$\tau(z) = B_0(z)s(z) \quad [11]$$

$$q_p = R_0 \frac{s(L)}{D} \quad [12]$$

$\boldsymbol{R_0}$ is the tip deformation parameter, representing the initial slope of the q_p curve as a function of $s(L)/D$ (Figure 6). This parameter can also be given directly in relation to $s(L)$ by setting the parameter $k_q = R_0/D$.

$\boldsymbol{B_0}$, also denoted $\boldsymbol{k_\tau}$**,** is the lateral friction mobilization parameter and therefore represents the initial slope of the lateral friction curve as a function of settlement (Figure 7).

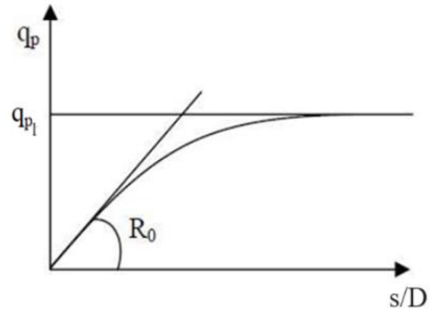


Figure 6. - Representation of the q_p – s(L)/D curve (Bouafia, 2018)

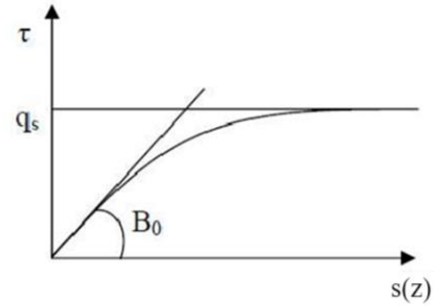


Figure 7. - Representation of the curve τ - s(z) (Bouafia, 2018)

In the case of a multilayer soil or a non-homogeneous monolayer soil, where the$B_0(z)$ profile is arbitrary, $B_0(z)$the soil is broken down into a set of sufficiently thin slices such that it can be assumed that the stress is practically constant in a given segment of the pile*,* and equation (10) can be integrated either by *the finite difference method* or by finding the analytical solution to this equation and imposing continuity at the interfaces of the slices. The latter procedure has been the basis for several computer programs such as PIVER and SETPIL (Bouafia, 2018).

- **Examples of monotonic t-z curves**

Since 1960, numerous monotonic **t-z** curves have been developed. The first of these curves was proposed by Coyle and Reese in 1966. In 2015, Bohn conducted a comparative study of eight monotonic t-z curves, examining the initial stiffnesses, soil type, and pile type for each curve. The results of this analysis are presented in Tables 2 and 3.

Table 2 . - Examples of monotonic t-z curves (Bohn, 2015)

