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Experimental evaluation of soft soil compaction process using Cement - Fly Ash - Marine Sand piles in Vietnam

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Abstract: *Cement fly ash marine sand (CFMS) piles are a new solution for improving soft soils in Vietnam, utilizing marine sand and fly ash to address riverbank erosion and environmental challenges caused by river sand extraction and waste from thermal power plants. Evaluating the effectiveness of this solution is essential for practical implementation. This study analyzes the theoretical aspects of soil compaction process and presents the results of field experimental evaluation. A square network of CFMS piles was constructed at Lai Cach Industrial Park, Hai Duong Province, Vietnam to evaluate the actual effectiveness of soil reinforcement for two types of soft soil: clay and sandy clay, both in their soft state. The CFMS piles, with a diameter of 300mm and a depth of 7.0m, were divided into three groups based on varying adhesive content (ADH) and the fly ash (FM)/cement (CM) ratio: Group 1 (ADH of 5% and FM/CM of 0%), Group 2 (ADH of 10% and FM/CM of 10%), Group 3 (ADH of 15% and FM/CM of 20%). Theoretical analysis indicated that two compaction processes occurred in soft soils: instantaneous compaction and time-dependent compaction. The effectiveness of the soft soil reinforcement was evaluated through laboratory tests on soil samples taken before and immediately after reinforcement. The results revealed the following changes: the moisture content decreased by 2.39% in clay and 7.07% in sandy clay, the unit weight increased by 1.04% in sandy clay and 1.06% in clay, the void ratio decreased by 2.22% in clay and 6.58% in sandy clay, the coefficient of compressibility decreased by 2.86% in clay and 3.13% in sandy clay. Dynamic penetration tests (DCP) were conducted at three different stages: before reinforcement, immediately after reinforcement and 10 days after reinforcement. The results indicated that the dynamic penetration resistance (q_d) increased by 9.8% in sandy clay and by 14.8% in clay immediately after reinforcement; after 10 days of reinforcement, q_d in Group 1 (q_{d-10-1}) increased by 19.3% in sandy clay and by 27.7% in clay; in Group 2 (q_{d-10-2}), it increased by 52.3% in sandy clay and by 47.9% in clay; and in Group 3 (q_{d-10-3}), q_d increased by 67.0% in sandy clay and by 59.1% in clay.*

Keywords: *cement - fly ash - marine sand pile; dynamic cone penetration test; soil compaction; soft soil reinforcement*

1. Introduction

With economic growth, construction activities have been rapidly advancing in Vietnam in recent years. However, this development is encountering various challenges. Among the most significant is the depletion of traditional construction materials and the difficulty of constructing works on soft soils. River sand resources are becoming increasingly scarce due to the construction of hydroelectric dams upstream, which has severally reduced alluvial sources in rivers. Furthermore, river sand extraction is causing environmental issues such as riverbank erosion and disrupting local planning efforts. In Vietnam, soft soil is widely distributed, especially in the Red River and Mekong Delta regions. Therefore, many infrastructure projects rely on soft soil reinforcement solutions. Several methods are being applied in Vietnam for soft soil improvement, such as prefabricated vertical drain, sand compaction piles, soil-cement columns (Diu et al., 2020) [3]; Trung and Young (2013) [10]; Thinh (2021) [11]).

Recently, some innovative solutions for soft soil improvement have emerged, focusing on replacing river sand and utilizing waste such as fly ash, bottom ash, rice husk ash. These solutions include mixing soft soil with cement and rice husk ash (Tham and Toan, 2008 [9]), combining soft soil with cement and lime (Hue, 2009 [5]), and using cement - fly ash - marine sand piles (Thinh, 2021 [11]). Among these methods, the cement - fly ash - marine sand (CFMS) pile is a new approach that has been proposed and is currently being researched and applied.

Soft soil reinforcement aims to enhance the stability and reduce settlement of the ground. The

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parameters of the ground improved by CFMS piles depend on both the properties of the soft soils and the CFMS piles themselves. The mechanical properties of soft soils are often related to factors such as the void ratio and particles size distribution of the soils. Meanwhile, the stability and settlement of the foundation are affected by the soil properties, as well as the strength of the piles. The process of increasing pile strength is evaluated through strength tests on CFMS pile samples conducted in the laboratory (Thinh, 2021 [11]; Dung et al., 2021 [4]). Therefore, the effectiveness of reducing settlement and enhancing the strength of soft soils should be evaluated based on construction of CFMS test piles, along with laboratory and in-situ tests.

In Vietnam, common in-situ testing methods used to evaluate the effectiveness of soft soil reinforcement solutions include the static penetration test (CPT), standard penetration test (SPT), and vane shear test (FVT). However, the execution of these tests in the field can significantly affect the soft soils due to counter load anchoring (with CPT) and drilling operations (with SPT, FVT), which can reduce their reliability. This paper presents the theoretical basis and results of evaluating the effectiveness of soft soil reinforcement using CFMS piles through laboratory tests and dynamic penetration tests (DCP) conducted in the field.

2. Theoretical basis

2.1. CFMS pile strengthening process

CFMS piles consist of a mixture of marine sand, cement, and fly ash in a dry state. During construction, these materials absorb water from the soft soils to form CFMS mortar, which then hardens over time. The hardening process of CFMS mortar increases the strength of the CFMS piles. The process is divided into two stages: the setting stage and the solidification stage. During the setting stage (first stage), the cement-fly ash mortar gradually loses its plasticity and thickens, but it has not yet developed significant strength. In the solidification stage (second stage), the hydration of the mineral components in the clinker occurs, primarily involving the following compounds: tricalcium silicate(C3S, $3CaO.SiO_2$), which accounts for 37% - 60%; dicalcium silicate (C2S, $2CaO.SiO_2$), which accounts for 15% - 37%; tricalcium aluminate (C3A, $3CaO.Al_2O_3$), which accounts for 10% - 18%; and tetracalcium aluminoferrite (C4AF, $4CaO.Al_2O_3.Fe_2O_3$), which accounts for 7% - 15%. In addition, there are several minor components, such as $5CaO.Al_2O_3$; $2CaO.Fe_2O_3$; MgO<4%; CaO<0.5%; SO_3<3.5% [9]. The chemical composition of fly ash includes approximately: 54.2% SiO_2, 23.3% Al_2O_3, 9.8% Fe_2O_3, 1.2% CaO, and 2.5% SO_3. The hydration process of the mineral components of the clinker and fly ash is represented by the following reactions:

$$3CaO.SiO_2 + nH_2O \rightarrow Ca(OH)_2 + 2CaO.SiO_2.(n-1)H_2O$$

$$2CaO.SiO_2 + mH_2O \rightarrow 2CaO.SiO_2.mH_2O$$

$$3CaO.Al_2O_3 + 6H_2O \rightarrow 3CaO.Al_2O_3.6H_2O$$

$$4CaO.Al_2O_3.Fe_2O_3 + nH_2O \rightarrow 3CaO.Al_2O_3.6H_2O + CaO.Fe_2O_3.mH_2O$$

