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Modeling and Analysis of Sheet Pile Wall Behavior by Accounting for Soil-Structure Interaction

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ABSTRACT

The stability of a sheet pile wall can be influenced by ground movements that induce forces at the interface between the soil and the structure. This phenomenon, known as soil-structure interaction, was modeled by Winkler to determine the structural movements associated with those of the ground. This article focuses on modeling and analyzing the behavior of a sheet pile wall subjected to lateral soil pressure, taking soil-structure interaction into account. An analytical approach, combined with numerical solutions, is applied to examine the behavior of the structure. The approach is based on the Winkler model, also known as the soil reaction coefficient method, which is a method for modeling soil-structure interaction. This model allows the wall to be treated as a self-supporting vertical beam, while the soil-assumed to be elastic and consisting of underconsolidated clay-is modeled as a set of independent springs with a horizontal reaction modulus of k_h . The results show variations in the structure's behavior under the effect of ground movements. A parametric study was conducted on the soil's pressuremeter modulus (Ménard modulus) and on the wall's stiffness to demonstrate the relationship between the properties of the soil and the structure and the structure's displacement. The variation in the Ménard modulus demonstrates that the stiffer the soil, the greater the reduction in the curtain's displacement and deflection, which confirms previous theories regarding the behavior of this type of structure.

Keywords: sheet pile walls, soil-structure interaction, Modeling, pressuremeter modulus, flexural stiffness

1. INTRODUCTION

In research on methods for calculating sheet pile walls, engineers generally use two main approaches: limit equilibrium methods and soil-structure interaction methods. The former, known for their analytical simplicity, estimate the forces exerted by the soil on the sheet pile wall based on classical theories of thrust and bearing. They therefore consider only soil shear parameters. The latter are exemplified by the reaction modulus method-also known as the Winkler model or the independent spring method-which is a soil-structure interaction model in which the reaction exerted by the soil at a given point is directly proportional to the displacement of that point[1]. Understanding Winkler's theory, which is based on the elastic soil model, offers an interesting approach to modeling sheet pile walls. Authors such as

Ménard and Balay, Schmitt, and the NF P94-282 standard have contributed to the formulation of the soil reaction coefficient and define it in terms of soil parameters. They highlight the interaction between the soil and the structure, which directly influences the distribution of lateral pressures, horizontal displacements, bending moments, and, ultimately, the overall stability of the structure.

However, it is imperative to understand the behavior of a sheet pile wall subjected to lateral earth pressure in order to predict its response during the design process.

It is in this context that this article aims to model and analyze the behavior of sheet pile walls while taking soil-structure interaction into account.

2. MATERIALS AND METHODS

2.1 Material Properties

The study focuses on a steel sheet pile wall of the AZ 52-700 type [2] driven into foundation soil characterized by the pressuremeter method. In accordance with the assumptions of Ménard and Bourdon, the analysis is conducted on a unit strip 1 m wide. All geometric and mechanical characteristics of the material and the soil are summarized in Table 1.

Table 1. Geometric and mechanical characteristics of the material and soil

Parameters	Parameter	Symbol	Value
Sheet pile wall	Modulus of elasticity (Young)	E	210 GPa
	Moment of inertia	I	$1330140.^{10^{-8}}$ m^4 /ml
	Studied bandwidth	B	1 m
Soil and geometry	Probe depth	D	5 m
	Limit pressure (pressiometric)	pl	0.2 MPa
	Rheological coefficient	α	0.5
	Geometric parameter	A	$\frac{2}{3}D$

D: Curtain wall profile

2.1 Soil and structure modeling

The principle is based on the fact that the reaction of one solid relative to another corresponds to a pressure profile whose intensity is directly proportional to the displacement of the interface between the two solids. Consequently, the interaction between these two solids can be modeled by a stiffness. Applying this concept to the field of retaining structures means that the horizontal soil pressure on the wall is proportional to the horizontal displacement of the wall. The coefficient of proportionality between these two quantities is called the soil reaction coefficient [3]. This parameter is not an intrinsic property of the soil; rather, it relates the pressure$p(z)$ to the displacement$y$ of the soil (Figure). The equation defined by this proportionality is as follows:

$$p(z) = k_h y \qquad (1)$$

The horizontal reaction modulusk_h is traditionally assumed to be independent of the displacement$y$ and, in most cases, constant throughout the thickness of a given soil layer [4].

The origin of this method likely lies in the fact that this interaction law leads, for elastic beams, to an ordinary differential equation:

$$EI\frac{d^4y}{dz^4} + p(z) = 0 \qquad (2)$$

From the equation of proportionality between the upstream soil pressure and the displacement of the structure, the following equation is derived:

$$EI\frac{d^4y}{dz^4} + k_h y = 0 \qquad (3)$$

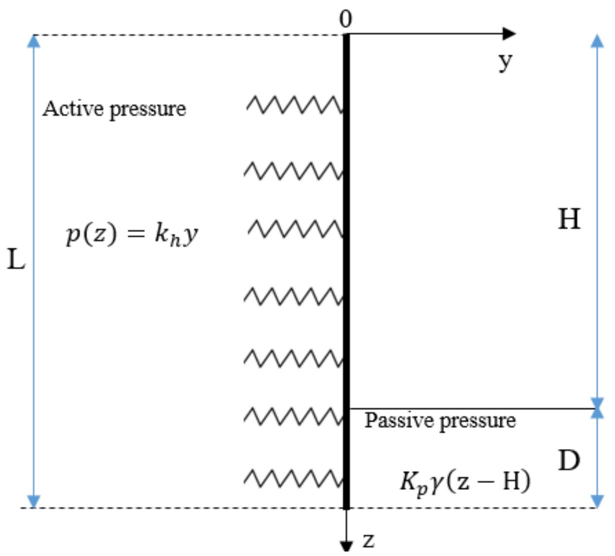
In the downstream section, the horizontal pressure f(z) is a soil bearing force and is defined by equation (4); this pressure is a Rankine mechanical force dependent on the Rankine bearing coefficient k_p.

