Evaluating Bentonite Sludge Suitability for Landfills and Developing a Coefficient of Permeability Prediction Model

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ABSTRACT: Construction in Bangkok, Thailand, often requires bentonite slurry for stabilization during deep foundation and diaphragm wall construction due to this region's thick, soft clay layers. However, the bentonite slurry creates environmental and logistical challenges after construction as it becomes a waste product. Thus, this study first examined the feasibility of repurposing flocculated bentonite sludge, treated with anionic polyacrylamide from deep foundation construction, for landfill construction. The research compared the physicochemical, swelling, and hydraulic properties of the bentonite sludge with the original bentonite powder, using the free swelling index and consolidation testing. The test results indicated that the microstructure of bentonite sludge, altered by the polyacrylamide, had reduced the swelling and barrier properties compared to the original material. Although unsuitable for the core material of geosynthetic clay liners, bentonite sludge could still be an effective compacted clay liner material with appropriate design and construction. Secondly, this study developed a more rapid method than the consolidation test to estimate permeability values based on the liquid limit and void ratio test results, providing a user-friendly, time-efficient approach. This research identified a sustainable solution for waste management in Bangkok or other similar regions with thick, soft clay layers, as well as offering a practical method for quick permeability estimation in landfill applications.

KEYWORDS: Bentonite sludge, Deep foundation, Landfill, Prediction, and Permeability.

1. INTRODUCTION

Bangkok, Thailand, presents unique geotechnical challenges for construction due to its soft clay layer, which is approximately 12–14 meters deep and complicates the establishment of stable foundations (Teparaksa et al., 1999). To address this, the wet-process, bored-pile construction method has been extensively used for over five decades, providing stability for buildings and infrastructure in the region (Aye et al., 2015). This method involves drilling, installation of reinforcement cages, and concreting, with bentonite slurry playing a critical role in preventing borehole collapse and stabilizing the construction process (Thasnanipan et al., 2002). However, the post-construction disposal of bentonite slurry raises substantial environmental and logistical challenges. Recent advances have addressed these challenges by applying polymers to extract bentonite particles from drilling fluid waste and altering its physicochemical properties (Guler et al., 2018). While cationic polymers, such as sodium polyacrylate, have been shown to enhance swelling properties and reduce permeability in treated bentonite (Prongmanee et al., 2018a), anionic polyacrylamides are more commonly used for flocculation in wastewater treatments (Poon & Chu, 1999). This process results in bentonite sludge (BS) with altered properties, warranting further investigation into its performance after polymer treatment, with the common properties used for evaluation being swelling and permeability tests (Prongmanee et al., 2018). For soils with very low permeability, such as clay or bentonite, the consolidation test is an indirect method for determining the coefficient of permeability (k). This test provides a range of k values comparable to direct measurement methods (Quang & Chai, 2015). However, based on the ASTM D2435 standard, this methodology is time-consuming and labor-intensive, necessitating multiple tests to determine the optimal bentonite mixtures for specific applications (Prongmanee et al., 2021). Based on these limitations, the present research had two main objectives. First, it evaluated the suitability of bentonite sludge for landfill applications by conducting laboratory tests, consisting of sieve analysis, Atterberg's limit test and tests for specific gravity, compaction, free swelling index, and a measurement of the indirect k-value based on consolidation testing. The goal was to explore the sustainable use of bentonite waste in landfills, assessing its potential as a material for clay liners or as a core material in geosynthetic clay liners (GCLs). This involved comparing the physicochemical, swelling, and hydraulic properties of the BS with those of the original bentonite specimens. Secondly, based on the consolidation test results and the existing literature, the study developed a mathematical model to predict the k-value in a time-efficient manner. The proposed empirical equation, validated using test results from the literature, has provided a novel approach for landfill methods. The findings, discussion, and empirical equation developed in this study should be important in landfill and waste management field applications.

2. MATERIALS AND TESTS

2.1 Bentonite Powder and Bentonite Sludge

Bentonil GTC4, a sodium bentonite commonly used in drilling operations, was utilized as bentonite powder (BP). Notably, it had an initial water content of 15%. On the other hand, the BS was a waste by-product resulting from wet-process, bored-pile construction. Before disposal, it had to undergo treatment that involved several steps, including the transportation of the bentonite slurry from construction sites to the decanter centrifuge machine for treatment alongside anionic polyacrylamide, separated sediment, and water in the drilling slurry. Then, the pH of the separated water was adjusted to be within the 5.5–9.0 before being discharged into the environment. After the treatment, the obtained bentonite sludge specimen was initially wet, as shown in Figure 1. The specimen was dried in an oven at 60°C for 48 hours as preparation. The drying temperature of 60°C is chosen to preserve the physical and chemical properties of bentonite sludge samples by preventing the degradation and phase transition of sensitive organic compounds like polyacrylamide. The dried specimen was then ground using a pestle and mortar and the resulting powder was stored in an airtight container until testing. Notably, the original water content of the bentonite sludge after polymer treatment was approximately 73.9%.

