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Swelling Soil Settlement Prevention by Stabilization with Quicklime Column in Diamniadio (Senegal)

Hamed Fall[1], Déthié Sarr[1], Ayoub Insa Correa[2], Modou Sarr[1], Seynabou Ndiaye[1]

[1]Département de Géotechnique, UFR Sciences de l'Ingénieur, Université Iba Der Thiam de Thiès, Thiès, Senegal
[2]Département de Génie Civil, UFR Sciences de l'Ingénieur, Université Iba Der Thiam de Thiès, Thiès, Senegal
Email: hamed.fall@univ-thie.sn

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Abstract

The risk of geotechnical drought, otherwise known as the risk of clay shrinkage and swelling, has long been identified by geotechnical engineers, and can be seen in many countries (USA, France, Canada, Ethiopia, etc.). In fact, clay soils show variations in volume when their water content varies. These variations affect the functioning of foundations and buildings in contact with the soil. This shrink-swell phenomenon is the cause of frequent disorders, which can range from a simple crack to considerable damage. The consequences are more spectacular in arid and semi-arid regions. In Senegal, this problem is of particular concern, especially in the context of the Diamniadio urban development project, which is based on geological formations prone to this phenomenon. However, it should be noted that the consequences of geotechnical drought on structures are conditioned by a range of factors of different kinds, which can be acted upon to prevent damage. One of these factors is the nature of the soil. Techniques for stabilizing soils by adding quicklime have been developed to deal with this risk. This article presents the reduction of settlements through the use of quicklime columns (from 1.85 mm to 1.054 mm). An analysis of the effect of column diameter and spacing on settlement is presented. The results show that settlement decreases as the columns are spaced closer together, giving a smaller settlement for a column spacing of 1 m.

Keywords

Settlement, Soil Swelling, Quicklime Column, Diamniadio

1. Introduction

Incessant population growth and urban expansion are driving the need for new infrastructure. However, intensive urbanization frequently leads to construction on problematic soils such as swelling clay soils, posing enormous geotechnical challenges. These soils are characterized by significant volumetric changes in response to moisture (known as shrink-swell), causing soil movements and deformations that are potentially damaging to structures. Shrink-swell of these swelling clay soils is a significant natural hazard. The phenomenon of shrink-swell, which generates differential settlement and damage to buildings, has been the subject of scientific research for many years. In France, for example, the damage caused by this phenomenon to buildings has been clearly identified since the 1990s. A number of studies have highlighted the scientific and technical concerns relating to the risk of geotechnical drought in the fields of geology, geotechnics, construction on clayey soils and risk prevention [1]. The mechanical consequences of the effects of drought on structures (buildings, engineering structures, etc.) are conditioned by several factors (nature of the soil, hydrogeological context, etc.) on which it is possible to act in order to prevent damage. For prevention, there are two possible solutions: adapting the building structure to soil movements, by taking into account practical constructive provisions, or adapting the soil to the structure by trying to modify its behavior with regard to swelling as noted [2]. The need to stabilize these soils is becoming crucial to guarantee the durability, safety and longevity of buildings. By understanding the complex particularities of swelling clay soils and developing appropriate stabilization solutions, the aim is to create a solid foundation for future construction. This research is part of the quest for sustainable and effective solutions to prevent the phenomenon of clay shrink-swell, which is considered a major geotechnical risk for structures built on these soils, requiring preventive measures to guarantee the durability and safety of constructions. Its main aim is to assess the impact of lime addition on the specific geotechnical parameters of the swelling clay soil at Diamniadio in Senegal. Consequently, the ultimate objective is to determine the most appropriate method for stabilizing this problematic soil type with lime, in order to demonstrate its usefulness for risk prevention.

2. Methods and Materials

2.1. Disorders Due to Geotechnical Drought Risk on Buildings

In the field of civil engineering, swelling and shrinkage phenomena are the cause of numerous disorders in both surface structures (buildings, surface foundations, retaining structures, embankments, etc.) and buried structures (tunnels, piles, pipes, deep foundations, etc.). There are many examples of disorders linked to the presence of swelling clays [3]. The disorders associated with foundation movement generally manifest themselves in the appearance of cracks on the house [4]. These cracks due to the shrink-swell phenomenon generally appear at the end of a dry period when the soil shows maximum settlement. They evolve according to climatic conditions: they enlarge during dry periods and may tend to close during wet periods.

2.2. Quicklime Stabilization Method

When swelling clay soils are subjected to variations in water content, they undergo major deformations known as shrink-swell phenomena, which can cause damage to structures built on them.

In general, there are two possible solutions for preventing building damage caused by the risk of geotechnical drought, as quoted [2].

- adapting the building structure to soil movements, by taking into account practical constructive measures.

- adapting the soil to the structure by trying to modify its behavior. This is called stabilization. The main purpose of soil stabilization is to alter the geotechnical properties of natural soils to meet specific engineering purposes. The soil's geotechnical properties can be improved by enhancing the shear strength parameters, increasing the tensile strength, and improving stiffness. It also aims to reduce plasticity, and minimize volumetric changes in expansive or fine-grained soils caused by moisture variability [5].

Stabilization techniques frequently used in civil engineering projects include the following: mechanical stabilization, thermal stabilization, chemical stabilization (using additives), stabilization by adding sand. In this work the chemical stabilization called lime stabilization is used. It is a well-established practice to utilize lime to enhance the engineering behavior of expansive clayey soils. Generally, fine-grained treated with exhibit decrease plasticity, enhancement of workability, and decrease volume variations characteristics. The three kinds of limes utilized to enhance the soil parameters include hydrated lime (calcium hydroxide-$Ca(OH)_2$, hydrated lime slurry, and quicklime (calcium oxide-CaO) [6]. **Figure 1** shows the chemical stabilization methods.