Tricalcium silicate (C3S, $3CaO.SiO_2$) is the most important component of hardening process due to its large proportion, high strength, rapid harden, and significant heat release. Tricalcium aluminate (C3A, $3CaO.Al_2O_3$) also hardens quickly in the early stages but develops low strength, releases the most heat, and is prone to causing cracking.

The solidification process of CFMS mortar can be divided into three stages:

1) Dissolution stage: During this stage, when cement and fly ash react with water, a chemical reaction between cement, fly ash and water occurs at the surface of the cement particles and fly ash. This reaction leads to the formation of new compounds that are soluble in water, such as $Ca(OH)_2$ and $3CaO.Al_2O_3.6H_2O$. These compounds immediately dissolve, creating a liquid that surrounds the surface of the cement particles and fly ash.

2) Colloidalization stage: During this stage, the newly formed $Ca(OH)_2$ and $3CaO.Al_2O_3.6H_2O$ compound from the hydration process becomes insoluble and exist in colloidal form. The insoluble dicalcium silicate (C2S, $2CaO.SiO_2$) will separate into small, dispersed particles within the solution, form dispersed colloids. As more colloids are generated, the relatively small dispersed colloidal particles begin to aggregate into larger colloidal particles, creating a viscous mixture. This causes the cement and fly ash to gradually lose their plasticity and solidify, though they do not yet develop significant strength.

3) Crystallization stage: During this stage, the compounds of $Ca(OH)_2$ and $3CaO.Al_2O_3.6H_2O$ transition from a colloidal to a crystalline form. The small crystals intertwine with each other, causing the cement and fly ash mixture to begin gaining strength. The compound $2CaO.SiO_2.mH_2O$ remains in a colloidal form for an extended period, transforming partially into crystals. As water continues to be lost, the colloid gradually dries, tightens and solidifies.

The stages of dissolution, colloidalization and crystallization do not occur independently; rather they happen simultaneously and alternately. In addition, the carbonation process also plays a role in the solidification of the cement.

2.2. The process of compacting soft soil

The CFMS pile construction method involves using a specialized device to press into the ground, creating holes, and then filling them with a dry CFMS mixture to form circular-section piles. This process causes the CFMS pile to replace the voids in soft soils, with water and air escaping from these voids due to pressure applied during construction. As a result, the volume of voids in the soil decreases, leading to soil compaction.

The nature of this compaction process is illustrated in Fig. 1a. In this method, four CFMS piles are constructed in a square grid with a distance of D between them. Each pile has a diameter of d, a depth of L in the ground, so the volume of a single pile in the ground is given by the formula $V_p = L\pi d^2/4$. Consider a soil mass with a square cross-section bounded by four piles. The initial volume of the soil mass is given by $V_o = L.D^2$. After constructing four CFMS piles, the volume of the soil mass decreases due to the volume occupied by the piles. The volume after constructing the piles is V_r, as shown in Fig. 1a. The volume of soil occupied by the pile is V_p (equal to the volume of one pile), which is yielded:

$$V_p = V_o - V_r \qquad (1)$$

In this soil mass, consider a soil element with a solid particle volume of 1 unit. The volume of the voids is represented by the void ratio of the soil (e_o). Therefore, the initial total volume of the soil element is ($1+e_o$). After the construction of the CFMS piles, the void ratio of the soil changes to e_r (as shown in Fig. 1a). The reduction in the void ratio due to the pile construction is given by:

$$\Delta e = e_o - e_r \qquad (2)$$

The relative volume change of the soil mass (Λ_s) is:

$$\Lambda_p = \frac{V_p}{V_o} = \frac{V_o - V_r}{V_o} = \frac{\Delta e}{e_o} = \frac{e_o - e_r}{e_o} = \frac{\pi d^2}{4D^2} \qquad (3)$$

From Eq. (3), the void ratio of the soil immediately after the compaction process due to construction is determined by the equation:

$$e_r = e_o\left(1 - \Lambda_p\right) = e_o\left(1 - \frac{\pi d^2}{4D^2}\right) \qquad (4)$$

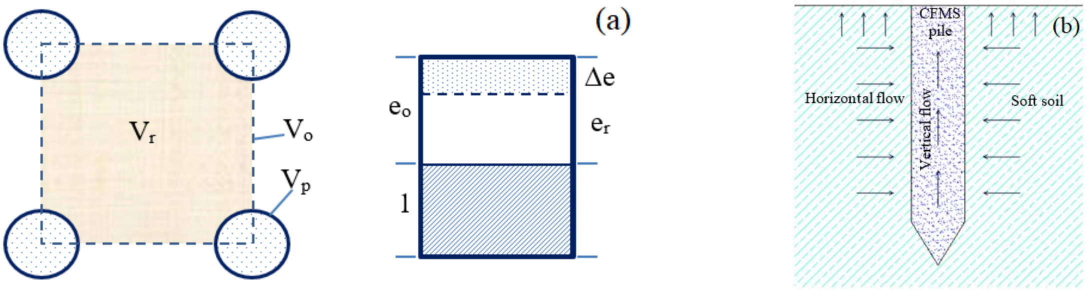


Fig. 1. (a) Diagram of soil compaction principle using CFMS pile; (b) Diagram of pore drainage in soft soil after CFMS pile construction

Eq. (4) indicates that the void ratio of the soil immediately after construction is influenced by the pile diameter d and the distance between piles D. In other words, the soil compaction process during pile construction depends on the ratio of the space occupied by the piles in the soil volume ($\Lambda_s = V_p/V_o$), which is determined by d and D. From the value of e_r, the soil compaction can be calculated using the compression curve.