Figure 1 Bentonite sludge after being treated with polymer

2.2 Physicochemical Properties Tests

A comprehensive set of tests assessed the physical properties of both the BS and BP specimens: determining the liquid limit (LL), plastic limit (PL), and plasticity index (PI) using the ASTM D423 and ASTM D424 standards, while ASTM D422 was utilized to perform grain size distribution analysis. A pycnometer was used to measure the specific gravity (G_s), following the guidelines set by ASTM D854. Notably, all soil particles were passed through a number 40 mesh before testing for Atterberg limits and specific gravity. The chemical analysis tests were performed on both the BS and BP specimens using X-ray fluorescence (XRF) and X-ray diffraction (XRD) to identify the chemical composition and microstructure. The specimens were sieved and passed through a number 200 mesh for these analyses. These tests helped to provide a thorough understanding of the physical and chemical properties of the specimens, which is crucial for further research and analysis.

2.3 Free Swelling Index (FSI) Test

The free swelling index (FSI) test procedure, following the ASTM D5890 guidelines, was used to evaluate the swelling capacity of the two types of specimens (BP and BS). Although this test is commonly used for bentonite materials in GCLs, in the present study, it was used to compare BP and BS and determine their potential suitability for GCL applications. To perform the test, 2.0 g of dried powder (either BP or BS), was incrementally added to 100 ml of deionized water at a rate of 0.1 g every 10 minutes. After 24 hours, the final volume of the expanded sample was measured. This process was repeated three times for each specimen to ensure accurate results. The FSI test is essential in assessing the suitability of materials for use in GCL applications because it indicates the extent to which a material can swell in the presence of water.

2.4 Compaction Test

The standard Proctor test is a widely used laboratory method for determining the maximum dry density and optimum water content of soils for engineering purposes (Prongmanee & Noulmanee, 2020). The test was conducted following the guidelines set out in ASTM D698. Once the test was completed, Equation (1) was used to determine the void ratio (e) of compacted soil:

$$e = \frac{\gamma_w \times G_s}{\gamma_d} - 1 \tag{1}$$

The detailed steps for the Proctor compaction test are as follows:

1) Water was gradually added to the prepared BS or BP sample with varying moisture contents within the expected range of optimum moisture content.

2) A cylindrical compaction mold (volume 944 cm³, diameter 10.16 cm, and height 11.43 cm) was filled with the soil specimen from step 1 in three equal layers. Compact each layer using a standard Proctor hammer, applying 25 evenly distributed blows. This hammer weighed 4.54 kg and was dropped from a height of 30.5 cm. When compacting each layer, it was ensured that the moisture content was uniform throughout the soil specimen by using a spatula to distribute excess soil. Then, the collar was removed and any excess soil was trimmed from the top of the mold using a straight edge.

3) The compacted soil specimen was removed carefully from the mold and a small representative sample was obtained. The sample's moisture content was determined based on weighing it before and after drying it in an oven at 110°C for 24 hours.

4) The compacted soil specimen's dry density (γ_d) was calculated. Then, γ_d was plotted against the corresponding water content (w) to determine the maximum dry density ($\gamma_{d,\,max}$, the highest dry density of the soil) and the optimum water content (OWC, the moisture content that produces in the maximum dry density).

2.5 Permeability Test

The coefficient of permeability (k) of the test specimens was evaluated using the consolidation test, performed using an oedometer apparatus, according to the guidelines of ASTM D2435. The k value was calculated using Taylor's (1948) method, based on the results of the oedometer testing. Notably, other research studies by Bohnhoff and Shackelford (2014), Quang and Chai (2015), and Prongmanee et al. (2021) have reported that the k value obtained from oedometer test results for low-permeability materials was comparable to directly measured values.

To prepare the test specimens, the following steps were conducted:

1. A slurry was formed by mixing the BP or BS dry powder with deionized water at twice its liquid limit, followed by curing for at least 24 hours.

2. The cured slurry was preconsolidated under an effective vertical stress of 50 kPa for approximately 3 days, using an oedometer ring with a 60 mm diameter and 20 mm height. Notably, adopting vertical stress of 50 kPa and a pre-consolidation period of 3 days in preparing bentonite specimens. This methodological optimization was determined through an extensive review of the existing literature and trial-and-error experiments.

3. The height of the preconsolidated specimen was measured (typically around 26 mm) and trimmed to a thickness of 20 mm. Excess BP or BS was removed using a wire saw.

4. The consolidation test was performed by doubling the stress at each subsequent step. The permeability was calculated from the consolidation test results using the method reported by Prongmanee et al. (2018b).

3. TEST RESULTS AND DISCUSSIONS

3.1 Physicochemical Characterization

Table 1 presents the physical properties of the BP and BS specimens. The sieve analysis results revealed that all the BP particles passed through a No. 200 sieve, while the BS specimen contained 60.2% fine particles, with the remainder as sand particles, having the largest particle size of 2 mm. The Atterberg limits (LL, PL, and PI) showed that BS had lower limit values than BP. Specifically, BS had a PL of 20.3%, LL of 51.9%, and PI of 31.6%, while BP had a PL of 112.5%, LL of 583.6%, and PI of 471.1%. The specific gravity results of both specimens were similar, with values of 2.81 and 2.76 for the BP and

BS, respectively. Based on the sieve analysis and Atterberg limits findings, the BP and the BS specimens could be classified under the Unified Soil Classification System (USCS) as high plasticity clay (CH).