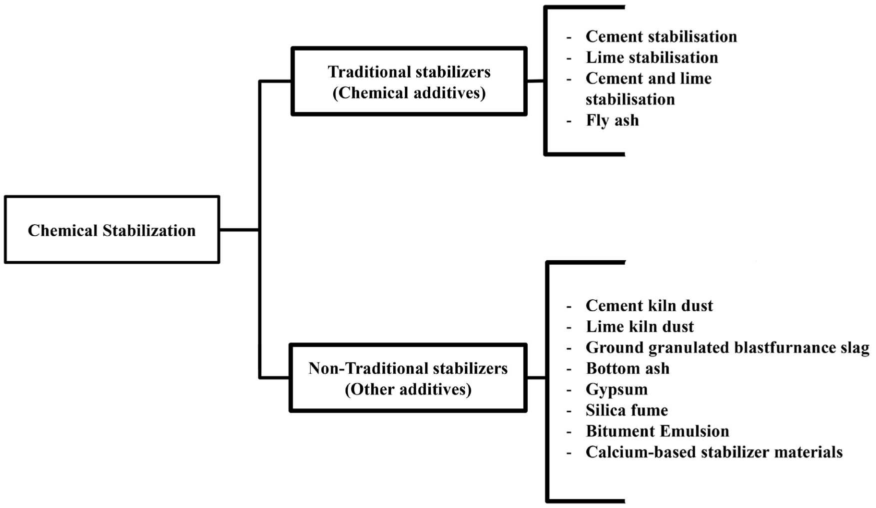


Figure 1. Chemical stabilizations methods [6].

The reaction diagram illustrates the process of stabilizing expansive soils using quicklime (calcium oxide, CaO). When quicklime is mixed with water (H_2O), it undergoes a reaction to form calcium hydroxide ($Ca(OH)_2$) a compound that is highly alkaline.

Quicklime is obtained by calcination given in Equation (1)

$$CaCO_3 + 50\ \text{kcal} \rightarrow CaO + CO_2 \tag{1}$$

It reacts in contact with water with a strong release of heat, then transforms into a white powder called slaked lime given in Equation (2)

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat} \tag{2}$$

In the presence of carbon dioxide, slaked lime can carbonate and become limestone again, according to this following chemical reaction following Equation (3)

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \tag{3}$$

Several methods are used to treat soils prone to swelling using lime. Among them, lime column technique. The lime column technique was introduced in Sweden in 1967 as a new type of foundation for light building (Breddenberg and Broms in [7]. The lime column technique has been applied successfully in recent years to improve the physical and mechanical properties of the soils [8]. Lime columns are constructed in-situ using a tool that functions like a giant "egg beater" (**Figure 2**). The principle is to screw down the tool into the soil in the required depth of the column. In the desired depth, the rotation of the tool is reversed and unslaked lime is forced under compression through openings placed just above the blades of the tool [8].

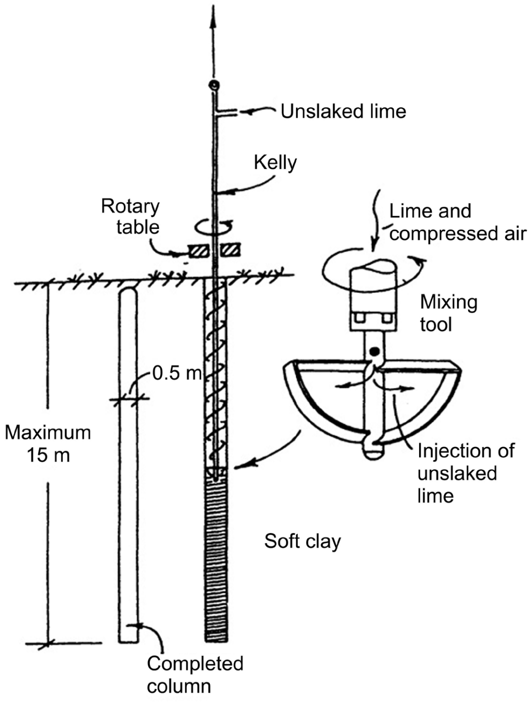


Figure 2. Manufacture of the lime columns [9].

3. Results and Discussions

3.1. Area Study Presentation

The material is extracted in Diamniadio, a town located approximately 30 km southeast of Dakar, more precisely in the department of Rufisque. The site is located next to the Diamniadio soccer field. The samples were collected using hand-dug wells approximately 1 m deep. Three (03) wells were dug and distributed around the site around the reference point: Latitude: 14.738931, Longitude: −17.232519 (**Figure 3**).

Figure 3. Aerial view of the sampling site (Google Earth Pro).

Figure 4 shows experimental procedure for soil stabilization.

