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Study of the Behavior of Deep Foundation Piles of a Bridge, Adapted to the Environment on Compressible Soils: The Case of the Bridge (420 ml) from the Porto-Novo Lagoon in Benin

Yémalin Daniel Agossou[1], Ernesto Houehanou[2], Judicaël Agbelele[3], Camel Godonou[1], Edmond C. Adjovi[1]

[1]Department of Civil Engineering, L2EGC-ENSTP/UNSTIM, Abomey, Benin
[2]Department of Civil Engineering, LAPIT-INSTI/UNSTIM, Abomey, Benin
[3]Department of Civil Engineering, ENSET/UNSTIM, Abomey, Benin
Email: yemdag@gmail.com

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Abstract

Bridge design has long been guided by purely economic criteria. However, today, given the potential impacts of climate change, it is imperative to adopt an approach that incorporates sustainable criteria at every stage of a bridge's life cycle, in order to protect the environment without compromising safety, functionality, and sustainable development. Adding to these concerns is the major challenge posed by the construction of bridges on compressible soils for socio-economic development. This study highlights the behavior of deep foundations on compressible soils subjected to axial loads from an environmentally adapted bridge on the Porto-Novo lagoon in Benin. It addresses the problems posed by climate change and, in particular, compressible soils. To achieve this, a multi-criteria analysis was conducted using a Matlab program for the ELECTRE I multi-criteria method, in order to determine the type of bridge suitable for our project, based on the principles of sustainable development. This analysis revealed that composite steel-concrete bridges of the twin-girder type with cantilevered sections are much more suitable. Furthermore, the soils (intended to support this bridge), identified as compressible, present risks of settlement and liquefaction, as revealed by geotechnical tests. Unable to improve the situation, we adopted a method using a group of piles, specifically driven cast-in-place piles. To determine the optimal dimensions, piles with diameters of 0.8 m, 0.9 m, and 1 m, ranging from 30 to 48 m in depth, were modeled using GEOFOND software. The results of this design allowed us to select piles with a diameter of 1 meter and a depth of 45 m, providing a bearing capacity of 3.19 MN. These dimensions, combined with a center-to-center spacing of 4.5 m between piles, demonstrate that the group of 6 piles is suitable, with an efficiency coefficient of 83%. Finally, the PLAXIS 3D software allowed us to understand the behavior of the group under axial load. Based on settlements, it was observed that the soil exhibits horizontal and vertical displacements, the largest values of which, 27 mm, are obtained in the underconsolidated surface layers.

Keywords

Composite Bridge, Multi-Criteria Analysis, Compressible Soils, Piles, Foundation Behavior

1. Introduction

The progress of a nation is undeniably facilitated by the construction of roads, which ensure the collection and distribution of goods, the connection between individuals on a human and social level, and long-distance transport [1]. Roads in very good condition are, therefore, one of the essential factors for facilitating free trade between populations. In Africa, road infrastructure carries more than 90% of land traffic, according to Malick Sané [2], and is vital for sustainable and socio-economic development. They therefore represent one of the key tools for contemporary mobility and must be integrated into the major future challenges facing territories [3].

Indeed, during the construction of these infrastructures, there may come a time when natural obstacles (breaches, waterways, etc.) or artificial ones (roads, railways, canals, etc.) are encountered. To ensure the continuity of these infrastructures, two solutions are available to the engineer: eliminating the obstacle (filling a breach, bypassing the obstacle) or constructing an engineering structure such as bridges, tunnels, viaducts, etc.

The design of engineering structures, particularly bridges, is constantly evolving thanks to the discovery of materials with rigorously controlled and improved performance, the development of both rapid and precise construction methods, the creation of innovative forms that provide new solutions to the problems posed by crossing obstacles of impressive dimensions, and computational tools that allow for the development of highly sophisticated behavioral models. This design requires a thorough understanding of the different types of load-bearing structures and foundations, as well as the fundamental principles of their dimensioning and the various construction methods. It is also essential to consider economic and environmental factors, which impose significant requirements, ranging from the harmonious integration of the structure into its site to the meticulous selection of building materials.

According to Ouézdou, certain data are necessary when designing a bridge; these include, on the one hand, natural data such as topographic, geotechnical, and hydraulic data, and actions of natural origin, and on the other hand, functional data such as data relating to the span, the obstacle crossed, and actions of functional origin [4]. The technical design of bridges, therefore, requires the solution to a complex brief that stipulates cost-effectiveness, technical quality, ease of maintenance, durability, site suitability, and aesthetic appeal [5].

To analyze all these parameters, the SETRA Guide proposes several types of bridge variants, meaning that certain characteristics listed in the tender documents (DCE) must be modified. Unlike technical proposals, these variants may require adjustments to the bill of quantities, the detailed cost estimate, and, of course, the Special Technical Specifications (CCTP) and the plans [6]. This allows the project to meet the identified needs, ensures the viability of the required investments, and ultimately enables the project to be authorized and registered. At this stage, in-depth analyses are conducted to examine the types of bridges under consideration better, as this decision is crucial to the project's continuation [7]. Until recently, all these variants were distinguished by the economic factor, which was the only one taken into account in the decision-making process concerning any type of bridge construction process.

Furthermore, recent studies have identified numerous potential impacts of climate change on bridges [8]. This is demonstrated by the resilience of the Saint-Etienne River bridge in the face of Cyclone Dumile in 2013, the collapse of the bridge at the entrance to Malanville in Benin due to torrential rains in 2018, etc. To anticipate these phenomena without compromising sustainable development, the objective is now to review the various sustainable methods and criteria used for decision-making at each phase of a bridge's life cycle, from the deck to the foundations, according to researcher Chaphakar [9].

Thus, the multi-criteria decision-making aid method appears as an alternative to classical optimization methods designed on the definition of a single function, expressed in economic (monetary) terms, and which reflects the consideration of numerous, often incommensurable, criteria [10]. The advantage of multi-criteria methods is that they consider a set of criteria of different natures (expressed in different units), without necessarily transforming them into economic criteria or a single function. The aim is not to find an optimum, but an intermediate solution that can take various forms, such as choices, allocations, or rankings. To improve the selection of bridge types, a process that reconciles the methods used in the guides with multi-criteria methods seems necessary. The more complex the models, the more susceptible they are to errors, both in their creation and during their use [11].

After the type of bridge is determined, the next step is the structural study of the deck and the foundation, which requires complex and time-consuming calculations. Indeed, the foundation is the part of the structure that connects the support to the ground. It transmits the weight of the structure and the normal and accidental loads applied to it to the ground [12]. However, soils, especially soft soils, present problems that generally manifest as low bearing capacity, significant deformations under static loads, or seismic loads, depending on the soil type [13].

For [14], the construction of civil engineering structures on soft, compressible soils presents a real challenge for geotechnical specialists. High settlement, low bearing capacity, and soil liquefaction are the most common problems encountered when constructing foundations in these soils.

To solve these problems, one of the first solutions considered is the use of deep foundations (pile foundations). However, the characteristics of foundations such as piles remain complex due to their highly nonlinear behavior in different soil types, particularly compressible soils [15].