Table 1 Physical properties of bentonite sludge (BS) and bentonite powder (BP)

Property	BS	BP
Sieve analysis		
Passing No.4 (4.75 mm) (%)	100.0	100.0
Passing No.10 (2.00 mm) (%)	99.1	100.0
Passing No.40 (0.425 mm) (%)	95.8	100.0
Passing No.100 (0.150 mm) (%)	78.4	100.0
Passing No.200 (0.075 mm) (%)	60.2	100.0
Liquid limit, LL (%)	51.9	583.6
Plastic limit, PL (%)	20.3	112.5
Plasticity index, PI (%)	31.6	471.1
Specific gravity, G_s	2.76	2.81
Soil classification (USCS)	CH	CH

Based on the chemical analysis, measured using triplicate samples, minor differences exist between the BS and BP samples. The concentrations of silicon dioxide (SiO_2) and calcium oxide (CaO) were marginally higher in BS than in BP, with a corresponding reduction in aluminum oxide (Al_2O_3). This discrepancy may be attributed to the contamination of the BS by surrounding soil and concrete during the bored pile construction, as detailed in Table 2.

Table 2 Chemical compositions of the bentonite sludge (BS) and bentonite powder (BP)

Component	BS				BP			
	#1	#2	#3	Avg.	#1	#2	#3	Avg.
Al_2O_3	8.73	8.74	8.64	**8.70**	11.25	11.38	11.41	**11.35**
SiO_2	82.31	82.41	82.48	**82.40**	76.35	78.05	77.32	**77.24**
K_2O	1.28	1.30	1.28	**1.29**	0.23	0.25	0.25	**0.25**
CaO	3.40	3.34	3.17	**3.30**	2.32	2.32	2.27	**2.30**
TiO_2	0.74	0.69	0.73	**0.72**	0.72	0.69	0.65	**0.69**
MnO_2	0.01	0.08	0.10	**0.06**	0.08	0.09	0.08	**0.09**
Fe_2O_3	2.61	2.60	2.70	**2.63**	3.88	3.95	3.80	**3.87**
ZrO_2	0.02	0.02	0.03	**0.02**	0.01	0.01	0.01	**0.01**

Nevertheless, the primary constituents of both BS and BP were aluminum oxide and silicon dioxide, consistent with the general composition of typical soil components. Furthermore, the subsequent treatment based on anionic polyacrylamide resulted in additional alterations to the microstructure of BS, as illustrated in Figure 2. The XRD patterns of BP (Figure 2a) and BS (Figure 2b) were distinctly different. The XRD pattern of BP revealed the presence of a montmorillonite peak (green color), while the XRD pattern of BS did not display a peak indicative of montmorillonite. Notably, montmorillonite is the primary mineral responsible for high values of swelling and water absorption capacity (França et al., 2022). Consequently, the BS underwent a complete transformation in the montmorillonite structure due to the polymer addition, which increased the anionic content on the clay surface, resulting in soil flocculation and structure alteration. This observation was consistent with the Atterberg's limits test results in Table 1.

3.2 Free Swelling Index (FSI)

Figure 3 presents the free swelling index (FSI) results for both the BP and BS specimens. There was a substantial difference between the swelling potential of BP and BS, with the FSI of the BS being substantially lower than for BP. Specifically, the swelling potential of BS decreased by approximately 86% compared to BP. The FSI value for the BS was only 4 mL/2 g, while the original BP had an FSI value of 28 mL/2 g. These findings suggested that the swelling properties of BS were considerably diminished after treatment with anionic polyacrylamide. The decline in the FSI value can be attributed to the anionic polyacrylamide causing cation exchange in the interlayer of soil particles, leading to a reduced absorption capacity and lower swelling potential. This change in swelling potential presents both advantages and disadvantages. For example, soil with a lower swelling potential offers excellent stability as backfilling material. In contrast, soil with a higher swelling potential possibly has improved barrier performance and is more suitable for a barrier layer. Based on the results of FSI, the BS did not qualify as a core layer for a geosynthetic clay liner, as it did not meet the required minimum value for the FSI of 24 mL/2 g (Prongmanee, 2018), as indicated in Figure 3.