$$f(z) = K_p\gamma(z - H) \qquad (4)$$

Substituting (4) into (2) yields the differential equation for the deformation at the curtain wall foundation; it is defined by:

$$EI\frac{d^4y}{dz^4} + k_h y = K_p\gamma(z-H) \qquad (5)$$

Where K_p : thrust coefficient, γ : soil density, H : clear height, z: depth, and EI: bending stiffness of the sheet pile wall.



L: Height of the sheet pile wall

H: Clear height (unfilled)

D: sheet depth

K_p : Soil resistance coefficient

γ : Soil density

Figure 1 . Geometric configuration of the model under study

2.3 Comparative study of the soil reaction modulus

The design of a retaining structure using the reaction coefficient method requires the determination of k_h, which represents the soil stiffness. It is this parameter, when incorporated into the calculation, that determines the soil-screen equilibrium. The underlying problem with determining soil stiffness, k_h, is that it is not an intrinsic parameter of the soil [5]. There are several methods for determining the parameter k_h; here, we compare three of these methods: the Ménard and Balay method, the Schmitt method, and the method specified in standard NF P94-282. Through comparison, we seek to evaluate which method yields the greatest deformation within the linear relationship linking soil pressure to its displacement y (Equation 1).

The Ménard and Balay method

The Ménard and Balay method was proposed for determining the soil reaction coefficient [5]. The initial expression described only the stiffness at the pile tip. Subsequent versions were proposed to provide a description over the entire height of the structure, notably by [6] and later by [7]. These modifications primarily concern the geometric parameter A, defined for construction stages of the structure that differ from those proposed in the initial version-for example, for geometric configurations with different pile dimensions or excavated area dimensions, or to incorporate the installation of supports (tie rods or struts) [8]. In this method, the expression for the soil reaction coefficient is defined as follows:

$$k_h = \frac{E_M}{0{,}5\alpha A + 0{,}133(9A)^\alpha} \qquad (6)$$

Where:

E_M: the pressuremeter modulus of the soil in the layer under consideration,

A: interaction height in m, which is the height over which the soil is subjected to bearing pressure from the structure

α: rheological coefficient of the soil.

The Schmitt Method

This method was proposed by [9] following a series of measurements on actual structures. It incorporates the mechanical characteristics of the structure into the calculation through its flexural stiffness. It is possible to use this method within the framework of the NF P94-282 standard [10] after performing a few additional checks. The main drawback is that it does not take into account the geometry of the excavation [7].

$$k_h = 2{,}1\frac{\left(\frac{E_M}{\alpha}\right)^{4/3}}{\left(\frac{EI}{B}\right)^{1/3}} \quad (7)$$

Where:
***E**_M*: soil pressure modulus,
EI: flexural stiffness of the sheet pile,
B: reference length taken as 1 m,

The NF P94-282 standard [11]

The NF P94-282 standard proposes the following formula for determining the reaction coefficient (derived from the Schmitt method):

$$k_h = 2\frac{\left(\frac{E_M}{\alpha}\right)^{4/3}}{\left(\frac{EI}{B}\right)^{1/3}} \quad (8)$$

With the following definitions:
$\boldsymbol{E_M}$: soil pressure modulus;
$\boldsymbol{\alpha}$: rheological coefficient depending on the nature and compactness of the soil;
EI: product of Young's modulus E and the moment of inertia I of the screen;
B: reference length taken as 1 m.

Figure , 3, and 4 present the results of a comparative analysis of methods for determining the parameter $\boldsymbol{k_h}$.

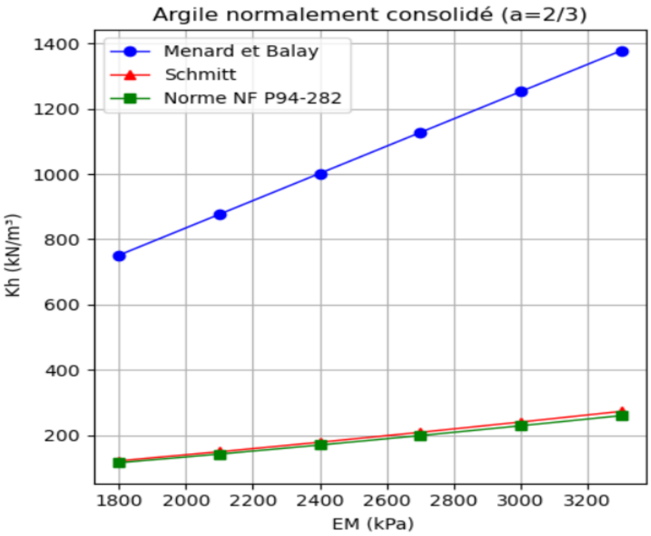


Figure 2. Variation of the reaction coefficient k_h as a function of the pressure modulus E_M for different calculation methods in the case of consolidated clay

