Lowland Technology International 2020; Volume 22(2): 228-238
International Association of Lowland Technology (IALT): ISSN 1344-9656

Research Paper

Engineering Characteristics of Khon Kaen Loess as Construction Material

P. Chindraprasirt [1], A. Kampala [2], A. Arngbunta [3], and S. Horpibulsuk[4]

ARTICLE INFORMATION

Article history:

Received: 16 December , 2019
Received in revised form: 4 June, 2020
Accepted: 20 June, 2020
Publish on: 06 September , 2020

Keywords:

Loess soil
Transportation soil
Consistency index
Engineering properties
Soil behavior
Strength parameters

ABSTRACT

In Khon Kaen province, the northeast of Thailand, the soil layer consists mostly of silty sand and silty clay with fine soil grains. This soil is a wind-blown deposit into layers of soil and is named “Loess” or “Khon Kaen Loess”. Thus far, there has been a limited studies conducted on its engineering properties and the use of this soil is also limited. Accordingly, this research studied the improvement of the basic and engineering properties of this Loess with addition of clay. The results indicated that Loess can be classified as inactive -silty-sandy clay. The yield stress of Loess-clay decreased with the increasing amount of clay and was directly related to the compressive strength. The compression index and swelling index also decreased with increasing consistency index. The ratio of $\left(e/e_{LL}\right)$ is significantly correlated with the coefficient of compressibility and permeability coefficient. Still, the effective cohesion of Loess is relatively low (approximately 20 kPa) and it will not change with the clay amount. The maximum of the effective internal friction angle is 25 degrees while the soaked bearing strength seems to decrease following the increasing amount of clay. Notably, the basic and engineering parameters of Khon Kaen loess are mutually connected via an experiential relationship whereas the predicted outcomes and the results from other studies are similar. The experiential equation from this research could be used to predict the engineering properties of silty soil or other types of low-swelling clay.

1. Introduction

The soil layers in Khon Kaen Province, northeast of Thailand, composes of 2 types of soil: 1) Silty Sand (SM) found at the depths of around 0-1.5 m and 5.0-8.0 m; and 2) Silty Clay (CL) found at the depth around 1.5 – 5.0 m or between the layers of SM.

The layer of SM consists of a wind-blown deposit of yellow fine-grain soil that have been piled up in the same area and is called “Loess” or “Khon Kaen Loess”. Typically, it is a fine-grained loam with a sizable amount of calcium. In the dry condition or compacted state with favorable humidity, it shows a good bearing capacity of over 1,000 kPa, the coefficient of permeability of $1\text{x}10^{-5}$ to $1\text{x}10^{-7}$ cm/sec, and low linear shrinkage of 1%. However, in wet

[1] Professor, Sustainable Infrastructure Research and Development Center, Department of Civil Engineering, Faculty of Engineering, Khon Kaen University, Khon Kaen 40002, Thailand; and Academy of Science, The Royal Society of Thailand, Dusit, Bangkok, 10300, Thailand, prinya@kku.ac.th
[2]*Assistant Professor, Department of Civil Engineering, Faculty of Engineering, Rajamangala University of Technology Isan. Khon Kaen Campus. 150 Srichan Street, Muang District, Khon Kaen, 40000, Thailand, apichit.ku@rmuti.ac.th
[3] Graduate Students, Department of Civil Engineering, Faculty of Engineering, Rajamangala University of Technology Isan. Khon Kaen Campus. 150 Srichan Street, Muang District, Khon Kaen, 40000, Thailand, minanukul@gmail.com
[4] Professor, School of Civil Engineering, Director, Center of Excellence in Innovation for Sustainable Infrastructure Development, Suranaree University of Technology, University Avenue, Muang District, Nakhon Ratchasima, 30000, Thailand, suksun@g.sut.ac.th
Note: Discussion on this paper is open until March 2021

condition or with 5-8% of moisture, this loess loses its strength and shows the bearing capacity less than 50 kPa while the linear shrinkage increases to 8-10% (Lommler and Bandini, 2015; Li P et al., 2014; Al-Rawas, 2000; Nouaouria et al., 2008; Phien-wej et al., 1992; Rogers et al., 1994; Rogers, 1995; Ryashchenko et al., 2008).

In contrast, CL is an inorganic silty soil with low to medium plasticity and its property is affects by the amount of clay. In terms of engineering property, CL has low shear strength, low permeability, and high consolidation (Horpibulsuk et al., 2007).

According to the previous studies, these two soils are problematic and not suitable for construction (Li P et al., 2014; Zhang F et al., 2013). Nevertheless, it is unavoidable that a number of infrastructures such as road, railway and building have to be constructed on these soils as illustrated in Figs. 1 and 2.

In the case of construction of road, railway and building groundwork, several engineering parameters as shown in Table 1 are required for the analysis of the geotechnical and foundation engineering designs. A number of studies presented the relationship between the engineering parameter and the basic property of soft clay soil (Horpibulsuk et al., 2007; Cox, 1971; Ladd et al., 1971; Balasubramanian,1975; Brenner et al., 1979; Akagi, 1981; Bergado, 1990; Kim et al., 1994; Horpibulsuk and Rachan, 2004). However, there are a few studies conducted on the layers of loess, silty sand, and silty clay (Sadia et al., 2012). In fact, the clay within the layers of Loess is also variable and affects the engineering properties of the soil. Consequently, it is necessary to use the equation to estimate the engineering parameters from the test on either physical properties or index properties in order to obtain the engineering properties for the construction design in a large area such as roadwork, railway work and piping work.

