Properties of Chiang Khruea Lateritic Soil and Their Applications in Civil Engineering

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Abstract

Sakon Nakhon is a city-municipality in Thailand whose infrastructure such as highways and buildings have been rapidly developing. Consequently, large amounts of construction materials such as crushed rock, sand, and laterite are being used in many construction projects. Chiang Khruea sub-district contains the main laterite quarries supplying construction sites in Sakon Nakhon province; the extracted laterite from this area is in demand for highway sub-base construction due to its good shear strength after compaction and its low price. However, the properties of Chiang Khruea lateritic soil are not yet maximally used in other civil engineering applications such as landfill liner, backfill material for retaining walls, and as a base layer for paved roads because engineers do not extensively understand its physical and engineering properties. Therefore, this study investigated and reported on the properties of Chiang Khruea lateritic soil to identify possibilities for its applications in civil engineering. The physical (i.e., specific gravity of soil, Atterberg limits, Los Angeles (L.A.) abrasion) and engineering properties (i.e., compacted soil density, California Bearing Ratio, shear strength parameters, permeability (k) and modulus of subgrade reaction) of Chiang Khruea lateritic soil were investigated through a series of laboratory and in-situ tests. The results reported could be useful for engineers as a reference for sustainable design and construction.

Keywords: Chiang Khruea Lateritic Soil (CKLS); Permeability; Compacted Soil; Shear Strength; California Bearing Ratio; Modulus of Subgrade Reaction

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Introduction

Sakon Nakhon province is located in northeastern Thailand, and forms part of the Sakon Nakhon Basin and the Phu Phan Range. Generally, infrastructure in this area has been developing around Nong Han Lake, the biggest natural lake in northeastern Thailand with an area of approximately 125.2 km^2. The lacustrine deposit surrounding Nong Han Lake consists of very fine soil particles. On the flood plain around Nong Han Lake, the soil has formed an alluvial deposit from 21 natural creeks before they discharge into this large lake. The soil in the undulating terrain is formed from residual Chiang Khruea lateritic soil (CKLS) through the decomposition of cretaceous sedimentary rocks such as shale or mudstone. Generally, the soil in this area is usually used for pottery, with the province's pottery village located at Ban Chiang Khrua. Along the Phu-Phan Range, most of the rocks are from the Cretaceous to the Jurassic eras, named Phu-Phan sandstone.

In August 2012, Sakon Nakhon became a city-municipality of Thailand, resulting in the rapid development of many types of infrastructure, with large amounts of construction material being utilized in many projects. Chiang Khruea is a sub-district of Sakon Nakhon province which has a large laterite quarry that supplies many construction projects in this area. Residual CKLS has decomposed from mudstone via leaching, with one possible leaching mechanism being the advection of the freshwater (rain and groundwater). This soil is loamy-skeletal with a reddish-brown color and is rich in iron oxide. Generally, the soil profiles in this area are composed of topsoil with a thickness of about 0.2 to 0.5 m. Beyond this soil layer is the laterite with large boulder particles and thick sheets of laterite. The soil conditions in this area are not good for agriculture, but the lateritic soils are good for road construction materials. For example, in 1964, the old Chiang Khruea airport was constructed into the US army military base during the Vietnam War because of its firm lateritic soil foundation. Then, 30 years later in 1994, this area was developed into a government university named the Kasetsart University Chalermphrakiat Sakonnakhon Province campus (KU-CSC). At present, the CKLS extracted from this area is generally used for road embankments, and as subgrade and subbase due to its good shear strength compaction and low price. However, it has been only limited to being used in road construction materials because its mechanical behavior has not been well-understood.

Thus, this study aimed to investigate and report on the physical and engineering properties of CKLS through a series of laboratory and in-situ tests. This study could provide essential information on the properties of CKLS for use in designing and construction of civil engineering projects.

Materials

The soils used in the analyses were collected from different locations on KU-CSC as shown in Figure 1. Test pit 1 (TP-1) was located at 17.287237 N, 104.106361 E, at a mean sea level (MSL) of +168.00 m, and test pit 2 (TP-2) was located at 17.290853 N, 104.115049 E, at an MSL of +166.70 m. Additionally, plate bearing tests were performed to evaluate the bearing capacity of the soil foundation. The location of the in-situ test site is shown in Figure 1.