The reduction in pore volume can be divided into two processes: the first occurs rapidly during construction, referred to as the instantaneous compaction process, and the second is closely related to the pore drainage process within the soil, similar to the consolidation process when vertical drainage is used (Fig. 1b). Since the pore drainage process in soft soil takes a long time, the soil compaction continues to gradually increases over time. When CFMS piles are used as an alternative to vertical drainage, the ADH content in the pile accelerates the consolidation process of soft soil (due to the amount of water involved in the chemical reaction), and the extent of this acceleration depends on the ADH content. Predicting the effectiveness of both instantaneous compaction and consolidated compaction is highly complex and requires experimental research to analyze the effects of each process.

3. Experimental study

3.1. CFMS strength enhancement evaluation

Testing materials

The strength of FCMS piles is influenced by the content of cement and fly ash. To assess the impact of cement and fly ash content on the strength of FCMS piles, material samples with varying cement and fly ash contents were prepared and tested in the laboratory.

The materials used in the testing include marine sand, cement, fly ash and water.

1) Cement: The cement used in the experiment was PCB40 But Son cement, which meets the technical specifications outlined in the TCVN 6260:2009 standard [6].

2) Marine sand: The sand used in the experiment was Hai Phong marine sand, collected from a completely saline land area. It is a fine-grained sand with a very low fineness modulus of 0.3. Most of the grains have a size smaller than 0.315 mm, with 74% of the grains being smaller than 0.014 mm. The technical characteristics of the marine sand used in the study are summarized in Table 1.

3) Fly ash: The fly ash used in the experiment was unselected fly ash from Formosa Thermal Power Plant, which meets the requirements of TCVN 10302:2014 [8] and ASTM C618-22 [1]. The fly ash meets the criteria of type F fly ash. The chemical composition, as well as the physical and mechanical properties of the Formosa fly ash, are presented in Tables 2 and 3.

Tab. 1. The technical characteristics of the Hai Phong marine sand

Property	Value
Specific gravity (g/cm^3)	2.48
Dry density (g/cm^3)	2.26
Saturated bulk density (g/cm^3)	2.35
Minimum bulk density (g/cm^3)	1.35
24-hour water absorption (%)	3.60
Percentage of grains smaller than 0.002mm (%)	9.20
Percentage of grains smaller than 0.075mm (%)	21.60

Tab. 2. The chemical composition of Formosa fly ash

Material	SiO_2	Al_2O_3	Fe_2O_3	SO_3	K_2O	Na_2O	MgO	CaO	P_2O_5	Lost on cal.
Fly ash	54.2	23.3	9.8	2.5	1.4	1.1	0.6	1.2	1.4	4.5
Cement	36.3	4.4	5.4	3.4	1.2	0.3	2.5	60.2	-	2.2

Tab. 3. The physical and mechanical properties of Formosa fly ash

Specific gravity (g/cm^3)	Amount of water required (%)	Specific surface area (cm^2/g)	Fineness on sieve 45 µm (%)	SiO_2 + Al_2O_3 + Fe_2O_3 (%)	Activity index: $M_a = \frac{Al_2O_3}{SiO_2}$	Alkaline index: $M_k = \frac{CaO + MgO}{SiO_2 + Al_2O_3}$	Total content: (CaO+MgO)
2.35	104.1	5,820	22.8	87.3	0.43	0.0232	1.8

Testing method

CFMS mixed material samples were designed with varying adhesive (ADH) contents, including cement (CM) and fly ash (FM) to evaluate the influence of ADH content on the compressive strength of the material samples. The mixed material samples were designed with three ADH contents: 5%, 10% and 15% by mass of marine sand. To better understand the effect of fly ash content on strength formation, the fly ash-to-cement mass replacement ratio (FM/ADH) was varied at three levels: FM/ADH of 0% (corresponding to 100% cement content); FM/ADH of 10% (corresponding to 90% cement content) and FM/ADH of 20% (corresponding to 80% cement content). For easer tracking, the samples were coded as follows: S(X; Y) where X represents the ADH content (with values of 5, 10 and 15), and Y represents the fly ash content in the ADH mixture (with values of 0, 10 and 20).

The water content remained consistent across all mixtures. The material composition of the samples was calculated based on the theory of absolute density. Table 4 provides the designed codes and composition for the mixed samples (for 1m^3 of mixture).

Cylindrical material samples with a height of 100mm and a diameter of 50mm were prepared and stored under laboratory conditions. The casting and hand compacting methods followed with the guidelines outlined in TCVN 9403:2012 [7].

The compressive strength of the sample was determined according to TCVN 9403:2012 [7] using a loading rate of 1.5mm/min (Fig. 2). For each mixture, the test was conducted on four samples after 28 days of curing.

Testing results

Table 5 presents the average compressive strength (R_p) results of four samples after 28 days of curing

Tab. 4. Design material composition for 1m^3 of material mixture

Samp. type	ADH/sand & FM/ADH ratio	Samp. code	ADH (kg)	CM (kg)	FM (kg)	Sand (kg)	Water (kg)	Water/ ADH ratio
Without fly ash	ADH 5% (100% CM, 0% FM)	$S_{(5;0)}$	75	75	0	1,500	300	4.00
	ADH 10% (100% CM, 0% FM)	$S_{(10;0)}$	145	145	0	1,450	300	2.07
	ADH 15% (100% CM, 0% FM)	$S_{(15;0)}$	205	205	0	1,367	300	1.46
With fly ash	ADH 5%, FM/ADH 10%	$S_{(5;10)}$	75	67.5	7.5	1,500	300	4.00
	ADH 10%, FM/ADH 10%	$S_{(10;10)}$	145	130.5	14.5	1,450	300	2.07
	ADH 15%, FM/ADH 10%	$S_{(15;10)}$	205	184.5	20.5	1,367	300	1.46
With fly ash	ADH 5%, FM/ADH 20%	$S_{(5;20)}$	75	60	15.0	1,500	300	4.00
	ADH 10%, FM/ADH 20%	$S_{(10;20)}$	145	116	29.0	1,450	300	2.07
	ADH 15%, FM/ADH 20%	$S_{(15;20)}$	205	164	41.0	1,367	300	1.46