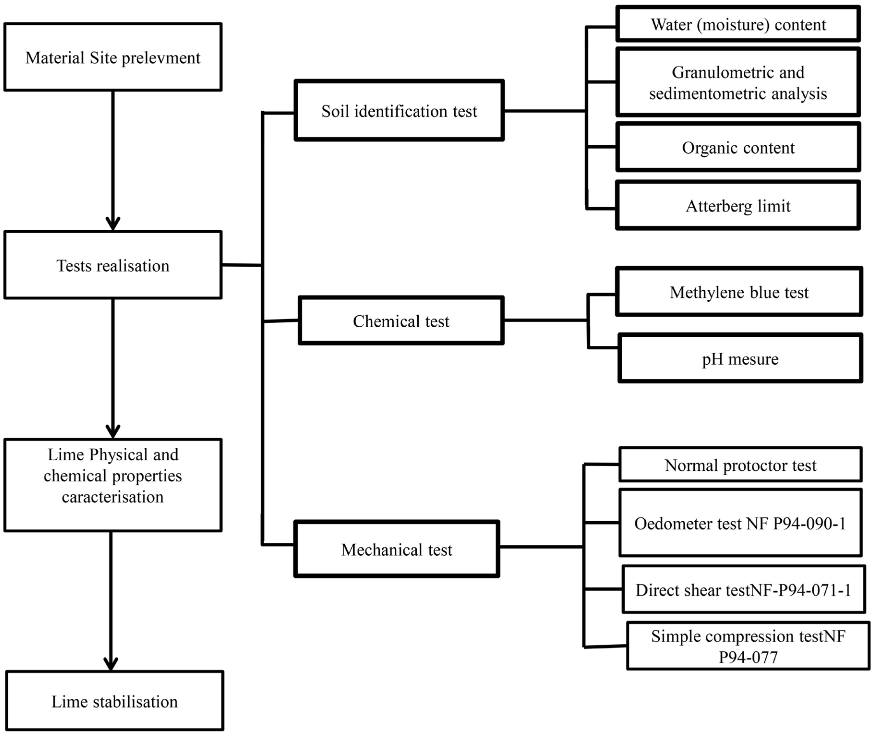


Figure 4. Soil stabilization process.

3.2. Soil Identification Test

Physical, Mechanical and chemical tests were carried out on the sample in order to determine the characteristics of the soil.

To determine the water (moisture) content w defined by Equation (4), three samples were used and Table 1 shows the water content average.

Table 1. Soil water (moisture) content.

Sample	1	2	3
w	10.32	15.79	12.95
$w_{average}$		13.02	

Where m_w and m_d are respectively the water mass evaporated during drying and the solid grains mass

$$w = m_w / m_d \tag{4}$$

The particle size analysis by sieving was carried out using the wet method in accordance with standard NF P 94-056, and the sedimentological analysis was carried out in accordance with standard NF P 94-057 (Figure 5). Table 2 shows C_u (uniformity coefficient) and C_c (curvature coefficient) values derived from the particle size distribution curve.

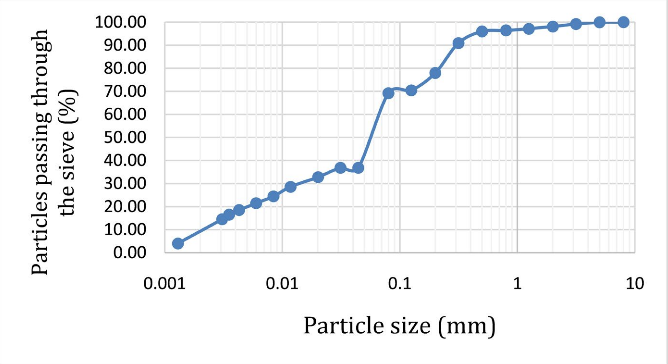


Figure 5. Granulometric curve.

Table 2. Uniformity and curvature coefficients.

C_u	C_c
30.43	1.40

In accordance with standard NF P 94-051, tests to determine the Atterberg limits were carried out, giving the values shown in Table 3.

Table 3. Atterberg limits values.

Liquid limit W_l	**51.1**
Plasticity limit W_P	**23.6**
Plasticity index $I_P = W_l - W_P$	**27.4**

With a plasticity index I_p of 27.4 between 15 and 40, the soil sample is considered plastic. The test according to standard NF P94-054 determined the specific weight γ_s to be 2.6 g/ml, which corresponds to clay soil. With regard to chemical tests, standard NF P 94-048 enabled the VBS methylene blue value to be calculated from two tests (**Table 4**).

Table 4. Methylene blue value.

Test No.	1	2
Methylene blue value	7.86	7.65
VBS average	**7.75**	

A pH value of 8.15 was determined using a pH meter in the chemistry department laboratory at Alioune Diop University in Bambey (Senegal).

The oedometer test according to standard NF P94-090-1 was also performed in order to plot the compressibility curve of untreated clay (**Figure 6**). This curve corresponds to the evolution of the void ratio as a function of the vertical stress applied to the clay.

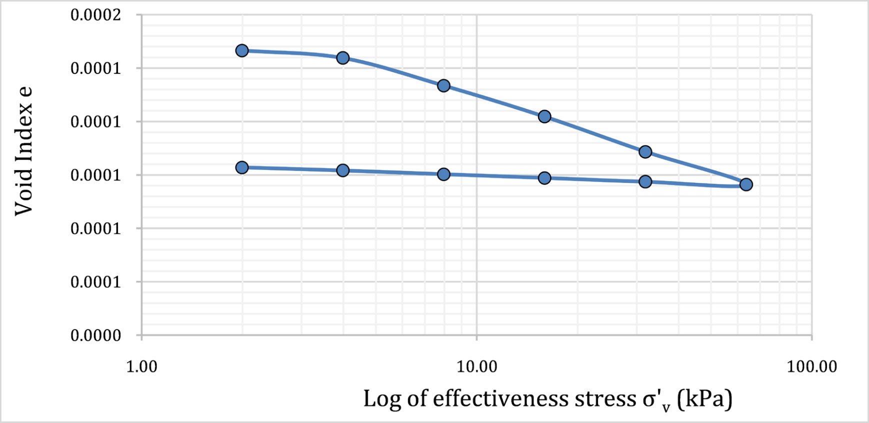


Figure 6. Compressibility curve of untreated clay.

Thanks to the compressibility curve, the compression index C_c defined by the equation and the swelling index C_G, were determined

$$C_c = (-\Delta e)/\Delta(\log \sigma'_z) \tag{5}$$

By analyzing the compressibility curve, we observe that the swelling index of the clay is 0.038. As C_G is lower than 0.05, we can conclude that the clay has low swelling properties. Furthermore, with a C_c value of 0.386, the clay in question is characterized by high compressibility.