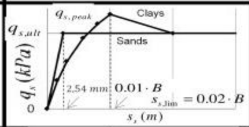
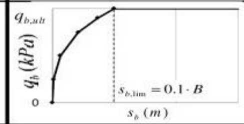
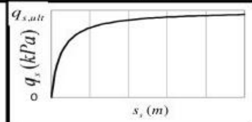
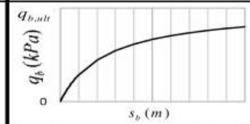
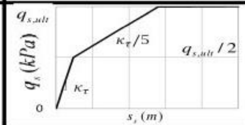
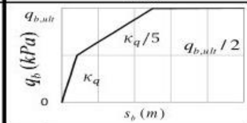
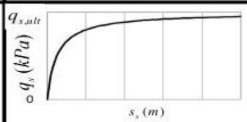
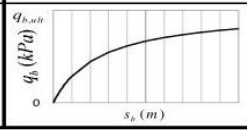
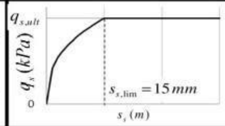
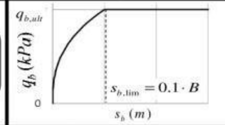
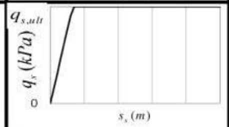
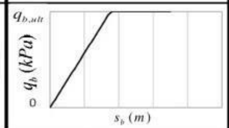
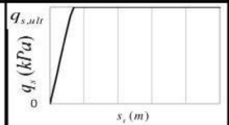
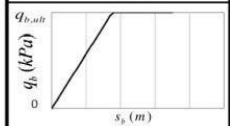
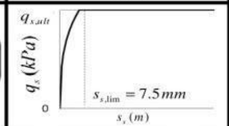
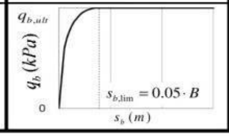
		Mathematical expression	Curve shape	Deformation parameter	Initial slope	Ground type	Pile type	Development based on pile load tests
API (1993): point by point curves	**Shaft**	Clay: s_s/D – $q_s/q_{s,ult}$ 0,0016 – 0,30 0,0031 – 0,50 0,0057 – 0,75 0,0080 – 0,90 0,0100 – 1,00 0,0200 – 0,70 to 0,90 ∞ – 0,70 to 0,90 Sand: s_s (mm) – $q_s/q_{s,ult}$ 0,00 – 0,00 2,54 – 1,00 ∞ – 1,00		Limit settlement $s_{s,lim}$: 0.02·B for clay, 0.00254 m fixed for sand	Clay: $\frac{q_{s,ult}}{0.0037\cdot B}$ Sand: $\frac{q_{s,ult}}{0.00254}$	Clay Non-carbonate sand	All	No information
	Tip	s_b/B – $q_b/q_{b,ult}$ 0,002 – 0,25 0,013 – 0,50 0,042 – 0,75 0,073 – 0,90 0,100 – 1,00		Limit settlement $s_{b,lim}$: 0.1·B	$\frac{q_{b,ult}}{0.008\cdot B}$			
Fleming (1992): hyperbolic curves	**Shaft**	$q_s=\frac{q_{s,ult}\cdot s_s}{M\cdot B+s_s}$		Parameter M (non explicit): 0.0005 for stiff soils to 0.004 for soft soils	$\frac{q_{s,ult}}{M\cdot B}$	All	All	4 tests
	Tip	$q_b=\frac{q_{b,ult}\cdot s_b}{0.6\cdot\pi\cdot\frac{B}{4\cdot E}\cdot q_{b,ult}+s_b}$		Young's modulus E (non explicit)	$\frac{4\cdot E}{0.6\cdot\pi\cdot B}$			
Frank & Zhao (1982) and Frank (1985): trilinear curves	**Shaft**	Fine-grained: $\kappa_\tau=\frac{2.0\cdot E_M}{B}$ Coarse-grained: $\kappa_\tau=\frac{0.8\cdot E_M}{B}$		Pressuremeter modulus E_M	Fine-grained: $\frac{2.0\cdot E_M}{B}$ Coarse-grained: $\frac{0.8\cdot E_M}{B}$	Fine-grained soil Coarse-grained soil	All	Approx. 30 tests
	Tip	Fine-grained: $\kappa_q=\frac{11.0\cdot E_M}{B}$ Coarse-grained: $\kappa_q=\frac{4.8\cdot E_M}{B}$		Pressuremeter modulus E_M	Fine-grained: $\frac{11\cdot E_M}{B}$ Coarse-grained: $\frac{4.8\cdot E_M}{B}$			
Hirayama (1990): hyperbolic curves	**Shaft**	$q_s=\frac{q_{s,ult}\cdot s_s}{0.0025\cdot B+s_s}$		Fixed parameter 0.0025	$\frac{q_{s,ult}}{0.0025\cdot B}$	Clay Non-carbonate sand	Large diameter bored piles	Several tests at one site
	Tip	$q_b=\frac{q_{b,ult}\cdot s_b}{0.25\cdot B+s_b}$		Fixed parameter 0.25	$\frac{q_{b,ult}}{0.25\cdot B}$			

Table 3 . - Examples of monotonic t-z curves (bis) (Bohn, 2015)