Several theoretical and experimental studies have been conducted to better understand the behavior and performance of pile foundations as a function of different variables, such as the number of piles [16], the foundation system (footing-pile, raft pile), the type of soil in place [15], and the calculation of the capacity of displaced piles in clay [17]. Other studies have focused on evaluating the effectiveness of individual pile types using different calculation methods on soft clays [18]. Soft clays, as well as compressible, flexible layers at depth, make the behavior of piles completely unpredictable.

Geotechnical studies are conducted at the various supports, and their main objective is to determine the nature of the foundation soils, define the foundations of the structure, establish the foundation bearing level, calculate the allowable bearing pressure, evaluate settlements, and determine the piezometric level [19]. To facilitate studies and take essential parameters into account, specific sizing and modeling software is offered and used by several designers. However, some aspects still need to be explored in greater depth, including advanced soil properties, the effect of the water table and pore pressures, the study of settlements, and in-depth analysis using digital tools.

2. Goals

The main objective is to develop a model capable of predicting the behavior of deep foundations on compressible soils.

Specifically, this refers to:

- Develop a MATLAB program for multi-criteria analysis of bridges;
- Determine, from geotechnical tests, the physical and mechanical properties of the different soil layers in the study area;
- Sizing the deep foundations of the bridge;
- Simulate the behavior of different foundation soils under loading.

3. Expected Results

- A sustainable, economical bridge that respects environmental and social standards is obtained;
- A bridge sizing program accessible to any designer is designed;
- Soil behavior is controlled for a solid and infallible foundation.

4. Methodology

4.1. Presentation of the Study Project

The project consists of widening the Sèmè-Porto-Novo road to two lanes over 10 km, followed by reinforcement of the existing roadway and the construction of a new 420 m long, 2 × 2 lane bridge [20].

To achieve the objectives of this research, the bridge over the Porto-Novo lagoon (35 km^2) from kilometer point 9 + 250 to kilometer point 9 + 696 will serve as a basis. This bridge will span the Porto-Novo lagoon, which separates the municipality of Sèmè-Podji (an area with strong industrial and economic potential) from Porto-Novo, the capital of Benin. It is located on National Interstate Route 1 (RNIE1), specifically at kilometer points 6˚27'43" north and 2˚37'12" east as shown in **Figure 1**.

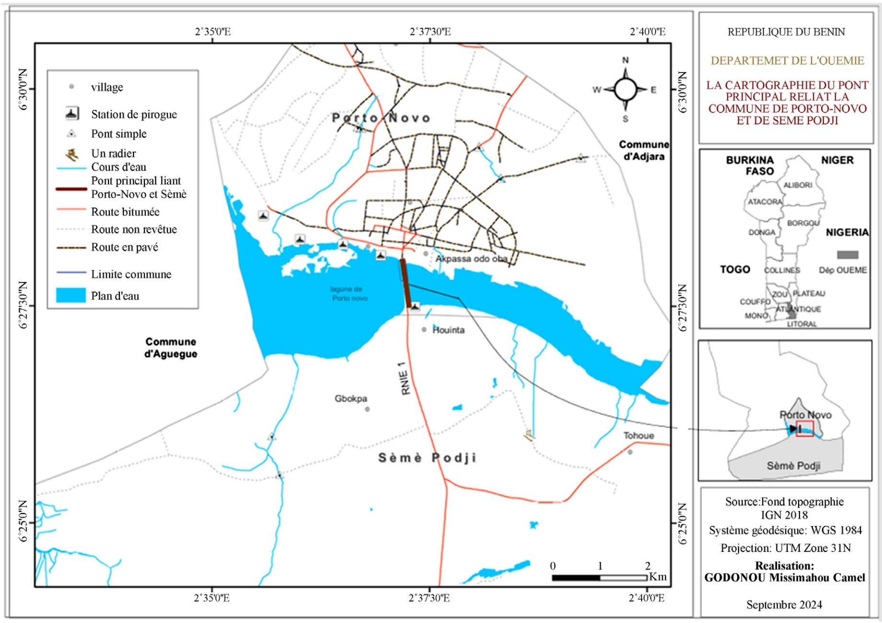

Figure 1. Location of the Sèmè-Porto-Novo axis bridge.

The bridge enjoys a sub-equatorial maritime climate under the influence of two dry seasons (December to February and August to September) and two rainy seasons (April to July and October to November) [21]. Average annual rainfall increases from west to east, oscillating around 1450 mm in Porto-Novo. Instantaneous rainfall intensities rarely exceed 100 mm/hour, with the most intense occurring at the beginning of the main rainy season [22].

4.2. Multi-Criteria Analysis of the Bridge

When designing a bridge project, it is essential to choose a bridge type based on various specific criteria. This choice is crucial and determines the project's success. To analyze these criteria, multi-criteria methods are of great importance in decision-making and can be classified into different groups according to similar characteristics [23].

Examples include:

- **Notation Methods:** These methods include Simple Additive Weighting (SAW) and Complex Proportional Evaluation (COPRAS). Their basis lies in evaluating alternatives using basic arithmetic operations. Both SAW and COPRAS yield the sum of weighted normalized values for all criteria. SAW is the older MADM method and allows for the consideration of maximization criteria.

- **Distance-Based Methods:** This group includes Goal Programming (GP), Compromise Programming (CP), Preference Ordering by Similarity to an Ideal Solution (TOPSIS), Multi-Criteria Optimization and Compromise (VIKOR), and Data Envelopment Analysis (DEA). These methods all aim to determine the distance between each alternative and a specific point. Within this group, there are two distinct philosophies. The goal of Goal Programming (GP) is to find the alternative that satisfies a set of objectives. The goal of Compromise Programming (CP) is to find the alternative closest to the hypothetical best alternative. While Data Envelopment Analysis (DEA) originates from GP, CP forms the basis of Multi-Criteria Optimization and Compromise (VIKOR) and Preference Ordering by Similarity to an Ideal Solution (TOPSIS) [24].
- **Pairwise Comparison Methods:** This group includes the Analytical Hierarchy Process (AHP), Analytical Network Process (ANP), and Measuring Attractiveness by a Categorical Evaluation Technique (MACBETH). These methods are very useful for determining the weighting of different criteria and comparing alternatives against a subjective criterion. The problem with these methods is that they rely solely on the decision-makers' knowledge. Furthermore, different decision-makers may have different perspectives on the same problem. The Analytical Hierarchy Process (AHP) was the first pairwise method presented and remains one of the most widely used in decision-making problems [25]. The Analytical Network Process (ANP) is a method that attempts to address the problem of criterion independence in the AHP. Measuring attractiveness using a categorical evaluation technique (MACBETH) is an alternative to the AHP.
- **Upgrading Methods:** The Preference Ranking Method for Enriching Evaluations (PROMETHEE) and the Elimination and Choice Method Expressing Reality (ELECTRE) both establish a preference relationship between a set of alternatives, indicating the degree of dominance among them. These methods can handle unclear and incomplete information, and their application results in a partial preference ranking of the alternatives, rather than a cardinal measure of their preference relationship.
- **Utility/Value Methods:** Multi-Attribute Utility Theory (MAUT) and Multi-Attribute Value Theory (MAVT) define expressions that determine the degree of satisfaction of criteria. These functions convert the scores that define the behavior of alternatives with respect to the criteria into their degree of satisfaction according to the method (MAUT or MAVT). The function expression can model different forms to link the scores and the degree of satisfaction.