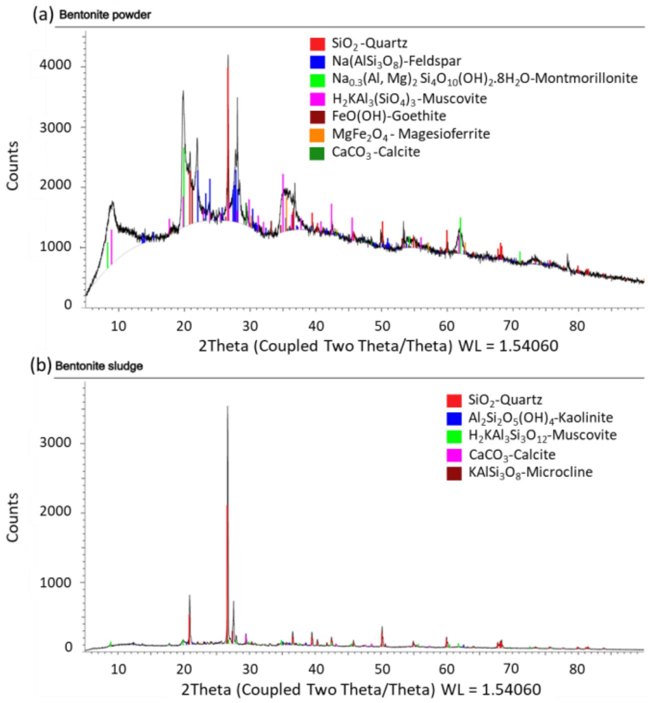

Figure 2 Results of XRD for: (a) BP and (b) BS, with presence of montmorillonite peak (green color)

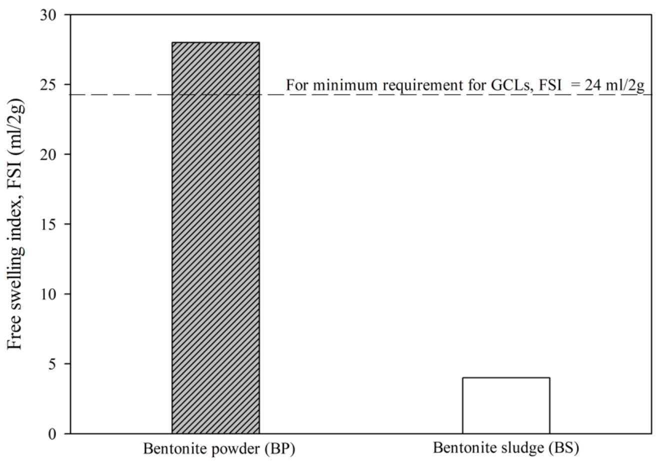

Figure 3 Results of FSI for BP and BS submerged in deionized water

3.3 Compaction Characteristic of the BS

Based on the physicochemical and swelling properties, BS was unsuitable as a core material in GCLs. Compaction tests were performed to assess its suitability for other landfill applications, as shown in Figure 4. The tests showed that the dry density (γ_d) increased with the water content (w) until the optimum water content (OWC) of 21% was reached, yielding a maximum dry density ($\gamma_{d,\,max}$) of 16.3 kN/m^3. Beyond this point, a further increase in w led to a decrease in γ_d. Based on the results presented in Figure 4, the void ratio (e) of compacted soil was determined using Equation (1), incorporating the specific gravity (G_s) value of 2.76, as reported in Table 1.