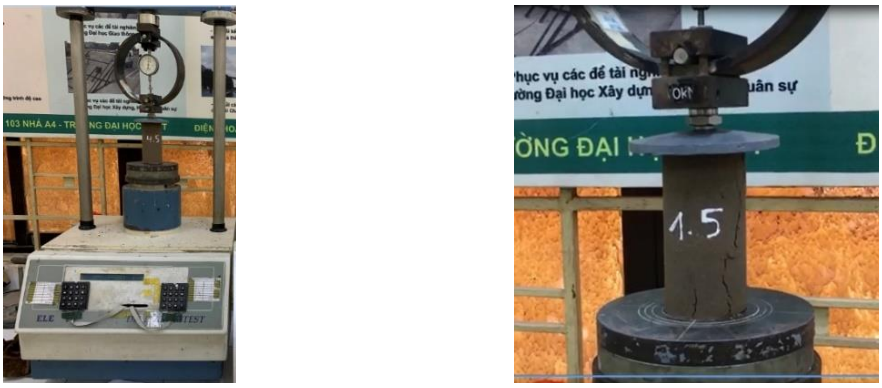

a) Sample at load time b) The sample at failure time

Fig. 2. Test to determine the compressive strength of cylindrical samples

Tab. 5. Average compressive strength test results of mixed samples after 28 days of curing

Sample type	ADH/sand and FM/ADH ratio	Sample code	R_p (MPa)
Without fly ash	ADH 5% (100% CM, 0% FM)	$S_{(5;0)}$	0.72
	ADH 10% (100% CM, 0% FM)	$S_{(10;0)}$	1.52
	ADH 15% (100% CM, 0% FM)	$S_{(15;0)}$	2.00
With fly ash	ADH 5%, FM/ADH 10%	$S_{(5;10)}$	0.68
	ADH 10%, FM/ADH 10%	$S_{(10;10)}$	1.40
	ADH 15%, FM/ADH 10%	$S_{(15;10)}$	1.74
With fly ash	ADH 5%, FM/ADH 20%	$S_{(5;20)}$	0.63
	ADH 10%, FM/ADH 20%	$S_{(10;20)}$	1.30
	ADH 15%, FM/ADH 20%	$S_{(15;20)}$	1.55

Tab. 6. R_p increase rate according to ADH content

ADH content (%)	R_p increase rate according to ADH content (times)		
	FM/ADH 0%	FM/ADH 10%	FM/ADH 20%
5	1.00	1.00	1.00

10	2.11	2.06	2.06
15	2.78	2.56	2.46

Comments:

Research results showed that as ADH content increased from 5% to15%, the compressive strength of the mixed material sample also increased. The R_p value rose from 2.06 to 2.78 times when the ADH ratio was increased from 10 to 15%. However, when FM is used to replace a portion of the cement content, the rate of strength gain tends to decrease as the FM replacement ratio increases. Specifically, without FM, the rate of increase ranged from 2.11 to 2.78 times. However, in the presence of FM (10-20%), the rate of increase was lower, rising by only 2.06 to 2.56 times (Fig. 3).

Fig. 4 more clearly illustrates the effect of FM on R_p. When the ADH content was 5%, the R_p of the sample with 10% FM decreased by approximately 6% compared to the sample without FM, while the R_p of the sample with 20% FM decreased by around 13% compared to the sample without FM. As the FM content increased from 10% to 20%, the 10% ADH sample experienced a decrease in R_p of 8% to 14%; while the 15% ADH sample saw a decrease in R_p of 13 to 23%. The columns corresponding to 10% ADH and 15% ADH were longer than those for 5% ADH, further highlighting the increased rate of strength loss with higher FM content. This can be explained by the fact that increasing the ADH content to 15% and raising the FM/ADH ratio, while maintaining the same amount of mixing water, may result in water deficiency during fly ash hydration.

In comparison to the strength of cement-soil mixtures after 28 days of curing (TCVN 9403:2012 [7]), the R_p of cement-soil mixture with 7% cement only reached 0.300-0.336 MPa, while the Rp of the cement-soil mixture with 12% cement ranged from 0.2 to 1.1 MPa. It is evident that the marine sand-cement-fly ash mixture in this study had higher R_p values (for the equivalent ADH ratio of 5-10%). Additionally, using 5%, 10% and 15% adhesive with a fly ash replacement rate of up to 20% is expected to significantly reduce the cost of mixed material piles while still meeting the strength requirements.

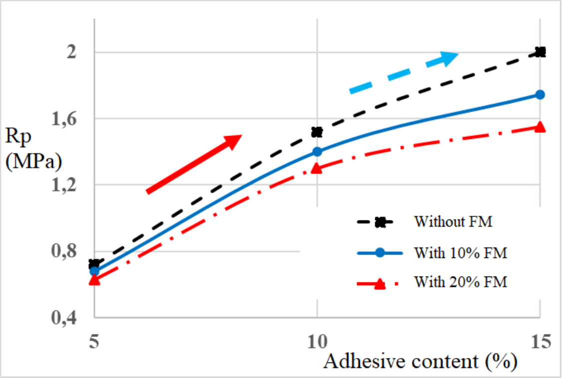


Fig. 3. Relationship between compressive strength and adhesive content at 28 days

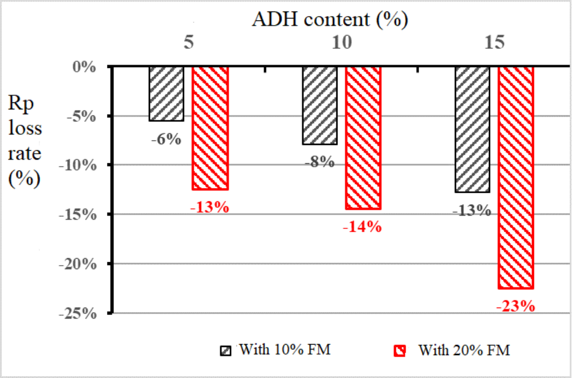


Fig. 4. Strength loss of mixtures of 5, 10 and 15% ADH with fly ash content of 10% and 20%

From the research results presented above, the following conclusions can be drawn as follows:

- CFMS material samples with 5% ADH content still exhibit an R_p greater than the R_p of cement-soil piles with 7% cement, thereby meeting the TCVN 9403:2012 standard [7].
- The rate of R_p increase tends to slow down as the ADH ratio increases from 10% to 15%.
- Increasing the FM/ADH content leads to a greater reduction in the strength of the material sample.
- The FM replacement ratio in the adhesive can reach 20% while still ensuring that the R_p meets the TCVN 9403:2012 standard [7].