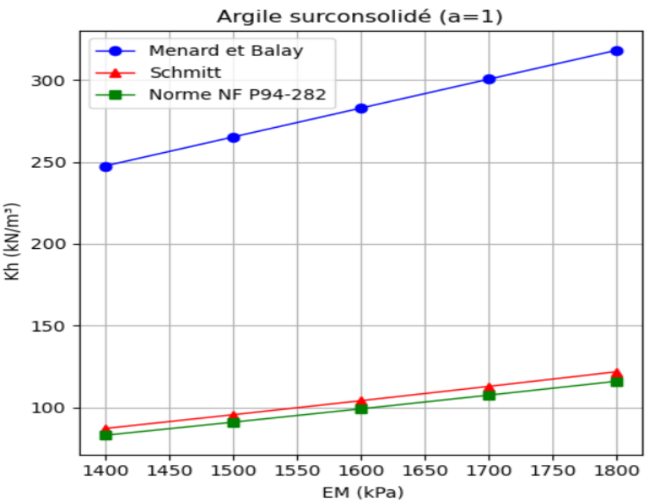


Figure 3. Variation of the reaction coefficientk_h as a function of the pressuremeter modulus E_M for different calculation methods in the case of overconsolidated clay

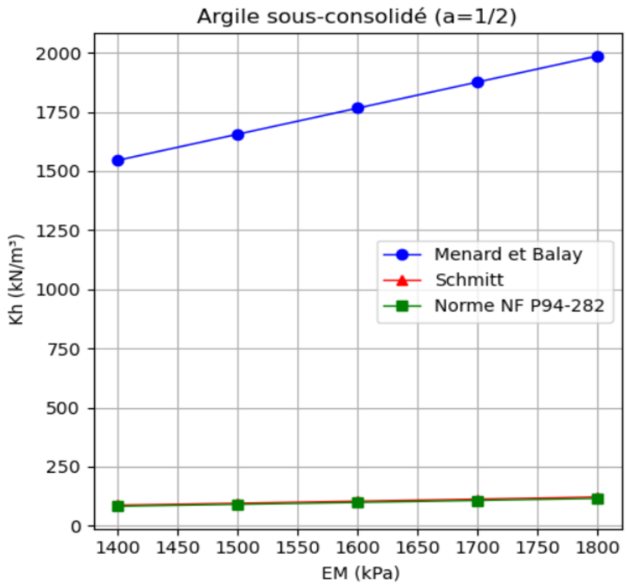


Figure 4 . Variation of the reaction coefficient k_h as a function of the pressuremeter modulus E_M for different calculation methods in the case of an underconsolidated clay

The stiffness of a soil varies as a function of its displacements. Thus, a soil will exhibit greater stiffness for small displacements than for large displacements [12]. The three authors-Ménard and Balay, Schmitt, and the NF P94-282 standard-link the soil reaction modulus to the soil's pressuremeter modulus. Ménard and Balay incorporate a dimensional parameter ***A*** into the reaction coefficient formula, whereas Schmitt and the NF P94-282 standard take into account the flexural stiffness of the screen.

The variation curves of the soil reaction coefficient defined by each author at different values of the pressuremeter modulus showed that the lowest values of the latter are obtained using the method specified in Standard NF P94-282. In what follows, we will use the expression for the soil reaction coefficient given by Standard NF P94-282 to solve the differential equation for the purpose of calculating the deformation and bending moment of the curtain wall.

2.4 Solution of the Model

The analysis of the mechanical behavior of the retaining wall is based on determining the horizontal displacement y(z), the bending moment M(z), the shear force V(z), and the rotation $\theta(z)$ over the entire height of the wall. Using the formulation of the soil reaction coefficient k_h from the ***NF P94-282*** standard**,** the flexural behavior of the structure under the **Winkler** elastic mass model is governed by the following system of differential equations:

$$\begin{cases} EI\dfrac{d^4y}{dz^4} + k_h y = 0\ , pour\ z \leq H & (a) \\ EI\dfrac{d^4y}{dz^4} + k_h y = K_p\gamma(z-H), pour\ z \geq H & (b) \end{cases} \qquad (9)$$

Where: $\boldsymbol{K_p}$: passive earth pressure coefficient,

$\boldsymbol{\gamma}$: Density of the soil (kN/m^3)

H : clear height of the retaining wall above the bottom of the excavation (m)

z: depth measured from the top of the sheet pile wall (m)

EI: bending stiffness of the sheet pile wall (kN.m^2/m)

Retaining section (z ≤ H)

In this section, the depth is less than the clear height of the sheet pile.

We have equation (a) of the system:

$EIy^{(4)}(z) + k_h y(z) = 0$ (10)

We set $\beta^4 = \frac{k_h}{EI}$

$y^{(4)}(z) + \beta^4 y(z) = 0$ (11)

The general solution is of the form:

$$y(z) = e^{\beta z} (A_1 \cos\beta z + A_2 \sin\beta z) + e^{-\beta z} (A_3 \cos\beta z + A_4 \sin\beta z) \quad (12)$$

Where:

A_i , i=1,…,4 integration constants determined from the boundary conditions.