As a result, this research purposively explored the basic properties of soil including specific gravity), grain size distribution, consistency index, and compaction; and the engineering properties including shear strength, permeability coefficient, and consolidation; and Khon Kaen Loess with variable clay amount.

Fig. 1. The Road Construction with Loess

The test results were analyzed and synthesized to construct the relationship between the engineering parameters and the parameters derived from the basic property test that is the main factor with an impact on the engineering properties of Khon Kaen loess. Above all, the benefit from this research would help engineer or designer with a simple method to evaluate different types of engineering property and the result could be used to create an appropriate and accurate design of the geotechnical engineering for the construction work.

Fig. 2. The Infrastructure and Pavement and Railway Construction with Loess

2. Materials and Methods

2.1 Materials

Khon Kaen Loess, and clay were taken from the borrow pit in Ban Phai, Muang District, Khon Kaen. The clay sample was mixed with the Loess to simulate the real soil used for construction. The soil samples were classified into 7 groups with clay contents of 0, 10, 20, 30, 40 50 and 100% of the dry weight of soil. The details of the soil content and basic properties of the 7 groups are given in Table 2. According to the USCS system, the loess was a silty sand (SM), loess with 10-50% clay was the low plasticity clays (CL); whereas, the clay was the high plasticity clay (CH), as shown in Fig. 3. The plasticity index and liquid limit of samples were affected by the amount of clay. However, the relationship between the plasticity index and liquid limit was a straight line above an A-line and between an A-line and U-line and could be described by Eqn. (1).

$$PI = 0.73(LL - 10) \tag{1}$$

Based on this relationship, the engineering properties of the loess-clay samples could be estimated from the consistency index. The utilization of this relationship would be discussed later. The soils (except 100% clay) were tested for their basic and engineering properties.

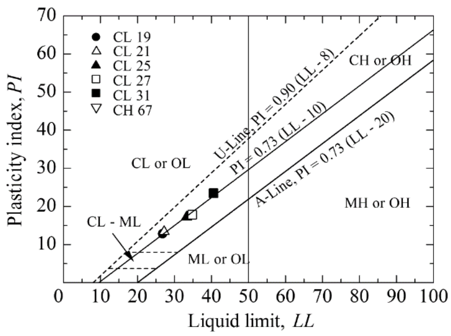

Fig. 3. The Relationship between Plasticity index and Liquid Limi

Table 1. The Parameters for Geotechnical and Foundation Engineering Designs and Analysis

Design	Parameter	Test
Bearing capacity Slope Stability	- Dry unit weight) γd(- Cohesion, c' - Internal Friction angle, ϕ' - Undrained Shear Strength, Su	- Standard Penetration test (SPT) - Plate bearing test (PBT) - Direct Shear Test - Unconfined Compressive Strength Test (UCS Test) - Triaxial Compression Test
Compressibility or Settlement	- coefficient of consolidation, Cv - Compression index, Cc - Swelling index, Cs) - Modulus of Deformation (E_{def})	- Triaxial Compression Test - Consolidation Test

Table 2. Basic Properties of Loess-Clay

Sample number	Sample names	Loess : Clay	% of clay fraction	Specific Gravity, (Gs)	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI (%)	Soil classification
1	CL 15	100 : 0	15.24	2.60	24.70	14.02	10.68	SM
2	CL 19	100 : 10	18.78	2.62	26.88	14.14	12.74	CL
3	CL 21	100 : 20	20.76	2.63	27.19	13.81	13.38	CL
4	CL 25	100 : 30	24.69	2.65	33.29	16.06	17.23	CL
5	CL 27	100 : 40	26.56	2.66	34.90	17.20	17.70	CL
6	CL 31	100 : 50	31.49	2.67	40.63	17.25	23.38	CL
7	CL 67	0 : 100	67.19	2.69	62.72	35.06	27.66	CH

2.2 Test on Physical Property

The soils were: tested for their physical properties as follows:

1. Specific gravity test in accordance with ASTM D 854.
2. Soil particle size distribution by wet sieving test in accordance with ASTM D 422.
3. Soil particle size distribution by hydrometer test in accordance with ASTM D 422-63.
4. Liquid limit and Plastic limit tests in accordance with ASTM D 4318.
5. Modified compaction test in accordance with ASTM D 1557

2.3 Test on Engineering Property

The tests engineering property consisted of the unconfined compressive strength test, bearing capacity test, consolidation test, and direct shear test.

1. The unconfined compressive strength test was performed following the ASTM D2166.The test was performed with samples compacted in a

10.16 cm (4'') standard mold under the modified compaction and the optimum water content (OWC). Before testing, the sample soil was soaked for 2 hours and then dried to a saturated surface dry stage. After that, it was tested in compression at the displacement rate of 1.0% of sample soil height per minute.