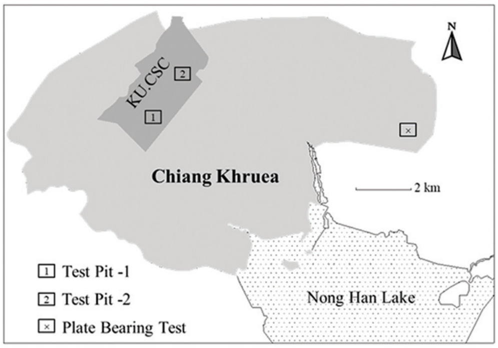

Figure 1 Location map of soil sampling and plate bearing testing

Generally, the lateritic soils in the study area are residual soil decomposed from mudstone; a typical soil profile is shown in Figure 2. The topsoil is -0.20 m in depth and has a loose state with a high amount of organic matter. From a depth of -0.20 to -1.20 m, the soil is lateritic with various sizes of soil particles up to 60 mm mixed with lateritic boulders. Just below -1.20 m, the white soil layer is plinthite with a thickness of about 0.8 m, being an iron-rich soil normally soft when wet but hardened when exposed to the air. Beyond -2.00 m is mudstone and building foundation material is usually sourced from this firm rock layer reaching to about -3.00 m below the ground level.

The lateritic soil was sampled using an earth auger from 0.3 to 0.5 m below the ground level. The soil samples were air-dried at room temperature for 2 to 3 days and then stored in plastic containers. The initial water content of the air-dried soil was about 2 - 5%.

Physical Properties Test

The physical properties of soil samples were analyzed using laboratory tests based on ASTM standards. The specific gravity (G_s) test was conducted following ASTM D854 [1]. Grain size distribution analysis was performed following ASTM D422 [2]. Atterberg limits test was applied following ASTM D423 [3], with ASTM D424 [4] used for both liquid

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limit (w_l) and plastic limit (w_p) testing. Loss Angeles (L.A.) abrasion test was conducted following ASTM C131 [5]. Each physical property test was repeated 3 times to calculate the average data.

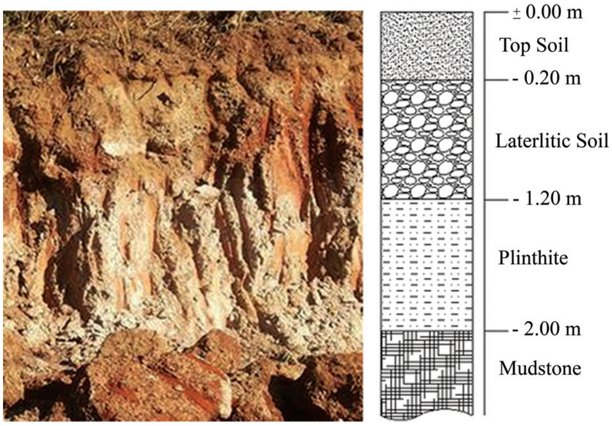

Figure 2 Typical soil profile in Chiang Khruea sub-district

Engineering Properties and In-Situ Test

The engineering properties of the lateritic soils from the two test pits (TP-1 and TP-2) were evaluated through a series of laboratory tests as follows; standard and modified compaction tests were conducted following ASTM D698 [6] and ASTM D1557 [7], respectively, and California Bearing Ratio (CBR) test was performed following ASTM D1883 [8]. In accordance with the DH-S 208/2532 standard, CBR tests were only conducted under soaked conditions. The specimens for CBR test were prepared at optimum water content obtained from the test result of modified compaction test. According to the DH-S 109/2517 standard, the CBR tests were only conducted under a soaked condition. It is well known that a soaked condition provides lower CBR values compared to that in unsoaked condition and that soil with low CBR values is more reliable for usage in construction materials.

For shear strength parameters, CKLS was analyzed using a series of direct shear tests following ASTM D3080 [9] (consolidated drained test, CD). The tested soil was passed through a No. 4 sieve due to the limitation of the test apparatus. Each CKLS sample was compacted into a shear box with dry density (γ_d) and optimum water content (OWC) obtained from the test results of standard compaction tests. Notably, only CKLS from TP-1 was investigated for soil shear strength.