$y^{(4)}(z)$ is the fourth derivative of the deformed function of the curtain.

Worksheet Section (z ≥ H)

In this section, we have the second equation of the system:

$EIy^{(4)}(z) + k_h y(z) = K_p\gamma(z - H)$ (13)

We can rewrite it in the following form:

$y^{(4)}(z) + \beta^4 y(z) = \frac{K_p\gamma}{EI}(z - H)$ (14)

Equation (8) has a general solution of the form:

$y(z) = y_h(z) + y_p(z)$ (15)

Where$y_h(z)$ is the solution to the homogeneous equation and $y_p(z)$ is the particular solution.

The solution to the homogeneous equation is given by:

$$Y(z) = e^{\beta z} (A_1 \cos\beta z + A_2 \sin\beta z) + e^{-\beta z} (A_1 \cos\beta z + A_2 \sin\beta z) + \frac{K_p\gamma}{K_h}(z - H) \quad (16)$$

Where A_i, i= 5,8 are integration constants to be determined from the boundary conditions.

The particular solution is given by:

$y_p(z) = rz + t$ (17)

Where ***r*** and ***t*** are constants to be determined.

$\frac{dy_p}{dz} = r$ and $\frac{d^2y_p}{dz^2} = 0$

The higher-order derivatives of $y_p(z)$,$\left(\frac{d^3y_p}{dz^3}\ et\ \frac{d^4y_p}{dz^4}\right)$, are also zero; therefore, by substituting equation (12) into equation (9) and performing identification, we obtain:

$r = \frac{K_p\gamma}{\beta^4 EI}$ and $t = -\frac{K_p\gamma H}{\beta^4 EI}$

We therefore obtain:

$y_p(z) = \frac{K_p\gamma}{K_h}(z - H)$ (18)

Hence, the general solution to the deformed equation becomes:

$$y(z) = e^{\beta z} (A_5 \cos\beta z + A_6 \sin\beta z) + e^{-\beta z} (A_7 \cos\beta z + A_8 \sin\beta z) \quad (19)$$

From equation (14), we can derive the bending moment (M), the shear force (V), and the rotation (θ).

$M(z) = \frac{EId^2y}{dz^2}$ (20)

$V(z) = \frac{d^3y}{dz^3}$ (21)

$\theta(z) = \frac{dy(z)}{dz}$ (22)

To determine the values of the constants, boundary conditions must be specified. To do this, we considered a self-supporting sheet pile wall whose top and bottom conditions are defined by equations (23) and (24).

At the top ($z = 0$), we have:

$\left.\frac{EId^2y}{dz^2}\right|_{z=0} = M(z = 0) = 0$ (23)

$\left.\frac{d^3y}{dz^3}\right|_{z=0} = V(z = 0) = 0$ (24)

At the point$z = H$, there is continuity for the deformed section, the rotation, the bending moment, and the shear force. Equations (25), (26), (27), and (28) led to a matrix system of eight (8) equations with eight (8) unknowns A_i , and$i = 1,2,,\ldots,8$, representing the integration constants to be determined.

$y(H^+) = y(H^-)$ (25)

$\theta(H^+) = \theta(H^-)$ (26)

$M(H^+) = M(H^-)$ (27)

$V(H^+) = V(H^-)$ (28)

$A_2 - A_4 = 0$

$A_1 - A_3 + 2A_4 = 0$

$\alpha_{3,5}A_5 + \alpha_{3,6}A_6 + \alpha_{3,7}A_7 + \alpha_{3,8}A_8 = f_3$

$\alpha_{4,5}A_5 + \alpha_{4,6}A_6 + \alpha_{4,7}A_7 + \alpha_{4,8}A_8 = f_4$

$\alpha_{5,1}A_1 + \alpha_{5,2}A_2 + \alpha_{5,3}A_3 + \alpha_{5,4}A_4 + \alpha_{5,5}A_5 + \alpha_{5,6}A_6 + \alpha_{5,7}A_7 + \alpha_{5,8}A_8 = f_5 = 0$
$\alpha_{6,1}A_1 + \alpha_{6,2}A_2 + \alpha_{6,3}A_3 + \alpha_{6,4}A_4 + \alpha_{6,5}A_5 + \alpha_{6,6}A_6 + \alpha_{6,7}A_7 + \alpha_{6,8}A_8 = f_6$
$\alpha_{7,1}A_1 + \alpha_{7,2}A_2 + \alpha_{7,3}A_3 + \alpha_{7,4}A_4 + \alpha_{7,5}A_5 + \alpha_{7,6}A_6 + \alpha_{7,7}A_7 + \alpha_{7,8}A_8 = f_7 = 0$
$\alpha_{8,1}A_1 + \alpha_{8,2}A_2 + \alpha_{8,3}A_3 + \alpha_{8,4}A_4 + \alpha_{8,5}A_5 + \alpha_{8,6}A_6 + \alpha_{8,7}A_7 + \alpha_{8,8}A_8 = f_8 = 0$
Let $C = (\alpha_{i,j})_{1\le i,j\le 8}$ be the matrix defined by the system above; then we have:
$C =$