2. The bearing capacity of California Bearing Ratio (CBR) test was based on ASTM D1883-99 Standard under the modified compaction strength at both soaked and un-soaked states. In terms of soaked stage, the sample was soaked for 4 minutes and swelling rate was measured. This test was performed by pressing an iron rod with a 19.35 cm diameter on the compacted sample at the speed of 1.27 millimeters per minute.
3. The consolidation test was also based on the ASTM D 2435 and performed with the sample 63 mm in diameter and 20 mm in height compacted under modified compaction and at OWC. The sample was pressed with the weights of 10, 20, 40, 80, 160, 320, 640, and 1280 kPa and rebounded to 100 kPa. During the test, the settlement rate was recorded at 0.25, 0.30, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, 960, and 1440 minutes.
4. The direct shear test was performed following the ASTM D 3080-98 with sample 63 mm in diameter and 20 mm in height compacted under the modified compaction and at OWC. it was later tested for its shear strength at the soaked stage where it was pressed throughout the consolidation period. After that, the sample was constantly sheared at the rectilinear motion rate of 1.27 - 2.54 mm. per minute. In all tests, five samples were tested under the same condition to obtain reliable results. The results under the same testing condition were reproducible with low mean standard deviation, SD ($SD/\bar{x}$ < 10%, where = mean strength value).

3. Test results

Fig. 4 presents the relationship between the plasticity index and activity index of loess-clay samples. The activity index increased with increasing amount of clay. The activity index ranged between 0.5 and 1.0 so the soil could be classified as an interactive normal clay (Skempton and Northey, 1953). Fig. 5 displays the relationship between free swelling ratio (FSR) and clay content based on the work of Prakash and Sridharan (2004). The free swelling ratio increased with increasing amount of clay. The FSR ranged between slightly less than 1.0 – 1.5. The soil was the low-swelling clay containing Kaolinite and Montmorillonite as the clay forming minerals (Prakash and Sridharan, 2004).

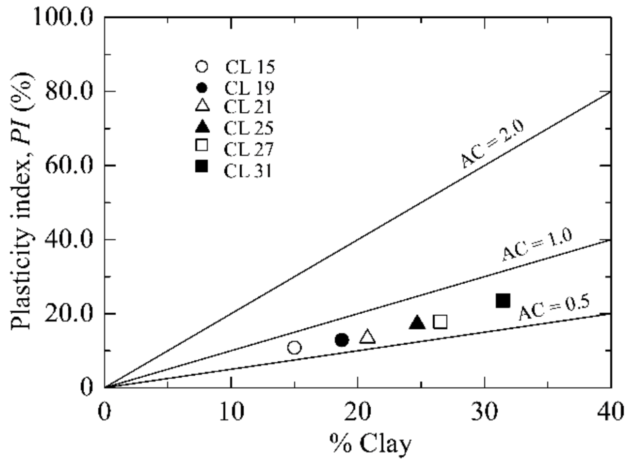

Fig. 4. Relationship between Plasticity Index and Activity Index of Loess-Clay

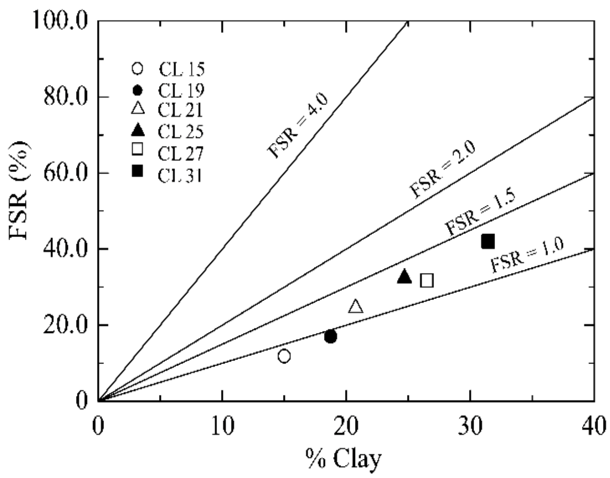

Fig. 5. Relationship between Free Swelling Ratio and Clay Mineral Classification Criteria of Loess-Clay

Fig. 6 presents the relationship between the dry unit weight and water content of loess-clay under the modified compaction. The maximum dry unit weight of loess (CL15) was the highest and decreased with the increasing amount of clay, whereas the OWC increased.

Fig. 7 portrays the relationship between the void ratio and the effective vertical stress for compressibility of loess-clay under the modified compaction. The sample with high amount of clay seemed to be more swelling if the humidity content was increased (after being soaked before testing). This swelling made (initial void ratio, e_o) and the compression curve of the sample clay CL67 (without loess soil) reached their maximum followed by CL31, CL27, CL25, CL21, CL19, and CL15 respectively. The test results also confirm that the maximum yield stress of Khon Kaen loess was the highest and it seemed to decrease significantly following an increasing amount of clay.

