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

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Abstract

Nowadays, Sakon Nakhon is a city-municipality in Thailand which infrastructures such as highway and building have been rapidly developing. Consequently, the large amount of construction materials such as crushed rock, sand and laterite have been being used for many construction projects. Chiang Khruea sub-district is the main laterite quarries supplied to construction sites in Sakon Nakhon province, and the extracted laterite from this area are singly used for highway sub-base construction due to it has good strength after compaction and low-priced. However, the properties of Chiang Khruea lateritic soil (CKLS) are not yet maximally used for other civil engineering applications such as, landfill liners, backfill material for retaining wall and base layer for paved road. This is because engineers have not yet been fully understanding about its physical and engineering properties. Therefore, this study aims to investigate and report the properties of the CKLS to find the possibility to apply this soil for several applications in civil engineering. The physical and engineering properties of the CKLS were investigated through a series of laboratory and in-situ test. The results reported in this study can be useful for engineers as a reference for sustainable design and construction.

Keywords: *Chiang Khruea Lateritic soil* (*CKLS*), *Civil Engineering, Physical Properties, Engineering Properties*

1. Introduction

Sakon Nakhon province is located in the northeast region of Thailand, and this province is situated on Sakon Nakhon Basin and Phu Phan Range. Generally, the infrastructures in this area have been developing around Nong Han Lake, where it is the biggest natural lake in the northeast region of Thailand with area about 125.2 km^2. The lacustrine deposit surrounded the Nong Han Lake is very fine soil particles. For flood plain area around the Nong Han lake area, the soil is an alluvial deposit that come from 21 natural creeks before discharge to this big lake. In case of the soil in undulating-terrain area, the residual Chiang Khruea lateritic soil (CKLS) is usually found, and it is normally decomposed from cretaceous sedimentary rocks named shale or mudstone. Generally, the soil in this area are usually using for pottery, and the province's pottery village is located at Ban Chiang Khrua. Along the Phu-Phan Range, mostly rocks are cretaceous to Jurassic era namely Phu Phan sandstone.

In August 2012, Sakon Nakhon became a city-municipality of Thailand, resulting in many infrastructures have been rapidly developing, and the large amount of construction materials have been being consume for many construction projects. Chiang Khruea is sub-district of Sakon Nakhon province which contains the large laterite quarry supplied for many construction projects in this area. The residual CKLS is decomposed from mudstone by leaching process, and one possible leaching mechanism is an advection of the freshwater (e.g., rain and groundwater). This soil is loaming-skeletal with reddish brown color, and riches in iron oxide. Generally, the soil profiles in this area are composed of the topsoil with thickness of about 0.2 to 0.5 m below ground surface. Beyond this soil layer, the lateritic soil with large boulder particles and thick sheet of laterite could be found. The soil conditions in this area are not good for agriculture, but the lateritic soil is good for road construction materials (Mengue et al., 2017). For example, in 1964, the old Chiang Khruea airport was constructed for US army military base during Vietnam War due to lateritic soil foundation is firmed, then 30 years later, this area had been developed to a government university in 1994 namely the Kasetsart University Chalermphrakiat Sakonnakhon Province Campus (KU-CSC). Presently, the CKLS extracted from this area is generally used for road embankment, subgrade, and subbase due to it has good strength after compaction and low-priced. However, the CKLS is only limit used for the material of road construction because its mechanical behavior is not well-understood.

Regarding to the limitation discussed above, this study aims to investigate and report the physical and engineering properties of the CKLS through a series of laboratory and in-situ test. This study is expected to provide an essential information of the CKLS properties in the design and construction of various civil engineering applications.

2. Materials

This study investigated on physical and engineering properties of Chiang Khruea lateritic soils (CKLS) through a series of laboratory and field tests. The soils were collected from different locations in Kasetsart University Chalermphrakiat Sakon Nakhon Province Campus (KU-CSC) as shown in Figure 1. The test pit 1 (TP-1) is located in N = 17.287237, E = 104.106361 with mean sea level (MSL) of +168.00 m, and the test pit 2 (TP-2) is located in N = 17.290853, E = 104.115049 with MSL of +166.70 m. Additionally, plate bearing test was performed to evaluate the bearing capacity of the soil foundation, and the location of the in-situ test is shown in the Figure 1.