The use of marine sand and fly ash from thermal power plants can help reduce dependence on the increasingly scarce river sand resources and make use of waste from thermal power plants. These findings provide a scientific basis for selecting and recommending suitable adhesive ratios in field studies.

3.2. Evaluation of soft soil compaction using CFMS piles

Implementation method

The objective of this experimental study is to evaluate the effectiveness of both immediate and time-dependent reinforcement when improving soft soils with CFMS piles containing varying ADH contents.

The experimental evaluation of the effectiveness of strengthening soft soils using CFMS piles was carried out at the Lai Cach Industrial Park site in Hai Duong province, Vietnam (Fig. 5).

Fig. 5. Location of Lai Cach Industrial Park, Hai Duong Province, Vietnam (https://tinhocnews.com/vector-ban-do-viet-nam/ and Google Earth Pro)

The field testing was carried out according to the diagram in Fig. 6. The content and sequence of the testing were divided into 3 steps:

- Step 1: Drilled the HK-1 borehole to a depth of 7.0m to determine the stratigraphy, collected soil samples, and carried out the DCP-1 test to a depth of 7.0m to determine the physical properties of the soft soil before constructing the CFMS piles;

- Step 2: Constructed 12 CFMS piles with a diameter (d) of 300mm, a depth of 7.0m, and a distance between piles (D) of 900mm using TP300 equipment (Fig. 7a). The piles were divided into 3 groups based on different ADH contents (Fig. 7b): Group 1 (including P-1, P-2, P-3, P-4) has ADH content of 5%, FM/ADH of 0%; Group 2 (P-5, P-6, P-7, P-8) has ADH content of 10%, FM/ADH ratio of 10%; Group 3 (P-9, P-10, P-11, P-12) has ADH content of 15%, FM/ADH ratio of 20%. Immediately after construction, DCP-2 was conducted, and HK-2 borehole was drilled with soil samples taken.

- Step 3: Conducted DCP tests in the 3 groups after 10 days of construction: DCP-3 in Group 1, DCP-4 in Group 2, and DCP-5 in Group 3.

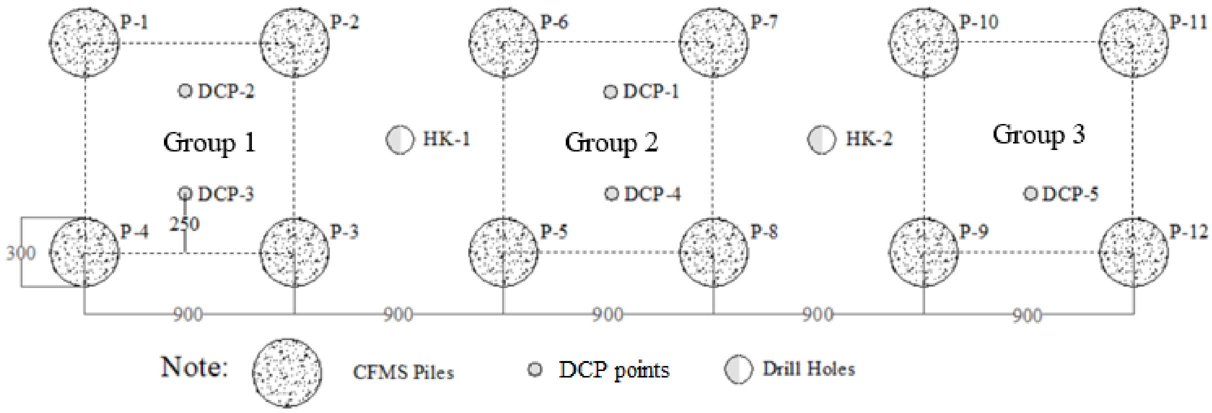


Fig. 6. Layout of CFMS piles, dill holes and DCP points

Fig. 7. a) TP300 pile pressing robot; b) CFMS pile after 10 days of construction

DCP tests were conducted at all three steps in the pile group areas with spacing similar to that of the piles. The DCP test equipment was manufactured by Tecnotest (Italy). The test was carried out by driving a metal cone into the ground using repeated blows from a 10 kg hammer dropped from a height of 500 mm. The penetration of the cone was measured after each blow and recorded. Dynamic penetration resistance (q_d) was determined using the following equation [2]:

$$q_d = \frac{gm^2H}{e(m+m\prime)}\frac{1}{A} \qquad (5)$$

where:
q_d is the dynamic penetration resistance (MPa),
A is the base area of the cone (m^2),
g is the gravitational acceleration (9.81 m/s^2),
m is the mass of the hammer (kg),
m’ is the mass of the shaft (kg),
e is the penetration depth (m).

Stratigraphy and geotechnical properties of soils at the experimental site

The drilling result at location HK-1 indicated that there were four distinct soil layers up to a depth of 7.0 m (Fig. 8). The groundwater level was found at a depth of 0.7 m. The geotechnical properties of soil layers are presented in Table 7.

The results of the DCP-1 test (conducted before CFMS pile construction) are shown in Fig. 8. Based on these results, soil layers 1, 2, and 3 all have low dynamic penetration resistance (q_d <4.0 MPa), while layer 4 has a significantly higher q_d range (5.52-9.71 MPa). As a result, soil reinforcement was only performed up to a depth of 4.0m, covering the soft soil layers 1, 2,and 3.

