International Journal of GEOMATE, April., 2021, Vol.20, Issue 80, pp.159-167
ISSN: 2186-2982 (P), 2186-2990 (O), Japan, DOI: https://doi.org/10.21660/2021.80.j2067
Geotechnique, Construction Materials and Environment

BEARING CAPACITY OF SHALLOW FOUNDATION ON COLLAPSIBLE KHONKAEN LOESS

Kokkrit Piyasawat, Ratamanee Nuntasarn, Dolrerdee Hormdee and Pongsakorn Punrattanasin

Faculty of Engineering, Khon Kaen University, Thailand

*Corresponding Author, Received: 28 Dec. 2020, Revised: 30 Jan. 2021, Accepted: 05 Feb. 2021

ABSTRACT: This research studied the bearing capacity of an isolated shallow foundation on wet and dry Khon Kaen loess. The bearing capacity was field-tested by a static load test at various depths for dry and wet conditions. The foundation size of 1.0x1.0 m was placed at a depth of 0.5, 1.0, and 1.5 m. Moreover, the foundation sizes of 1.2x1.2 m and 1.5x1.5 m were placed at only a depth of 1.0 m. Four LVDTs were installed at the four corners of the foundation to measure the settlement due to the loading. The load of 25 mm settlement was observed and defined as a failure in this study. The triaxial test under unconsolidated undrained was investigated as the shear strength parameters of this study. Moreover, the soil structures and soil composition minerals were determined by scanning electron microscopy (SEM) and energy dispersive X-ray spectroscopy (EDX), respectively. The estimation of the bearing capacity of dry Khon Kaen loess from Terzaghi's theory was closer to the field test than Meyerhof's theory. However, the estimation of the bearing capacity of wet Khon Kaen loess from Terzaghi's theory and Meyerhof's theory was much higher than the bearing capacity in the field. The field test showed that the bearing capacity of Khon Kaen loess was not dependent on the foundation size and depth. Nevertheless, this study found that Khon Kaen loess's bearing capacity was affected by its degree of saturation.

Keywords: Shallow foundation, Bearing capacity, Collapsible soil, Khon Kaen loess, Static load test

1. INTRODUCTION

Khon Kaen is a province in Northeastern Thailand. The photography of Khon Kaen is the plateau. Therefore, almost Khon Kane soil is a windblown deposit, which the majority composition is sand and silt with a low percentage of clay and call Khon Kaen Loess.

The structure of Khon Kaen Loess is a honeycomb structure, which a loose structure. Khon Kaen Loess's bearing capacity is high in a dry condition and low in a wet condition. The different bearing capacity between wet and dry conditions causes trouble during construction work.

A building's foundation is one of the essential structures because the building's vertical load is transferred to it. Therefore, the bearing capacity has to carry the applied load from the building.

In the past, the foundation of buildings at Khon Kaen University were often shallow due to the high soil strength during the dry season. Without considering wet conditions' parameters, many Khon Kaen buildings were damaged due to water leaks.

Hence, this study investigated the bearing capacity and settlement of shallow foundations on Khon Kaen loess at various depths and sizes in natural and wet conditions.

Udomchoke [1] details that Khon Kaen loess's shear strength parameters decrease when the moisture content increases and that Khon Kaen loess is a collapsible soil.

Gasaluck and Nuntasarn [2] explain that Khon Kaen loess is moderately collapsible. However, its collapse index decreases through compaction at 95% through a modified method. Khon Kaen loess's strength is very high when the moisture content is low; increased moisture content denotes decreased cohesion and friction angle, damaging the building.

The research by Yodsa-nga, Gasaluck, and Punrattanasin [3] involved field testing and numerical modeling. It studied the pattern of stress distribution in loess due to surcharge loading. Plate bearing tests in the field were assessed with a natural deposit of loess. Using a hardening soil model in a finite element analysis could lead to more realistic stress predictions. The contract pressure pattern should also be considered in the finite element model since several workers observed that uniform contact pressure assumption was unreasonable.

Tungsheng, Shouxin, and Jiaomao [4] found that aeolian soil, defined as loess, was found in approximately 80% of central China. As this soil had a low permeability, it settled quite quickly. The water flowed through the surface and eroded into furrows, causing the soil to collapse.

El-kady and Badrawi [5] offered an alternative foundation scheme that reduced foundation costs by reducing reinforcing steel and reducing tensile stress. Their study investigated the folded isolated

foundation using 5 samples, the isolated foundation, and the folded foundation at 10, 20, 30, and 40 degrees. Then a comparative experiment was conducted using the program (ABAQUS). This study used a physical modeling with a scale of 1:4 and then a comparative experiment with the program. It was concluded that the folded foundation had low subsidence. The isolated foundation had the most influence at an angle of 30 degrees. The maximum tensile strength, the stress in the reinforcement, decreased by approximately 48%, and the subsidence was 57% lower.

Shahein and Hefdhallah [6] performed an analysis of settlement by giving an example of a power plant. Analysis results suggested that neighboring foundations' effect might be significant for transforming the foundation system from a single foundation to a spread foundation.

This system, spread foundation, could reduce settlement about 4 to 5 times. When subsidence was more than acceptable, switching from a single foundation system to a spread foundation to resolve the issue might be necessary.