In this study, only the ELECTRE I method is used because we are in a selection situation, specifically, we want to choose a type of bridge from a batch of bridges.

This method, proposed by Roy (1968), solves multi-criteria decision problems.

It identifies the subset of actions offering the best possible compromise. It is often used in the selection of competing projects to identify the most efficient subset based on the considered criteria. In the case of the ELECTRE I method, true criteria are defined, and the notion of competition is also present; selecting the best [26].

i) Definition of Actions: Types of Bridges

The results of tests carried out on the site prove that the soils in place consist of a succession of layers of peat, clayey sand, sand, and clay up to 55 m below the surface [27]. We are therefore dealing with marshy soils with relatively low resistance, and where the resistant soil is located at a considerable depth. Two solutions are then available to us for choosing the bridge structure:

- Increasing the number of supports reduces the load transmitted to the ground by the piles. This directly results in a decrease in the span length between supports and an increase in foundation costs.

Indeed, according to the client's requirements, the bridge must allow for relatively easy navigation, which leaves us no choice but to reduce the span length. This solution is therefore not appropriate.

- Opting for lightweight structures means structures whose construction requires lightweight materials, which allows us to maintain the distance between supports but also to reduce the weight of the structure.

Next, the bridge must be built in close proximity to an existing, trafficable bridge. Therefore, the installation and construction work must not disrupt traffic flow in the surrounding area. This necessitates considering not only a lightweight structure but also the use of prefabricated elements.

We are therefore looking for a lightweight road bridge whose construction requires many more prefabricated elements.

Our options are therefore limited to prestressed concrete bridges, mixed steel-concrete bridges, metal bridges, and cable bridges.

Regarding cable bridges, not only is the cost high and the technical expertise required for their construction lacking, but the owner also does not consider such an option.

We are therefore left with three types of bridges, namely prestressed concrete bridges, mixed steel-concrete bridges, and metal bridges, which will then be subjected to multi-criteria analysis.

Note:

- **P1:** Prestressed concrete bridge.
- **P2**: Steel-concrete composite bridge.
- **P3:** Metal bridge.

ii) Definition of Criteria and Weighting per Criterion

A criterion is a function defined on the set of actions representing the user's preferences from their perspective. We will proceed to determine the criteria that can lead us not only to the choice of a given structure but also to a sustainable structure capable of withstanding climate change. This is because there is significant empirical evidence demonstrating that considerable changes have already occurred in the climate system, and climate model projections predict continued evolution in the future [8]. Each criterion is associated with a value k_j that will determine the weight of the criterion.

Several methods are possible for setting a weighting. They are divided into subjective (non-numerical methods) and objective (numerical methods).

For this paper, bridge types are evaluated based on 4 criteria, which are:

- **Criterion 1 (C_{r1}): The Economy**

This is one of the major criteria, encompassing not only the initial cost of the work (site preparation, earthworks, construction), but also long-term maintenance costs. An economically viable solution must optimize expenses without compromising the safety or durability of the structure.

- **Criterion 2 (C_{r2}): The Environmental and Social Impact**

The choice of bridge type must limit environmental impacts (deforestation, disruption of waterways, noise, pollution) and take into account the effects on local communities (relocations, accessibility, safety). A responsible project will seek to minimize these impacts while maximizing benefits for the community.

- **Criterion 3 (C_{r3}): Execution Time and Durability**

The construction timeframe is a crucial factor, particularly in situations where traffic needs to be restored quickly or when weather conditions reduce the construction window. The type of bridge chosen must therefore allow for a reasonable completion timeframe.

- **Criterion 4 (C_{r4}): Aesthetics**

The integration of the bridge into its surrounding landscape and visual environment is an important criterion, especially in urban or tourist areas. The architectural aspect can influence the project's acceptance by users and local populations.

These criteria were chosen based on the realities and requirements of the project owner and are now the most important in civil engineering projects [11] [28].

The importance of each criterion in the decision-making process is reflected in a weighting k_j as shown in the following (**Table 1**) [10]. These weightings are based on real-world conditions, the client's priorities, and subjective assessments of the studies conducted.

Table 1. Weighting of criteria.

Criteria	C_{r1}	C_{r2}	C_{r3}	C_{r4}
Weight ($\boldsymbol{k_j}$)	35	20	30	15

iii) Determining the Performance Table

The weighting of parameters is of particular importance in multi-criteria evaluation [29]. We then established a rating scale based on life cycle assessment documents and bridge performance data according to materials. **Table 2** shows the bridge ratings for each criterion.

Table 2. Bridge ratings based on criteria (performance).

Type of Work	C_{r1}	C_{r2}	C_{r3}	C_{r4}
Weight (%)	35	20	30	15
P1	8	7	7	6
P2	10	6	8	7
P3	7	5	9	7

These ratings are based on current assessments in bridge engineering and may vary depending on the specifics of the projects and construction sites.

To calculate the concordance and discordance matrices, a MATLAB program was developed for speed and accuracy. This program takes as input the number of choices or actions, the number of criteria and their respective weights, as well as the scores for each action following each criterion, to produce the decision graph.

The results of the analysis are presented in the results and discussion section.

4.3. Loading Program

Once the type of bridge has been chosen, it is imperative to assess the load on the structure, including road traffic surcharges.

The reference document for the load and test program is booklet 61, title II of the CPC [30]. Indeed, the load-bearing regulations for bridges used in Benin, as in most French-speaking sub-Saharan African countries, are French regulations. These are compiled in booklet 61, titles I, II, and III of the Common Specifications Document (CPC). These titles relate respectively to railway bridges, road bridges, and canal bridges.

The purpose of load calculation is to evaluate the loads and overloads acting on the bridge.

Thus, to justify the safety and durability of the structures to which the booklet applies, the limit state method is used [31].

The individual actions to which the structure will be subjected are classified as permanent actions, cyclical actions, intermittent actions, and accidental actions, according to the general characteristics of their distribution over time. The characteristic value of an action is the one that has an accepted a priori probability of being reached or exceeded on the side of the most unfavorable values during a defined period, called the reference period.

In the combinations of actions to be considered, a distinction is made, on the one hand, between actions considered to be of short duration or accidental, and on the other hand, between actions considered to be of long duration.

Accidental actions arise from rare phenomena (earthquakes, shocks, etc.). When actions are considered short-term or accidental, the focus is generally on the peak values of the individual actions likely to be applied to the structure. When actions are considered long-term, the focus is on the values of other actions likely to be applied concurrently. Permanent actions are always considered long-term. Cyclical and intermittent actions are considered, depending on the combination, as either short-term or long-term, with different characteristic values. Accidental actions are never considered long-term [32].

4.4. Geotechnical Program

To better understand the behavior of the supports for better bridge stability, it is essential to carry out geotechnical studies.

According to Uribe, geotechnical studies are essential to guarantee the safety, functionality, and durability of construction projects by ensuring appropriate interaction between the soil and the structure of the works [33]. These studies are based on a combination of *in situ* tests carried out on the site itself, as well as laboratory tests carried out on soil samples taken by coring.