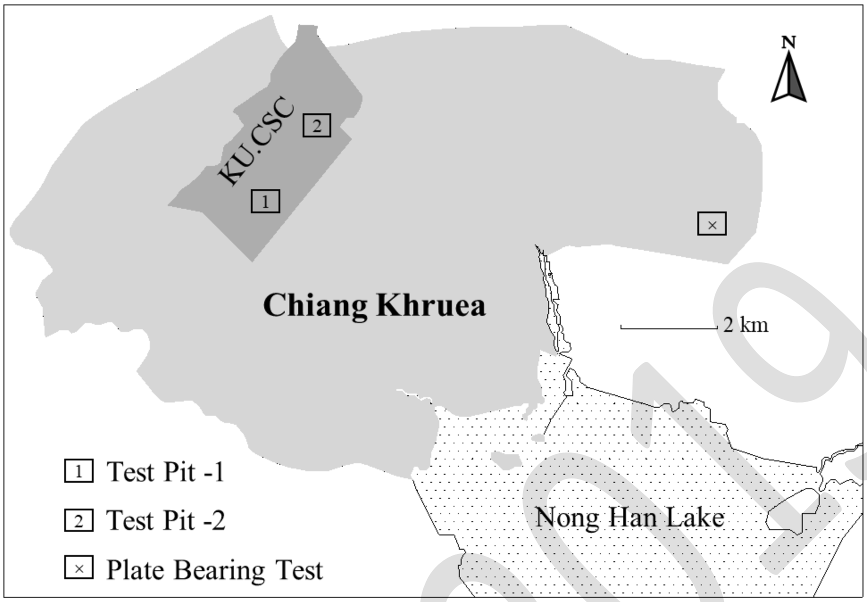

Figure 1 The soil sampling and plate bearing test location

Generally, the lateritic soils in this study are residual soil decomposed from the mudstone, and the typical soil profile are shown in Figure 2. The topsoil was found in the depth from ground level 0.00 m to 0.20 m in depth. This soil layer was loose state and contains high amount of organic. Then, from the depth of -0.20 m to -1.20 m, it is the lateritic soil with various sizes of the soil particles mixed with laterite boulders. After -1.20 m, the white soil layer was plinthite with thickness about 0.8 m, where its properties were iron-rich soil that was normally soft when wet, and it becomes hard when exposures to the air. Beyond -2.00 m, it was mudstone and the building foundation in this area generally sited on this firmed rock layer with depth about -2.00 to -3.00 m below ground level.

For soil sampling in this study, the lateritic soils were sampling by earth auger from -0.3 to -0.5 m below ground level. Then, the soil samples were air-dried in the room temperature about 2 to 3 days, and then the air-dried soil were storage in the plastic container. The initial water content of the air-dried soil was about 2 to 5%.

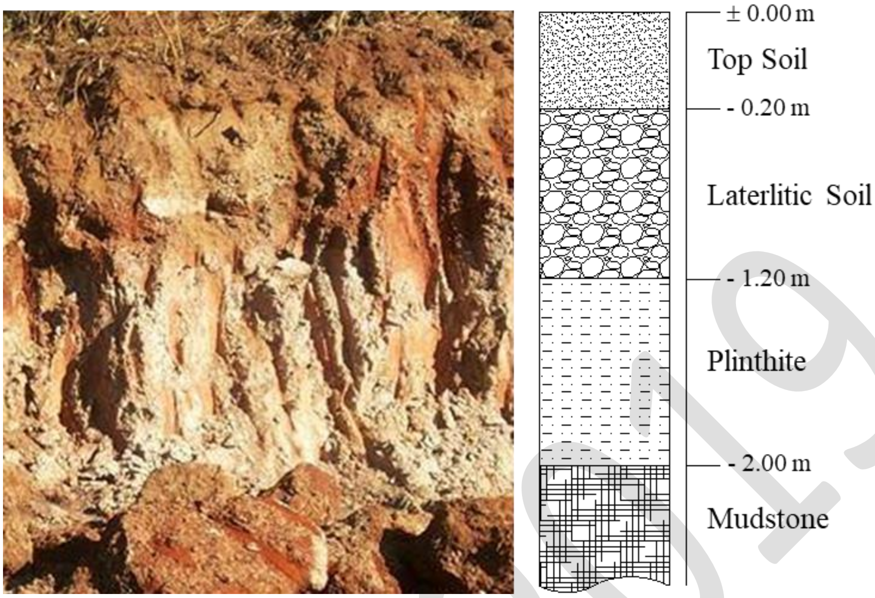

Figure 2 A typical soil profile in Chiang Khruea sub-district

3. Test Methods

3.1 Physical Properties Test

The physical properties of soil were conducted as following the basic soil properties in the soil laboratory test based on the ASTM standard. Specific gravity (G_s) test was conducted following ASTM D854. Grain size distribution analysis was performed following ASTM D422. The Atterberg's limit test was investigated following ASTM D244 and ASTM D427 for liquid limit (w_l) test, and plastic limit (w_p) test, respectively. Loss Angeles (L.A.) abrasion test was conducted following ASTM C131.