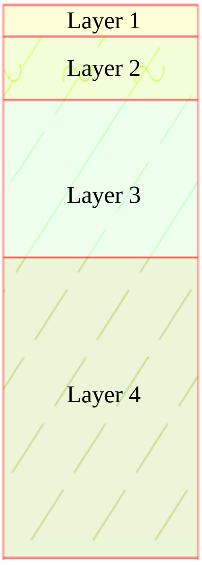
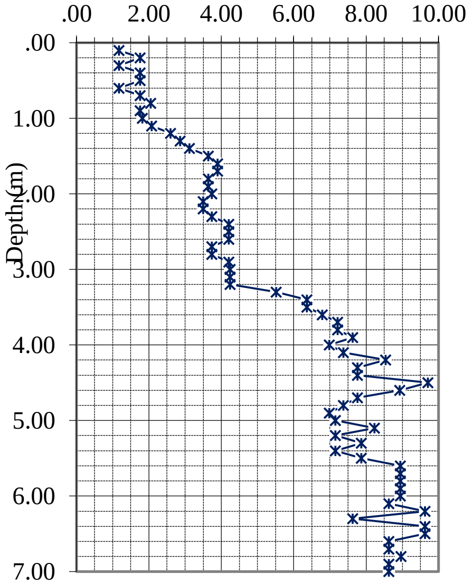
Stratigraphy	Depth (m)	q_d (MPa)	q_d value (MPa)		
			Min	Max	Average
	0-0.4		1.18	1.76	1.47
	0.4-1.2		1.18	2.60	1.88
	1.2-3.2		2.86	4.24	3.81
	3.2-7.0		5.52	9.71	8.03

Fig. 8. The results of the DCP-1 test

Tab. 7. Geotechnical properties of soil layers before reinforcement

No.	Property	Layer 1	Layer 2	Layer 3	Layer 4
1	Soil name (USCS)	Filling sand	Soft, grey sandy Clay	Soft, grey lean Clay	Medium dense, grey clayey Sand
2	Natural water content (W), %	-	31.1	33.5	32.3
3	Natural unit weight (γ), kN/m^3	-	18.84	18.44	18.44
4	Dry unit weight (γ_d), kN/m^3	-	14.32	13.83	13.93
5	Void ratio (e)	-	0.886	0.901	0.830
6	Coeff. of compression (a), cm^2/N	-	0.0033	0.0036	0.0036
7	Internal friction angle (φ), degree	-	12^{o}45'	10^{o}15'	16^{o}15'
8	Cohesion (c), kN/m^2	-	15.8	14.8	11.3

Results of efficiency evaluation by laboratory tests

After constructing the CFMS piles, HK-2 borehole was drilled and three soil samples were taken from three soil layers 2, 3, and 4, at depths of 0.8m, 1.8m, and 3.4m, respectively. These samples were then tested in the laboratory according to current TCVN standards. The results of comparing some physical and mechanical properties of the soil before and after reinforcement are presented in detail in Table 8. Accordingly, the reinforcement efficiency in all types of soil was clearly demonstrated through various properties, such as a reduction in water content, an increase in unit weight, an increase in density (with γ_d increasing and e decreasing), a reduction in compressibility, and an increase in strength. Among the soil layers, the reinforcement efficiency was highest in sandy soil (layer 4,), followed by sandy clay (layer 2), and finally clay (layer 3).

From Table 8, it can be observed that the compaction efficiency of the CFMS piles is clearly demonstrated through several key properties: the water content decreased by 2.39% to 9.91%, the natural unit weight increased by 1.04% to 1.06%, the dry unit weight increased from 1.42% to 3.52%, the void ratio decreased by 2.22% to 7.27%, the coefficient of compressibility decreased by 2.86% to 3.13%, the internal friction angle increased by 2.36% to 8.47%, the cohesion increased by 3.97% to 15.65%. The highest compaction efficiency was observed in the medium dense clayey Sand (layer 4), followed by the soft sandy Clay (layer 2), and finally the soft lean Clay (layer 3).

Tab. 8. Results of determining and comparing the physical and mechanical properties of the soils before and after reinforcement

Sampling time	Water content	Natural unit weight	Dry unit weight	Void ratio	Coef. of compres.	Internal friction angle	Cohesion
	W	γ	γ_d	e	a_{1-2}	φ	c
	%	kN/m^3	kN/m^3	-	cm^2/N	Degree	kN/m^2
Layer 2: Soft, grey sandy Clay							
Before reinf.	31.1	18.84	14.32	0.836	0.0032	12°45'	15.79
After reinf.	28.9	19.03	14.81	0.781	0.0031	13°11'	16.58
Efficiency	-7.07%	1.04%	3.42%	-6.58%	-3.13%	5.30%	4.97%
Layer 3: Soft, grey lean Clay							
Before reinf.	33.5	18.44	13.83	0.901	0.0035	10°15'	14,81
After reinf.	32.7	18.64	14.03	0.881	0.0034	11°01'	15,40
Efficiency	-2.39%	1.06%	1.42%	-2.22%	-2.86%	8.47%	3.97%

Sampling time	Water content	Natural unit weight	Dry unit weight	Void ratio	Coef. of compres.	Internal friction angle	Cohesion
	W	γ	γ_d	e	a_{1-2}	φ	c
	%	kN/m^3	kN/m^3	-	cm^2/N	Degree	kN/m^2
Layer 4: Medium dense, grey clayey Sand							
Before reinf.	32.3	18.44	13.93	0.880	0.0035	10°15'	11,28
After reinf.	29.1	18.64	14.42	0.816	0.0034	10°39'	13,05
Efficiency	-9.91%	1.06%	3.52%	-7.27%	-2.86%	2.36%	15.65%

Results of efficiency evaluation by DCP test

The DCP test was used to evaluate the efficiency of soil compaction and strength increase immediately after pile construction and 10 days after pile construction, with varying ADH content.

Immediately after FCMS pile construction, the DCP-2 test was conducted in Group 1. The results showed the following increase in q_d: Layer 1 - 15%, Layer 2 - 9.8%, and Layer 3 - 14.80%. Thus, the instantaneous compaction capacity was highest in the filling sand (Layer 1), and lowest in the soft sandy Clay (Layer 2).