The tests carried out essentially include:

- 12 pressuremeter boreholes up to 55 m deep, in accordance with standard NF EN ISO 22476-4 of May 2015 [34];
- 12 core drillings up to a depth of 55 m according to standard EN ISO 22475-1 [35];
- 24 intact samples obtained during surveys and drilling.

In addition to these tests, there are 24 laboratory identification tests, namely particle size analysis [36], the determination of the density of solid particles [37], the limits of Atterberg [38], the determination of water content [39], and the oedometer test.

Furthermore, often limited by cost, time, and technicality during the execution of projects, it is then possible to find parameters such as the coefficient of grain friction, the shear modulus, the cohesion from the net limit pressure, and the pressuremeter modulus.

➢ Undrained cohesion *Cu*

Several authors have proposed empirical rules based on observations of foundation behavior. The most notable works are those of Ménard (1957), Amar and Jézéquel (1972), and Baguelin and Jézéquel (1973).

According to its authors, CASSAN definitively proposed in 2005 [40], the undrained cohesion, which is estimated by the following correlations:

- For: $p_{LM}^{*} \le 0.3\ \text{MPa}$; $C_u = \dfrac{p_{LM}^{*}}{5.5}$.
- For: $0.3 \le p_{LM}^{*} \le 1\ \text{MPa}$; $C_u = \dfrac{p_{LM}^{*}}{12} + 0.03(\text{MPa})$.
- For: $p_{LM}^{*} \ge 1\ \text{MPa}$; $C_u = \dfrac{p_{LM}^{*}}{35} + 0.085(\text{MPa})$.

➢ The angle of friction of the soils φ

To calculate the friction angle, Ménard proposes the following equation:

$$p_{LM}^{*} = b \times 2^{(\varphi - 24)/4}$$

After mathematically developing Ménard's equation, we find:

$$\varphi = 24 + 4 \times \frac{\ln P_{LM}^{*} - \ln b}{\ln 2}$$

with $b = 1.8$ for moist soils with loose structure, and $b = 3.5$ for a dry and structured soil.

4.5. Pile Sizing

The pile calculations were performed using the GEOFOND 1.22 software developed by GEOS INGENIEURS CONSEILS. This software calculates the bearing capacity of deep foundations, allowing the user to choose from existing reference documents: Fascicule 62 title V, DTU 13.2, and the French application standards of EUROCODE 7, specifically standard NF P 94.262.

Settlements are calculated using the Frank & Zhao method, depending on the type of tests available: Pressuremeter or static penetrometer (Geofond User Manual 2014).

Its Deep Foundations MODULE allows you to check displacements, forces, and maximum moments in a group of piles.

The software also provides an approach to bearing capacities and settlements using probabilistic methods, that is, not in the form of a deterministic value, but in the form of a Gaussian distribution, in order to be able to retain values of bearing capacities or settlements according to a probability of exceedance [41].

The calculation consists of verifying that the axial stress, including possibly negative friction, remains less than $Q_{\max}$ (**Table 3**) determined in the two limit states (ULS and SLS) [30].

Table 3. The values of $Q_{\max}$ limitating axial stress calculations [30].

Ultimate Borderline States	$Q_{\max}$
Fundamental Combinations	$\frac{Q_u}{1.40}$
Accidental Combination	$\frac{Q_u}{1.20}$
Service Limit States	
Rare Combinations	$\frac{Q_c}{1.10}$
Quasi-Permanent Combinations	$\frac{Q_c}{1.40}$

The expression for the limit loads Q_u in compression and tension of a deep foundation element is as follows:

$$Q_u = Q_{pu} + Q_{su} \text{ en compression} \quad (1)$$

$$Q_u = Q_{su} \text{ en traction} \quad (2)$$

The maximum peak load is given by:

$$Q_{pu} = A * q_{pu} \quad (3)$$

The ultimate lateral friction load is given by the following formula:

$$Q_{su} = \sum_{i=0}^{n} P_i * q_{si} * e_i \quad (4)$$

The creep loads in compression Q_c and tension of a deep foundation element are evaluated, in the absence of in-situ testing, from Q_{pu} and Q_{su} by the following relationships:

– For foundation elements installed by excavating the ground:

$$Q_c = 0.5 * Q_{pu} + 0.7 * Q_{su} \text{ en compression}$$

$$Q_c = 0.7 * Q_{su} \text{ en traction}$$

– For foundation elements installed with soil displacement:

$$Q_c = 0.7 * Q_{pu} + 0.7Q_{su} \text{ en compression}$$

$$Q_c = 0.7 * Q_{su} \text{ en traction}$$

4.6. Study of the Behavior of Piles and Soils

Using Plaxis 3D software, we were able to model various scenarios, thus evaluating soil settlement.

Indeed, PLAXIS 3D is a three-dimensional finite element program developed for the analysis of deformation, stability, and groundwater flow in geotechnical engineering. It is part of the PLAXIS product line, a suite of finite element programs used worldwide for geotechnical engineering and design [42].

This step enabled a detailed analysis of the interactions between the piles and the supporting soil, offering valuable insights into the overall performance of the structure.

5. Results and Discussion

The results presented in this section are those of support 6 (the most used support).

5.1. Multi-Criteria Analysis of the Bridge

Based on the criteria, weights, and scores defined for each type of bridge, the concordance and discordance matrices obtained by the Matlab programming are as illustrated in Table 4 and Table 5.

Table 4. Concordance matrix.

	P1	P2	P
P1	0	0.2	0.55
P2	0.8	0	0.7
P3	0.45	0.45	0

Table 5. Discordance matrix.

	P1	P2	P3
P1	1	0.4	0.4

Continued

P2	0.2	1	0.2
P3	0.4	0.6	1

To determine the concordance and discordance thresholds, we will take the value just below the highest concordance index and the value above the lowest discordance index. This will yield a non-optimal, rather than a compromise, solution. According to the program, we find: and $c = 0.7$ and $d = 0.4$.

Table 6 presents the result of Upgrading:

Table 6. Upgrading table.

	P1	P2	P3
P1	**'Nothing'**	**'fake'**	**'fake'**
P2	**'TRUE'**	**'Nothing'**	**'TRUE'**
P3	**'fake'**	**'fake'**	**'Nothing'**

In **Table 6**, we observe that variant P2, representing composite bridges, outperforms both other options. Composite bridges thus emerge as the most cost-effective and reasonable option for optimal performance, considering environmental factors, geotechnical aspects, time constraints, and aesthetics. Therefore, we can confidently assert, using the ELECTRE I multi-criteria analysis method, that a steel-concrete composite bridge is suitable for our project.

This choice aligns with the two variables studied during the preliminary design phase. These variables are a prefabricated prestressed girder bridge and a composite two-girder steel bridge, both of which were equivalent in terms of price (less than a 5% price difference in favor of the composite bridge). However, by adding other criteria such as environmental and social impact, durability, construction time, and aesthetics, the composite two-girder steel bridge outperforms the prefabricated prestressed girder bridge and becomes the ideal solution. This solution also remains consistent with the choice made by engineers Ariel Elegbede and Sourou Azian, based on the ELECTRE I method, in their respective reports.

Then, in this range of bridges, following the SETRA guide, taking into account the span and width of the bridge, which are respectively 35 and 14 m, the twin girder type structures with bridge pieces with cantilevers are much more suitable for our project.