The value of coefficient permeability (k) was calculated from the test results of multiple stage loading (MSL) oedometer testing using the method of Taylor, D. [10]. The oedometer tests were conducted following ASTM D2435 [11]. Studies in the literature have reported that for a low permeability material (less than 10^{-6} m/s), the value of k deduced from an oedometer test result is comparable with direct measurement of k [12] - [13]. The air-dried CKLS was passed through a No. 40 sieve, then mixed with deionized water using a mechanical mixer. The amount of water was about 1.2 times its liquid limit (w_l). Then, the mixed slurry was wrapped and cured for 24 h. After that, the slurry was pre-consolidated in the consolidation ring under a vertical stress of 17 kPa for 24 h. The pre-consolidation pressure of 17 kPa was obtained from trial and error, where a sample could be successfully formed. Subsequently, the pre-consolidated sample was trimmed to 20 mm in height. Then, the specimen was installed in the consolidation test apparatus. After applying the vertical pressure (σ'_v), the settlement with elapsed time was recorded until the specimen achieved a steady state for each σ'_v value (for 24 h). The test was carried out with σ'_v from 10 to 320 kPa, by doubling the stress for each subsequent step. It should be noted that soil permeability was investigated only in the CKLS from TP-1.

The in-situ plate-bearing test was conducted following ASTM D1194 [14] under a dry condition to determine the soil bearing capacity for footing construction of a pedestrian bridge on highway route No. 22 Sakon Nakhon-Nakhon Phanom, Thailand. The set-up equipment installation for the plate bearing test is shown in Figure 3. First, the soil was excavated to the design depth. Then, a thin layer of fine sand was spread over the tested surface to ensure a proper seating between the plate and the ground surface was achieved. After that, the hydraulic excavator was used as counterweight. A circular steel plate (300 mm) was installed on top of the soil surface under the bottom frame of a hydraulic excavator 150 tonne hydraulic piston/pump machine (700 bars) and three dial gauges were installed. Then, the hydraulic piston was used to apply load and the test commenced. The load displacement was recorded during the elapsed time. It should be noted that the size of steel plate was quite different from the size of the foundation, which means that the stress distribution from two conditions may be different. Even though the plate bearing test has some limitations, it is still widely used to determine the bearing capacity and settlement of the soil foundation.

Figure 3 Equipment for plate bearing test

Results of Physical Properties

Some test results of physical properties are listed in Table 1 for specific gravity (G_s), Atterberg limits, and L.A. abrasion of CKLS from two sources. The test results showed that the G_s value of the soils varied from 2.7 to 2.8. The value of G_s of the lateritic soil from TP-1 was about 2.8 which was higher than from TP-2 (which was about 2.7). This may have been because of different soil chemical compositions for the CKLS samples from the different locations. It is well known that high values of G_s for lateritic soil are due to the high content of iron oxides (such as aluminum oxide, iron (III) oxide). The Atterberg limits test results showed that the lateritic sample from TP-1 had higher values for w_l of 36%, plastic limit (w_p) of 18% and plasticity index (I_p) of 18% compared to the lateritic soil from TP-2 with values of w_l of 51%, w_p of 28% and I_p of 23%.

Table 1 Summary test results of CKLS

Soil Property	Unit	Source	
		TP-1	TP-2
Specific Gravity	-	2.80	2.70
Liquid Limit, w_l	%	36.00	51.00
Plastic Limit, w_p	%	18.00	28.00
Plasticity Index, I_p	%	18.00	23.00
L.A. Abrasion	%	42.30	42.80