Dixit and Patil [7] studied groundwater levels' influence on bearing capacity and settlement parameters by the Terzaghi method and the IS Code method. Using a rectangular foundation as a test foundation, they found that as the foundation depth increased. The bearing capacity would increase accordingly, making it safer. The bearing capacity percentage at depths of 1.2 m, 1.5 m, and 1.8 m increased to 8.67%, 16.97%, and 25.11% compared with a depth of 0.9 m. Furthermore, the bearing capacity percentage was reduced due to groundwater levels of 12.18% and 29.43% following the IS Code method, and 8.45% and 29.43% following the Terzaghi method.

Belgaonkar and Thakai [8] studied the settlement of isolated foundations by testing three types of soils: dense sand, hard clay, and gravel. They concluded that the depth of the foundation was inversely proportional to the depth of the settlement. However, rectangular foundations had less depth of settlement compared to square foundations. Moreover, for low SBC soils, a raft foundation was more economical and safer.

Gaone, Gourvenec, and Doherty [9] presented field test data from four instrumented rigid square pad foundations on soft clay brought to failure under concentric vertical loading. The test program comprised two unconsolidated undrained (UU) foundation tests and two consolidated undrained (CU) tests. The two foundations were preloaded to achieve a proportion of the UU capacity, and the soil was allowed to consolidate before being brought to undrained failure. In this paper, the site works and testing procedures were presented along with the load- and time-settlement responses of all four foundations. Horizontal stress and pore pressure data were presented for the two CU tests. The undrained and consolidated undrained load-settlement responses were shown to agree well with theoretical and numerical predictions. The UU tests were the subject of a prediction exercise, summarized in a companion paper presented on this particular issue.

Lee and Eun [10] presented the effects of multiple-footing configurations in the sand on bearing capacity, which was investigated using field plate load tests and finite element analyses. Both strip and spread footings were considered in the finite element analyses. They found that the load responses of multiple footings were similar to those of the single footing at distances greater than three times the footing width. Design equation and correlation parameters, necessary for quantifying the bearing capacity ratio values for the different multiple-footing configuration, were derived. Experimental test results from the literature were selected and used in verifying the proposed method.

ASTM D2487-06 [11] describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of the particle-size characteristics, liquid limit, and plasticity index. It shall be used when precise classification is required.

ASTM D1194-94 [12] covers the estimation of soil bearing capacity in place through field loading tests. This test method can be used in soil investigation for foundation design. It gives information on the soil only to a depth equal to about two diameters of the bearing plate and considers only part of the effect of time.

European Committee for Standardization [13] recommend the settlement of isolate foundation for serviceability was 25 mm.

2. EQUATIONS

Das [14] Terzaghi presents the equation to estimate the ultimate bearing capacity of square and circular foundations.

$$q_u = 0.867c'N_c + qN_q + 0.4\gamma BN_\gamma \quad (1)$$

$$q_u = 0.867c'N_c + qN_q + 0.3\gamma BN_\gamma \quad (2)$$

In Eq. (1), B equals the dimension of each side of the foundation. In Eq. (2), B equals the foundation's diameter.

c' = cohesion
q = effective stress at the level of the bottom of the foundation
γ = unit weight of the soil
B = width of the foundation
N_c, N_q, N_γ = bearing capacity factors

Meyerhof presents that the ultimate bearing capacity. Equations (1) and (2) are only for square

and circular foundations. They do not address the case of a rectangular foundation ($0 < B/L < 1$). Also, the equations do not consider the shearing resistance along the soil's failure surface above the foundation's bottom. Besides, the load on the foundation may be inclined. To account for all these shortcomings, Meyerhof suggests the following form of the general bearing capacity equation.

$$q_u = c'N_cF_{cs}F_{cd}F_{cc} + qN_qF_{qs}F_{qd}F_{qc} + 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma c} \quad (3)$$

In this equation,

c' = cohesion
q = effective stress at the level of the bottom of the foundation
γ = unit weight of the soil
B = width of the foundation
$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors
$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors
$F_{cc}, F_{qc}, F_{\gamma c}$ = compressibility factors
N_c, N_q, N_γ = bearing capacity factors

The equations for determining the various factors given in Eq. (3) are described briefly in the following sections. Note that the original equation for ultimate bearing capacity is derived only for the plane-strain case. The shape, depth, and load inclination factors are empirical factors based on the experimental data.

3. APPARATUS

In this study, the precast concrete sizes of 1.0 x 1.0 x 0.25 m, 1.2 x 1.2 x 0.25 m, and 1.5 x 1.5 x 0.25 m were used to isolate shallow foundations, as shown in Fig. 1. The counterweight was a group of pile size 0.35 x 0.35 x 5.0 m, as shown in Fig. 3. Moreover, the load column, with a capacity of 500 kN, was used to measure the applied load. The settlement of the foundation was examined by using four Kyowa LVDTs. All electronic devices were connected to the UCAM data logger, as shown in Fig. 2, to record all data.

The ground surface settlement was also observed using Leica sprinter digital levels, as shown in Fig. 4.