In conclusion, through the ELECTRE I multi-criteria analysis, composite bridges, particularly twin-girder bridges with cantilevered sections, are ideal for the project.

5.2. Loading Program

The total load is obtained by combining the actions at the limit states; it is directly derived from the type of bridge chosen previously (a composite steel-concrete bridge of the two-girder type with cantilevers). **Table 7** shows the calculation of the load transfer to the footing of pier 6:

Table 7. Loads exerted on the base.

	Weight (t/m^3)	Height (m)	Section (m^2)	Weight (t)
Weight of Batteries (t)	7.85	16.6	0.460	59
Sole Weight (t)	2.5	1.76	55.55	244
Weight (Batteries + Sole + Protection) (t)		487.52		
Reaction R1 Due to Weights with Apron (t)		827.74		
Reaction R2 Due to Operating Costs (t)		328.4		
Reaction R3 Due to the Exceptional Load (t)		352		
Wind Reaction R4 (t)		36		
ELECTED in MN		14.32		
ELS in MN		12.34		

The total load exerted on the footing of pier 6 is 14.32 and 12.34, respectively, at the ULS and SLS.

5.3. Geotechnical Studies

– **Pressuremeter Test**

Table 8 presents the results of the essential parameters from the pressuremeter test.

Table 8. Results of the pressuremeter test.

Depth (m)	Pressuremeter Modulus (MPa)	Net Limit Pressure (MPa)	Creep (MPa)	Em/Pl Ratio
1	3.4	0.21	0.17	16.19
2	1.9	0.23	0.18	8.26
3	3.9	0.23	0.18	16.96
4	2.1	0.23	0.18	9.13
5	3.5	0.21	0.17	16.67
6	3.2	0.17	0.14	18.82
7	2.5	0.18	0.14	13.89
8	3.5	0.28	0.22	12.50
9	4	0.75	0.60	5.33
10	6.9	0.32	0.26	21.56
11	3.7	0.2	0.16	18.50
12	3.3	0.28	0.22	11.79
13	6.1	0.31	0.25	19.68
14	12.8	0.25	0.20	51.20

Continued

15	3.8	0.21	0.17	18.10
16	12.9	0.14	0.11	92.14
17	16.7	1.42	1.14	11.76
18	14.6	2.19	1.75	6.67
19	29.7	1.29	1.03	23.02
20	40	2.58	2.06	15.50
21	32.1	2.27	1.82	14.14
22	33.6	1.88	1.50	17.87
23	14.8	1.01	0.81	14.65
24	21.2	1.75	1.40	12.11
25	24.5	2.14	1.71	11.45
26	82.9	2.71	2.17	30.59
27	43.6	2.42	1.94	18.02
28	52.8	4.79	3.83	11.02
29	33.6	5.53	4.42	6.08
30	26.1	1.64	1.31	15.91
31	26.5	2.25	1.80	11.78
32	31.7	2.21	1.77	14.34
33	20	1.8	1.44	11.11
34	29.8	2	1.60	14.90
35	28.1	3.57	2.86	7.87
36	98.4	3.04	2.43	32.37
37	32.6	5	4.00	6.52
38	34.8	2.36	1.89	14.75
39	48.4	4.12	3.30	11.75
40	48.8	4.21	3.37	11.59
41	41.9	4.68	3.74	8.95
42	60.3	5.12	4.10	11.78
43	60.4	5.76	4.61	10.49
44	83.4	5.28	4.22	15.80
45	139.1	4.6	3.68	30.24
46	174	6.97	5.58	24.96
47	149.9	6.88	5.50	21.79
48	83.4	4.59	3.67	18.17
49	84	4.59	3.67	18.30
50	90.4	4.58	3.66	19.74

Depending on the length of the bridge, a total of twelve points were identified for the pressuremeter test. **Figure 2** presents the main parameters, which are the deformation moduli, limit pressures, and creep pressures at different depths of the support 6.

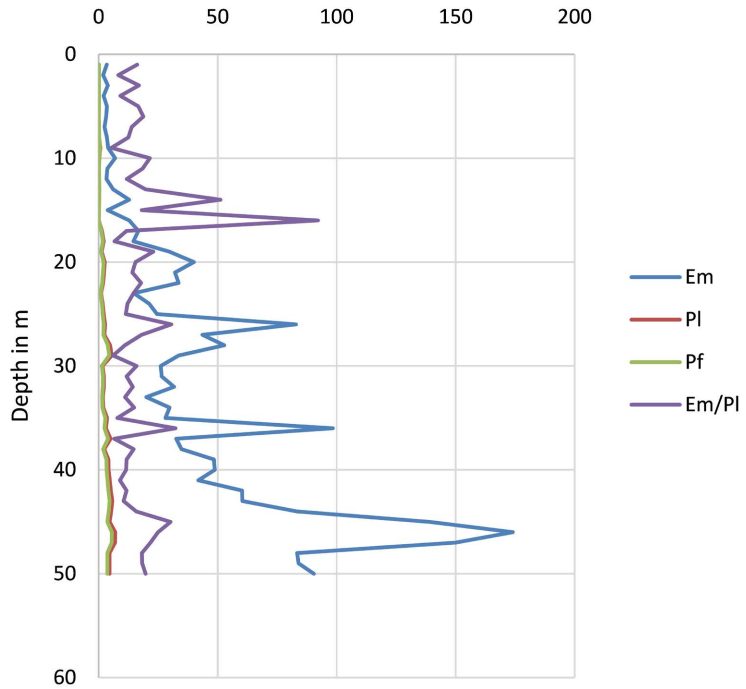

Figure 2. Pressuremeter test parameters.

The results (**Figure 2**) show that the pressuremeter modulus, net limit pressure, and creep generally increase with depth, indicating an improvement in soil stiffness and bearing capacity. Indeed, the surface layers exhibit low values for all parameters, thus characterizing loose soils with very low bearing capacity. From 17 meters, the values increase significantly, reaching high levels of up to 174 MPa for the pressuremeter modulus at a depth of 46 m, which corresponds to soils with low stiffness. The high net limit pressures and relatively significant creep values at these depths present conditions favorable to the solution of deep foundations.

– **Description of Core Sampling Samples**

The results of the geotechnical investigations by core drilling, carried out at the location of support 6 and of laboratory tests on samples taken, allow for a detailed description of the different layers of the subsoil encountered (**Table 9**).

Table 9 presents the soil stratigraphy to a depth of 0 to 53 m. It shows that the soil is composed of several layers, including silt, clayey sands, sandy clays, and peats, with their respective thicknesses. The nature of these soils corresponds to compressible soils according to the SETRA and LCPC guidelines. However, the deep clayey sand layers between 44 and 53 m can offer excellent stability.

Table 9. Soil stratigraphy.

Depth (m)	Thickness (m)	Nature of the Soils Crossed
0 to 8	8	Blackish vase
8 to 17	9	Greyish sandy mud
17 to 25	8	Greyish clayey sand
25 to 27	2	Beige sandy clay
27 to 29	2	Coarse beige sand
29 to 37	8	Beige sandy clay
37 to 39	2	Blackish peat
39 to 44	5	Beige clayey sand
44 to 53	9	Coarse beige clayey sand

– **Particle Size Analysis**

The results of the particle size analysis are presented in the form of particle size curves (**Figure 3**), illustrating the variation of the percentages of particles as a function of their diameter.