The results of the grain size distribution (GSD) are shown in Figure 4. It can be clearly seen that most soil particles for both CKLS samples were sand. The results indicated that the CKLS from TP-2 contained fine particles (passing through sieve No. 200) of more than 12.5%, while the CKLS from TP-1 had just 6%. The results of GSD from Figure 4 indicated that the CKLS from TP-2 was outside the specifications reported in the Department of Highway (DOH) standard for selected materials class A, while the CKLS from TP-1 satisfied these specifications. These %passing sieve No. 200 values were in good agreement with the results of the Atterberg limits (Table 1), where there was an indication that higher amounts of fine particles pointed to higher Atterberg limits. Considering the gravel content (%retained No. 4 sieve), the CKLS from TP-1 had average gravel content of about 64.0%, while CKLS from TP-2 had average gravel content of 34.5%. For total gravel and sand content (100% minus %passing No. 200 sieve), the CKLS from TP-1 gave an average content of 94.2%, while the CKLS from TP-2 gave an average content of 86.2%. By comparing the gravel content and half of total gravel and sand content, it was found that the CKLS from

TP-1 was classified as gravel (G) and the CKLS from TP-2 was classified as sand (S). The GSD results demonstrated that the values of the coefficient of uniformity (C_u) and coefficient of curvature (C_c) of the CKLS from TP-1 varied from 44.44 to 55.00 and from 2.78 to 4.65, respectively, while no corresponding measurements were made for TP-2 as the percentage of particles passing the No. 200 sieve was higher than 12.5 %. Based on the Unified Soil Classification Standard (USCS) and the results of GSD and Atterberg limits, the CKLS from TP-1 could be classified as GW-GC or GP-GC, while the soil from TP-2 was classified as SC.

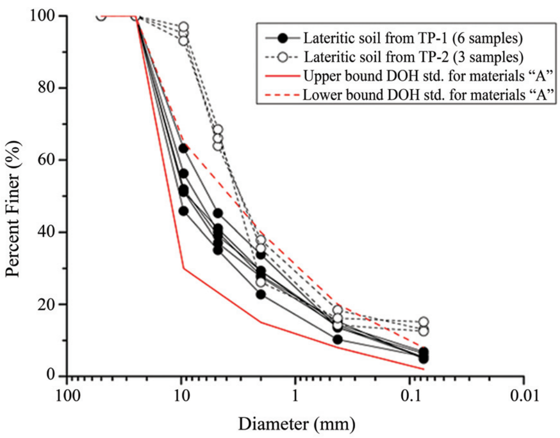

Figure 4 Particle size distribution of lateritic soils

The differences in the physical properties of the CKLS samples from the two locations could have been due to different geographical features between the two test pits (for example, TP-1 had a higher elevation than TP-2). Furthermore, the ground water could have flowed under gravitational forces from TP-1 to TP-2, resulting in a higher ground water level flow around the TP-2 zone (higher advection in fresh water). Therefore, the CKLS sample from TP-2 may have had a higher degree of weathering compared to TP-1.

The L.A. test results are listed in Table 1 and were similar for the two test sites. While these results seemed high, this could be explained by the CKLSs originating from decomposed rock from weathering processes which may have caused the low particle strength.

Compaction Test Results

The relationships of dry density (γ_d, max) and water content (w) of the standard and modified compaction of the CKLS from the two test pits are plotted in Figure 5. The compaction characteristics of CKLS followed a general tendency of an increase in compaction energy with an increase in $\gamma_d, \max$ and a decrease in optimum water content (OWC). The results clearly showed that the CKLS from TP-1 had higher $\gamma_d, \max$ values and lower OWC values from both compaction methods compared with those from TP-2.

Specifically, the values of $\gamma_d, \max$ of the CKLS from TP-1 varied from 2.16 to 2.20 t/m^3 with OWC values in the range of 8.59 - 8.99 %, while the $\gamma_d, \max$ values of the CKLS from TP-2 varied from 2.09 t/m^3 to 2.12 t/m^3 with OWC in the range of 9.64 - 9.72 %. The standard compaction, $\gamma_d, \max$ values varied from 1.87 t/m^3 to 1.92 t/m^3 and from 1.76 to 1.81 t/m^3 for TP-1 and TP-2, respectively, while the OWC values were in the ranges of 11.45 - 13.00%, and 14.23 - 14.25% for TP-1 and TP-2, respectively.