		Mathematical expression	Curve shape	Deformation parameter	Initial slope	Ground type	Pile type	Development based on pile load tests
Krasiński (2012): root curves	**Shaft**	$q_s = \min\left(q_{s,ult}\left(\frac{s_s}{s_{s,\lim}}\right)^{0.38}; q_{s,ult}\right)$		Limit settlement $s_{s,\lim}$: 0.015 m fixed	∞	Sand	Screw displacement piles	Approx. 10 tests
	Tip	$q_b = \min\left(q_{b,ult}\left(\frac{s_b}{s_{b,\lim}}\right)^{0.38}; q_{b,ult}\right)$		Limit settlement $s_{b,\lim}$: 0.1·B	∞			
Randolph & Wroth (1978): linear curves	**Shaft**	$q_s = \min\left(\frac{G}{B/2 \cdot \ln\left(\frac{r_m}{B/2}\right)} \cdot s_s; q_{s,ult}\right)$		Shear modulus G (non explicit)	$\frac{G}{B/2 \cdot \ln\left(\frac{r_m}{B/2}\right)}$	All	All	No
	Tip	$q_b = \min\left(\frac{8 \cdot G}{\pi \cdot B \cdot (1-\nu)} \cdot s_b; q_{b,ult}\right)$		Shear modulus G (non explicit)	$\frac{8 \cdot G}{\pi \cdot B \cdot (1-\nu)}$			
Verbrugge (1981): linear curves	**Shaft**	$q_s = \min\left(0.22 \cdot \frac{a \cdot b \cdot E}{B} \cdot s_s; q_{s,ult}\right)$ with $E = 3600 + 2.2 \cdot q_c$; $a = \begin{cases}1 \text{ for bored piles} \\ 3 \text{ for driven piles}\end{cases}$; $b = \begin{cases}1 \text{ for normally consolidated soils} \\ 2 \text{ for overconsolidated soils}\end{cases}$		Young's modulus E from CPT correlations	$0.22 \cdot \frac{a \cdot b \cdot E}{B}$	Fine-grained soil Coarse-grained soil	Replacement piles Driven piles	Yes (no details)
	Tip	$q_b = \min\left(\frac{a \cdot b \cdot E}{0.32 \cdot B} \cdot s_b; q_{b,ult}\right)$		Young's modulus E from CPT correlations	$\frac{a \cdot b \cdot E}{0.32 \cdot B}$			
Vijayvergiya (1977): root curves	**Shaft**	$q_s = \min\left(\left(2 \cdot \sqrt{\frac{s_s}{s_{s,\lim}}} - \frac{s_s}{s_{s,\lim}}\right) \cdot q_{s,ult}; q_{s,ult}\right)$		Limit settlement $s_{s,\lim}$: 0.0075 m fixed	∞	Sand	Driven piles	2 tests
	Tip	$q_b = \min\left(\left(\frac{s_b}{s_{b,\lim}}\right)^{1/3} \cdot q_{b,ult}; q_{b,ult}\right)$		Limit settlement $s_{b,\lim}$: 0.05·B	∞			

3.4. Numerical methods

Numerical methods have become powerful tools for modeling and solving soil/foundation interaction problems, with increasing use of the finite element method or the finite difference method. Soil/pile interaction is modeled by an axisymmetric mesh composed of plane elements, which can be studied using any general finite element or finite difference calculation program. Recent years have seen the emergence of powerful software dedicated to modeling geotechnical problems, such as **Plaxis, Crisp**, and **Flac,** as well as structural analysis software such as **Robot Structural Analysis** with its **REX Pile** extension**,** which allow the in-situ soil to be modeled to better account for soil/foundation interaction. In addition to being a powerful tool for research into pile behavior, the finite element method is commonly used in pile foundation projects with specific aspects, to take into account soil/pile interaction more realistically.

4. DEFLECTION OF A PILE UNDER LATERAL LOADING

4.1. Principles for calculating the displacement of a transversally loaded pile

The response of a pile subjected to lateral loading depends on the mechanical characteristics of the pile, the nature and behavior of the soil, and the loading law. Consequently, there are many different parameters to consider during design, including boundary and initial conditions, geometry, and the properties of the pile and soil. To effectively study a pile under lateral loading, it is necessary to:

- Have an appropriate soil behavior law
- Use a model to define the soil-pile interaction (**P-y** curves)
- Employ an adequate numerical solution technique
- Perform realistic modeling of the pile and soil geometry
- Consider the boundary loading conditions

In most cases, the design criterion is not the ultimate lateral or axial capacity of the pile, but the maximum displacements at the head. Several methods have been developed to analyze piles under lateral loads on this basis. The classical rigid-plastic theory presented in 2.9 assumes that the soil is completely in a state of failure in the bearing and counter-bearing zones. This theory allows the limit load for a pile to be determined, but does not describe its displacement behavior. Furthermore, for a group of piles, it does not take into account the presence of neighboring piles, which often leads to an overestimation of the limit load. Poulos & Davis (1980) propose various solutions for isolated piles depending on the different boundary conditions. In the case of plane stresses, the soil-pile system is considered as a rigid block, and the ultimate load of the assembly is that of a vertically buried footing.