With regard to mechanical testing, the direct shear test in accordance with standard NF P94-071-1 was used to evaluate the shear strength of soils, focusing mainly on cohesion and friction angle. **Figure 7** shows the Mohr-Coulomb diagram, which indicates that the clay has a cohesion of 27.495 kPa and a friction angle of 7.744.

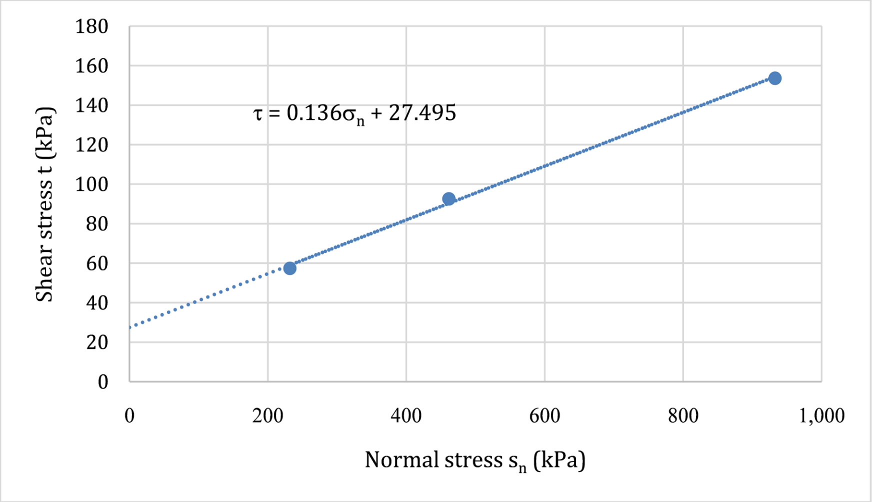


Figure 7. Mohr Coulomb curve.

The simple compression test according to standard NF P94-077 was used to determine the simple compressive strength using Equation (6), which corresponds to a value of 0.69 MPa.

$$R_C = F_{\max} / S \tag{6}$$

The material was treated with lime, whose chemical and physical characteristics are given in Table 5 and Table 6, respectively.

Table 5. Lime chemical analysis.

CaO total	CaO available	CO_2	MgO	SiO_2	Al_2O_3	Fe_2O_3	S
94.60%	90.20%	3.30%	0.50%	0.70%	0.35%	0.20%	0.10%

Table 6. Lime physical properties.

Particles passing through the sieve	0.163 mm	35%
	2 mm	90%
	3.15 mm	98%
Reactivity T_{60}	3.1 mm	
Density	$0.9 < d < 1$ g/ml	

3.3. Soil Stabilization

This part of the paper present the results of soil stabilization regarding to the percentage of used lime.

3.3.1. Influence of Lime on pH Value

The pH value of the Diamniadio clay studied in this article, which is initially

8.15, increases after the gradual addition of lime to a value of 12.27 corresponding to 2% lime. For a lime percentage above 2%, the pH value stabilizes despite the continuous addition of lime (**Figure 8**). Thus, we can conclude that the initial percentage for stabilizing Diamniadio clay soil is 2%. The starting dosage of 2% is retained because it is at this threshold that the pH reaches approximately 12.3 and stabilizes, conditioning pozzolanic activation, with a simultaneous decrease in Atterberg limits (and therefore plasticity) and swelling indicators.

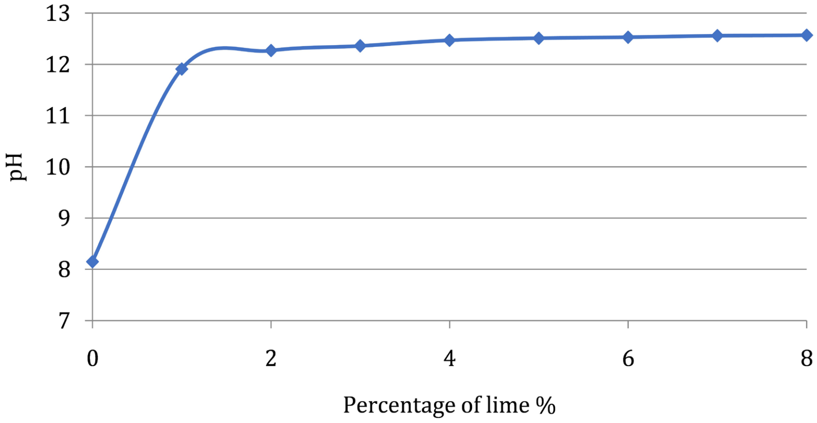


Figure 8. Influence of lime on pH.

3.3.2. Influence of Lime on Atterberg Limits

Figure 9 shows a decrease in the liquidity limit from 51.1% for untreated clay to 40.7% when the proportion of lime reaches 8%. The curve shown in **Figure 10** illustrates the change in the plasticity limit as lime is added. The curve shows an increase in the plasticity limit as the percentage of lime increases, from 23.6% in the absence of lime to 36.9% when the percentage of lime reaches 8%.

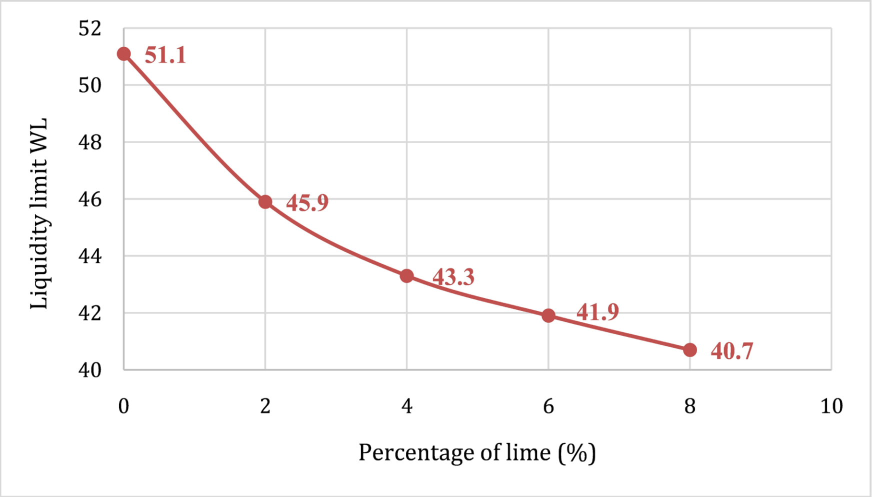


Figure 9. Influence of lime on the liquidity limit.