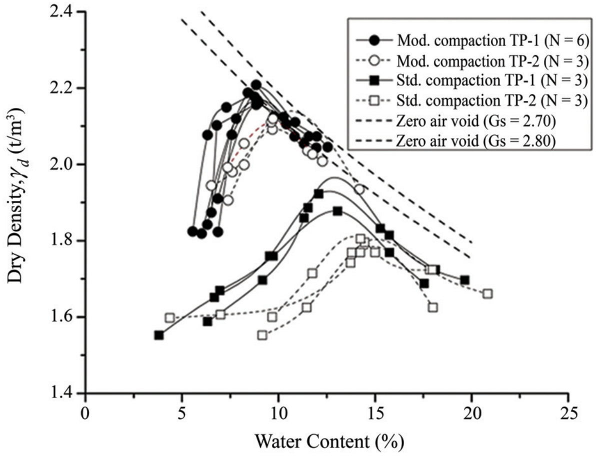

Figure 5 Results of modified and standard compaction tests

California Bearing Ratio (CBR)

The test results of CBR and swelling are shown in Figure 6 and Figure 7, respectively. The CBR results clearly showed that as the density (γ_d) increased, the CBR increased, while the swelling ratio (%) decreased with an increase in γ_d values. Considering maximum dry density ($\gamma_d, \max$) value (section 5.1), it was clear that the CKLS from TP-1 had higher CBR values and lower swelling values compared to TP-2. Based on the test results, the CBR values were approximately 55% and 28% with swelling of 0.7% and 1.1% for compacted CKLS from TP-1 and TP-2, respectively.

According to the properties reported in the sections 5.1 to 5.2, it is indicated that the soil from TP-1 is more suitable for construction materials comparison with the soil from TP-2. Therefore, the soil from TP-1 was selected to further investigate its shear strength parameters and permeability in the next sections.

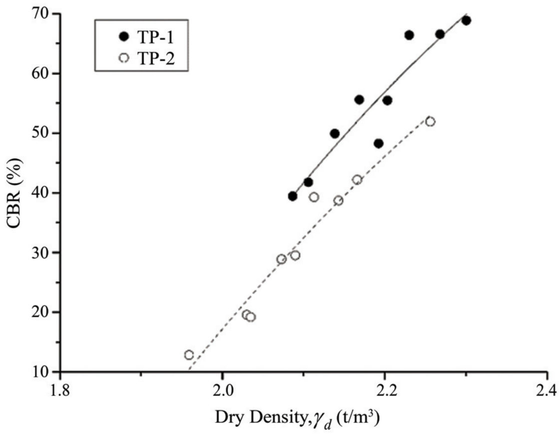

Figure 6 Relationships between CBR and γ_d of compacted CKLS from TP-1 and TP-2

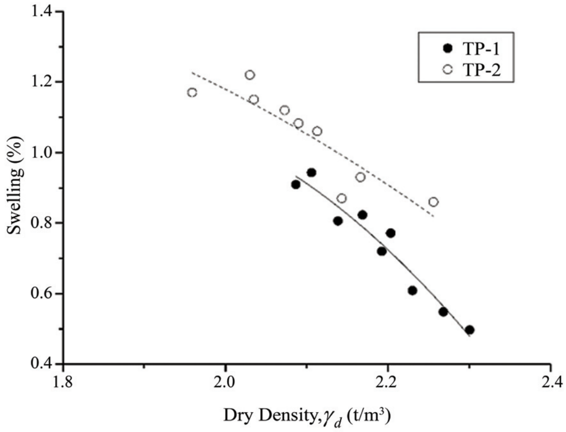

Figure 7 Results of swelling testing for TP-1 and TP-2

Direct Shear Test

The relationships between the shear stress (τ) and horizontal displacement (δ_x) of the compacted CKLS samples are plotted in Figure 8. The results indicated that the shear behavior of compacted CKLS was found in the strain softening, where the shear stress increased with an increase in displacement until a peak shear stress was reached, and then the shear stress value decreased with an increase in displacement to a residual state. Based on the results of τ with σ_n, the Mohr-Coulomb failure envelope can be created as shown in Figure 9. The direct shear test results showed that the ϕ'_{peak} and $\phi'_{residual}$ values were 31.7° and 27.2°, while the c'_{peak} and $c'_{residual}$ values were 68 and 38.2 kPa, respectively. It should be noted that the residual state can be reached when the structures have large displacement, and for the conservative design, the $\phi'_{residual}$ and $c'_{residual}$ values are recommended.