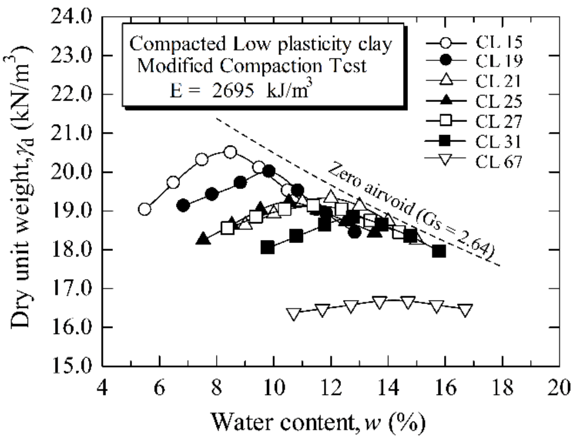

Fig. 6. Relationship between Dry Unit Weight and Water Content of Loess-Clay

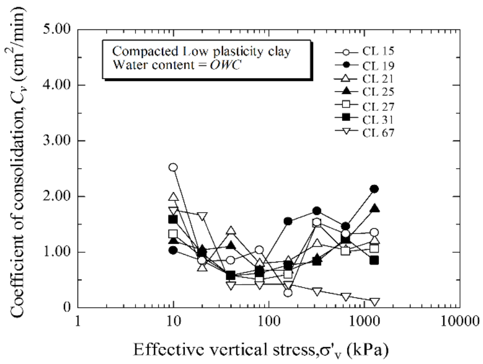

Fig. 8. Relationship between Coefficient of Consolidation and Effective Vertical Stress of Loess-Clay

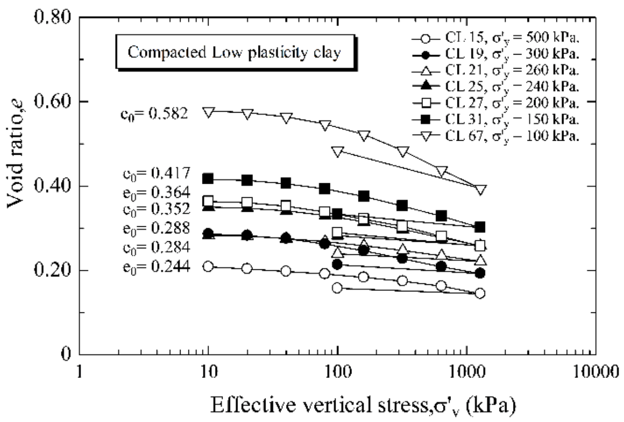

Fig.7. Compressibility of Loess-Clay

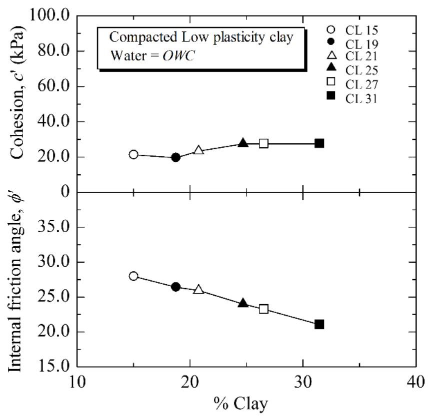

Fig. 9. Relationship between Effective Cohesion and amount of Clay, and Internal Friction Angle and Amount of Clay

Fig. 8 shows the relationship between coefficient of consolidation and effective vertical stress of loess-clay under the modified compaction. The coefficient of consolidation of sample CL67 (100%clay) decreased with the increasing effective vertical stress. For the Loess with clay, the coefficient of consolidation initially decreased with increasing effective vertical stress and at around the effective vertical stress of 100 kPa, the coefficient of consolidation started to increase with increasing effective vertical stress.

Fig. 9 shows the relationship between the effective cohesion and amount of clay; and internal friction angle and amount of clay under the modified compaction and the direct shear test for the consolidation settlement. According to the Fig., the effective cohesions of all samples were small with an average around 20 kPa. There was a tendency that the effective cohesion slightly increased with the increasing amount of clay. The internal friction angle, however, increased with the increasing amount of clay with the maximum at 27 degree.

Fig. 10 describes the relationship between the unconfined compressive strength and amount of clay under the modified compaction tested at both soaked and un-soaked states. The unconfined compressive strength of un-soaked samples was higher than those of the soaked samples. For the un-soaked condition, the unconfined compressive strength of Loess-clay increased with the increasing amount of clay. Whereas, it decreased with the increasing amount of clay for the soaked conditions.

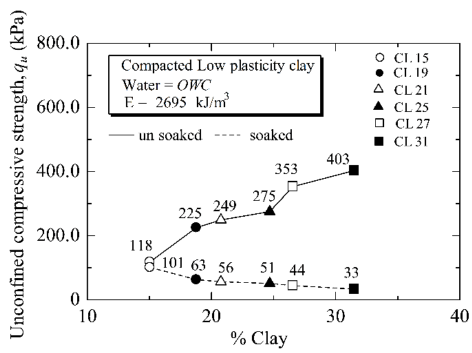

Fig. 10. The Relationship between Un-soaked and Soaked Strength and Clay Amount of Khon Kaen Loess with Variable Clay Amount

Fig. 11 presents the relationship between the California bearing ratio (CBR) and amount of clay in both soaked and un-soaked states under the modified compaction. The Fig. shows that the results of CBR followed the same trend as those of unconfined compression test. This was as expected as the two tests were related to the strength condition of the samples.

Fig. 12 describes the relationship between the swelling ratio and amount of clay under the CBR test at the soak state. The test result confirmed that the swelling ratio of Loess was low at around 0.2%. This swelling ratio increased significantly with the increasing amount of clay.