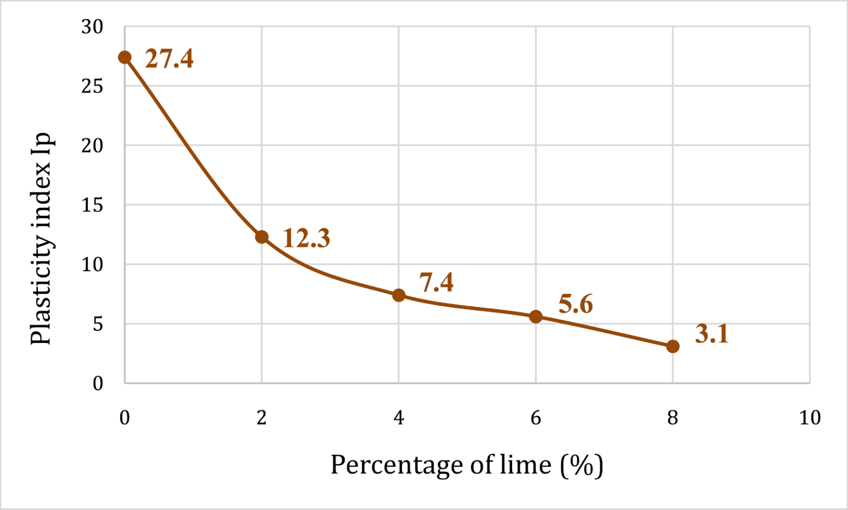


Figure 10. Influence of lime on the plasticity index.

The curve shows a reduction in the plasticity index as the percentage of lime increases, from 27.4% without lime to 3.1% when the lime content reaches 8%.

3.3.3. Influence of Lime on the Swelling Index

As for the swelling coefficient, it decreases as the percentage of lime increases, reaching a minimum of 0.0094 at 4% lime. Above this proportion, the swelling potential begins to increase again in proportion to the addition of lime (Figure 11).

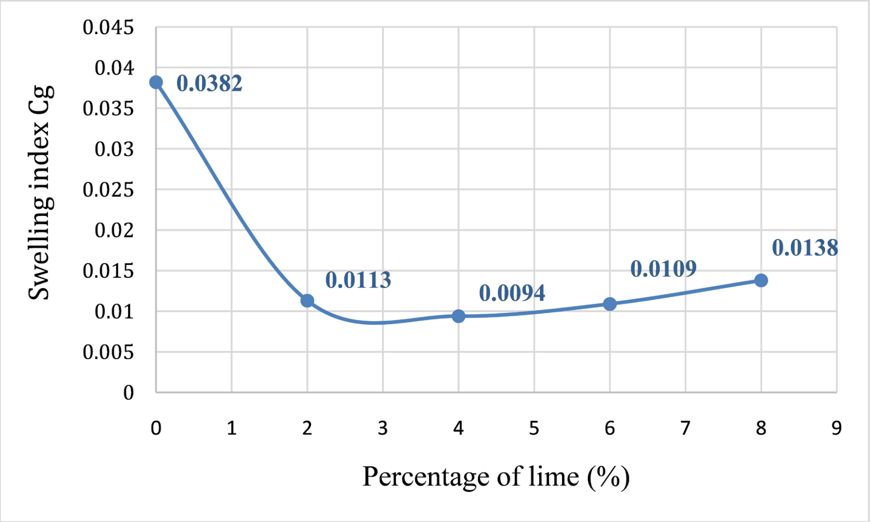


Figure 11. Influence of lime on the swelling index.

3.3.4. Influence of Lime on Cohesion and Friction Angle

Figure 12 shows the evolution of undrained cohesion as a function of the lime addition rate. These results show an improvement in cohesion until the percentage of lime added reaches 4%. However, above this concentration, cohesion decreases.

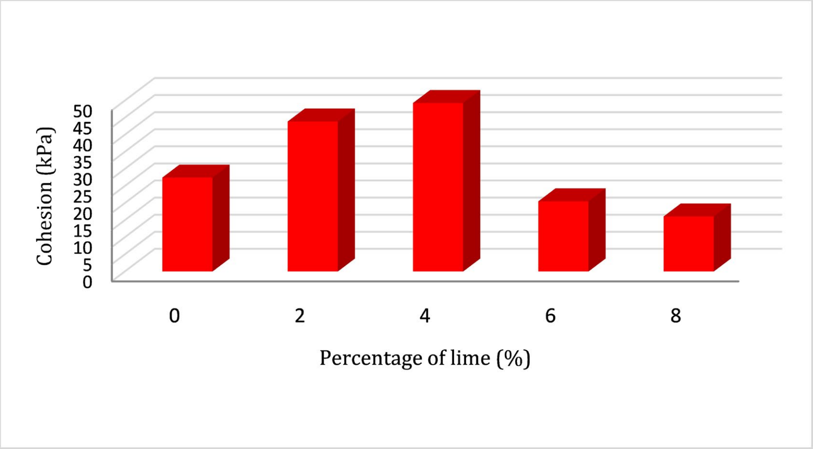


Figure 12. Influence of lime on cohesion.