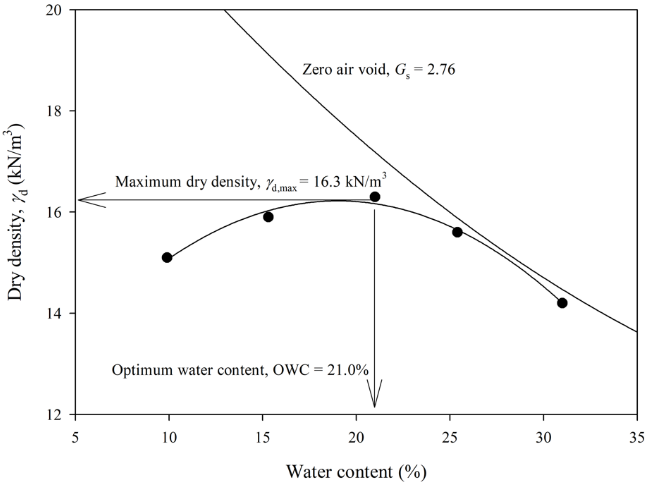

Figure 4 Compaction results for BS

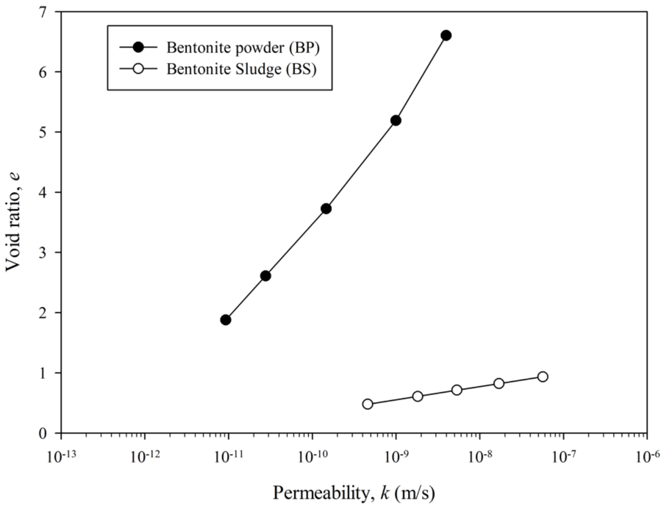

Figure 5 Plot of calculated *k* versus *e* values for BP and BS

3.4 Results of Coefficient of Permeability (*k*)

Figure 5 demonstrates the correlation between e and k based on the oedometer test results. The test results revealed that for an e value of 0.66, the k value was 0.3×10^{-9} m/s for BS. A decrease in the e value generally led to a reduction in the k value. At equivalent e values, BS had a higher k value than BP, indicating that the anionic polymer-modified bentonite structure reduced its barrier properties. Notably, the minimum requirement for compacted clay liners is a k value of 1×10^{-9} m/s (Prongmanee, 2018). With the optimum water content and following the standard Proctor energy procedure, the compacted BS provided k values marginally lower than this minimum requirement. Consequently, BS could be considered suitable for use as a compacted clay liner (CCL) material, with the estimated e value of compacted soil at the maximum dry density condition being approximately 0.66. The present results suggested that when compacted at the optimum water content using the standard Proctor method, BS could be utilized as a clay liner material with a k value of approximately 0.3×10^{-9} m/s. This application could be particularly beneficial for water storage in irrigation systems in regions with limited water storage and supply, such as Northeastern Thailand. However, further investigation would be required to validate these test results through direct measurements. Critically, the initial water content of BS (w = 73.9%) exceeds its liquid limit (LL) value of 51.9% and the optimum water content of 21%. Thus, if used as a landfill liner, the water content should be reduced before application. In addition, full-scale testing is recommended before implementing BS as a construction material.

To fully understand the impact on the environment of using BS, it is essential to consider several factors beyond just the eco-friendliness of the end product. One critical aspect is conducting laboratory testing to check for residual polyacrylamide from the process of anionic polyacrylamides commonly used in wastewater treatment (Tepe & Çebi, 2019). While anionic polyacrylamides are generally considered low-toxicity substances (Smith & Oehme, 1991), they may contain traces of acrylamide, a neurotoxin and a potential carcinogen (King & Noss, 1989). This raises concerns about environmental health, especially if the treated BS is introduced into the environment or used in a way that could expose the surrounding area to contamination. To ensure safe and efficient management, it is necessary to investigate potential chemical contamination and the environmental implications of using treated BS. While this paper primarily focused on the swelling and barrier properties of BS, an extensive examination of ecological aspects is crucial and requires further research. The ultimate goal of this field of study is responsible usage and disposal of BS with a primary safety concern.

Table 3 Results for constants "*a*" and "*b*" and for coefficient of determination (r^2), based on curve fitting data from Figure 6

No	LL	*a*	*b*	r^2	Remark
1	583.60	0.753	20.939	0.9929	Bentonite powder with deionized water (Present study)
2	51.90	0.096	2.557	0.9984	Bentonite sludge with deionized water (Present study)
3	621.00	1.489	44.731	0.9896	Bentonite with deionized water (Prongmanee et al., 2018b)
4	499.00	1.182	36.342	0.9625	Polymerized bentonite with deionized water (Prongmanee et al., 2018b)
5	394.68	0.967	29.021	0.9950	Bentonite with 0.1 M NaCl (Prongmanee et al., 2018b)
6	372.66	0.751	22.457	0.9869	Polymerized bentonite with 0.1 M NaCl (Prongmanee et al., 2018b)
7	314.17	0.930	26.612	0.9950	Bentonite with 0.1 M $CaCl_2$ (Prongmanee et al., 2018b)
8	164.14	0.453	12.893	0.9934	Polymerized bentonite with 0.1 M $CaCl_2$ (Prongmanee et al., 2018b)
9	267.35	0.557	16.701	0.9894	Bentonite with 0.6 M NaCl (Prongmanee et al., 2018b)
10	140.91	0.272	8.642	0.9963	Polymerized bentonite with 0.6 M NaCl (Prongmanee et al., 2018b)
11	185.16	0.515	14.698	0.9972	Bentonite with 0.6 M $CaCl_2$ (Prongmanee et al., 2018b)
12	132.82	0.334	9.319	0.9916	Polymerized bentonite with 0.6 M $CaCl_2$ (Prongmanee et al., 2018b)
13	38.00	0.098	2.443	0.9935	Lateritic soil with deionized water (Prongmanee et al., 2021)
14	47.80	0.115	2.969	0.9972	Lateritic soil mixed 2% bentonite with deionized water (Prongmanee et al., 2021)
15	50.10	0.139	3.658	0.9625	Lateritic soil mixed 4% bentonite with deionized water (Prongmanee et al., 2021)
16	56.20	0.142	3.930	0.9777	Lateritic soil mixed 8% bentonite with deionized water (Prongmanee et al., 2021)
17	66.50	0.176	4.946	0.9944	Lateritic soil mixed 16% bentonite with deionized water (Prongmanee et al., 2021)
18	29.35	0.083	2.037	0.9762	Lateritic soil mixed 4% with 0.3 M NaCl (Prongmanee et al., 2021)
19	22.94	0.073	2.009	0.9844	Lateritic soil mixed 4% bentonite with 0.6 M NaCl (Prongmanee et al., 2021)

3.5 Coefficient of Permeability Prediction Model Development

This section was developed in response to the lengthy duration required by consolidation tests to determine the coefficient of permeability (k). Instead, this study proposed a rapid and efficient method to predict k, leveraging the results from the LL and void ratio (obtained through compaction testing). Consequently, a mathematical model was formulated using regression analysis, incorporating data from the present study and existing literature (Quang & Chai, 2015; Prongmanee et al., 2018b; Prongmanee et al., 2021). The section outlines the methodology and concepts underlying the development of the model, including a step-by-step procedure for its application in design projects. The ensuing details provide further insight into this process, elucidating the approach taken and discussing the results.