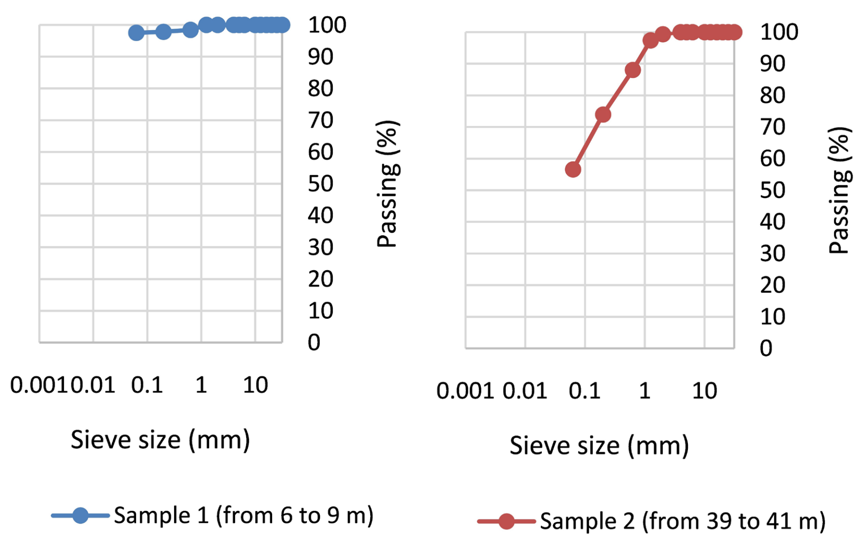

Figure 3. Particle size analysis curves.

Particle size analysis revealed that the in-situ soils contained 97.52% and 56.52% particles smaller than 80 micrometers, respectively, at depths of 9 and 41 meters. These high percentages indicate a predominance of fine particles, meaning a high fines content. They also fall within the particle size distribution range proposed by [43] and [44] for compressible soils.

– **The Limits of Atterberg**

This test is essentially based on two parameters: the liquid limit (W_L) and the plasticity limit (W_P).

The results shown in **Table 10** reveal that the plasticity indices of the different samples (16.96% and 22.25%) range from 12% to 25%. These values, combined with the fine material percentages ranging from 35% to 100%, allow us to conclude that the in-situ soil is class A2 according to the Earthworks and Backfilling Guide [45]. This class consists of fine clayey sands, silts, clays, marls, and grus. According to the plasticity diagram of [46], this is the class of clays with low plasticity and a potential means of swelling.

Table 10. Atterberg limit values.

Depth (m)	Liquidity Limit (WL) %	Plastic Limit (WP) %	Plasticity Index (PI) %
6 to 9	47.51	24.67	22.84
39 to 41	32.69	15.73	16.96

– **Essay Oedometric**

The fundamental parameters, which are void ratio, settlement, compression ratio, recompression ratio, effective stress, and preconsolidation stress, are grouped in **Table 11**.

Table 11. Results of the oedometer test.

Depth (m)	6 to 9	39 to 41
Index of voids e	1,375	0.236
Settlement (mm) ΔL_s	8,941	3,162
Compression index C_c	0.599	0.138
Recompression index C_s	0.163	0.012
Effective stress in place σ'_{vo} (kPa)	10	10
Void index of the *in-situ* soil e_o	2.796	0.449
Effective pre-consolidation stress σ'_p (kPa)	1	29
Pre-consolidation void index e_p	2,871	0.446

Based on the results in **Table 11**, we observe, firstly, that the consolidation coefficients C_c vary between 0.138 and 0.599. Secondly, we note that the effective in-situ stress σ'_{vo} is greater than the effective preconsolidation stress σ'_p in the first ten meters but becomes less than the latter as the depth increases. We therefore deduce that we are indeed dealing with moderately compressible soils that are underconsolidated at the surface and overconsolidated at depth. According to [47], these are soils in the process of consolidating under their own weight. They are generally unsuitable for construction without special treatment; they continue to deform even without additional load [48]. These soils are particularly dangerous for the foundations of lightweight constructions.

In conclusion, the analysis of the various parameters using the tests carried out revealed the low bearing capacity of the soil in place. Indeed, we are dealing with underconsolidated compressible soils composed of silts and clays with high fines and water content. These are loose, problematic soils with a high risk of deformation and settlement, and are associated with a low load-bearing capacity. Nevertheless, an improvement in the parameters is observed as the depth increases. To minimize the cost of the foundations, piles are therefore necessary to transmit the loads by lateral friction to the underlying layers down to the layer with relatively high bearing capacity.

5.4. Sizing Results

– **Isolated Pile under Axial Loads**

Given the soil type and the construction conditions, we opted for a driven diaphragm pile. Table 12 shows the bearing capacity of the piles as a function of diameter and depth, calculated using the Geofond software at the ultimate limit state (ULS) and serviceability limit state (SLS). It is important to remember that the piles only act through lateral friction; end bearing effects (since suitable soil is not reached and soil behavior remains unpredictable with depth), and negative friction are neglected. We then calculated the soil bearing capacity for piles with diameters of 0.8 m, 0.9 m, and 1 m at depths of 30 m, 35 m, 40 m, 45 m, and 48 m.

Table 12. Soil bearing capacity for piles of different diameters and depths.

Diameter (m)	Depth (m)	Load Capacity (MN)	
		ULS	SLS
1	30	2.71	1.9
	35	3.16	2.21
	40	3.66	2.56
	45	4.56	3.19
	48	5.09	3.57
0.9	30	2.44	1.71
	35	2.85	1.99
	40	3.29	2.3
	45	4.1	2.87
	48	4.58	3.21
0.8	30	2.17	1.52
	35	2.53	1.77
	40	2.93	2.05
	45	3.64	2.55
	48	4.07	2.85

According to Table 12, the maximum capacity is obtained for piles with a diameter of 1 m and a depth between 40 and 48 m.

Curve 4 (Figure 4) shows the evolution of the capacity of these piles at the SLS between 40 and 48 m depth.

This curve shows the progression of capacity as a function of depth. We will use a depth of 45 m for the pile group checks.

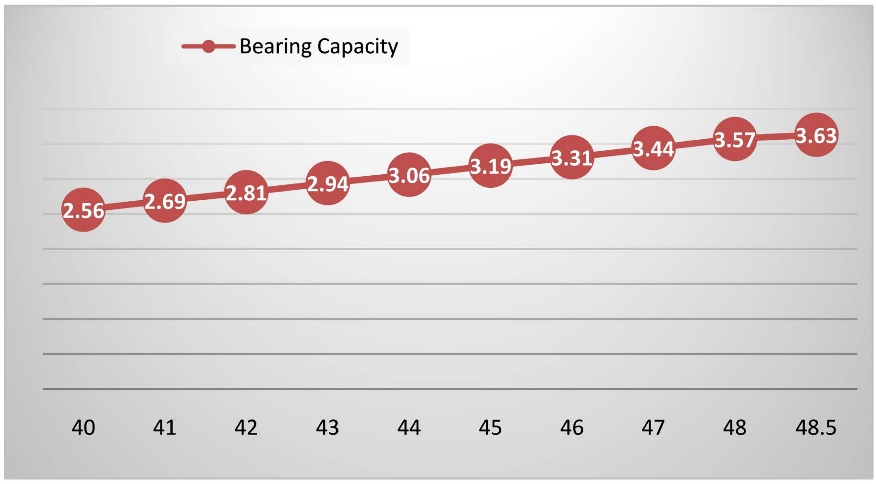

Figure 4. The evolution of soil bearing capacity as a function of depth.