The elastic continuum method is based on the solution of Mindlin's equations (1936), which describe the displacement induced by a point force in a semi-infinite elastic massif. The soil is modeled as a continuous elastic material. Another approach is to use the finite element method to model the soil. This method is powerful and can handle complex cases (multilayers, 3D, etc.), but requires precise knowledge of the soil's behavior laws. In addition, reproducing the initial stress state around the pile while respecting the possible symmetries of the problem can be tricky, and the calculation times are often prohibitive. Finally, the most commonly used method in practice is based on modeling the soil using a series of closely spaced springs without coupling between them, based on Winkler's theory (1867). This method is simple to use because it directly links the behavior of the soil (P reaction or p pressure**)** to the displacement y of the pile under lateral loading (Figure 8).

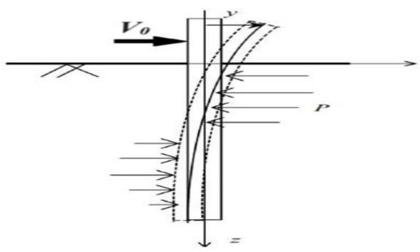


Figure 8. - Pile mobilizing the lateral reaction of the soil (Winkler, 1867)

The pile's **L** value is then compared to this transfer length. If the pile's L value is greater than $3l_0$, the pile is said to be flexible; if it is less than l_0, the pile is said to be rigid; and if **L** is between l_0 and $3l_0$, the pile is said to be semi-rigid.

$$l_0 = \sqrt[4]{\frac{(4E_pI_p)}{E_s}} \quad [13]$$

Where:

E_s is the soil reaction modulus, also known as the frontal pressure mobilization modulus,

E_p is the elastic deformation modulus of the pile,

I_p is the moment of inertia of the pile.

4.2. Methods for calculating piles under lateral loads

Several studies have been conducted to better understand the behavior of piles under lateral loads. These studies have led to the development of several approaches that can be classified into four categories:

- Reaction modulus method (Winkler, 1867) ;
- **p-y** curve method (Matlock, 1970) & (Reese.C, 1974);
- Elastic continuum method (Poulos & Davis, 1980) ;
- Numerical methods using finite elements or finite differences.

Although somewhat complex, finite element and finite difference methods are often used.

4.2.1. The reaction modulus method

- **General principle**

The Winkler method is the oldest analytical method for predicting lateral soil reaction. It consists of modeling the interaction between the soil and the pile using a series of independent springs with variable stiffness. The stiffness allows the lateral soil reaction (**p**) to be directly linked to the lateral displacement of the pile (**y**) under lateral loading (Figure 8). This method forms the basis of **p-y** curves**,** with the "springs" exhibiting non-linear behavior. The Winkler model defines the soil as a stack of independent layers. Each soil layer is modeled by a lateral spring (Figure 9) on which the pile rests.

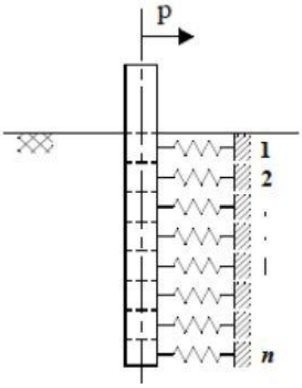


Figure 9. - Representation of the Winkler model (Winkler, 1867)

The pressure **p** on a "slice" of soil depends only on the lateral displacement of the slice and on a soil reaction coefficient, called thek_h , in the case of lateral loading.

$$\boldsymbol{p = k_h(z).y} \quad [14]$$

Where:

k_h : The horizontal soil reaction coefficient is expressed in MN/m^3 also denoted k_s.

y: Lateral displacement of the pile

This equation is also expressed as:

$$\boldsymbol{P = E_s.y} \quad [15]$$

Where:

P : Soil reaction per unit length of pile given by equation 16

E_s : Soil reaction modulus given by equation 17

D: Pile diameter

$$\boldsymbol{P = p.D} \quad [16]$$

$$E_s = k_h.D \quad [17]$$

The pile is idealized as an elastically loaded beam. The soil is modeled by horizontal springs, independent of each other, with stiffnessE_S . Thus, the behavior of the pile is governed by the equation of a beam on elastic supports:

$$\boldsymbol{E_pI_p\frac{d^4y}{dz^4} = -p.D} \quad [18]$$

The effect of axial loading on the pile is neglected. According to equations (14) and (18), the equation describing the displacement of a pile under lateral loading is:

$$\boldsymbol{E_pI_p\frac{d^4y}{dz^4} + k_h(z).D.y = 0} \quad [19]$$

The solutions to this equation can be obtained either by an analytical method or by a numerical method. The main advantage of this method is that the soil-pile interaction can be defined at any point along the pile. However, this definition is restricted by the assumption that the pressure at a point is a linear function of the displacement at that point and by its dependence on the choice of the profile of the soil k_h values.

- **Expression of the reaction modulus E_s**

Defining the reaction modulus profile is the main difficulty with the reaction modulus method. It depends on many parameters such as the stiffness of the pile, the load level, the nature of the soil, etc…Hadjadji (1993) lists most of the formulations published in the literature. He concludes that the soil's reaction modulus E_s can be determined if Young's modulus E has been obtained through laboratory tests or the pressiometric modulus E_M through in-situ tests. According to various authors, we note the following:

Terzaghi (1955)

For sands, the author proposes the following relationship:

$$\frac{E_S}{E} = \frac{1}{1{,}35} = 0{,}74 \quad [20]$$

With:

$$E = A.\gamma.z \quad [21]$$

where γ is the soil density and A is a dimensionless coefficient depending on the sand density (Table 4).

Table 4 . - Values of coefficient A according to Terzaghi (1955)

Sand Density	Loose	Medium	Dense
Value of A	100 - 300	300 - 1000	1000 - 2000

Ménard, Bourdon, and Gambin (1969)

$$\frac{E_s}{E_M} = \begin{cases} \dfrac{3}{\frac{2}{3}\left(\frac{D_0}{D}\right)\left(2{,}65.\frac{D}{D_0}\right)^{\alpha} + \frac{\alpha}{2}} & pour\ D > D_0 \\ \dfrac{18}{4.2{,}65^{\alpha} + 3\alpha} & pour\ D < D_0 \end{cases} \quad [22]$$

With:

D_0: reference diameter equal to 0.6

α : Rheological coefficient depending on soil type (Tables 5 and 6)

E_M : Soil pressure module

Table 5 . - Rheological factor α for various soil types

Degree of consolidation						
	Clay		Silt		Sand	
	$\frac{E_M}{p_l}$	α	$\frac{E_M}{p_l}$	α	$\frac{E_M}{p_l}$	α
Over consolidated	> 16	1	> 14	$\frac{2}{3}$	> 12	$\frac{1}{2}$
Normally consolidated	9 - 16	$\frac{2}{3}$	8-14	$\frac{1}{2}$	7-12	$\frac{1}{3}$
Altered and reworked or loose sub-consolidation	7-9	$\frac{1}{2}$	5 - 8	$\frac{1}{2}$	5-7	$\frac{1}{3}$

Table 6 . - Rheological factor α for rock (Fascicule 62, 1992)

Rock type	Rheological coefficient value α
Very little fracturing	2/3
Normal	1/2
Highly fractured	1/3
Very altered	2/

Poulos (1971)

$$\frac{E_s}{E} = 0{,}82 \quad [23]$$

Matlock (1970) and Reese (1974)

$$E_s = 1{,}3 \sqrt[12]{\frac{E.D^4}{E_p I_p}} \frac{E}{(1-\nu^2)} \quad [24]$$

E: soil modulus of elasticity

$E_p I_p$: pile stiffness

ν : soil Poisson's ratio.