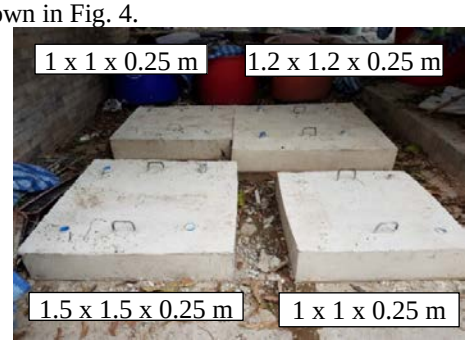

Fig. 1 Shallow foundations

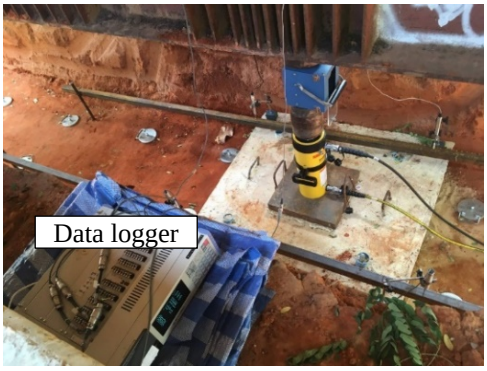

Fig. 2 Electronic devices

4. METHODOLOGY

4.1 Static Load Test

Khon Kaen loess was excavated at a depth of 0.5, 1.0, and 1.5 m with a width of 1.8 m. Then the precast concrete as isolated spread footing was installed in the pitch. There were three foundations sizes of 1.0 x 1.0 x 0.25 m, 1.2 x 1.2 x 0.25 m, and 1.5 x 1.5 x 0.25 m. The first foundation was studied at a depth of 0.5 m, 1.0 m, and 1.5 m in the bolt's natural and wet conditions. The second and third foundations were studied at a depth of 1.0 m in the bolt's natural and wet conditions. Four LVDTs were installed at the four corners of the foundation to measure the settlement due to loading. A hydraulic jack was installed at the center of the foundation to increase pressure during testing. The incremental loading was 20 kPa. The data was read from the data logger at 0, 1, 2, 4, 8, 15, 30, and 60 minutes, and every tested pressure was repeated. Also, footplates were installed at the shallow foundation's side to measure ground surface displacement by Leica sprinter digital levels. The ground surface settlement was determined at every loading. The foundation was soaked for three hours, as shown in Fig. 5, after the pressure reached 200 kPa. The pressure at soaked condition was maintained at 200 kPa. Meanwhile, the foundation's settlement was also measured every 30 minutes.

Fig. 3 Counterweight

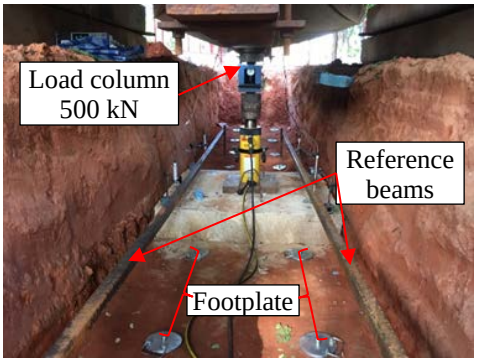

Fig. 4 The installation of the devices

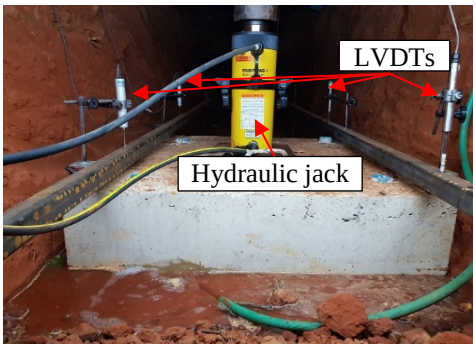

Fig. 5 Soaked

4.2 Triaxial Test

Khon Kaen loess's friction angle and cohesion were observed from the multistage triaxial test under unconsolidated undrained (UU) test at three different conditions: saturation, transition, and residual regimes. The soil conditions were defined from the soil water characteristics curve (SWCC) by the initial saturation degree. Hence, the initial saturation degree of the sample had to admit the range of each regime. The degree of saturation of the saturated sample had to be higher than 85%. Moreover, the initial saturation degree of transition and the residual sample had to admit 39% to 85% and less than 39%, respectively. Cell pressures of 150, 300, and 600 kPa were applied to the sample for the shearing stage. Undisturbed Khon Kaen loess samples were trimmed to a diameter of 5.0 cm and a height of 10.0 cm. All samples had to achieve a saturation process to get a different degree of saturation. The B-value of a saturated sample had to be higher than 95% before shearing.

Moreover, the transition sample preparation was saturated until the degree of saturation was between 85% and 39% to increase to the desired moisture content before shearing started. The residual samples are sheared at the initial moisture content by shearing soil samples at a speed of 1 mm/min, as shown in Fig. 6.