The permeability coefficient (k) can be indirectly measured based on consolidation test results. Other literature suggested a linear correlation between void ratios (e) and the logarithmic value of k (Quang & Chai, 2015; Prongmanee et al., 2018b; Prongmanee et al., 2021). The correlation between e and the natural logarithm of k, is depicted in Figure 6. An empirical model was developed based on the present test results and the results from Prongmanee et al. (2018b) and Prongmanee et al. (2021), as shown in Figure 6. Notably, those findings showed excellent linearity between the e and log k values for each scenario. Thus, a non-linear logarithmic equation was proposed to fit the test results accurately, as shown in Equation (2):

$$e = a \cdot \ln\left(k\right) + b \tag{2}$$

where e is the void ratio, k is the coefficient of permeability, the constant "a" determines the slope of the logarithmic curve, and the constant "b" represents the vertical shift of the curve.

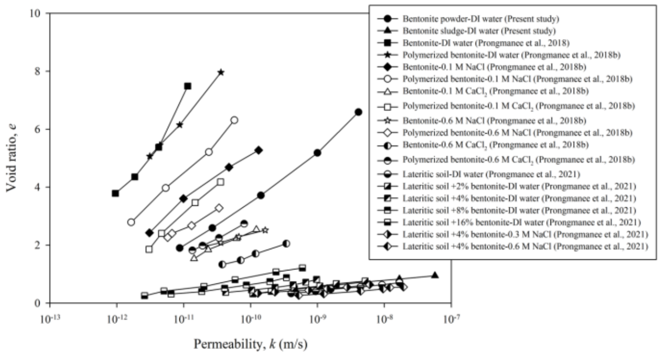

Figure 6 Plot of calculated k versus e values for BP and BS

The present study used Equation (2) as the basis to calculate the constant parameters and coefficient of determination (r^2) as listed in Table 3. The investigation posited that the water content within soil pores plays a decisive role in the relationship between the void ratio (e) and the logarithm of the permeability coefficient (log k). Thus, this study proposed that the constants "a" and "b" are influenced by the liquid limit (LL) value. A higher water content absorbed capacity, indicated by a higher LL value, leads to a steeper slope (a higher "a" value), presumably because specimens with higher water absorption exhibit greater initial e values under surcharged load consolidation, leading to easier water expulsion, more substantial volume change, and decreased permeability.

In addition, a higher e value correlates with a higher y-intercept, influenced by the constant "b." Based on the results and analysis presented in Table 3, a correlation was established between the constants "a" and "b" with their corresponding liquid limit (LL) values, as illustrated in Figures 7A and 7B. The predicted model agreed with the test results from the present study and literature studies with r^2 values in the range 0.9694–0.9707 for constants "a" and "b", respectively.

It was evident from the data that both the constants "a" and "b" increase linearly with an increase in LL value, as depicted in Figures 7a and 7b. This observation confirmed our hypothesis, indicating that porous materials, such as a clay with higher water-holding capacity, have higher values of the constants "a" and "b", suggesting increased sensitivity to surcharge loads, particularly in materials with higher original void ratios. Consequently, based on the present results, expressions for the constants "a" and "b" were developed, as shown Equations (3) and (4), respectively:

$$a = 0.0023LL + 0.013 \tag{3}$$

$$b = 0.0698LL - 0.102 \tag{4}$$

where LL denotes the liquid limit.

Substituting Equations (3) and (4) into Equation (2), a new equation can be derived, presented as Equation (5):

$$k = e^{\left(\frac{-70LL+1000e+1020}{23LL+130}\right)} \tag{5}$$

Determining the coefficient of permeability (k) for different known void ratios (e) and liquid limit (LL) values can be done quickly using Equation (5). The value of e for compacted soil can be obtained using a compaction test. At the same time, the liquid limit (LL) value can be easily determined using the traditional Casagrande method or other standard tests. The present developed method is highly recommended since it reduces testing time to a single day for determining water content, in contrast to at least 1 week required for a consolidation test.

Before applying the proposed Equation (5) in design work, it must be validated based on reliable test results. The present study derived the k value using data from Quang and Chai (2015) consolidation tests on Ariake clay and dredged mud, for which the LL values were 133 and 147, respectively.