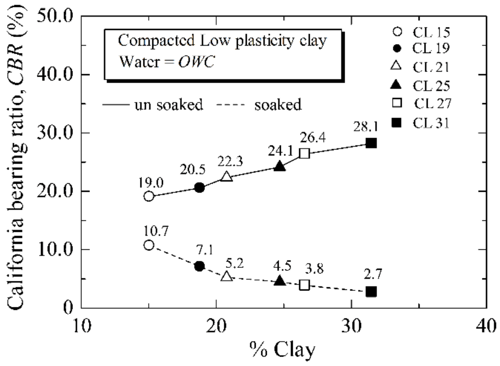

Fig.11. Relationship between CBR and Amount of Clay

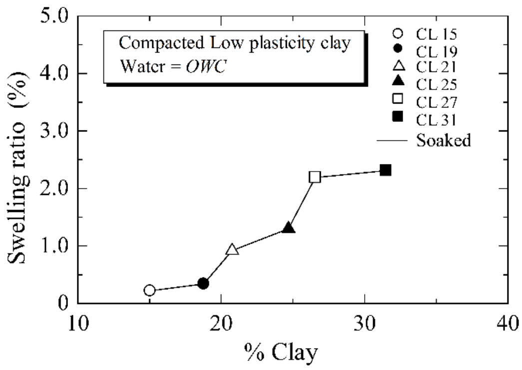

Fig. 12. Relationship between Swelling Ratio and Amount of Clay (%)

4. Discussion

Based on the consistency index (Skempton and Northey, 1953) and free swelling ratio test using the Prakash and Sridharan's method (Prakash and Sridharan, 2004) on the Loess with variable amount of clay (See Fig. 4), it was found that the activity indices of sample soils were between 0.5 and 1.0. As a result, the soils could be designated as inactive – normal clay. This was similar to the results from the FSR test (See Fig. 5). The FSR values ranged between 1.0 and 1.5 and according to could be classified as inactive clay containing Kaolinite and Montmorillonite as its forming minerals.

According to the consolidation test, the dry unit weight and OWC were related to the amount of clay. As the clay within the layers of loess was an inactive one and its liquid limit (LL) controlled the water content for the consolidation (Horpibulsuk et al., 2008; Nagaraj et al., 2006). Moreover, the result from the data synthesis could be used to form an equation for the relationship between the optimum water content and LL of the loess under the modified compaction strength as displayed in Fig. 13. Based on this relationship, the OWC could be predicted from Equation (1) (See Table 3).

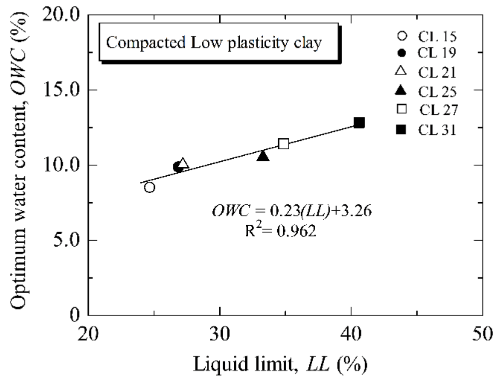

Fig. 13. Relationship between OWC and LL of Loess-Clay

From the compacted soil's property, the dry unit weight was typically varied by the OWC so it was possible to form an equation for the relationship between the dry unit weigh and OWC as shown in Fig. 14. Accordingly, the relationship between the maximum dry unit weigh and OWC could be described by Equation (2) (See Table 3).

According to the relationship between the OWC and LL and the relationship between the dry unit weigh and OWC, it was possible to predict the maximum dry unit weight and OWC of Loess from the result of the Atterberg Limit test.

The key parameters for clay settlement were the coefficient of consolidation, coefficient of volume compressibility, compression index, and swelling index. These parameters could be obtained from the consolidation test of the water-saturated clay (Nagaraj and Srinivasa Murthy, 1986; Nagaraj et al., 1994).

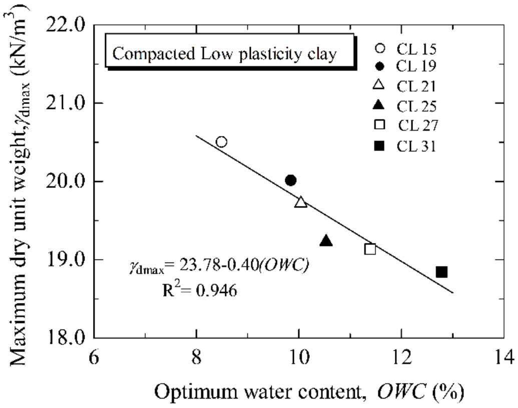

Fig. 14: Relationship between Maximum Dry Unit Weight and OWC of Loess-Clay

The result from the consolidation test of the loess with variable clay amount indicated that the clay within the sample swelled after being soaked in water so that the initial void ratio increased with increasing amount of clay while the yield stress or pre-consolidation pressure decreased. This notable behavior indicated that the control variables of clay (the liquid between the pore fluid and the surface area) depending on the types of clay minerals) that would also decrease the unconfined compressive strength of the soaked clay in the same manner (See Fig. 10).

In addition, the yield stress and unconfined compressive strength were closely related since these two parameters varied with the strength of soil. The relationship between yield stress and unconfined compressive strength of the soaked sample shown in Fig. 15 could be described by Equation (3) (See Table 3).