Figure 13 shows the evolution of the internal friction angle as a function of the increase in the rate of lime addition.

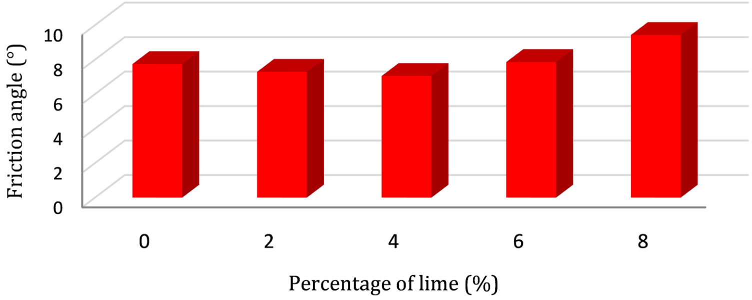


Figure 13. Influence of lime on friction angle.

3.3.5. Influence of Lime on Compressive Strength

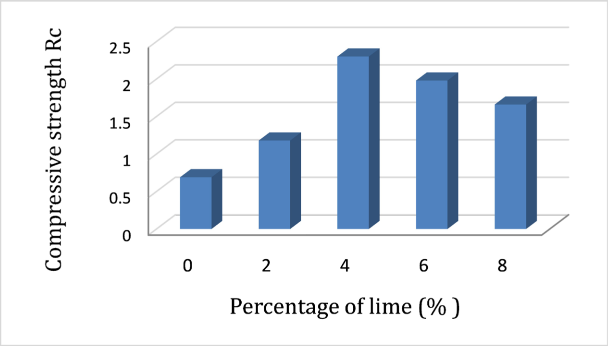


Figure 14. Influence of lime on compressive strength.

Figure 14 illustrates the significant change in tensile strength following the addition of lime during a 28-day curing period. The observation shows that incorporating lime into clay soils makes them more resistant to compression. By gradually increasing the percentage of lime in the mixture (lime and clay), increased resistance is observed up to a threshold of 4%.

3.4. Settlement Prevention with Quicklime Column

This section of the article presents a numerical analysis of the stabilization of the swelling clay soil in Diamniadio using lime columns to reduce settlement. PLAXIS 3D CE V20 was used.

Based on the lithology of the Diamniadio site, we observe a layer of black clay on the surface, followed by a layer of marl-limestone at depth (Figure 15). Therefore, for modeling purposes, the HSM model for the clay and marl-limestone layer was used to account for the swelling effect. Table 7 and Table 8 show the parameters for the clay and marl layers, respectively.

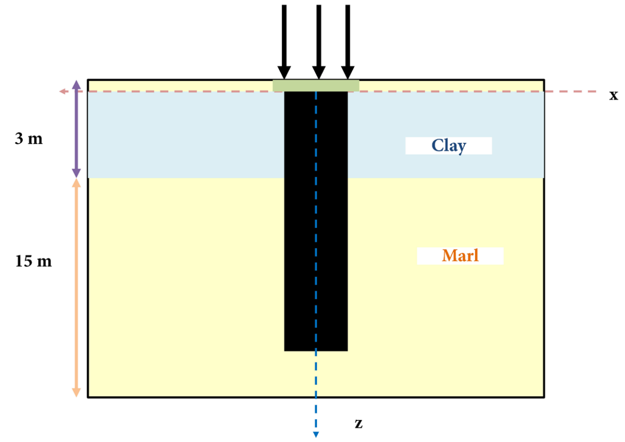


Figure 15. Model geometry.

Table 7. Clay HSM model parameters.

C_c	C_s
0.386	0.045

E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	E_{ur}^{ref} (kPa)	φ	c (kPa)	ν_{ur}	K_0^{NC}	Υ_h (KN/m^3)	Υ_d (KN/m^3)	$k_x = k_y$ (m/s)
8.832×10^5	9.81×10^5	3.533×10^6	7.74	27.49	0.2	0.865	20.69	18.15	6.834×10^{-10}

Table 8. Marl HSM model parameters.

C_c	C_s
0.026	0.012

E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	E_{ur}^{ref} (kPa)	φ	c (kPa)	ν_{ur}	K_0^{NC}	Υ_h (KN/m^3)	Υ_d (KN/m^3)	$k_x = k_y$ (m/s)
2.42×10^5	9.98×10^4	7.26×10^5	19	45	0.2	0.293	16.6	11.3	6.834×10^{-10}

E_{ur}^{ref} the unloading/reloading elastic modulus (by default $E_{ur}^{ref} = 3E_{50}^{ref}$), E_{50}^{ref}: the reference secant modulus in a triaxial situation, E_{oed}^{ref} the reference tangent modulus under edometric stress, C: cohesion, φ: friction angle et, R_f le ratio de rupture (par défaut R_f = 0.9), ν_{ur}: elastic Poisson's ratio (by defaut ν_{ur} = 0.2), K_0^{NC}: K_0-consolidation (by defaut $K_0^{NC} = 1 - \sin\phi$).

For this study, a circular column with a diameter *B* of 0.8 m, a length *L* of 12 m, and a thickness *E* of 1.2 m was used. Table 9 shows the Mohr Coulomb parameters for the column.

Table 9. Mohr Coulomb parameters for the column.