0	1	0	-1	0	0	0	0
1	0	-1	2	0	0	0	0
0	0	0	0	$\alpha_{3,5}$	$\alpha_{3,6}$	$\alpha_{3,7}$	$\alpha_{3,8}$
0	0	0	0	$\alpha_{4,5}$	$\alpha_{4,6}$	$\alpha_{4,7}$	$\alpha_{4,8}$
$\alpha_{5,1}$	$\alpha_{5,2}$	$\alpha_{5,3}$	$\alpha_{5,4}$	$\alpha_{5,5}$	$\alpha_{5,6}$	$\alpha_{5,7}$	$\alpha_{5,8}$
$\alpha_{6,1}$	$\alpha_{6,2}$	$\alpha_{6,3}$	$\alpha_{6,4}$	$\alpha_{6,5}$	$\alpha_{6,6}$	$\alpha_{6,7}$	$\alpha_{6,8}$
$\alpha_{7,1}$	$\alpha_{7,2}$	$\alpha_{7,3}$	$\alpha_{7,4}$	$\alpha_{7,5}$	$\alpha_{7,6}$	$\alpha_{7,7}$	$\alpha_{7,8}$
$\alpha_{8,1}$	$\alpha_{8,2}$	$\alpha_{8,3}$	$\alpha_{8,4}$	$\alpha_{8,5}$	$\alpha_{8,6}$	$\alpha_{8,7}$	$\alpha_{8,8}$