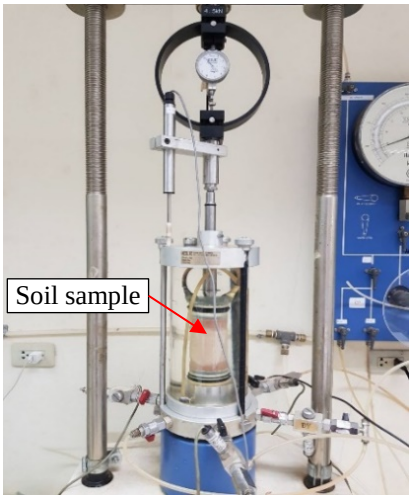

Fig. 6 Triaxial shear test apparatus

5. LABORATORY TEST RESULTS

5.1 Basic Property of Khon Kaen Loess

Table 1 presents the basic properties of Khon Kaen loess. It is primarily sand and has been classified as silty sand (SM) [11]. Figure 7 shows that the grain size distribution curve was determined from sieve and hydrometer analyses. The Atterberg, liquid, and plastic limits showed a plasticity index of 2.6%. The specific gravity was 2.66.

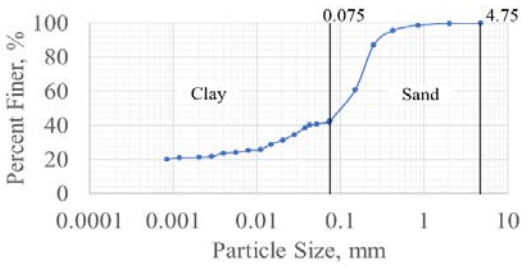

Fig. 7 Grain size distribution curve

Table 1 Basic property of Khon Kaen loess

Property	
USCS	SM
Specific gravity, G_s	2.66
Liquid limit, LL (%)	16
Plastic limit, PL (%)	13.4
Plasticity index, PI	2.6
Maximum dry density, γ_d (t/m^3)	1.63
% Passing No. 4 sieve	100
% Passing No. 200 sieve	42.74
% Sand	57.26
% Silt	21.75
% Clay	20.99

5.2 Scanning Electron Microscopy

Scanning electron microscopy (SEM) was used to determine the structural characteristics of the Khon Kaen loess samples. Figure 8 shows that the soil grain structure has a high porosity with a binder between the soil grains.

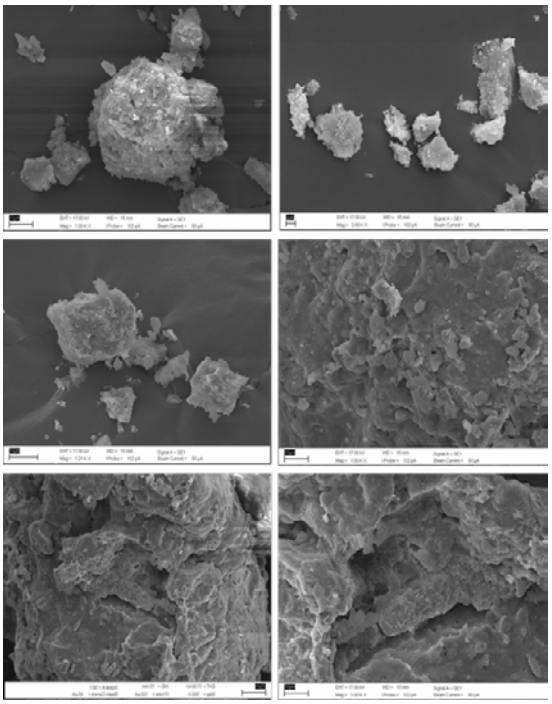

Fig. 8 Khon Kaen loess structure in an undisturbed condition

5.3 Energy-Dispersive X-ray Spectroscopy

Energy-dispersive X-ray spectroscopy (EDX) was used to analyze various minerals in the Khon Kaen loess soil samples. It was found that Khon Kaen loess had iron (Fe) of 2.54% by weight. Table 2 shows that iron is a binder between the soil grain and various minerals.

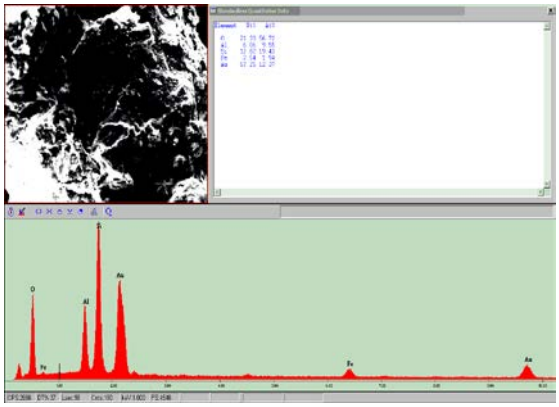

Fig. 9 EDX of Khon Kaen loess

Table 2 EDX of Khon Kaen loess

Element	Weight (%)	Atomic (%)
O	21.33	56.72
Al	6.06	9.55
Si	12.82	19.43
Fe	2.54	1.94
Au	57.25	12.37

5.4 Soil-Water Characteristic Curve

The pressure plate test results for determining the soil-water characteristic curve (SWCC) found that the initial dry density was 1.67 t/m^3, while the matric suction at which air began to enter the soil (Air Entry Value) was 4.4 kPa. The matric suction of the saturation regime was 0.0 4.4 kPa, the matric suction of the transition regime was 4.4 – 20.5 kPa (S_r was between 85% and 39%), and the matric suction of the residual regime was greater than 20.5 kPa (S_r was less than 39%), as shown in Fig. 10.