From the data obtained from this experiment, the compression index and swelling index of Loess with variable amount of clay were related to the consistency index as illustrated in Fig. 16 which could be described by Equations (5) and (6) (See Table 3).

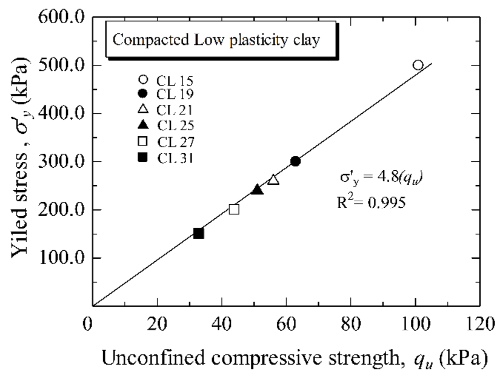

Fig. 15. Relationship between Yield Stress and Unconfined Compressive Strength of Loess-Clay

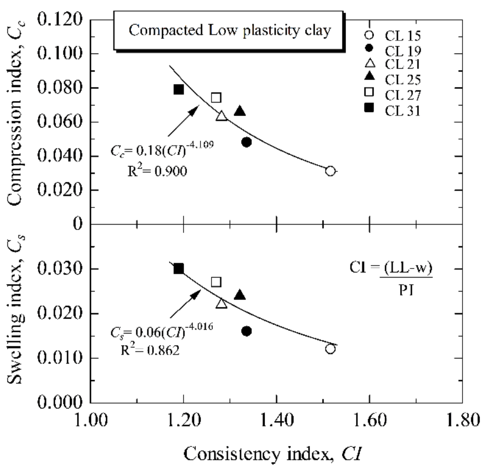

Fig. 16. Relationships between a) Compression Index, and Consistency Index; and b) Swelling Index and consistency Index of Loess-Clay

In utilizing the Intrinsic state line (ISL) introduced by Nagaraj et al. (1998) and Horpibulsuk et al., (2007), the relationship between normalized void ratio and effective vertical stress of remolded high plasticity clay were shown to be linear. The relationship for Loess with variable amount of clay was also linear as shown in Fig. 17. The position of $(e/e_{LL}, \sigma'_{vo})$ Loess was below the intrinsic state line and this behavior confirmed that Loess was a structural soil that became more stable after compaction. Consequently, the relationship between the natural state and intrinsic state could be described by Equation (4) (See Table 3).

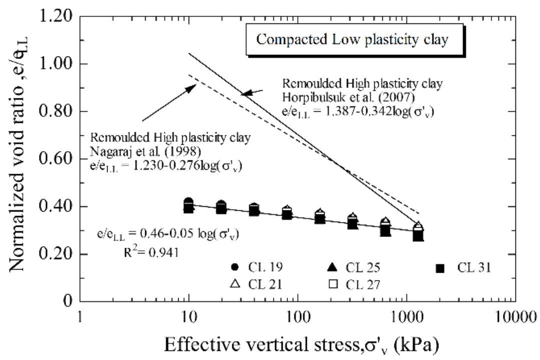

Fig. 17. Relationship between Natural State and Intrinsic State of Loess with Variable Amount of Clay

Since the relationship between PL and LL of clay between the layers of Khon Kaen loess was homogeneous (See Fig. 4) and consistent with the study outcome by Allen (2016) and Hughes (2002), PI and LL could be the key parameters for the behavioral analysis on the consolidation of Loess with variable clay amount. When (e/e_{LL}) was considered for its relationship with other engineering parameters including the coefficient of volume compressibility and permeability coefficient, their relationships were highly significant as illustrated in Figs. 18 and 19, respectively. The relationship was very beneficial for the estimation of coefficient of volume compressibility and permeability coefficient since they are the key parameters for the analysis of settlement and settlement rate which usually requires complicated method of testing and analyzing. In this regard, the relationships between the normalized void ratio and coefficient of permeability; and the coefficient of volume compressibility and coefficient of permeability were described by Equations (7) and (8) (See Table 3).

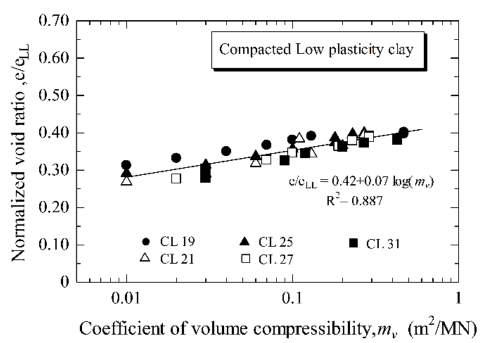

Fig. 18. Relationship between Normalized Void Ratio (e/e_{LL}) and Coefficient of Volume Compressibility of Loess with Variable Clay Amount

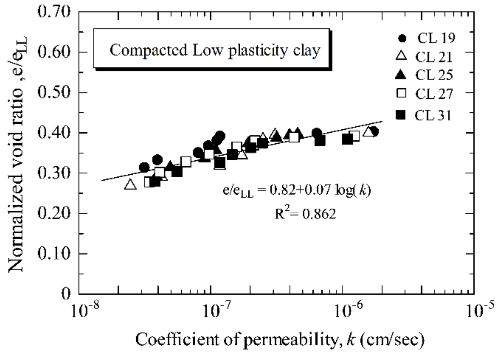

Fig. 19. Relationship between Normalized Void Ratio (e/e_{LL}) and Coefficient of Permeability of Loess with Variable Clay Amount

Even though the coefficient of consolidation might be either increased or decreased with the increasing effective vertical stress (See Fig. 7), it is still the key parameter with linear relationships with the coefficient of volume compressibility, changes in the effective vertical stress, and the permeability coefficient. The relationship between the coefficient of consolidation/coefficient of permeability (C_v/k) and the coefficient of volume compressibility (m_v) is shown in Fig. 20 and described by Equation (9) (See Table 3).