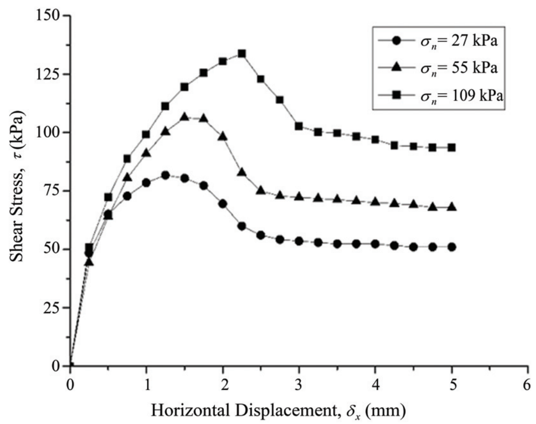

Figure 8 Test results of stress-displacement of compacted CKLS from TP-1

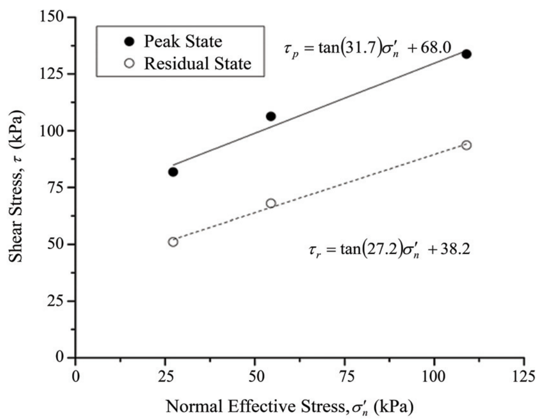

Figure 9 Mohr-Coulomb failure envelope of compacted CKLS from TP-1

Permeability (Consolidation Test)

The relationship between permeability (*k*) and the void ratio (*e*) of the CKLS is shown in Figure 10, indicating that the value of *k* on a log scale was directly proportional to the *e* values, where the *e* values decreased when the *k* values decreased The *k* values of the CKLS samples varied from 1.5 x 10^{-8} to 6.8 x 10^{-10} m/s with *e* values varying from 0.86 to 0.38, respectively. The test result clearly showed that the *k* values in this study are comparable to the test results from another study, which used a direct measurement method to determine *k* values [15].

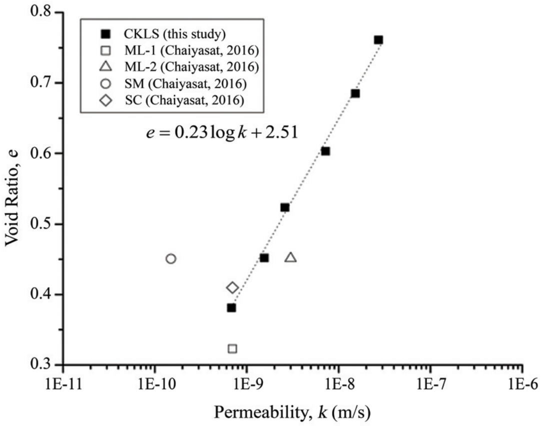

Figure 10 Relationship between void ratio (*e*) and *k* (log) of CKLS

Plate bearing

The plate bearing test results are reported in Figure 11 and show that the maximum bearing capacity or more than 180 t/m^2 can be achieved with displacement of about 9 mm. However, it should be noted that the plate bearing pressure may reduce when testing in wet conditions. The test results showed that the pate bearing pressure increased with an increase in settlement. After the plate bearing pressure reached 160 t/m^2, the plate bearing pressure increased with little settlement because the soil foundation was the very strong mudstone layer. During the test, the capacity of the reaction frame was not enough to support the applied pressure. For safety purposes, the test was terminated before the settlement reached the maximum intended value of 25 mm. Considering if modulus of subgrade reaction (*K*), the plate bearing pressure corresponding to the settlement of 1.25 was 42 t/m^2, then the value of *K* can be calculated following equation (1). The obtained *K* in this study was 33,600 t/m^3.

$$K = P / 0.00125, \tag{1}$$

where *P* is plate bearing pressure (t/m^2) at the settlement of 1.25 mm.