5.5 Triaxial Test (UU Test)

The Undisturbed Khon Kaen sample was accomplished in the summer season. Therefore, the initial moisture content and dry unit weight were too low as 3% and 16 kN/m^3, respectively. Five samples determined the cohesion (c′) and friction angle (ϕ′) at various degrees of saturation. Sample no. 1 was tested at the saturation regime. Sample no. 2 and 3 were tested at the transition regime, while sample no. 4 was tested at the residual regime. The test results are shown in Table 3, and the Mohr's circle of each sample are illustrated from Figs. 11-14.

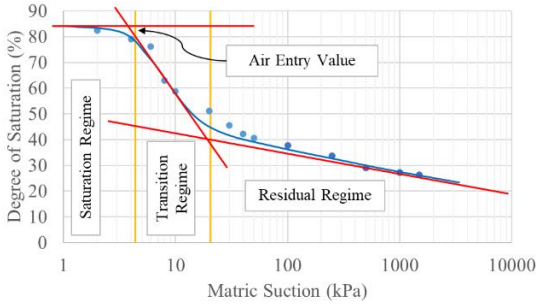

Fig. 10 SWCC of Khon Kaen loess

Table 3 Triaxial test result

Sample No.	Condition	Initial moisture content	S_r (%)	γ_d (g/cm^3)	c (kPa)	ϕ (deg)
1	Transition	3.5	100	1.63	34	0
2		3.2	75.9	1.61	0	15
3		3.1	58.6	1.58	0	21
4	Residual	2.8	8.8	1.54	60	24

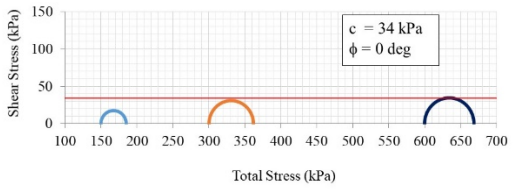

Fig. 11 Mohr's circle of sample no. 1

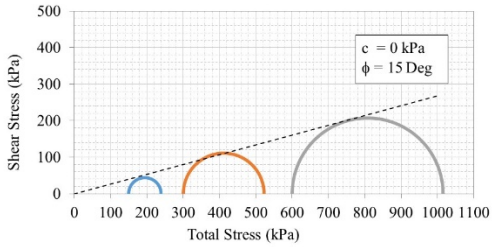

Fig. 12 Mohr's circle of sample no. 2

6. FIELD TEST RESULT

6.1 Static Load Test

The foundation size of 1.0 x 1.0 x 0.25 m was studied at a depth of 0.5 m, 1.0 m, and 1.5 m in both dry and wet conditions. However, the foundation size of 1.2 x 1.2 x 0.25 m and 1.5 x 1.5 x 0.25 m were studied at a depth of 1.0 m in dry and wet conditions. The maximum applied load in this study was 200 kPa. The failure is defined as a 25 mm settlement. The relationships between applied pressures and settlement of foundation on Khon Kaen loess at the residual regime and transition regime are illustrated in Figs. 15-16, respectively. The ultimate bearing capacity (q_{ult}) of foundations is presented in Table 4. Moreover, the relationship between the degree of saturation and ultimate bearing capacity (q_{ult}) is presented in Fig. 24.

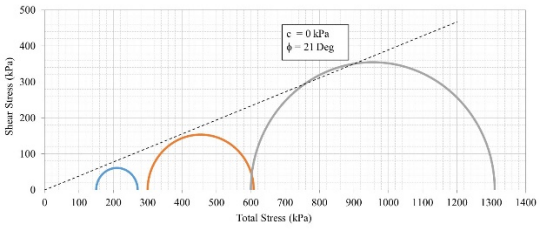

Fig. 13 Mohr's circle of sample no. 3

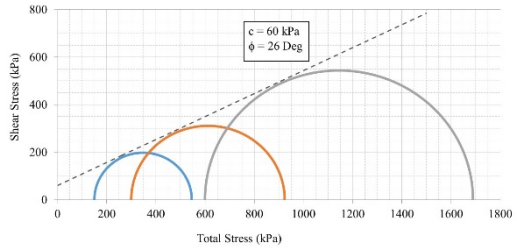

Fig. 14 Mohr's circle of the sample no. 4

Table 4 Static Load Test

Sample	Size (m)	Depth (m)	S_r (%)	q_{ult} (kPa)
Residual	1 x 1	0.5	6.6	2145.4*
	1 x 1	1.0	6.9	713.2*
	1 x 1	1.5	20.4	513.1*
	1.2 x 1.2	1.0	26.5	99.1
	1.5 x 1.5	1.0	23.0	158.9
Transition	1 x 1	0.5	53.5	23.5
	1 x 1	1.0	39.0	83.4
	1 x 1	1.5	42.7	48.1
	1.2 x 1.2	1.0	50.5	84.4
	1.5 x 1.5	1.0	49.7	63.8

* Predicted from the graph load and settlement

6.2 Ground Surface Displacement

The graph presents the bearing capacity failure mode almost as a punching shear failure, which agrees with the ground surface displacement, as shown in Figs. 17–23.