Ten days after CFMS pile construction, DCP-3, DCP-4 and DCP-5 tests were conducted in the three groups, as shown in the diagram in Fig. 6. The results indicated that the following increase in q_d: Group 1 (Layer 1 - 75.0%, Layer 2 - 19.3%, and Layer 3 - 27.7%); Group 2 (Layer 1 - 80.0%, Layer 2 -52.3%, Layer 3 - 47.9%); Group 3 (Layer 1 - 130.0%, Layer 2 - 67.0%, Layer 3 - 59.1%). The detailed results are presented in detail in Table 9. The increase in q_d with depth before and after reinforcement in the pile groups is illustrated in detail in Fig. 9.

Tab. 9. Evaluation of the effectiveness of the compaction process according to the DCP test

Layer	Values	Dynamic penetration resistance, q_d (MPa)				
		Before reinforcement (q_d)	Immediately after reinforcement (q_{d-0})	10 days after reinforcement (q_{d-10})		
				Group 1	Group 2	Group 3
Layer 1	Min	1.18	1.18	1.76	2.35	1.18
	Max	1.76	2.06	2.94	3.53	4.70
	Average	1.47	1.69	2.57	2.64	3.38
	Increment	-	15.0%	75.0%	80.0%	130.0%
Layer 2	Min	1.18	1.76	1.76	2.35	2.35
	Max	2.60	2.60	2.60	3.65	4.70
	Average	1.88	2.06	2.24	2.86	3.14
	Increment	-	9.8%	19.3%	52.3%	67.0%
Layer 3	Min	2.86	3.12	3.12	4.68	4.68
	Max	4.24	5.61	5.84	6.25	7.01
	Average	3.81	4.38	4.87	5.64	6.07
	Increment	-	14.8%	27.7%	47.9%	59.1%

Based on the results in Table 9, the efficiency of increasing dynamic penetration resistance in soil layers over time and according to ADH content and composition can be evaluated, as shown in Fig. 10.The DCP test results at three stages reveal that the q_d increase process can be divided into two processes (Fig. 11): the compaction process immediately after FCMS pile construction, where the pile material occupies the voids of soft soil (q_{d-0}) and the strength increase process over time (after 10 days), resulting from the consolidation effect of soft soil and the adhesive effect in the piles (Δq_{dc}). Thus, Δq_{dc} represents the difference between the measured penetration resistance after 10 days (q_{d-10}) and q_{d-0}, calculated for the soil layers and summarized in Table 10.

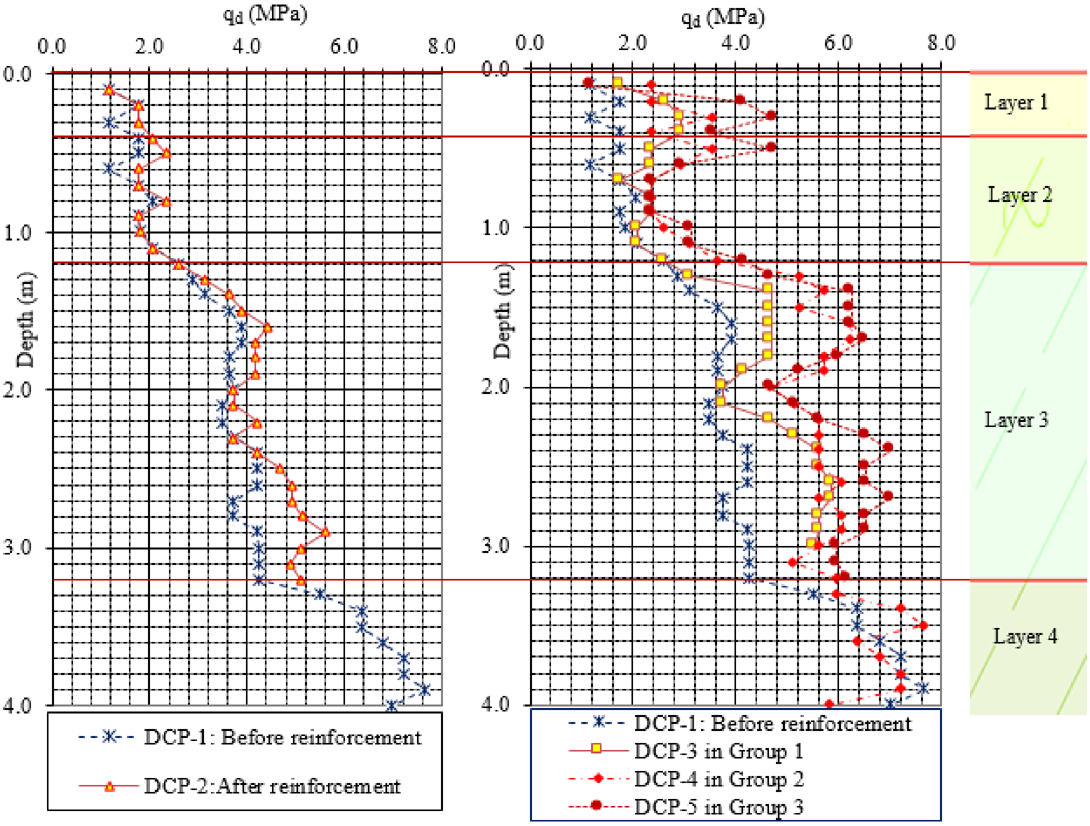


Fig. 9. Comparison chart showing the increase in q_d of the ground with depth in pile groups containing different ADH content

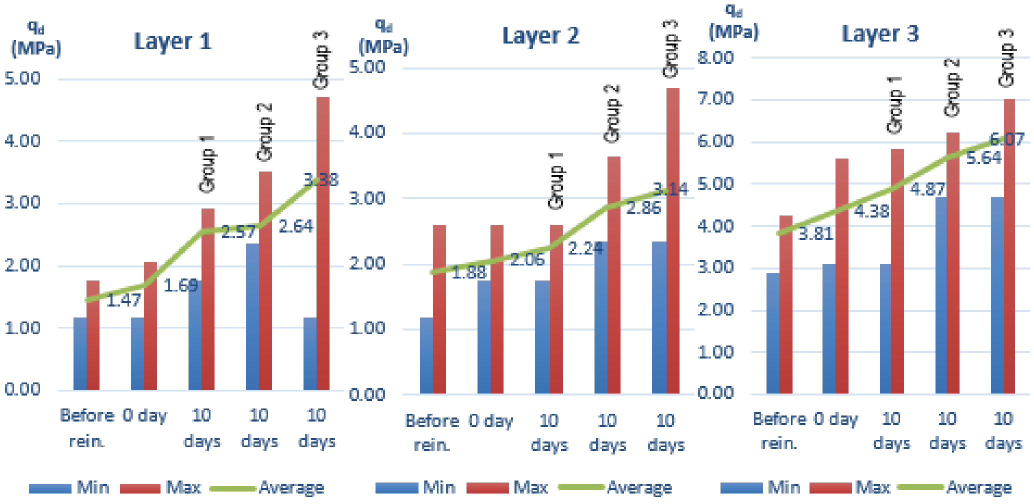


Fig. 10. Graph showing the increase in q_d according to ADH content in soil layers