Pavement Application

According to DH-S 208/2532 and DH-S 209/2532, the CKLS samples could be generally classified as selected material A or B for soil foundations in pavement construction. Considering the size of particles as shown in Figure 4, the CKLS sample from TP-1 followed the specifications, whereas the CKLS sample from the TP-2 was outside the specifications.

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Only the CKLS sample from TP-1 had w_l and I_p values that satisfied the minimum requirements for Atterberg limits of a class A material (w_l < 40% and I_p value < 20%), while TP-2 had values higher than those of the requirements. To reduce the w_l and I_p values, one effective method which is easy and cheap is to mix in dust stone. Mahent, R. and Joshi, R. [16] reported that the Atterberg limit of lateritic soil decreased with an increase in the amount of dust stone, resulting in increased maximum dry density ($\gamma_d, \max$).

Both CKLS samples met the minimum CBR specification requirement (CBR value > 25%) following DH-S 208-2532. Additionally, the swelling values of both CBR samples had lower values than the required minimum (swelling < 3%). These test results confirmed that the CKLS sample from TP-1 was suitable as class A or class B material for the subbase layer. However, neither sample satisfied the requirements for use as base material, according to DH-S 201/2544, and their properties would need to be improved. In particular, the minimum L.A. requirement is less than 40%, but both CKLS samples had values exceeding this requirement. However, their properties could be improved by treating with cementitious materials (such as cement or geopolymer), so that high-quality materials such as crushed rock would not need to be transported from other supply zones which would help to minimize the construction budget. It has been reported that lateritic material mixed with cement can replace crushed rock for road construction [17]. Based on test results, it can be recommended that CKLS is good for road subgrade and subbase material. However, the properties of CKLS would need to be improved (e.g., mixing with cement or lime) before being used in base material.

Foundation

This section discusses the bearing capacity of the CKLS samples and their application in soil foundations. A shallow foundation or spread footing, commonly used in residential building construction, is a major component of a building that transmits the building loads to the soil foundation. The ultimate bearing capacity (q_u) of the shallow foundation is based on the simplified idea developed by Terzaghi, K. [18], where q_u can be determined using equation (2) for circular footing.

$$q_u = 1.3c'N_c + q'N_q + 0.3\gamma' BN_\gamma, \tag{2}$$

where c' is the cohesion of soil, q' is the overburden pressure on the footing (unit weight of soil multiplied by the depth of footing), γ' is the unit weight of the soil, and N_c, N_q, N_γ are Terzaghi's bearing capacity factors that depend on the soil friction angle (ϕ').

Based on the test results in section 5 (c' = 3.8 t/m^2, q' = 3.2 t/m^2 considering a depth of footing of 2.0 m, γ' = 1.6 t/m^3, B = 0.3 m (plate bearing), where N_c, N_q and N_γ = 29.24, 15.9 and 11.6, respectively), q_u can be estimated as approximately 199 t/m^2. It is very important to note that the values of the shear strength parameters were obtained from the compacted CKLS sample under standard compaction energy, while the actual shear strength parameters in the field may be lower or higher than the values reported in this study. According to the U.S. Army Corps of Engineers [19], the recommended factor of safety (FS) for shallow foundation is 3. By using an FS value of 3, the allowable bearing capacity (q_a) from Terzaghi's bearing capacity is 66 t/m^2. According to the results of plate bearing testing (Figure 11), considering a plate bearing pressure at 66 t/m^2, the vertical settlement would be only about 2 mm, which is much lower than the maximum allowed value of 25 mm. Based on this analysis, the CKLS would be good for use in soil foundation. Even though CKLS is suitable for shallow foundations, this soil could still collapse due to the leaching process. Fresh water could leach the iron oxide component out of the lateritic soil, resulting in disaggregation between the lateritic soil particles and resulting in the soil strength being diminished. In fact, there are several buildings on KU-CSC which are currently facing this problem as shown in Figure 12 and the differential settlement over a long period can require a large expenditure on building repairs. Therefore, further investigation on the effects of leaching and exchanging cations as well as ways to improve engineering properties in CKLS is required.