Bowles (1977)

$$E_s = K\left(0{,}308 + 1{,}584\frac{D}{L}\right)\frac{z}{r.L} \quad [25]$$

With:

r = D + D·tanβ

L: longueur du pieu

z: la profondeur

β: angle de dispersion, il varie entre φ/4 et φ

φ: angle de frottement du sol

K: paramètre du sol donné dans le tableau suivant (Tableau 7)

Table7 . - Value of parameter K (Bowles, 1977)

Soil type	K parameter
Dense sand	200-400
Medium-dense gravel	150-300
Medium-dense sand	100-250
Fine sand	80-200
Stiff clay	60-180
Saturated stiff clay	30-100
Plastic clay	10-80
Clay	2-30

One of the disadvantages of this method is that it cannot be extended to a stratified soil medium, and the influencing factors cannot be calculated using Mindlin's equation (1936). Mindlin's equation does not apply to a non-homogeneous stratified medium. It should also be noted that the assumption that the pile is a rectangular strip embedded in the soil is only approximately valid if the pile has a square or I-shaped cross-section**.** In the case of piles with a circular cross-section, this idealization is more approximate, but still seems reasonable. Despite these imperfections, this method is often used in practice.

4.2.2. The p-y reaction curve method

The **P-y** method is a generalization of the Winkler model. It is useful for taking into account the nonlinear behavior of soils and for studying the behavior of piles under lateral loads. It is a semi-empirical method because the prediction and construction of curves for the study of an isolated pile is based on laboratory tests or in situ tests. Each soil is represented by a series of **P-y** curves**.** The soil is treated as linear or nonlinear elastic supports (commonly referred to as springs), represented by (P**,** y**)** diagrams (Figure 10.b), i.e., by relationships between the lateral reaction, P**,** and the lateral displacement, y**.** These diagrams are called **P-y** reaction curves**.**

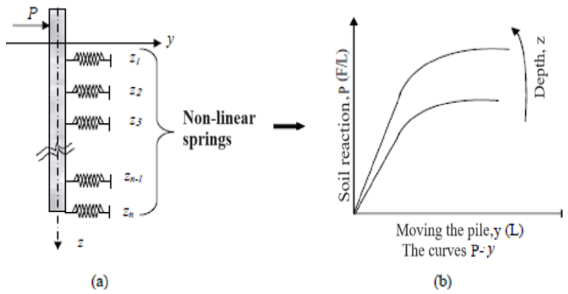


Figure 10. - (a) Winkler model in the case of lateral loading and (b) Reaction curves (p-y) (Winkler, 1867)

For a soil-pile system subjected to lateral loading, consider what happens at a section (or slice of the pile) located at a depth **z**. At rest, after installation, the section is subjected to lateral earth pressure whose resultant is zero (Figure 11.b).

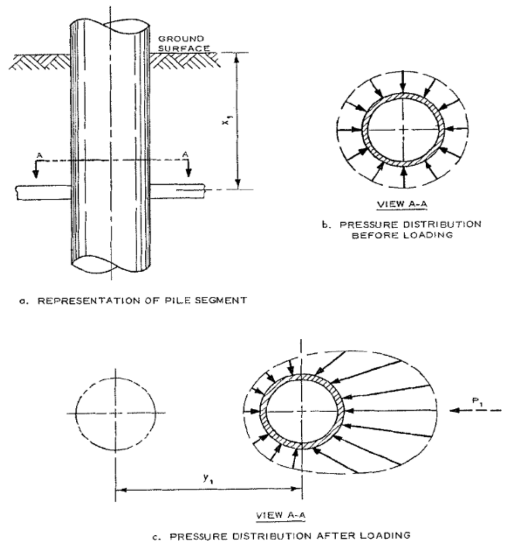


Figure 11. - Graphical definition of p and y (Reese & Sullivan, 1980)

When the pile is subjected to lateral loading, the section in question moves laterally by $\mathbf{y_i}$ and the stress state is modified so that the lateral resultant on the section in question has a direction opposite to the displacement $\mathbf{y_i}$ (Figure 11.c). Over the entire height of the pile, for a given depth, similar behavior with varying intensities can be observed. This makes it possible to study the entire pile for any load and any type of soil. Nonlinear **P-y** curves that vary with depth and soil type are obtained over the entire length of the pile (Figure 12).