3.2 Engineering Properties and In-Situ Test

The lateritic soils from two test pits (TP-1 and TP-2) were evaluated their engineering properties through a series of laboratory test as follows; standard and modified compaction tests were conducted following ASTM D698 and ASTM D1557, California Bearing Ratio (CBR) test was performed following ASTM D1883. According to the DS-H 208/2532 standard, the CBR tests were only conducted under a soaked condition.

For shear strength parameters, the CKLS was conducted through a series of direct shear test following ASTM D3080. The tested soil was sieved passing through the No. 4 sieve due to the limitation of the test apparatus. The CKLS were compacted into the shear box with control the dry density (γ_d) and optimum water content (OWC) obtained from the test results of standard compaction test. It is worth to note that only CKLS from the TP-1 was investigated the shear strength of the soil.

The value of coefficient permeability (k) was calculated from the test results of multiple stage loading (MSL) oedometer testing using Taylor's (1948) method [1]. The oedometer tests were conducted following ASTM D2435. There are publications in the literature 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 a directly measured k [2,3]. The air dried CKLS was sieved passing through No. 40 sieved, then mixed with deionized water by 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 the vertical stress of 17 kPa, the pre-consolidation was kept for 24 h. Subsequently, the pre-consolidated sample was trimmed into 20 mm in height. Then, the specimen was installed in the consolidation test apparatus. After applied the vertical pressure (σ_v'), the settlement with elapsed time was recorded until the specimen achieved the steady state for each σ_v' value (about 24 h). It is noted that only the CKLS from the TP-1 was investigated the soil permeability.

The in-situ plate-bearing test was conducted following the ASTM D1194 under the dry condition to determine soil bearing capacity for footing construction of pedestrian bridge on highway route No. 22, Sakon Nakhon-Nakhon-Phanom, Thailand. The set-up equipment installation for plate bearing test is shown in Figure 3. For installation steps, firstly, the soil was excavated to the design depth. Then, thin layer of fined sand was place on top of the tested surface to make sure the proper seating between plate and ground surface. After that, the hydraulic excavator was used as counterweight. The circular steel plate (300 mm) was installed on top of the soil surface under the bottom frame of hydraulic excavator. The 150-ton hydraulic piston/pump machine (700 bars) and three dial gauges were installed. Then, the hydraulic piston was applied the load and the test was started. The load-displacement were record with elapsed time.

Figure 3 Equipment for plate bearing test

4. Results of Physical Properties

Table 1 Summary test results of CKLS

Soil properties	Unit	Resources TP-1	TP-2
Specific gravity, G_s	-	2.8	2.7
Liquid limit, w_l	%	36	51
Plastic limit, w_p	%	18	28
Plasticity index, I_p	%	18	23
L.A. abrasion	%	42.3	42.8

Some test results of physical properties such as, specific gravity (G_s), Atterberg's limits, L.A. abrasion of the CKLS from 2 resources are listed in the Table 1. The test results show that the G_s of the soils are varying from 2.7 to 2.8. The value of G_s of the lateritic soil from TP-1 was about 2.8 which is higher than that of the G_s from TP-2 with the value of about 2.7. This may because the CKLS from different locations may have the different chemical compositions. It is well known that the higher values of the G_s than the typical values of G_s of other soils is because the lateritic soil contains high content of the iron oxide (e.g., aluminium oxide, Iron (III) oxide, etc). The results of the Atterberg's limit show that the lateritic from TP-1 had the w_l of 36 %, plastic limit w_p of 18 % with plasticity index (I_p) of 18 %, while the lateritic soil from TP-2 had w_l of 51 %, w_p of 28 % and I_p of 23 %. The results show that the CKLS from TP-1 had higher w_l, w_p and I_p values compared to those of the CKLS from the TP-2.