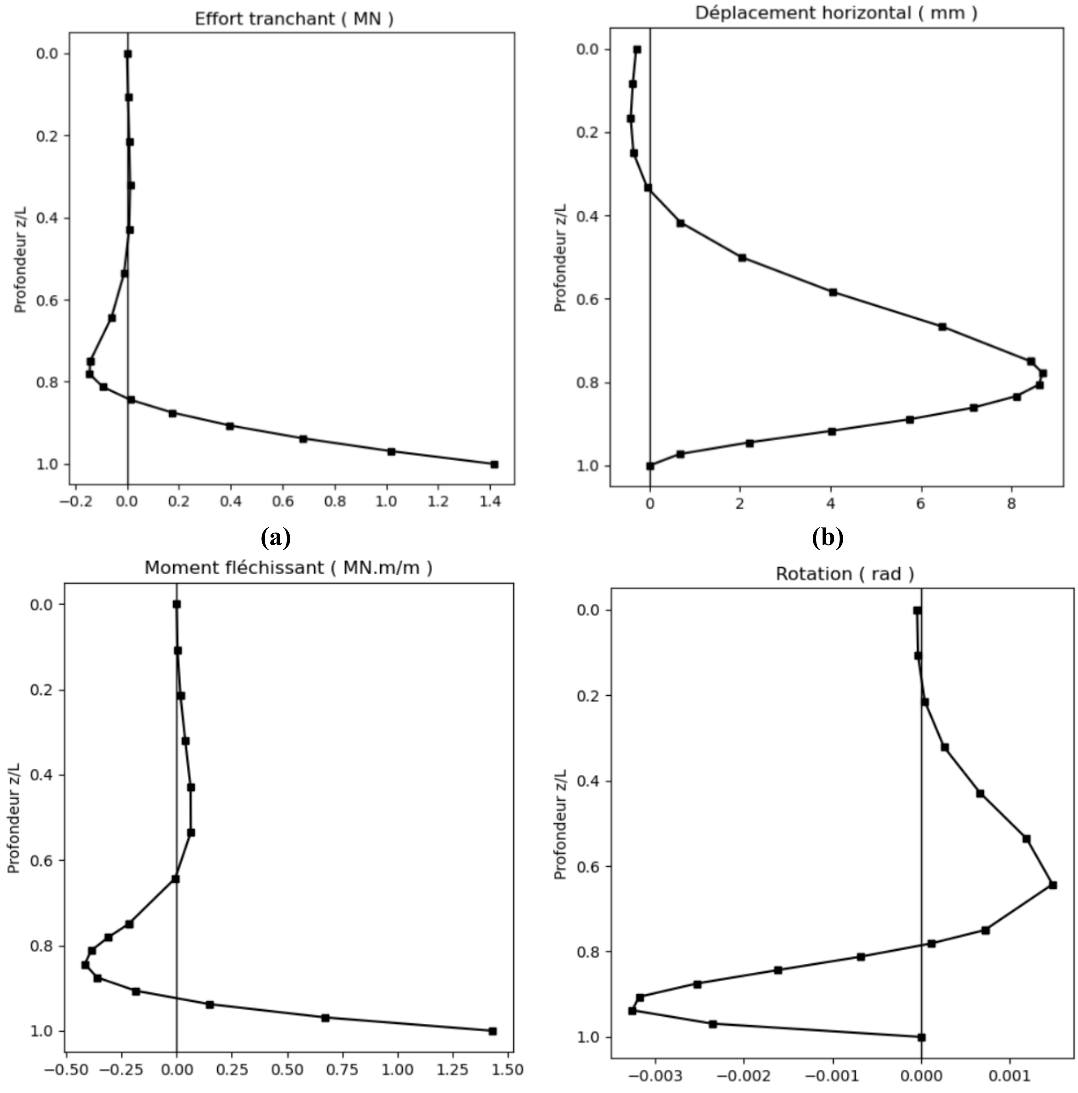
The expressions for the coefficients $(\alpha_{i,j})_{1\le i,j\le 8}$ of matrix (C) are defined by:
1e row of matrix C:
$\alpha_{3,5} = e^{\beta L}\cos\beta L$,
$\alpha_{3,6} = e^{\beta L}\sin\beta L$,
$\alpha_{3,7} = e^{-\beta L}\cos\beta L$,
$\alpha_{3,8} = e^{-\beta L}\sin\beta L$
4e row of matrix C:
$\alpha_{4,5} = \beta e^{\beta L}(cos\beta L - sin\beta L)$,
$\alpha_{4,6} = \beta e^{\beta L}(cos\beta L + sin\beta L)$
$\alpha_{4,7} = -\beta e^{-\beta L}(cos\beta L + sin\beta L)$
$\alpha_{4,8} = \beta e^{-\beta L}(cos\beta L - sin\beta L)$
5e row of matrix C:
$\alpha_{5,1} = e^{\beta H}\cos\beta H$,
$\alpha_{5,2} = e^{\beta H}\sin\beta H$
$\alpha_{5,3} = e^{-\beta H}\cos\beta H$,
$\alpha_{5,4} = e^{-\beta H}\sin\beta H$
$\alpha_{5,5} = -e^{\beta H}\cos\beta H$
$\alpha_{5,6} = -e^{\beta H}\sin\beta H$
$\alpha_{5,7} = -e^{-\beta H}\cos\beta H$,
$\alpha_{5,8} = -e^{-\beta H}\sin\beta H$
6e row of matrix C:
$\alpha_{6,1} = \beta e^{\beta H}(cos\beta H - sin\beta H)$,
$\alpha_{6,2} = \beta e^{\beta H}(cos\beta H + sin\beta H)$
$\alpha_{6,3} = -\beta e^{-\beta H}(cos\beta H + sin\beta H)$
$\alpha_{6,4} = \beta e^{-\beta H}(cos\beta H - sin\beta H)$
$\alpha_{6,5} = -\beta e^{\beta H}(cos\beta H - sin\beta H)$
$\alpha_{6,6} = -\beta e^{\beta H}(cos\beta H + sin\beta H)$
$\alpha_{6,7} = \beta e^{-\beta H}(cos\beta H + sin\beta H)$,
$\alpha_{6,8} = -\beta e^{-\beta H}(cos\beta H - sin\beta H)$
7e row of matrix C:
$\alpha_{7,1} = -e^{\beta H}sin\beta H$
$\alpha_{7,2} = e^{\beta H}cos\beta H$
$\alpha_{7,3} = e^{-\beta H}sin\beta H$
$\alpha_{7,4} = -e^{-\beta H}cos\beta H$
$\alpha_{7,5} = e^{\beta H}sin\beta H$
$\alpha_{7,6} = -e^{\beta H}cos\beta H$
$\alpha_{7,7} = -e^{-\beta H}sin\beta H$
$\alpha_{7,8} = e^{-\beta H}cos\beta H$
8e row of matrix C:$\alpha_{8,1} = e^{\beta H}(cos\beta H + sin\beta H)$
$\alpha_{8,3} = e^{-\beta H}(cos\beta H - sin\beta H)$
$\alpha_{8,4} = e^{-\beta H}(cos\beta H + sin\beta H)$
$\alpha_{8,5} = e^{\beta H}(cos\beta H + sin\beta H)$
$\alpha_{8,2} = e^{\beta H}(cos\beta H - sin\beta H)$
$\alpha_{6,6} = -e^{\beta H}(cos\beta H - sin\beta H)$
$\alpha_{6,7} = -e^{-\beta H}(cos\beta H - sin\beta H)$
$\alpha_{6,8} = -e^{-\beta H}(cos\beta H + sin\beta H)$
$f_1 = f_2 = f_5 = f_7 = f_8 = 0$,$f_3 = -\frac{K_p\gamma}{\beta^4 EI}[L-H]$, $f_4 = -\frac{K_p\gamma}{\beta^4 EI}$, and $f_6 = \frac{K_p\gamma}{\beta^4 EI}$
From this, we deduce that the matrix A and the function F are such that:
$A = [A_1 \quad A_2 \quad A_3 \quad A_4 \quad A_5 \quad A_6 \quad A_7 \quad A_8]^T$ and
$F = [f_1 \quad f_2 \quad f_3 \quad f_4 \quad f_5 \quad f_6 \quad f_7 \quad f_8]^T$
Hence, we have the matrix equation defined by:

$$CA = F$$

RESULT

Due to the complexity of analytically solving the system, a numerical approach using

Python was chosen. This program allows us, on the one hand, to determine the integration constants, and on the other hand, to obtain the profiles of horizontal displacement, bending moment, shear force, and rotation. The corresponding results are shown in Figure 5.