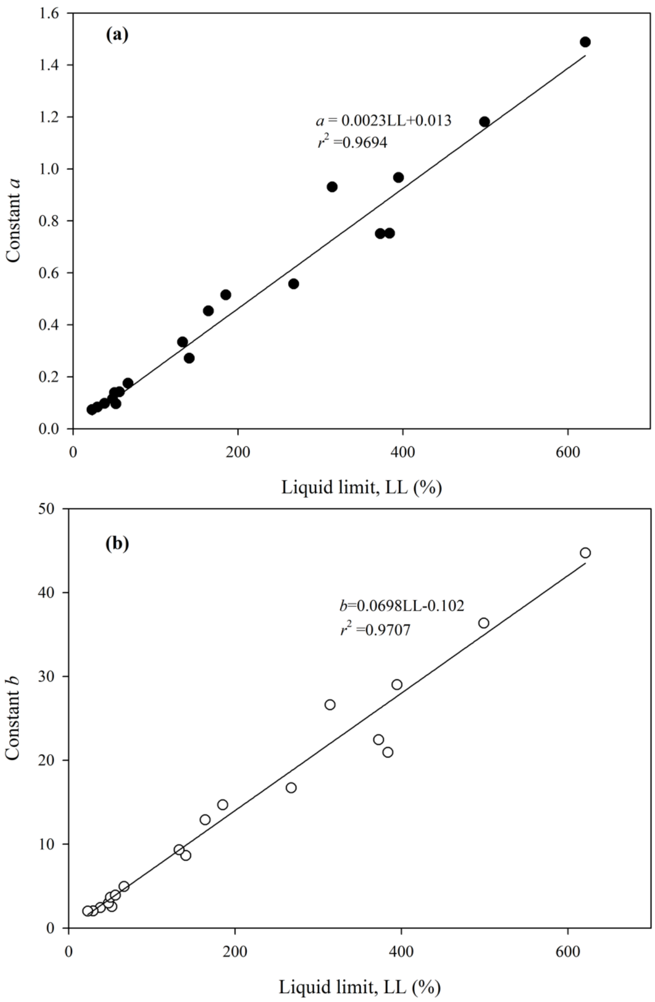

Figure 7 (a) Relationship between LL and constant "*a*" and (b) relationship between LL and constant "*b*"

Figures 8a and 8b compare the predicted results with those from the literature. While the model predictions did not perfectly align with the results reported by Quang and Chai (2015), they were within an acceptable range. The r^2 values for the Ariake clay and dredged mud specimens were 0.7031 and 0.8699, respectively. The present study demonstrated a speedy and straightforward prediction method that uses only *e* and LL values, making it practical for fieldwork. Notably, soil particles are more densely packed at lower void ratios and tend to orient parallel. This arrangement enhances the applicability of classical consolidation and permeability theories, yielding predictions that more accurately reflect observed outcomes. In contrast, the soil adopts a flocculation structure at higher void ratios, markedly increasing its complexity. This structural intricacy, evident through a more complicated pore network and fluid flow pathways, diverges theoretical predictions and actual measurements. Additional parameters could be incorporated to produce an even better fit. With these verified results, the model presented in Equation (5) can be used with confidence to determine the appropriate amendment mixtures for compacted clay liners. This study presents a procedure based on the results of Prongmanee et al. (2021) to demonstrate this approach further. The methodology was exemplified using a compacted clay liner case study, based on data from Prongmanee et al. (2021), which involved mixing bentonite in proportions in the range 2–16%. The proposed method determines the appropriate bentonite mixture for compacted clay liners to achieve *k* values of less than or equal to 1×10^{-9} m/s. The suggested prediction procedure is as follows:

1. Soil samples were collected and prepared for laboratory examination. Liquid limit and compaction tests were administered to determine the values for the void ratio (*e*) and liquid limit (LL). After mixing lateritic soil with varying amounts of bentonite (0%, 2%, 4%, 8%, and 16%), the resulting *e* values were 0.56, 0.59, 0.67, 0.73, and 0.83, respectively. The corresponding LL values were 38%, 47.8%, 50.1%, 56.2%, and 66.5%.

2. Equation (5) was utilized to determining the value of *k*, entailing substituting the values of *e* obtained from Step 1. To quickly determine *k*, it is sufficient to utilize the *e*-values of 0% and 16% mixtures at the maximum dry density. The relationship between the calculated *k* and *e* values versus the percentage of the mixtures can be ascertained based on Figures 9a and 9b.

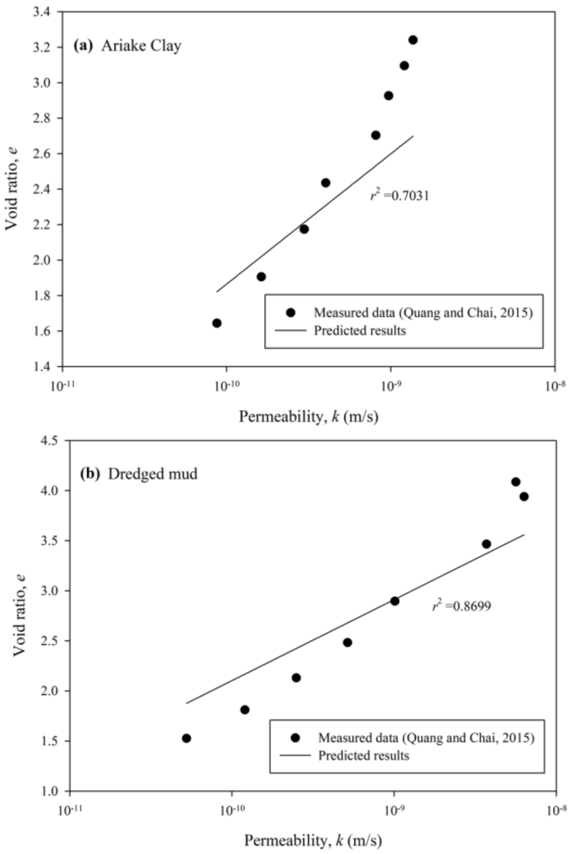

Figure 8 Verified predicted model using results from Quang and Chai 2015 for (a) Ariake clay and (b) Dredged mud