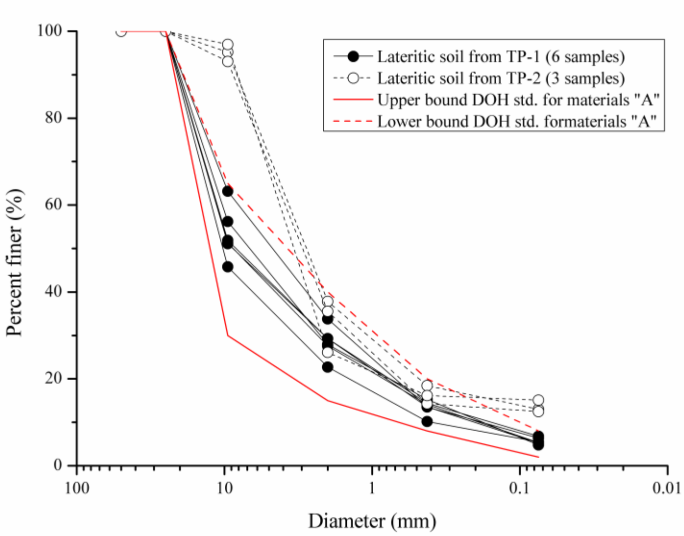

Figure 4 Particle size distribution of lateritic soils

The results of the grained size distribution (GSD) are shown in the Figure 4. It can be clearly seen that the majority of soil particle for both CKLS were sand. The results indicate that the CKLS from the TP-2 contained the fine particle (passing No. 200) more than 12.5%, while the CKLS from TP-1 had percent passing No. 200 just about 6%. The results of GSD from Figure 4 indicate that the CKLS from TP-2 is out of specification reported by DH-S standard for a selected materials class A, while the CKLS from TP-1 is following the specification. The results of %finer passing No. 200 sieve are good agreement with the results of Atterberg's limits (Table 1), where the higher fine particle, the higher Atterberg's limits values can be obtained. The results of the GSD demonstrate that the values of coefficient of uniformity (C_u) and coefficient of curvature (C_c) of the CKLS from the TP-1 varied from 44.44 to 55.00, and C_c values varied from 2.78 to 4.65, while the TP-2 was not determined the C_u and C_c due to the %finer passing No. 200 higher than 12.5%. For the soil classification under to the Unified Soil Classification Standard (USCS), based on the results of GSD and Atterberg's limits, the CKLS from TP-1 can be classified as SW-SC or SP-SC, and the soil in TP-2 was classified as SC.

The different in physical properties of the CKLSs from two locations can be described as; there is one possible mechanism explained the test results which is the reaching process. Due to the different geography (i.e., the TP-1 had higher elevation than the TP-2). The ground water can be flowing from area around TP-1 to TP-2 area, resulting in higher ground water level around the TP-2 which can be higher amount of freshwater advection into the soil layer. Therefore, the CKLS around TP-2 zone may have higher degree of weathering compared to that of the CKLS around the TP-1.

For test result of L.A. as listed in the Table 1, it reports that the CKLS from the TP-1 had the value of L.A. almost the same as the CKLS from the TP-2. The results of the L.A. values seem high, and the reason may because the CKLSs are a decomposed rock from weathering process which may be low the particles strength.

5. Results of Engineering Properties

5.1 Compaction Test

The relationships of dry density ($\gamma_{d,\ max}$) and water content (w) of the standard and modified compaction of the CKLS from 2 test pits are plotted in the Figure 5. The compaction characteristic of CKLS are following general tendency that the increasing of compaction energy results in increasing in $\gamma_{d,\ max}$ and decreasing in optimum water content (OWC). The results are clearly shown that the CKLS from TP-1 had higher $\gamma_{d,\ max}$ values and lower OWC values for both compaction methods compared with those of the CKLS from TP-2.

For the values of $\gamma_{d,\ max}$ and OWC from modify compaction, it can be summary that the values of $\gamma_{d,\ max}$ of CKLS from TP-1 are varying from 2.16 to 2.20 t/m^3 with OWC values in the range of 8.59 to 8.99 %, while $\gamma_{d,\,max}$ of CKLS from TP-2 had lower values with the $\gamma_{d,\,max}$ values varying from 2.09 t/m^3 to 2.12 t/m^3 with OWC of about 9.64% to 9.72%. For the results of the standard compaction, $\gamma_{d,\,max}$ values are varying from 1.87 t/m^3 to 1.92 t/m^3 and 1.76 to1.81 t/m^3 for TP-1 and TP-2, respectively, while the OWC values are varying from 11.45% to 13.00%, and 14.23 to 14.25 % for the TP-1 and TP-2, consequently.