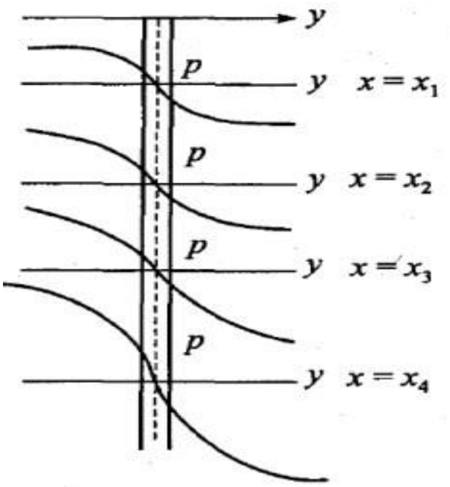


Figure 12. - Series of p-y curves for a pile (Reese & Sullivan, 1980)

Since a **p-y** curve accurately represents the behavior of a pile at a given section, and therefore for different sections of the entire pile, assuming that the sections are independent, several researchers (Ménard, Gambin, and Bourdon (1969), Dunand (1981), Briaud, Smith, and Tucker (1985), Baguelin, Jezéquel, and Shields (1978), Robertson, Hughes, and Campanella (1984)) have proposed methods for determining them in order to design piles. For the shape of the pile section, tests carried out by Roscoe (1957) show that the shape has very little influence on the distribution of lateral pressure and on the ultimate strength of the piles. What matters is the width of the section perpendicular to the direction of loading, which is in contact with the ground. These methods have been developed and refined by numerous researchers using various approaches: in situ tests, laboratory tests, physical modeling, and numerical modeling. The diversity of these approaches leads to as many **P-y** reaction curves**.** Some are recognized and adopted in pile design codes (Fascicule 62, A.I.P., P.H.R.I., etc.).

4.2.3. Linear elasticity method (elastic continuum)

Poulos (1971) used the elastic continuum approach to develop a model for analyzing laterally loaded piles. The elastic continuum method is based on the solution of Mindlin's equations (1936). Mindlin established the displacement induced by a point force in a semi-infinite elastic massif. The soil mass is considered to be a continuous elastic material. Poulos describes the pile in this mass as a vertical rectangular plate of width d, length L, and constant stiffness $\boldsymbol{E_p I_p}$. The pile is divided into **(n+1)** elements of equal length, each element being subjected to a constant horizontal stress p across its width (Figure 13). Each of these elements is subjected to a uniform lateral stress that is constant across the width of the pile. The soil is considered to be an ideal elastic material that is homogeneous, semi-infinite, and isotropic. The soil properties are considered to be unaffected by the presence of the pile, which is a reasonable assumption.

In his basic model, Poulos did not consider the interface between the pile and the soil. However, he also presented an approximate method that takes this interface into account. The basic principle of Poulos' model is Mindlin's equation for the displacement of a point in a semi-infinite continuum caused by a point load in the mass. Under elastic conditions, the horizontal displacements of the pile and the soil are compatible. So that these displacements are assimilated to the centers of the elements, except for the extreme elements (at the base and head of the pile).

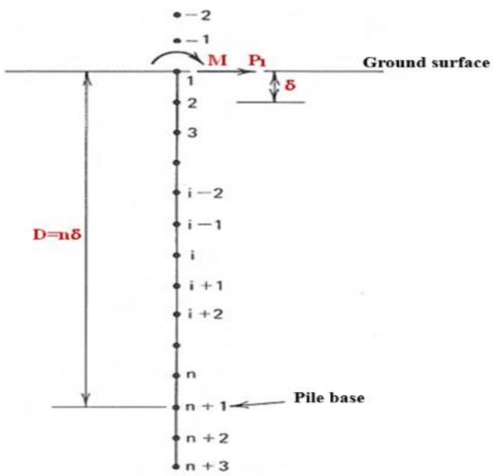


Figure 13. - Finite difference discretization for the analysis of laterally loaded piles (Poulos and Davis, 1980)

Thus, the soil displacements for all points along the length of the pile are expressed by the following equation:

$$\{\boldsymbol{P_s}\} = \frac{\boldsymbol{B}}{\boldsymbol{E_s}}[\boldsymbol{I_s}]\{\boldsymbol{p}\} \qquad \textbf{[26]}$$

Where:

- ✓ $\{P_s\}$: lateral soil displacement vector
- ✓ $\{p\}$: lateral load vector
- ✓ $[I_s]$: factor influencing displacement