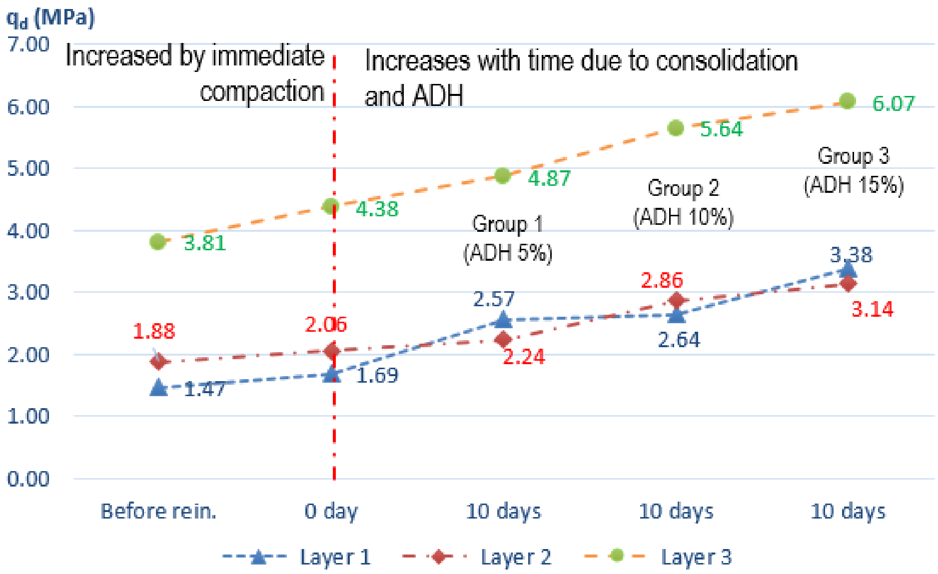


Fig. 11. Graph showing the q_d increase process in soil layers immediately and 10 days after FCMS pile construction

Tab. 10. Increase in q_d due to instantaneous and time-dependent compaction in soil layers

Soil layer	Increase in q_d due to instant. compaction (q_{d-0})	Increase in q_d due to the consolidation effect of soft soil and the effect of adhesive in the pile (Δq_{dc})		
		Group 1 (ADH of 5%, FM/ADH of 0%)	Group 2 (ADH of 10%, FM/ADH of 10%)	Group 3 (ADH of 15%, FM/ADH of 20%)
Layer 2 (soft sandy Clay)	9.8%	9.6%	42.5%	57.3%
Layer 3 (soft lean Clay)	14.8%	12.9%	33.2%	44.3%

From the data in Table 10, it can be observed that the process of increasing q_d due to instantaneous compaction ranged from 9.8% to 14.8%. The increase in q_d due to consolidation and the ADH effect in CFMS piles was significantly influenced by the ADH content. With 5% ADH, Δq_{dc} ranged from 9.6% to 12.9%. When the ADH content increased to 10% and 15%, Δq_{dc} increased substantially, reaching 42.5% and 57.3% in layer 2, and 33.2% and 44.3% in layer 3, respectively. This clearly demonstrates d the role of ADH in enhancing the efficiency of strengthening soft soil with CFMS piles. The fly ash component in ADH served to reduce the cement mass without significantly decreasing the strength of the pile (as discussed above), thereby maintaining the effectiveness of the solution.

4. Conclusion

Based on the above research results, the following conclusions can be drawn:

The compressive strength of CFMS samples with ADH content of 10% and 15% was 2.06 - 2.78 times greater, respectively, compared to the CFMS samples with ADH of 5%. Replacing cement (CM) with fly ash (FM) at an FM/ADH ratio of less than 20% decreased the R_p of the CFMS material mixture, but the decrease was not significantly. Specially, the R_p of CFMS piles with ADH content greater than 5% and an FM/ADH ratio less than 20% still remains above the value specified in TCVN 9403:2012 (R_p of soil-cement material with 7% CM).

The effectiveness of strengthening soft soil with CFMS piles is demonstrated through two processes: instantaneous compaction and consolidation. The effectiveness of each process was evaluated in the field with three groups of CFMS D300 piles containing different ADH contents. The results showed that the instantaneous compaction process led to the following changes:

The water content of soil decreased by 2.39% in Clay and 7.07% in sandy Clay.

The natural unit weight increased by 1.04% in sandy Clay and 1.06% in Clay.

The void ratio decreased by 2.22% in Clay and 6.58% in sandy Clay.

The coefficient of compressibility decreased by 2.86% in Clay and 3.13% in sandy Clay.

Dynamic penetration resistance (q_{d-0}) increased by 9.8% in sandy Clay and increased by 14.8% in

Clay.

After 10 days of reinforcement, the increase in q_d was observed as follows:

In Group 1 (q_{d-10-1}), q_d increased by 19.3% in sandy Clay and 27.7% in Clay.

In Group 2 (q_{d-10-2}), q_d increased by 52.3% in sandy Clay and 47.9% in Clay.

In Group 3 (q_{d-10-3}), q_d increased by 67.0% in sandy Clay and 59.1% in Clay.

The consolidation process was significantly influenced by the ADH content. With 5% ADH, Δq_{dc} ranged from 9.6% (sandy Clay) to 12.9% (Clay). When the ADH content increased to 10% and 15%, Δq_{dc} increased notably, reaching 42.5% and 57.3% in sandy Clay, 33.2% and 44.3% in Clay.

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Conflicts of Interest

The authors declare no conflict of interest.

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