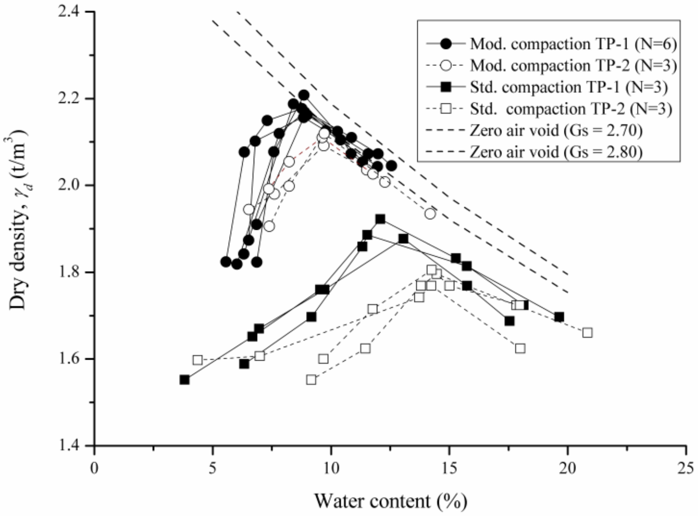

Figure 5 Results of modify and standard compaction test

5.2 California Bearing Ratio (CBR) test

The results of CBR clearly show that the density (γ_d) increases, resulting in the CBR values increases, while the swelling ratio (%) decreases with increasing of the γ_d value. Considering under the same γ_d value, it is clearly seen that the CKLS from the TP-1 had higher CBR value, and lower swelling value compare to that of the CKLS from the TP-2. Based on the test results, the CBR values are about 55% and 28% with swelling of 0.7 % and 1.1 % for compacted CKLS from the TP-1 and the TP-2, respectively.

5.3 Direct Shear Test

The relationships between the shear stress (τ) and horizontal displacement (δ_x) of the compacted CKLS are plotted in Figure 8. The results indicate that the shear behavior of the compacted CKLS is found the strain softening after reached the peak state, while the τ values increase with increasing of the normal stress (σ_n). Based on the results of τ with σ_n, the Morh-Coulomb failure envelope can be created as shown in Figure 9. The test result of direct shear test found that the ϕ_{peak} and $\phi_{residual}$ values were 38.2°and 27.2°, while the c_{peak} and $c_{residual}$ values were 68 and 31.8 kPa, respectively. It is noted that the residual state can be reached when the structures had large displacement, and for the conservative design, the $\phi_{residual}$ and $c_{residual}$ value are recommended.

5.4 Permeability (Consolidation Test)

Relationship of permeability (*k*) and void ratio (*e*) of the CKLS is shown in Figure 10. The relationship indicates that the values of *k* in log scale is directly proportional to the *e* values, where the *e* values decrease, the *k* values decrease. The *k* values of the CKLS are varying from 1.5×10^{-8} to 6.8×10^{-10} m/s with the *e* values varying from 0.86 to 0.38, respectively.