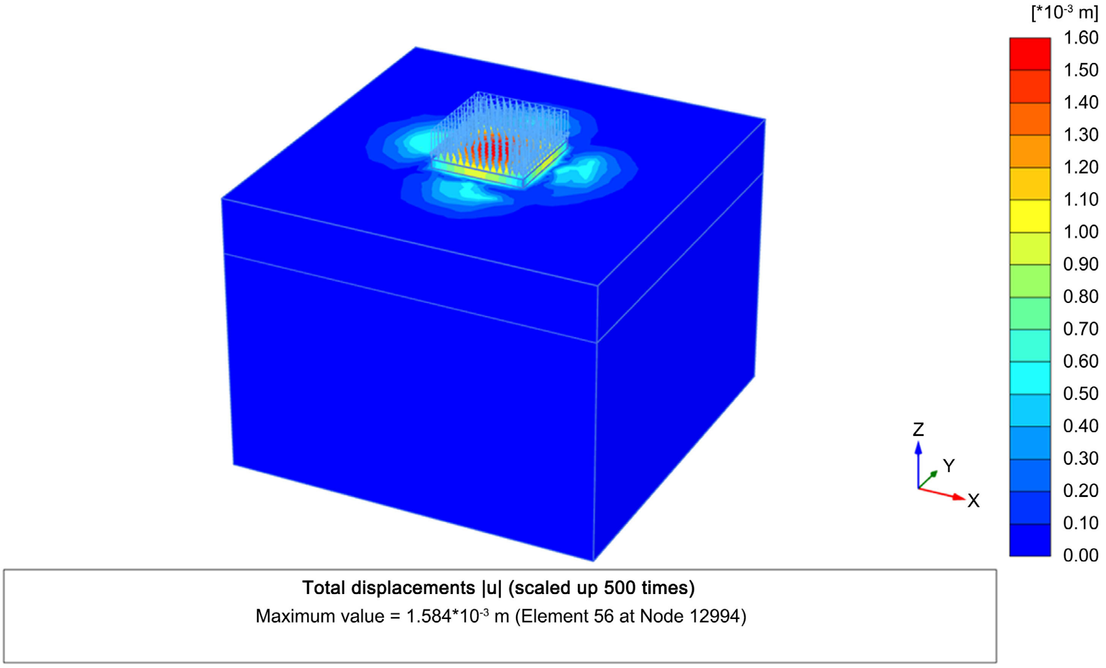
E_{50} (kPa)	Φ	*c* (kPa)	*ν*	$Υ_h$ (KN/m^3)	$Υ_d$ (KN/m^3)	$k_x = k_y$ (m/s)
2.944.10^6	7.051	49.31	0.2	20.26	16.8	0

The foundation in this study is a concrete raft slab with a length *L* of 2.5 m, a width l of 2.5 m, and a thickness *E* of 0.45 m, a long-term modulus of elasticity of concrete $E_b = 10$ GPa, a Poisson's ratio $\upsilon_p = 0.15$, and a density of concrete $\gamma_d = 24$ kN/m^3. The applied load is a surface load of 55.9 kN/m resulting from the descent of loads from a four-story building.

As part of this study, four calculation stages were taken into account:

- Phase 1: corresponding to the application of a load on the foundation without swelling
- Phase 2: corresponding to the application of a load on the foundation with a column without swelling
- Phase 3: corresponding to the application of a load on the foundation with swelling and a group of columns without swelling
- Phase 4: corresponding to the application of a load on the foundation with swelling

Soil treatment with lime columns indicates that the use of lime reduces soil settlement. Less settlement is observed when columns are installed compared to the footing alone under load, as shown in Figures 16-18.



Total displacements |u| (scaled up 500 times)
Maximum value = 1.584*10^{-3} m (Element 56 at Node 12994)

Figure 16. Settlement of the foundation in phase 1.

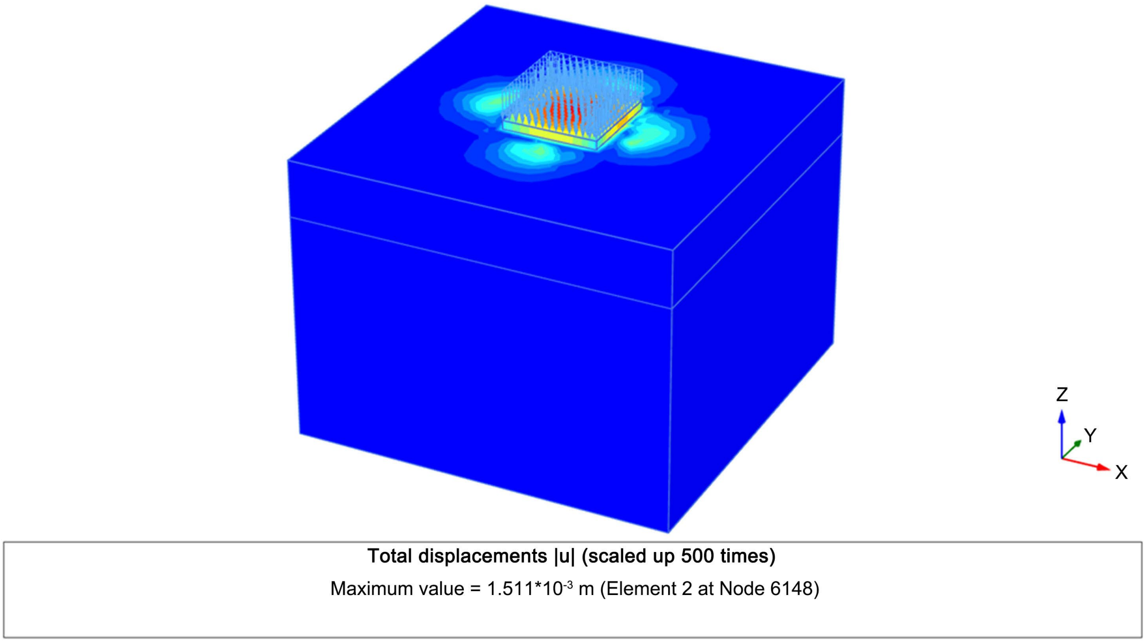


Figure 17. Settlement of the foundation with a column.