The elements of the matrix $[I_s]$ are evaluated by integrating Mindlin's equation over a rectangular surface. Pile displacements are calculated using Euler-Bernoulli's differential equation for the bending of a prismatic beam. The beam equation, after applying the finite difference method, is given by (Poulos and Davis, 1980):

$$\{\boldsymbol{p}\} = \frac{\boldsymbol{E_p I_p n^4}}{\boldsymbol{BL^4}}[\boldsymbol{L}]\{\boldsymbol{p_p}\} + \frac{\boldsymbol{E_p I_p n^4}}{\boldsymbol{BL^4}}\{\boldsymbol{A}\} \qquad \textbf{[27]}$$

Where :

- ✓ $\{p_p\}$: displacement matrix
- ✓ $\{A\}$: vector representing moments and loads in the pile
- ✓ $n+1$: number of divisions used for pile discretization
- ✓ $[L]$: finite difference coefficient matrix
- ✓ B : foundation width

The system of equations becomes:

$$\left[[\boldsymbol{I}] + \frac{\boldsymbol{E_p I_p}}{\boldsymbol{E_s L^4}}[\boldsymbol{L}].[\boldsymbol{I_s}]\right].\{\boldsymbol{P}\} = \{\boldsymbol{P_L}\} \qquad \textbf{[28]}$$

- ✓ $\{P_L\}$ represents the vector composed of the load applied to the pile.

Based on this model and after solving Mindlin's equations, Poulos gives an expression for the displacement $\mathbf{y}$ and rotation $\boldsymbol{\theta}$ of the pile. The parameters governing these expressions are mainly the slenderness of the pile L/D and the flexibility factor of the pile K_R defined by:

$$\mathbf{K_R} = \frac{\mathbf{E_p I_p}}{\mathbf{E_s L^4}} \qquad \textbf{[29]}$$

Tables based on these parameters and the connection and loading conditions at the top of the pile can be used to calculate the displacement and rotation of the pile at the top.

4.2.4. Numerical methods

These methods can model soil-pile interaction more rigorously and include effects such as vertical movement at the soil-pile interface, linear soil non-rigidity, and the condition of sloping ground surfaces, etc. Numerical calculation using the finite element or finite difference method offers better alternatives for studying soil-pile interaction and analyzing the response of laterally loaded piles. Among the approaches used to model the problem of a laterally loaded pile are the axisymmetric approach, in plane and three-dimensional deformation.

- **Finite element method**

This approach consists of modeling the soil using the finite element method. It is a powerful tool for representing complex cases (multilayer, 3D, etc.). However, this modeling requires knowledge of the appropriate soil behavior laws. In addition, it is difficult to reproduce the initial stress state of the massif around the pile. Finally, even when taking into account the possible symmetries of the problem to be modeled, the calculation times are often significant. The finite element method can be used to study the behavior of piles. In most cases, it is a useful tool, particularly for better visualizing the phenomenon. This tool allows the problem to be examined in its entirety and in a single calculation integrating both elements, namely the soil and the foundation. It thus provides interesting results on the behavior of piles and the soil mass. Several authors have developed and used this method to study the lateral response of piles under lateral loads. This method requires discretization of the domain under study, and the response depends on this discretization.

- **Finite difference method**

This method is less widely used than the finite element method. Like the latter, it requires discretization in space and time.

CONCLUSION

The first part of this chapter presents the characteristics of the soil-structure interface in terms of rheological and numerical modeling. It highlights the importance of understanding the mechanisms of friction mobilization and their relationship with volumetric deformation behavior for accurate modeling. Studies show that the interface plays a significant role in defining the stability conditions of the structure. The second part describes the response of a pile under static axial stress and presents methods for calculating settlement. For correct dimensioning, an accurate estimate of the bearing capacity and displacement of the pile is necessary. The NF P 94-262 standard recommends the transfer curve method, in particular the Frank and Zhao (1982) model, for calculating settlement. With regard to piles under lateral loading, the study shows the complexity of the analysis. Theoretical and experimental approaches often give divergent results. Measurements of displacements and moments in the pile help to understand the influence of various parameters (soil type and stiffness, pile type, presence of vertical load). The most commonly used method is that of Winkler (1867). The use of advanced numerical methods, such as finite elements, shows promise for solving these problems.

Declaration by Authors

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