Leica Digital Levels results indicate that the Khon Kaen loess ground surface graph shows both local shear failure and punching shear failure characteristics. Nevertheless, it can be concluded that Khon Kaen loess has a punching shear failure in line with the load-settlement curve.

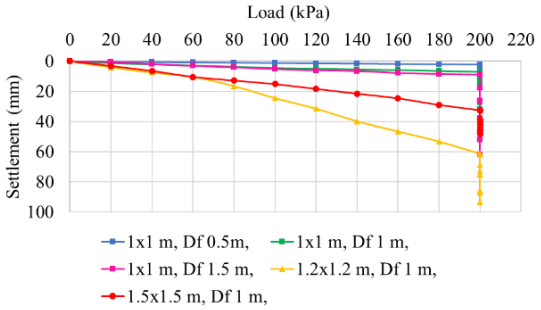

Fig. 15 Load-settlement curve of foundation on residual regime

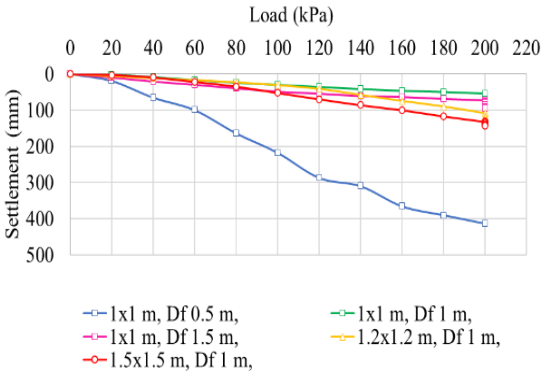

Fig. 16 Load-settlement curve of foundation on transition regime

7. DISCUSSION

7.1 Field Test

The plotted data between ultimate bearing capacity and degree of saturation shown in Fig. 24 present that the bearing capacity of foundation on dry soil or soil at residual regime is higher than the wet soil or soil at transition regime. Moreover, the foundation's size and depth did not affect collapsible soil bearing capacity in a transition regime, as shown in Figs. 25-26, respectively. However, the foundation's size affects the soil in the residual regime. The bearing capacity of the foundation reduces with the depth of the foundation.

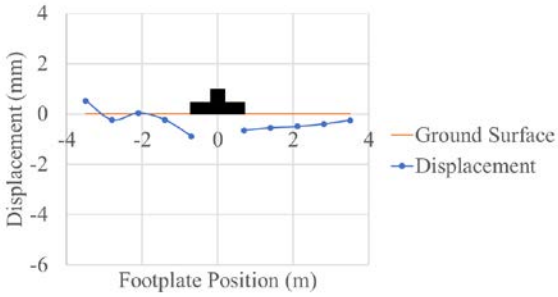

Fig. 17 Surface settlement at residual condition (foundation: B = 1.0 m and D_f = 0.5 m)

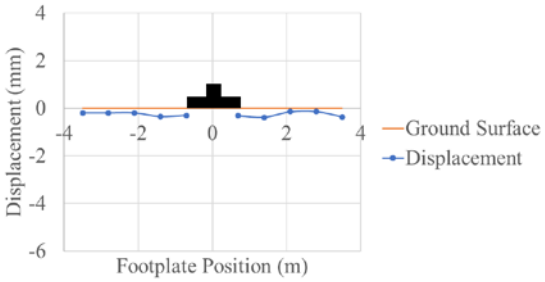

Fig. 18 Surface settlement at residual condition (foundation: B = 1.0 m and D_f = 1.0 m)

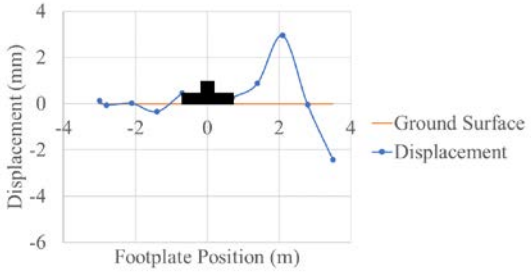

Fig. 19 Surface settlement at residual condition (foundation: B = 1.0 m and D_f = 1.5 m)

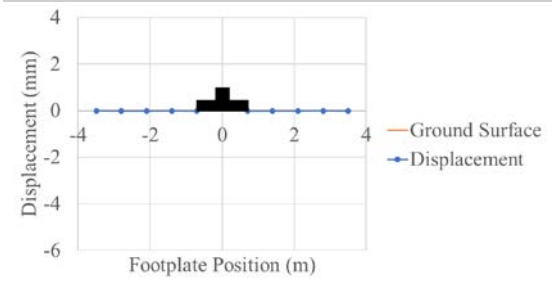

Fig. 20 Surface settlement at residual condition (foundation: B = 1.2 m and D_f = 1.0 m)