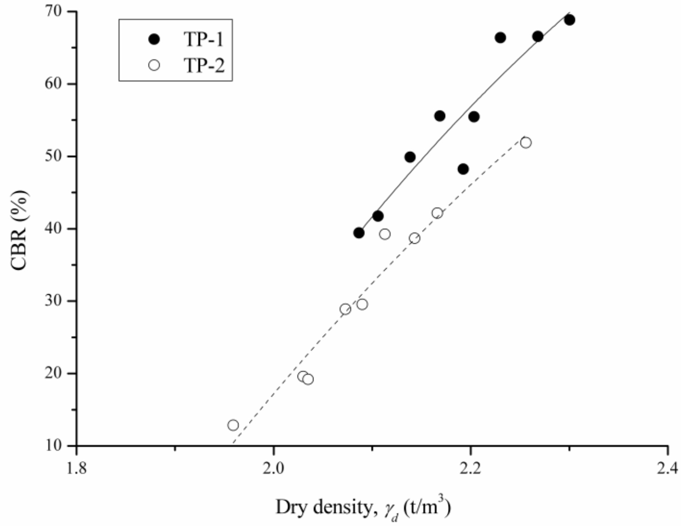

Figure 6 Relationships between CBR and γ_d of compacted CKLS from TP-1 and TP-2

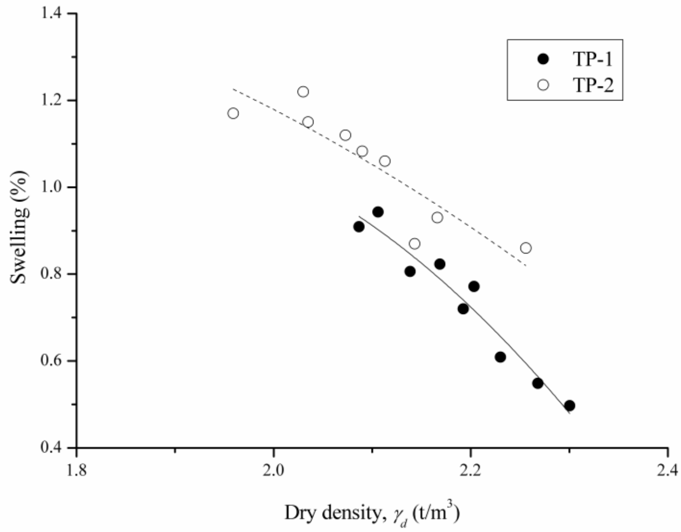

Figure 7 The results of the swelling of the TP-1 and TP-2

5.5 Plate bearing

The test results of plate bearing test is reported in Figure 11, and the result shows that the maximum bearing capacity can be reached to more than 180 t/m^2 with the displacement of about 9 mm. However, it is noted that the plate bearing pressure may reduce when testing in wet condition. The test result shows that the pate bearing pressure increase with increasing of the settlement. After the plate bearing pressure reached to 160 t/m^2, it is worth to note that the value of plate bearing pressure increase with small settlement because the soil foundation was very strong mudstone layer. During the test, the capacity of the reaction frame was not enough to support the applied pressure. For safety purpose, the test was early stopped before the settlement reached to the maximum value of 25 mm.

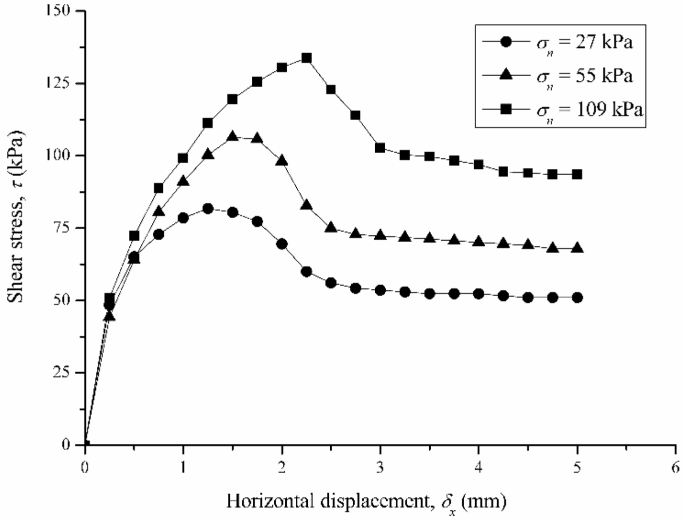

Figure 8 The test results of the shear stress-displacement of the compacted CKLS from TP-1

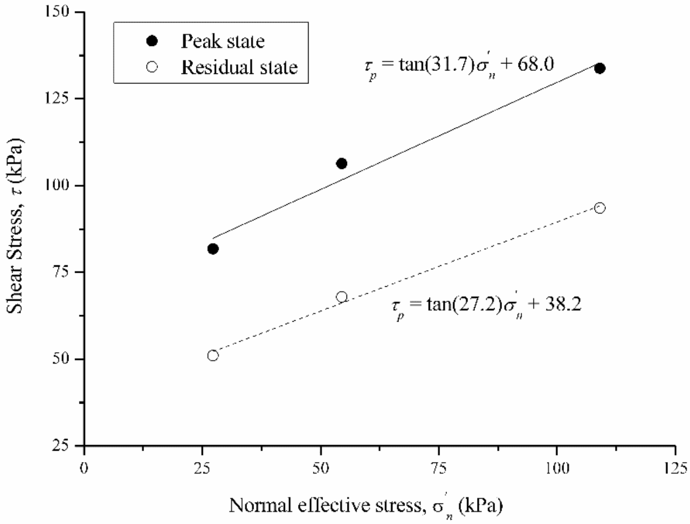

Figure 9 Mohr-Coulomb failure envelope of the compacted CKLS from TP-1

6. Applications in Civil Engineering

6.1 Pavement

According to the DH-S 208/2530 and DH-S 209/2532, the CKLS are generally classified as a selected material A or B for soil foundations in pavement construction. Considering the size of particles as shown in Figure 4, the CKLS from TP-1 was following the specification, whereas the CKLS from the TP-2 out of specification. For results of Atterberg's limits, only CKLS from TP-1 had the w_l and I_p values follows the minimum requirement of the class A material (w_l < 40% and I_p value < 20%), while the TP-2 had value higher than that of the requirement. To reduce the w_l and I_p value, one effective method which easy and cheap is mixing with the dust stone. [4] reported that the Atterber's limit of the lateritic soil decrease with increasing of the amount of dust stone, resulting in maximum dry density ($\gamma_{d,\,max}$) increased.