– **Behavior of the Pile Group**

According to the calculation of the load transfer, the load that the footing must support is 12.34 MN at the SLS and 14.32 MN at the ULS.

For a pile with a bearing capacity of 3.19 MN at the SLS and 4.56 MN at the ULS, a minimum of four (04) piles is required to ensure the stability of the foundation.

For this purpose, for better security of our structure, we opt for a group of six (06) piles of one (01) meter in diameter and 45 m deep.

Based on these values and Figure 5, the efficiency coefficient of the pile group is written according to the Converse LABARRE formula:

$$C_e = 1 - \frac{2 * \arctan\left(\frac{B}{S}\right)}{\pi}\left(2 - \frac{1}{n} - \frac{1}{m}\right)$$

with:

B: Diameter of the piles;

S: Center distance;

n: Number of pile columns;

m: Number of rows of piles.

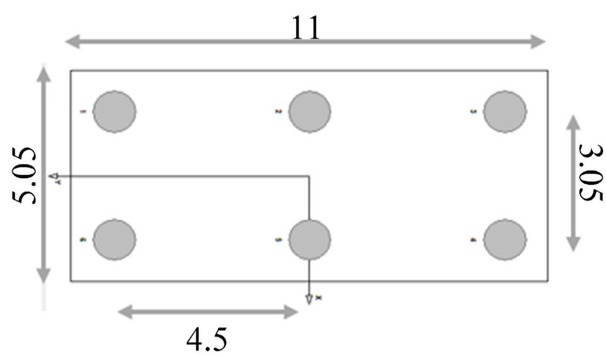

Figure 5. Plan view of the pile arrangement under the footing.

$$C_e = 1 - \frac{2 * \arctan\left(\frac{1}{4.5}\right)}{\pi}\left(2 - \frac{1}{2} - \frac{1}{3}\right) = 0.83$$

That's 83%.

Thus, according to booklet 62, for the verification of the pile group, we have:

$$\sum_{j=1}^{N} Q_j \leq C_e N Q_{max}$$

where:

Q_j : Axial load on the pile;

$Q_{\max} = 3.19$: Maximum compressive load;

N : The number of piles.

Thus, we have $\sum_{j=1}^{N} Q_j = 14.274\ \text{MN} < C_e N Q_{\max} = 15.888\ \text{MN}$.

Therefore, the pile group is verified for an efficiency of 83%. This coefficient value falls within the range found by Nguyen Ngoc-thanh (less than 1 or between 80 and 95%) for marrow clays [49].

Table 13 presents the results of the pile group calculation according to Geofond:

Table 13. Bearing capacity of piles in groups.

	Stake No.	$Q_{\max}$ (kN)
ULS Fundamental	1	4517
	2	4517
	3	4517
	4	4517
	5	4517
	6	4517
Accidental ULS	1	5270
	2	5270
	3	5270
	4	5270
	5	5270
	6	5270
SLS Quasi-Permanent	1	3162
	2	3162
	3	3162
	4	3162
	5	3162
	6	3162
SLS Frequent	1	3162
	2	3162

Continued

SLS Frequent	3	3162
	4	3162
	5	3162
	6	3162
Rare SLS	1	4025
	2	4025
	3	4025
	4	4025
	5	4025
	6	4025

5.5. Study of Pile and Soil Behavior

The results of the behavioral analysis performed with Plaxis 3D V20 are crucial for a better understanding of soil behavior in this study. To highlight the impact of the pile groups, simultaneous simulations were conducted on the in-situ soil, allowing conclusions to be drawn.

Let us recall that the study focused on the soils below pile 06. The soil was modeled as an elasto-plastic volumetric element following the soft soil model appropriate for our case (intermediate soils) and the available tests.

The saturation and non-saturation densities are respectively 26 and 24.6 kN/m^3 and the dilatancy Ψ' is equal to 0.

The footing and pile group are modeled as a reinforced concrete slab element using the linear plasticity model with E = 27,000 MPa, $\gamma = 25$/kN/m^3, and $\upsilon = 0.15$. The diameter of the piles is 1 m, and the total length of the shaft is 53.4 m, of which 45 m is driven into the ground.

Based on the tests carried out, the stratification and the approximation relationships, the modeling parameters are summarized in Table 14.

Table 14. Layer properties.

Layer	Soil Type	Depth (m)	Thickness (m)	e_0	C_c	C_s	$C'\left(\frac{\text{kN}}{\text{m}^3}\right)$	$\varphi'(^\circ)$
1	Blackish vase	8	8	3.23	0.61	0.038	15	12
2	Greyish sandy mud	17	9	1,375	0.59	0.163	10	16
3	Greyish clayey sand	25	8	1.15	0.28	0.05	20	24
4	Beige sandy clay	37	8	0.57	0.15	0.03	30	26
5	Blackish peat	39	2	0.236	0.13	0.012	10	15
6	Beige clayey sand	44	5	0.582	0.17	0.06	10	29
7	Coarse beige clayey sand	55	9	0.964	0.17	0.01	5	30

The saturation and non-saturation densities are respectively 26 and 24.6 kN/m^3, and the dilatance Ψ' is equal to 0.

In the continuation of this study, the results of the soil behavior analysis are presented. Particular attention is paid to a set of key parameters, including displacements.

– **Travel U_z**

A vertical displacement of the deflections is observed, concentrated at the levels where the load is applied (**Figure 6**). This vertical displacement is the settlement, the maximum value of which is obtained for the pile group is 27 mm. This maximum value is obtained at the level of the underconsolidated surface layers; it nevertheless remains lower than the maximum value recommended by Eurocode 7 (50 mm maximum) [50]. Indeed, in the overconsolidated layers at a depth of 45 m, this maximum value is reduced to 8.436 mm, which falls within the 5 to 10 mm range established by the researcher [51]. This margin is set at 15 mm by Denis in his study on the Impact of Support Settlements on the Design of Medium-Span Bridges [52].

– **Travel and U_xU_y**

Excessive horizontal displacement of the piles can lead to a redistribution of loads within the pile group. Some piles may then bear more load than others, which could lead to imbalance and progressive failure of the foundation system. **Figure 7** shows the horizontal displacements observed in the different layers, of soil around the group of piles.