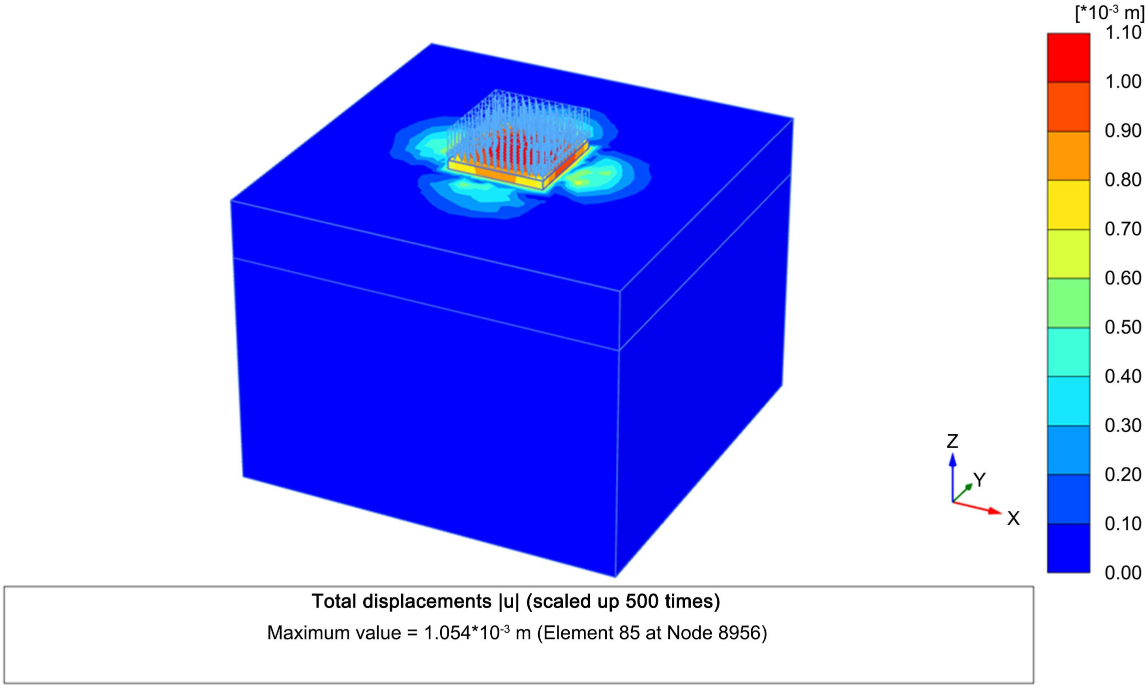


Figure 18. Settlement of the foundation with a group of columns.

The obtained results are summarized in Table 10. We can see how the use of lime column impacted the settlement of the foundation when the swelling is not taken account (settlement from 1.584 mm to 1.511 mm) and also when the swelling is taken account.

The impact of column diameter on settlement was analyzed. The results obtained are shown in Figure 19.

Table 10. Summary of results.

Phases		Settlement (mm)
Foundation subjected to loading without soil swelling	without the use columns	1.584
	with the use of a column	1.511
Foundation subjected to loading with soil swelling	with the use of a column	1.322
	with the use of a group of column	1.054

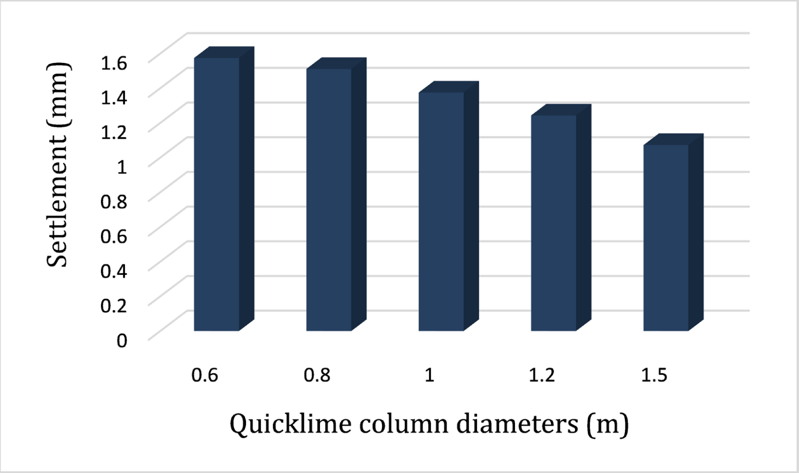


Figure 19. Influence of column diameter on settlement.

A study of the influence of spacing on settlement between columns was also carried out. The results obtained are presented in **Table 11**.

Table 11. Settlement in different cases of spacing between columns.

Spacing between columns (m)	1	1.2	1.5
Settlement (mm)	0.844	1.054	1.107

The results indicate that settlement decreases as the columns get closer together, resulting in less settlement when the spacing between columns is 1 m.

3.5. Discussions

The numerical analysis carried out on the stabilization of the swelling clay soil in Diamniadio using lime columns demonstrates the beneficial effect of lime on reducing soil settlement, which decreases from 1.85 mm for the base on the swelling soil to 1.054 mm after treating the soil with lime columns. Following the application of volumetric deformation to integrate swelling, settlement decreased from 1.511 mm to 1.322 mm. This reduction in settlement is due to the swelling of the clay soil, which was reduced by the lime column.

4. Conclusion

The action of lime columns on the surrounding soil begins with a consolidation and dehydration process in which quicklime (CaO) absorbs moisture from the surrounding soil, causing the lime to swell and form slaked lime. After this initial consolidation phase, an ion exchange process takes place: Ca^{2+} ions from the slaked lime are absorbed by the clay surface, promoting the binding of clay particles and resulting in a significant increase in shear strength. Finally, pozzolanic reactions occur and contribute to increasing long-term shear strength. In addition, it has been observed that when the diameter of the columns is large and the spacing between them is reduced, settlement decreases further.

Conflicts of Interest

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

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