For CBR specification, both CKLS, meet the minimum requirement (CBR value >10%) following the DH-S 208-2532. Additionally, the swelling values of both CBR specimen had lower values than that of the minimum requirement (swelling < 3%). Based on the test results in this study, it is confirmed that the CKLSs from TP-1 and TP-2 are suitable for class A and class B material for subbase layer, respectively. Considering for the class of construction materials for a base layer, both soils are not suitable for the base material according to the DH-S 201/2544, and their properties need to be improved. Especially, the L.A. abrasion, minimum requirement of the L.A. value should be less than 40%, but both CKLSs had the values exceed the minimum requirement. However, their properties can be improved by treating with cementitious materials (e.g., cement, geopolymer), where the high-quality materials such as, crushed rock do not need to transport from other resources which can be minimized the construction budget. [5] reported that the lateritic mixed with cement can be replacing the crushed rock for road construction.

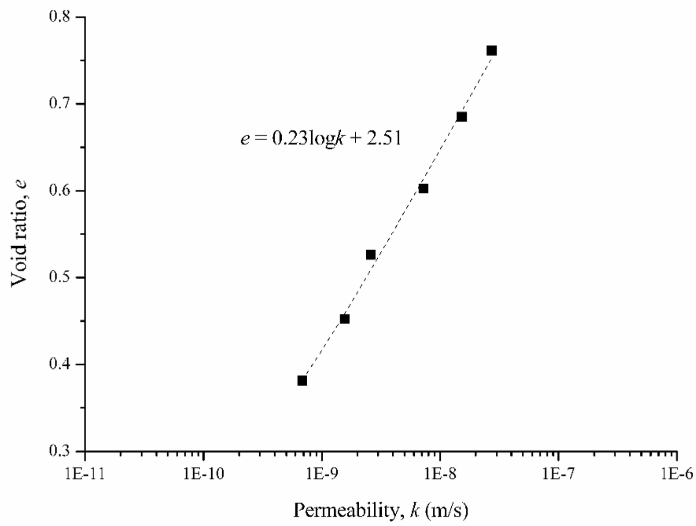

Figure 10 Relationship between void ratio (*e*) and log*k* of CKLS

6.2 Foundation

This section discussed the bearing capacity of the CKLS, and its application for soil foundation. Shallow foundation or spread footing is a major component of a building that transfers the building loads to the soil foundation, and it is commonly used for residential building in this area. The ultimate bearing capacity (q_u) of the shallow foundations is mostly based on the simplified idea developed by [6], where the q_u can be determined by equation 1 for circular footing.

$$q_u = 1.3c' N_c + q' N_q + 0.3\gamma' B N_\gamma \tag{1}$$

where c' is cohesion of soil, q' is overburden pressure on footing (unit weight of soil multiply by depth of footing), γ' is unit weight of soil, while N_c, N_q, N_γ are Terzaghi's bearing capacity factors depend on 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 dept 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), the calculated q_u can be estimating, and the calculated q_u is approximately of 199 t/m^2. It is very important to note that the values of shear strength parameters were obtained from the compacted CKLS specimen under standard compaction energy, while the shear strength parameters in the field may lower of higher than the values reported in this study. According to the U.S. Army (1992) [7], the recommendation value of factor of safety (FS) for shallow foundation is 3. By using the FS of 3, the allowable bearing capacity (q_a) from Terzaghi's bearing capacity is 66 t/m^2. According to the result of plate bearing (Figure 11), considering the plate bearing pressure at 66 t/m^2, the vertical settlement just only about 2 mm, which much lower than the maximum requirement value of 25 mm. Based on this analysis, it can be reported that the CKLS is good for soil foundation. Even through the CKLS is suitable for shallow foundations, but this soil still can be collapsed due to the leaching process. The fresh water can be reaching the iron oxide component out from lateritic soil, resulting in disaggregate between lateritic soil particles (i.e., the strength of soil was diminished). Thus, there are several buildings in this area which are currently facing on this problem as shown in Figure 12. The differential settlements can be found in all buildings in KU-CSC after long-term condition, and the university spends a lot of money to repair the building. Therefore, the investigation of the effect of reaching and exchange cation of the CKLS as well as the way to improve its engineering properties against the reaching mechanism are needed to be further investigation.