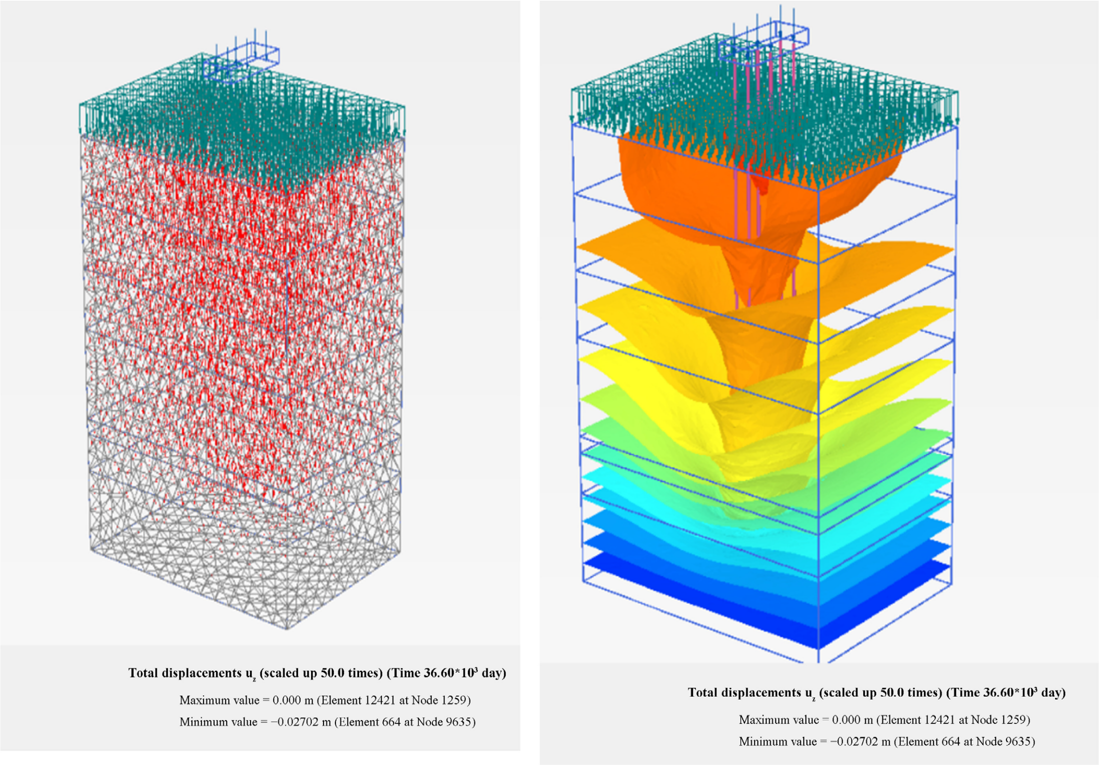

Figure 6. Total soil settlement under the effect of the loaded pile group.

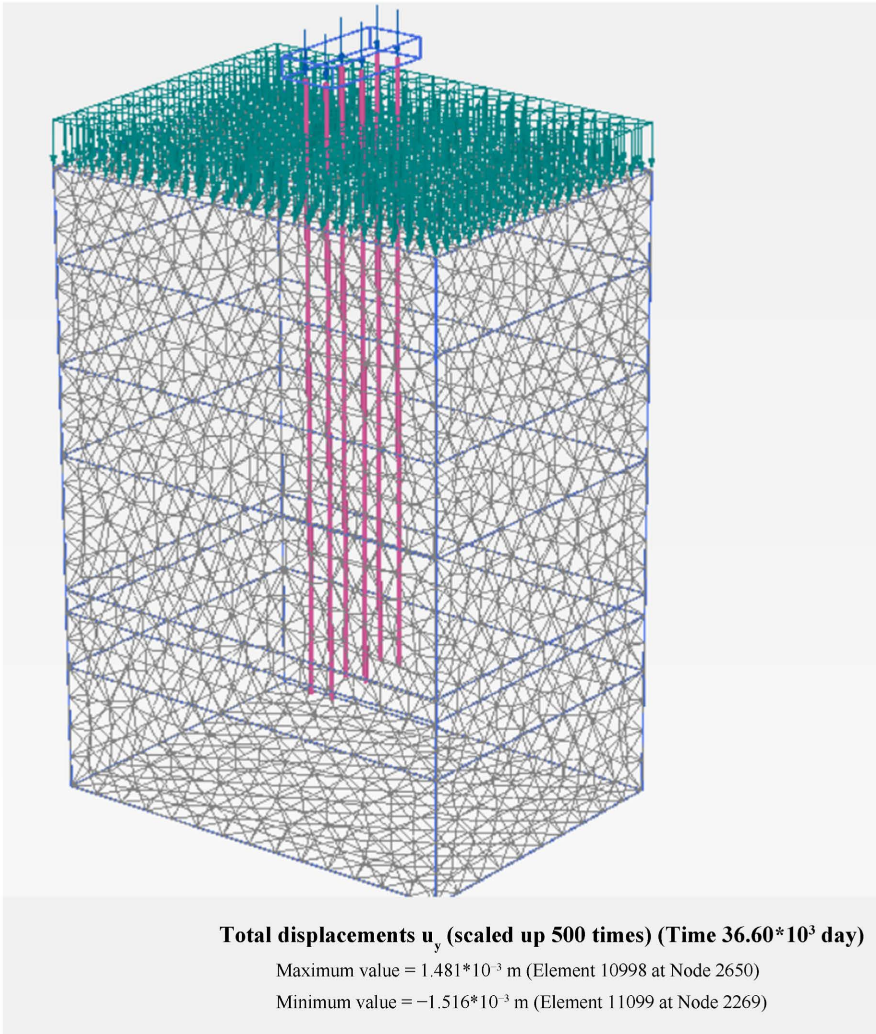

Figure 7. Horizontal displacements under the effect of the loaded pile group.

Along the horizontal plane, displacements are also observed, with a maximum value of 1.516 mm. This value remains below the acceptable value proposed by researcher David Remaud in 2021 (5 mm at 10% of the diameter) [53].

6. Conclusions

This research presents a numerical study of the behavior of deep bridge foundations on compressible soils. Geotechnical studies demonstrated the soil's inability to support the bridge with shallow foundations. These studies also classified the supporting soil as compressible, with the associated problems. Unable to improve its condition, we adopted a method using pile groups. In this case, considering the soil type and maintenance issues, we opted for driven diaphragm piles.

To determine the optimal dimensions for these pile groups, the GEOFOND software was used, based on the results of the pressuremeter test. The results of this sizing analysis led to the selection of piles with a diameter of 1 m and a depth of 45 m. These values, combined with a center-to-center spacing of 4.5 m between piles, demonstrate that the pile group has an efficiency coefficient of 83%.

Finally, the PLAXIS 3D software allowed us to understand the group's behavior under axial load. Based on settlement measurements, it was observed that the soil exhibited horizontal and vertical displacements, with the largest values obtained in the underconsolidated surface layers. Nevertheless, these values remain within the range recommended by various guidelines and recent research.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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Nomenclature

p_{LM}^{*} : Net limit pressure, MPa

E_m : Pressuremeter module, MPa

C_u : Undrained cohesion of the grains

Q_u : Limit load in compression or tension, MN

Q_{pu} : Peak load limit, MN

HAS: Cross-sectional area of the pile at the tip, m^2

Q_{su} : Lateral friction limit load, MN

q_{pu} : Peak limiting resistance, MPa

q_{si} : Unit lateral friction, MPa

C_e : Pile group efficiency coefficient

PK: Kilometer point, km

P_i : Perimeter of section i of the pile, m

e_i : Layer thickness i, m