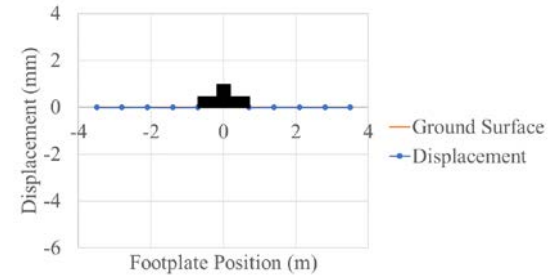

Fig. 21 Surface settlement at residual condition (foundation: B = 1.5 m and D_f = 1.0 m)

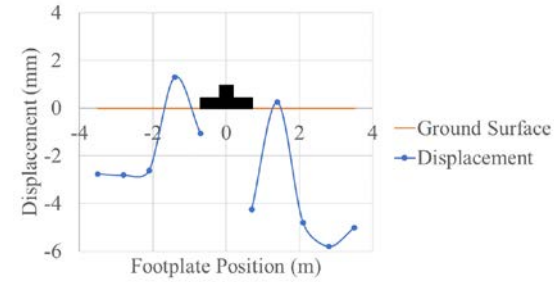

Fig. 22 Surface settlement at transition condition (foundation: B = 1.0 m and D_f = 1.0 m)

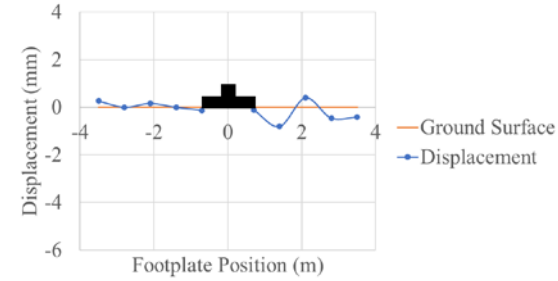

Fig. 23 Surface settlement at transition condition (foundation: B = 1.0 m and D_f = 1.5 m)

7.2 Triaxial Test

The relationship between cohesion (c) and friction angle (ϕ) and degree of saturation (S_r) is shown in Figs. 27-28, respectively.

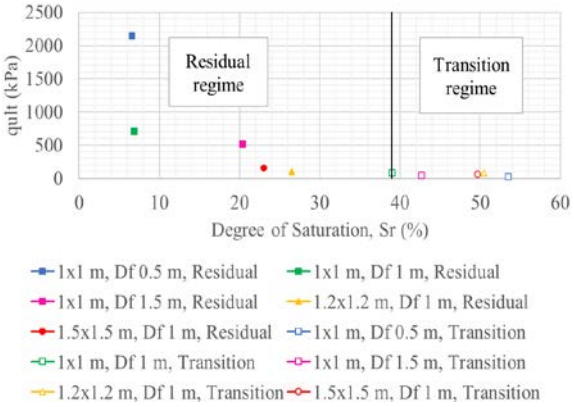

Fig. 24 Field bearing capacity – degree of saturation of foundations

Figure 28 presents a good relationship between ϕ and S_r in terms of the exponent. The friction angle increases with increasing matric suction. However, the cohesion (c) was a relatively constant value of each regime. These were used to estimate the bearing capacity by the equation of Terzaghi and Meyerhof in the next section.

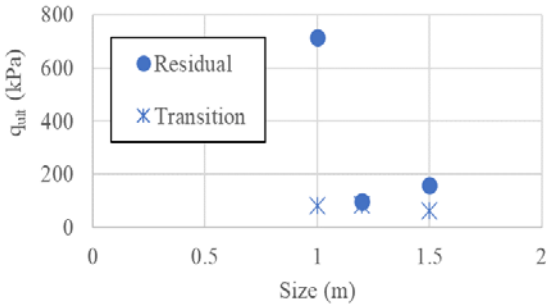

Fig. 25 Field bearing capacity — the size of foundations

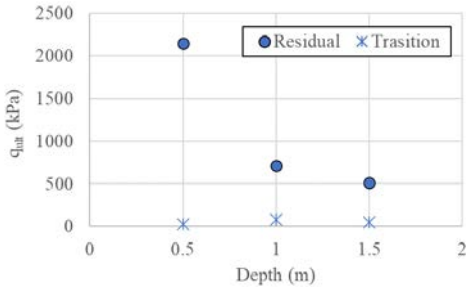

Fig. 26 Field bearing capacity — the depth of foundations

7.2 Bearing Capacity Equation

The predicted c and ϕ values from Figs. 25-26 used to calculate the ultimate bearing capacity by equations of Terzaghi (Eq. (1)) and Meyerhof (Eq. (3)) are shown in Table 5.

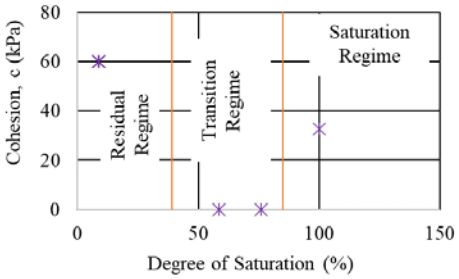

Fig. 27 The prediction of the cohesion by a degree of saturation.

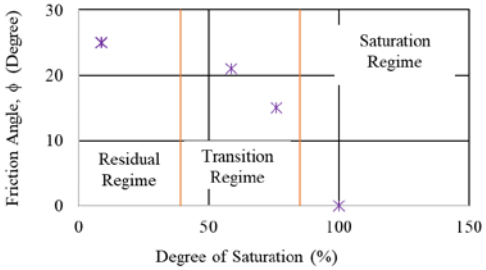

Fig. 28 The prediction of the friction angle by a degree of saturation.