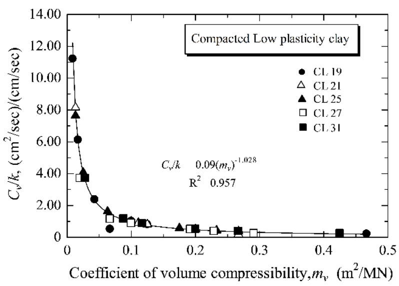

Fig. 20. The Relationship between (C_v/k) and Coefficient of Volume Compressibility (m_v) of loess with Variable Clay Amount

At this point, the bearing capacity of loess is analyzed in term of the effective strength parameter which is a specific value for each loess with varying amount of clay. The test result affirmed that the clay had no influence on changes in the effective cohesion but it could explicitly change the coefficient of internal friction angle of soil grains (See Fig. 9) because the clay forming minerals was an indicator of the properties of clay within Loess. The result from the FSR test also suggested that Kaolinite and Montmorillonite were the main elements of clay (See Fig. 7) and after testing at the water-saturated state, the presence of water made the clay swell as the internal friction angle was lowered with the increasing amount of clay. This behavior created the relationship between the internal friction angle and the plasticity index (PI) since PI defined the level of plasticity of clay. The clay with high plasticity index would show a high-volume change (contraction or swelling) when the humidity was changed. The relationship between the internal friction angle $(\sin\ \phi')$ and Plasticity Index (PI) as illustrated in Fig. 21 could be described by Equation (10) (See Table 3).

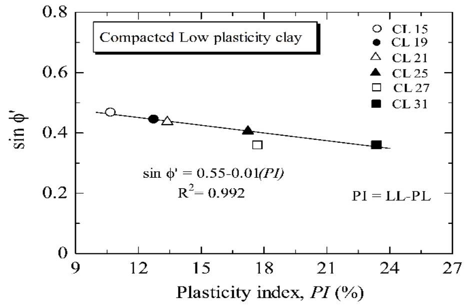

Fig. 21. Relationship between Internal Friction Angle $(\sin\ \phi')$ and Plasticity Index (PI) of Loess with Variable Clay Amount

For this Loess, the amount of clay had a direct influence on the shear strength and the consistency index (CI) or the liquidity index (LI) became the key parameters controlling the consistency state of soil and had a direct relationship with the undrained shear strength (Nagaraj et al., 1990; Wood, 1990). Due to this behavior, it was possible to estimate the unconfined compressive strength at the soaked state and the consistency index (CI) as shown in Fig. 22 while the relationship between the unconfined compressive strength and the consistency index (CI) could be explained by Equation (11) (See Table 3). In addition, the previous studies (Nagaraj et al., 2018; Rashid et al., 2014) confirmed that it was possible to create the relationship between the unconfined compressive strength and the bearing capacity and similarly CBR could be estimated from this relationship as shown in Fig. 23 where the bearing capacity was slightly less than 10% of the unconfined compressive strength. The relationship between the California bearing ration and the unconfined compressive strength could be explained with Equation (12) (See Table 3).

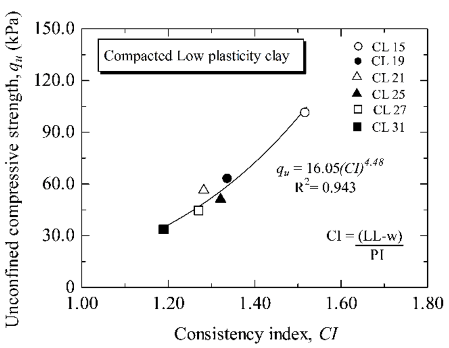

Fig. 22. The Relationship between Unconfined Compressive Strength and Consistency Index of Loess with Variable Clay Amount

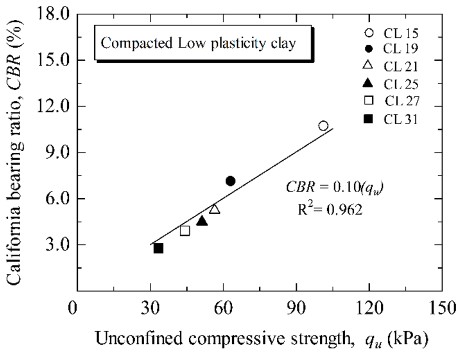

Fig. 23. The Relationship between California Bearing Ratio and Unconfined Compressive Strength of Loess with Variable Clay Amount

Fig. 24 shows the comparison between the results from this test and those predicted by the experiential equation including the OWC, maximum dry unit weight, void ratio, compression index, swelling index, and internal friction angle of both silty clay and low swelling clay that were reported by Punrattanasin (2005), Anantasech et al (2005), Ma et al. (2017), Kim et al., (2013) and Yodsa-nga et al. (2012). The outcome indicated that the ratio between the laboratory result and the predicted one (Exp / Pre) was very good with 9 test results falling very close to 1.0 and all 14 results were between 0.8 – 1.2 affirming that the equation was accurate. This outcome could also be extended with the experiential equation used in this study could be used to predict other types of silty clay and clay of low plasticity.