(a) **(b)** **(c)** **(d)**

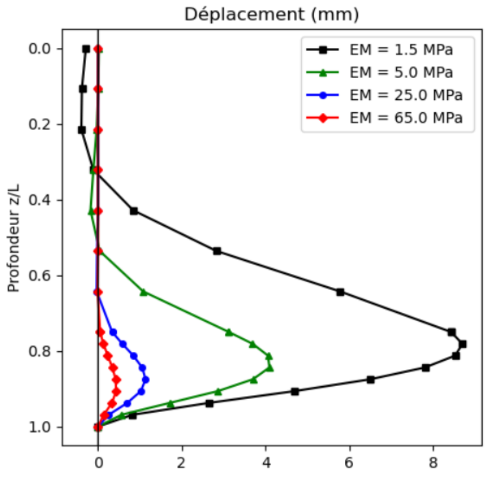
Figure 5 . Evolution of stresses on the sheet pile wall: (a) horizontal displacement, (b) bending moment, (c) shear force, and (d) rotation.

The parametric study conducted in this article focuses on the soil's pressuremeter modulus ($\boldsymbol{E_M}$) and the wall's flexural stiffness (***EI***). These parameters were varied for different values (Figure and**Figure**), with the aim of studying their influence on the structure's mechanical behavior. The geotechnical and structural data used for these calculations are summarized in

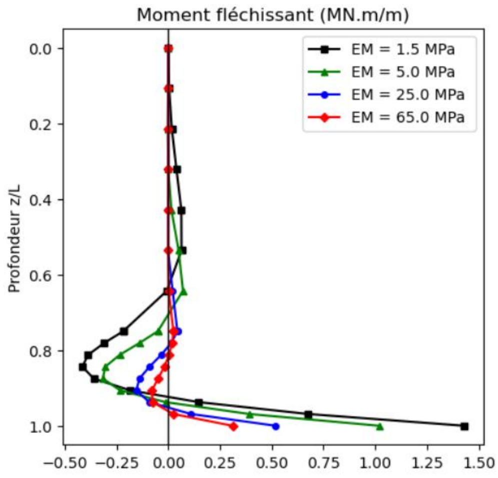
Table 2.

Table 2. Calculation data for the parametric study of the soil's pressure modulus and the sheet pile's stiffness

L (m)	$\boldsymbol{\varphi}$**(degrees)**	$\boldsymbol{\gamma(kN/m^3)}$	$\boldsymbol{\alpha}$	**E (GPa)**
20	18	18	1/2	210



(a)



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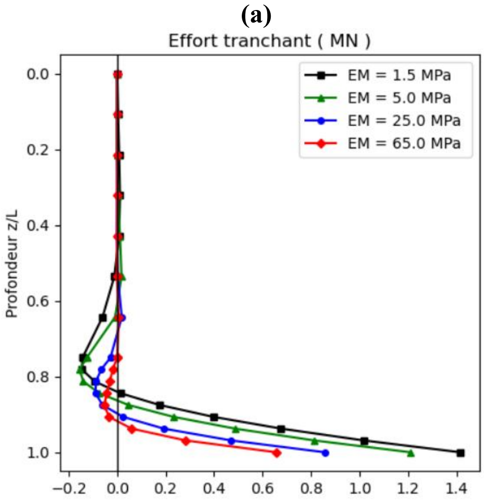
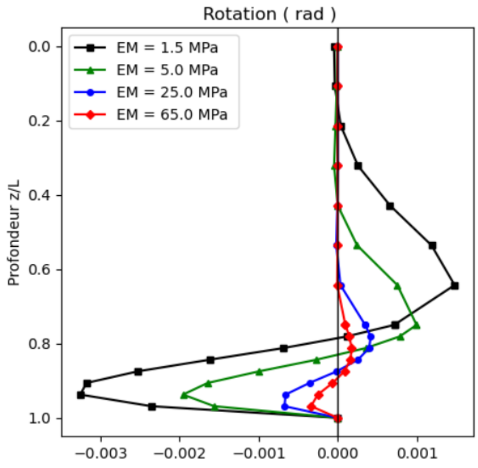
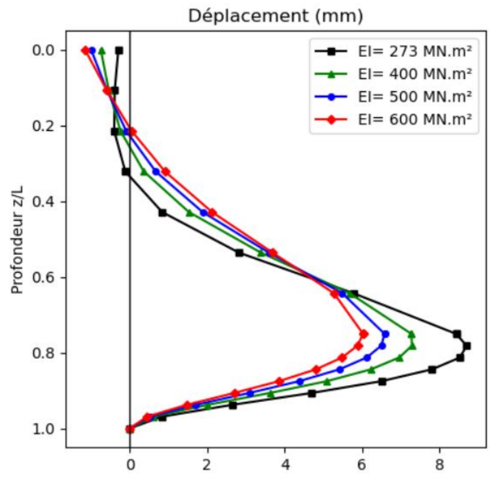


(c)

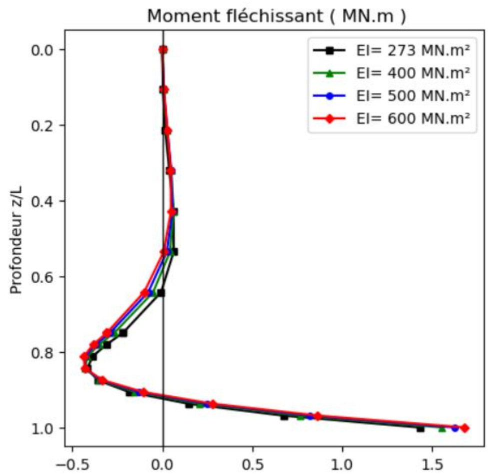


(d)