Other Applications in Civil Engineering

The use of CKLS for other applications such as in landfill lining and as back fill material were briefly discussed. It is well known that the major parameters controlling the flowability through the compacted clay for waste landfill or impervious core for earth dam are the permeability (k) and the k value that is a function of the void ratio (e).

The e values of the compacted soil can be estimated by considering the k values from the test results in section 5. For example, considering the results of the standard compaction test on the sample from TP-1 (Figure 5), the $\gamma_{d,\max}$ value of compacted soil varied from 1.87 t/m^3 to 1.92 t/m^3 with a G_s value of 2.8. Based on the given data, the estimated e values would be 0.31 to 0.33. By using the proposed equation in Figure 10, the k values can be deduced, producing estimated k values of approximately 2.72 x 10^{-10} to 3.32 x 10^{-10} m/s, which are lower than the typical value of k for landfill liner application (<1 x 10^{-9} m/s) [20]. This analysis indicates that CKLS could be used as a construction material for landfill liner applications. However, the calculated *k* value from the current study was limited only by the permeability of the soil-water interaction, whereas in the field, many chemical compounds could be dissolved resulting in an increased *k* value, especially with the dissolved cations in a solution [21]. Moreover, it is very important to note that in field construction, the *k* value can be higher or lower than the reported *k* values because the grain size distribution of a material is difficult to control, resulting in the *k* value increasing when there are more coarse-grained particles or reducing when there are more fine-grained particles compared to the test results reported in the this study. Therefore, quality checking during construction is very important.

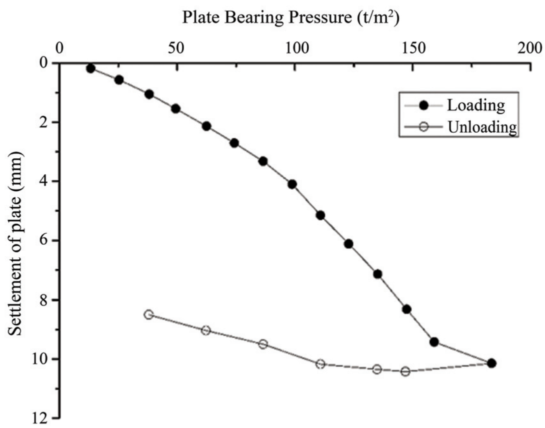

Figure 11 Test results of plate bearing

It is important to note that the calculated *k* values in this study only considered *k* in the vertical direction, whereas for the core material in earth dam applications both the horizontal and vertical directions of *k* should be considered. Thus, the values of *k* reported in this study are not sufficient to assess the *k* value for earth dam applications; more experiments are required to investigate this issue.

CKLS is a high plasticity soil with a low *k* value, and using this soil as backfill could be problematic due to high excess water pressure against the wall due to poor drainage. However, nowadays, geosynthetic materials that can improve soil drainage and increase the shear strength of the soil are widely used [22]. The use of geosynthetics can save on construction costs since marginal soil can be substituted for high-quality backfill material. However, further investigation is needed in the use of geosynthetics to improve the properties of CKLS for backfill material.

Conclusions

This study investigated the properties of Chiang Khruea lateritic soil (CKLS) through a series of laboratory and in-situ tests. Based on the test results, the following conclusions were drawn:

1. The physical, compaction, and CBR test results for the CKLS samples from the two different sites (TP-1 and TP-2) were different. One possible reason for these differences is that the CKLS samples from different locations may have been subjected to different degrees of weathering due to the leaching process.
2. The physical parameters controlling the compaction and CBR values were the grain size distribution and the Atterberg limits (w_l, w_p, and I_p), where a higher fine content (< 0.075 mm), higher Atterberg limits values, and lower maximum dry density resulted in lower CBR values.
3. Shear strength and permeability (k) of CKLS was only investigated at TP-1. The test results showed that the CKLS sample from there had high shear strength parameters and low k values.
4. Based on these test results, CKLS would be good as construction material for the subbase, landfill liner, and soil foundation, but its properties need to be improved before being used for some applications such as backfill and base layer.

Figure 12 Structural cracks on KU CSC buildings due to settlement

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