Table 3. The Experiential Relationship between Basic Parameters and Engineering Parameters of Loess with Variable Clay Amount

Equations No.	Engineering Parameter	Empirical relationships	Independent variables
[1]	Optimum water content, *OWC* (%)	$OWC = 0.23(LL) + 3.26$	*LL* (%)
[2]	Maximum dry unit weight, $\gamma_{d\max}$ (kN/m^3)	$\gamma_{d\max} = 23.78 - 0.4(OWC)$	OWC (%)
[3]	Yield stress, σ'_y	$\sigma'_y = 4.79(q_u)$	q_u (kPa)
[4]	Compressibility curve	$\frac{e}{e_{LL}} = 0.46 - 0.05\log(\sigma'_v)$	e_{LL}, σ_v'
[5]	Compression index, C_c	$C_c = 0.18(CI)^{-4.109}$	$CI = \frac{LL - w}{PI}$
[6]	Swelling index, C_s	$C_s = 0.06(CI)^{-4.016}$	$CI = \frac{LL - w}{PI}$
[7]	Coefficient of volume compressibility, m_v (m^2/kN)	$\frac{e}{e_{LL}} = 0.42 + 0.07\log(m_v)$	e_{LL}, e
[8]	Coefficient of permeability, k (cm/sec)	$\frac{e}{e_{LL}} = 0.82 + 0.07\log(k)$	e_{LL}, e
[9]	Coefficient of consolidation, c_v (cm^2/sec)	$\frac{c_v}{k} = 0.09(m_v)^{-1.028}$	k (cm/sec) m_v (m^2/MN)
[10]	Internal friction angle, ϕ'	$\sin\phi' = 0.55 - 0.01(PI)$	$PI = LL - PL$
[11]	Unconfined compressive strength, q_u (kPa)	$q_u = 16.05(CI)^{4.48}$	$CI = \frac{LL - w}{PI}$
[12]	CBR (%)	$CBR = 0.1(q_u)$	q_u (kPa)

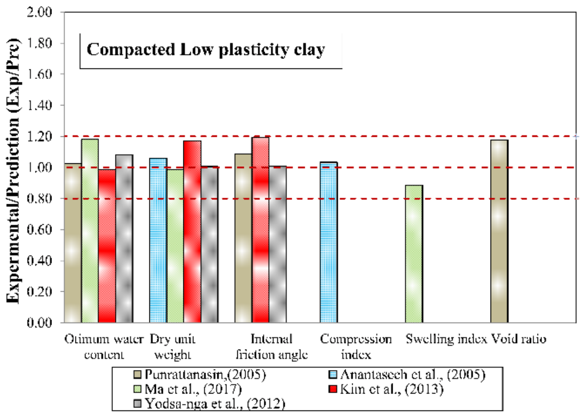

Fig. 24. Prediction of Compacted Low Plasticity Clay

5. Conclusions

Based on the study, the following conclusions could be made.

1) The loess in Khon Kaen is classified as silty sand (SM). Mixed with clay, the soil samples are defined as the low plasticity clay (CL). The sample soil showed its activity index between 0.5 to 1.0 and the free swelling ratio ranged from 1.0 to 1.5 so it could be classified as the inactive and normal clay.

2) The maximum yield stress of compacted loess was the highest and decreased with the increasing amount of clay. This yield stress was directly related to the unconfined compressive strength and their relationship was linear.

3) The compression index and the swelling index of loess with variable clay amount were significantly decreased by an increasing level of the consistency index

4) The coefficient of volume compressibility and the permeability coefficient of Khon Kaen loess with variable clay amount have a significant relationship with the void ratio and the void ration at the consistency state.

5) The maximum effective cohesion of Khon Kaen loess compacted was relatively low and stable averagely at 20 kPa; it cannot be changed by clay amount. Besides, the maximum internal friction angle was 25 degree and can be gradually decreased by an increasing amount of clay.

6) The unconfined compressive strength and the shear strength of Khon Kaen loess compacted with the maximum dry unit weight increases following the increasing clay amount; meanwhile, the unconfined compressive strength and the shear strength at the soaked state seems to decrease when the clay amount is increasing.

7) Both basic and engineering parameters of Khon Kaen loess have an experiential relationship to one another; meanwhile, the predicted parameters and the test results from several previous studies are notably similar. Furthermore, the experiential equations proposed in this research work can be useful for the prediction on other types of silty or inactive clay.

Acknowledgements

The authors acknowledge the financial supports provided by Rajamangala University of Technology Isan KhonKaen Campus. The first author would like to acknowledge the support of the Thailand Research Fund through Distinguished Research Professor Grant DPG 6180002.

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