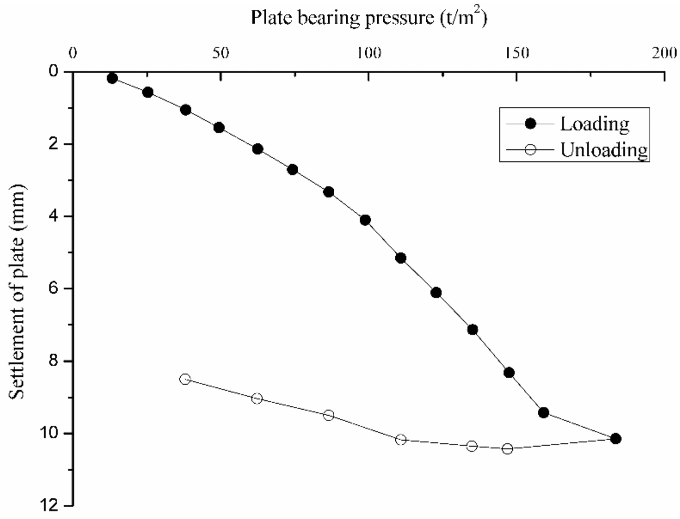

Figure 11 Test results of plate bearing

6.3 Other applications in Civil Engineering

The overviews of using CKLS for other applications such as landfill liner and back fill material were briefly discussed. It is well known that the major parameter controlled the flow ability through the compacted clay for waste landfill or impervious core for earth dam is the permeability (*k*), and the *k* value is a function of the void ratio (*e*).

Considering test result in section 5, The *e* values of the compacted soil can be estimated. For example, considering the results of standard compaction test of the TP-1 (Figure 5), it is indicated that the compacted soil had $\gamma_{d,\,max}$ varied from 1.87 t/m^3 to 1.92 t/m^3 with G_s value of 2.8. Based on the giving data, estimated *e* values were 0.31 to 0.33. By using the proposed equation in Figure 10, the *k* values can be deduced, and the estimated *k* values were about 2.72×10^{-10} to 3.32×10^{-10} m/s, which lower than the typical value of *k* for landfill liner application ($<1\times10^{-9}$ m/s) [8]. Regarding to the analysis, it can be reported that the CKLS can be using as a construction material for the landfill liner applications. However, the calculated *k* value from this study are limit only for permeability of soil-water interaction, but in the field many chemical compositions can be dissolve, and it can be increasing the *k* value, especially the dissolved cation in a solution [9]. Moreover, it is very important to note that in the field construction, the *k* value can be higher or lower than the reported *k* values because the grain size distribution of the material are difficult to control, in which the *k* value may increase when it contains higher coarse-grained particles or the *k* value may decrease when it contains higher fine-grained particles compared to the test result reported in this study. Therefore, the quality checking during construction is very important.

It is important to note that the calculated *k* values in this study were only *k* in vertical direction. For the core material in earth dam applications, it should be considering in both directions (horizontal and vertical directions) of *k* value. Thus, the values of *k* reported in this study are not sufficiently used to assess the *k* value for earth dam applications, and more experiments are required to investigation.

For backfill materials, the CKLS is a high plasticity soil with low *k* value, by using this soil as a back fill, it can be facing the high excess water pressure beside the wall due to it poorly drainage. However, nowadays, geosynthetic materials are widely used, and it can be improving the soil drainage and increasing the shear strength of soil [10-12]. By using the geosynthetics, it can be saving the construction cost by using the marginal soil instead of the high quality backfill material. However, the further investigation of using geosynthetics improved the properties of the CKLS for backfill materials is needed to study in the future.

Figure 12 Structures cracks on the building inside KU CSC due to the settlement.

7. Conclusions

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

1. Regarding to the results of physical, compaction, and CBR tests, the CKLSs from two different resources (TP-1 and TP-2) had different in physical properties, compaction behaviour, and CBR values, and one possible mechanism described this phenomenon is

the CKLS from different locations may have the difference in degree of weathering due to the leaching process.

2. The physical parameters controlled the compaction and CBR values are the grained size distribution and the Atterberg's limits (w_l, w_p, and I_p), where the higher fine content (<0.075 mm), higher the Atterberg's limits values, lower maximum dry density resulting in lower CBR values.
3. For the investigation of shear strength and permeability (k) of the CKLS, only the CKLS from TP-1 was investigated, and the test results show that the CKLS had high values of the shear strength parameters, and low k values.
4. Based on test results, it can be reported that the CKLS is good for construction materials for subbase, landfill liner and soil foundation, but some applications such as backfill, and base layer need to improve its properties before used.

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