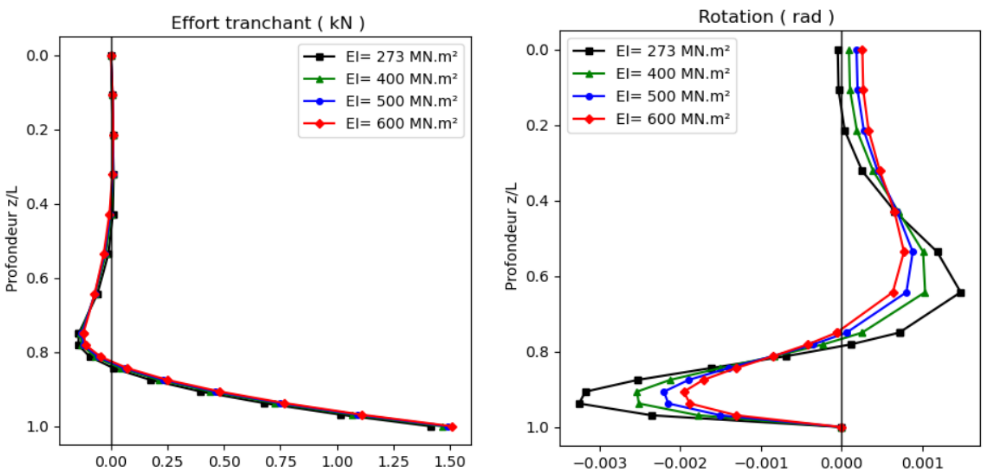
Figure 6 . Influence of the pressure modulus on the behavior of the curtain wall: (a) Horizontal displacement, (b) Bending moment, (c) Shear force, and (d) Rotation.



(a)



(b)



(b) **(d)**

Figure 7 . Effect of sheet pile stiffness on the behavior of the retaining wall: (a) Horizontal displacement, (b) Bending moment, (c) Shear force, and (d) Rotation.

Figure 5 shows the distribution of horizontal displacement $y(z)$, bending moment $M(z)$, shear force$V(z)$, and rotation$\theta(z)$ over the normalized height (z/L) of the sheet pile wall. It can be seen that at the top of the structure (z/L=0), the bending moment is zero ($M = 0$), reflecting the free-end condition. The horizontal displacement increases along the exposed section, reaching its maximum amplitude at a relative depth of $z/L \approx 0{,}85$. At the toe of the wall (z/L = 1), the displacement is zero, consistent with the fixed support condition imposed at the base.

The distribution of the bending moment reveals a zone of negative bending at the pile tip ($z/L \approx 0{,}85$), where $M \approx -0{,}40 MN.m/ml$, induced by the soil's abutment reaction. The moment then reverses direction to reach its absolute maximum at the toe of the structure, directly generated by the fixed support. The shear force remains virtually zero in the upper section (z/L < 0.5), then increases sharply in the anchorage zone to reach its maximum value at the base (z/L = 1).

The rotation profile confirms the overall kinematics of the wall, showing a maximum amplitude atz/L ≈ 0,65 (0.0015 rad), followed by a reversal toward a negative peak atz/L ≈ 0.90 (0.0030 rad). The cancellation of rotation at the base ($\theta(z) = 0$ at z/L=1) confirms the rotational locking characteristic of the fixed support.

The pressuremeter modulus is a geotechnical parameter obtained from the pressuremeter test (Ménard type). Its variation directly causes a variation in the soil reaction modulus. The results (Figure) show that the higher the pressuremeter modulus, the smaller the horizontal displacement. This is because the pressuremeter modulus represents the stiffness of the soil that is, its ability to resist lateral deformation. A soil with a high pressuremeter modulus is rigid and therefore deforms very little under pressure, resulting in small displacements. In contrast, soil with a low pressiometric modulus is flexible or compressible and deforms easily. The results show that an increase in the pressiometric modulus does not have a significant effect on the shear stress in the upper and lower parts of the wall, but does cause a decrease in rotation.

The results obtained in the study "

(b)

(d)

Figure) show that an increase in the wall's intrinsic stiffness primarily alters its horizontal displacement behavior. When the flexural stiffness increases and the wall is fixed at the base, the displacements decrease with depth precisely when the z/l ratio exceeds 0.6. The wall's resistance to bending increases, allowing the structure to better resist lateral soil pressure; the same applies to rotation: as stiffness increases, rotation decreases and becomes zero at the base, which corresponds to the wall's fixed support. In contrast, in the retaining section (z/l < 0.6), a gradual increase in displacements was observed. A slight decrease in the bending moment and shear force is also observed along the length of the curtain wall.

CONCLUSION

This article examined the behavior of unanchored sheet pile walls through modeling that incorporates soil-structure interaction. The results obtained using Winkler's analytical approach and numerical solutions with Python demonstrated that the soil reaction coefficient method is an effective technique for calculating the behavior of sheet pile walls used for earth retention. The parametric study conducted on the soil's pressure modulus and the structure's stiffness also revealed that variations in these parameters related, respectively, to the properties of the soil and the wall can have significant effects on the stability of the structure. The lessons learned from this analysis highlight the significant impact of geotechnical and structural characteristics on the overall stability of the structure. It is therefore essential to account for the changes in behavior induced by these parameters on the sheet pile walls during calculations.

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