3. To attain a *k*-value of 1×10^{-9} m/s or less, it is imperative to ascertain the exact percentage of the mixture required. This in-depth analysis revealed that the calculated percentage mixture was 5.36, which was marginally higher than the 5.24 reported by Prongmanee et al. (2021), with a percentage of error of approximately 2.3%. Consequently, this study recommends a mixture percentage of 6% as the optimal percentage of the mixture. Implementing this approach would substantially curtail the required tests, as only two would suffice, allowing valuable reductions in time and human resources. Notably, lateritic soil with a 6% mixture of bentonite needs to be checked for its value of *k*, with additional field quality checking of the field density and permeability recommended.

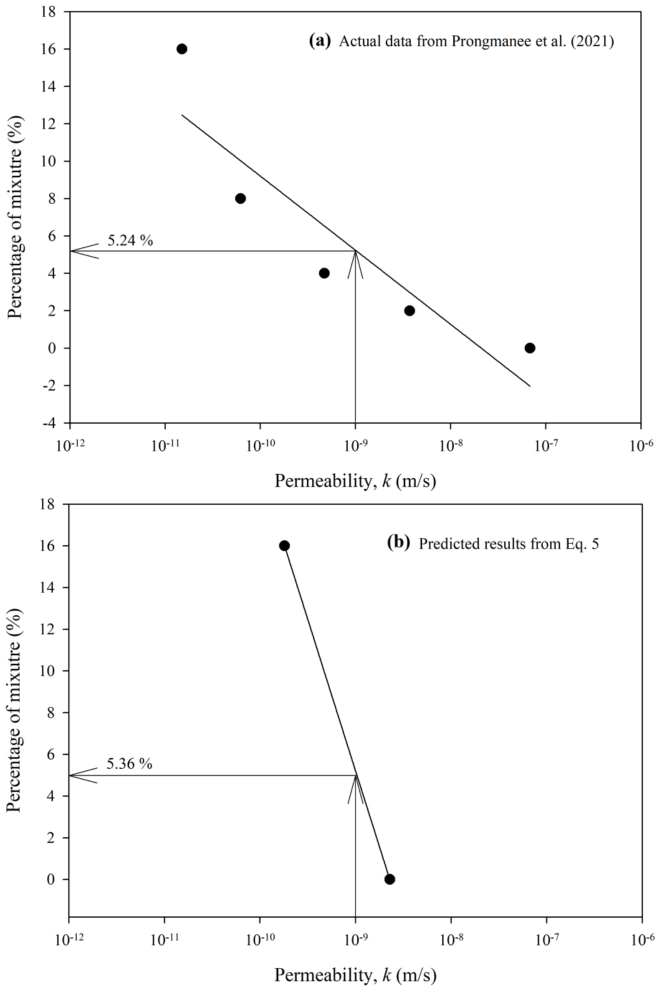

Figure 9 Percentage of mixture versus *k* value (a) Actual data and (b) Predicted result

4. CONCLUSIONS

This study assessed the feasibility of utilizing waste bentonite sludge from bore piles and diaphragm wall construction as landfill construction material. A thorough investigation was conducted based on a series of laboratory tests, with an empirical model being developed to estimate the coefficient of permeability (*k*) from the liquid limit (LL) and void ratio (*e*) acquired from compaction tests. Through an in-depth analysis of the results obtained, the following conclusions were drawn:

The analysis of the physicochemical, swelling, and hydraulic properties of the bentonite sludge (BS) revealed reduced swelling and barrier properties compared to original bentonite specimens. This reduction was likely due to the anionic polyacrylamide used in the flocculation process, which altered the microstructure of the BS. Despite these changes, the results from compaction and permeability testing indicated that BS could be effectively used as a compacted clay liner, especially in water storage structures within irrigation systems. Using BS in this manner could substantially reduce waste and enhance the sustainability of landfill construction.

The present study successfully developed an empirical equation that can efficiently determine *k* values using the LL and *e* results. This presents a highly effective alternative to traditional methods and has been verified against other research results. The present model had an acceptable range in the coefficient of determination between 0.7031 and 0.8699, indicating its suitability for predicting *k* values. Then, the predicted model was used to predict the percentage of bentonite mixture-to-lateritic soil for determining the optimal condition for compacted clay liner material with a *k* value less than or equal to 1×10^{-9} m/s. The model predicted an amount of bentonite of 5.36% against the actual test result of 5.24%, with an error percentage of only 2.3%. The proposed method is user-friendly and can simplify the determination of suitable conditions for landfill construction. Consequently, this mathematical model should be an invaluable tool for practical landfill applications.

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