Table 5 Comparison of the theoretical bearing capacity estimates with the field test

Foundation size (m^2)	Depth (m)	Condition	S_r (%)	c′ (kPa)	ϕ′ (Degree)	Field q_{ult} (kPa)	Terzaghi q_{ult} (kPa)	Meyerhof q_{ult} (kPa)	Terzaghi : field ratio	Meyerhof : field ratio
1 x 1	0.5	Residual	6.6	60	26.23	2145.4*	877.5	1897.5	0.41	0.88
1 x 1	1.0	Residual	6.9	60	26.19	713.2*	925.9	2016.4	1.30	2.83
1 x 1	1.5	Residual	20.4	60	24.61	513.1*	923.9	1864.0	1.80	3.63
1.2 x 1.2	1.0	Residual	26.5	60	23.90	99.1	839.3	1660.7	8.47	16.76
1.5 x 1.5	1.0	Residual	23.0	60	24.31	158.9	844.5	1724.6	5.31	10.85
1 x 1	0.5	Transition	53.5	0	20.74	23.5	46.0	76.0	1.96	3.23
1 x 1	1.0	Transition	39.0	0	22.47	83.4	92.6	175.2	1.11	2.16
1 x 1	1.5	Transition	42.7	0	22.01	48.1	121.5	242.7	2.53	5.16
1.2 x 1.2	1.0	Transition	50.5	0	21.09	84.4	81.9	150.6	0.97	1.78
1.5 x 1.5	1.0	Transition	49.7	0	21.19	63.8	85.6	155.6	1.34	2.44

*The ultimate bearing capacity was estimate from the graph as shown in Figure 15

Table 5 presents that the bearing capacity from the static load test is lower than Terzaghi's equation and Meyerhof's equation, as illustrated in Eq. (1) and (3), respectively. Moreover, the bearing capacity calculated with Terzaghi's equation, as shown in Eq. (1) was lower than the bearing capacity calculated with Meyerhof's equation, as shown in Eq. (3).

8. CONCLUSIONS

The limitation of this study is that the applied load under residual conditions did not reach a failure. However, this paper can be concluded that:

- The mode of failure of isolated foundation on Khon Kaen loess under residual and transition condition was punching shear failure.
- The foundations' depth and size were not entirely affected by Khon Kaen loess's bearing capacity.
- The primary effect on the bearing capacity of Khon Kaen loess is the degree of saturation or moisture content of the soil.
- The bearing capacity from the static load test is lower than the theory.

9. ACKNOWLEDGMENTS

Acknowledgment is given to the Sustainable Infrastructure Research and Development Center (SIRDC), Khon Kaen University, for the support of this research.

10. REFERENCES

[1] Udomchoke V., Origin and Engineering Characteristics of the Problem Soil a in the Khorat Basin, Northeastern Thailand, Ph.D. dissertation, Asian Institute of Technology, Bangkok, Thailand, 1991.

[2] Gasaluck W. and Nuntasarn R., Behavior of Khon Kaen Loess due to High Moisture Content, Proceeding of the Eighth National Convention on Civil Engineering, The Engineering Institute of Thailand under H.M. The King's Patronage and Department of Civil Engineering, Khon Kaen University, Khon Kaen, pp. GTE 458–463, 2002.

[3] Yodsa-nga P., Gasaluck W. and Punrattanasin P., Stress Distribution in Khon Kaen Loess under Spread Footing. Electronic Journal of Geotechnical Engineering, 17, 3753–3769, 2012.

[4] Tungsheng L., Shouxin Z. and Jiaomao H., Stratigraphy and Paleoenvironmental Changes in the Loess of Central China, Institute of Geology, Academia Sinica, Beijing, China, 5, 489–495, 1986.

[5] El-kady M.S. and Badrawi E.F., Performance of isolated and folded footings. Journal of Computational Design and Engineering, 4, 150–157, 2017.

[6] Shahein M. and Hefdhallah A., Effect of Neighboring Footings on Single Footing Settlement, International Conference on Case Histories in Geotechnical Engineering, 19(1.27a), 2013.

[7] Dixit M.S. and Patil K.A., Effect of Depth of Footing and Water Table on Bearing Capacity of Soil, Indian Geotechnical Conference, GEOtrendz, 16–18 December 2010, Mumbai, IGS Mumbai Chapter & IIT Bombay, 2010.

[8] Belgaonkar P.P. and Thakai R.J., Parametric Study on Settlement of Isolated Footings, Civil and Environmental Research, 8(9), 2016.

[9] Gaone F.M., Gourvenec S. and Doherty J.P., Large-scale shallow foundation load tests on soft clay — At the National Field Testing Facility (NFTF), Ballina, NSW, Australia, 253–268, 2018.

[10] Lee J. and Eun J., Estimation of bearing capacity for multiple footings in sand, 36, 1000–1008, 2009.

[11] American Society for Testing and Materials. (D2487-06). Standard practice for classification of soils for engineering purposes (Unified Soil Classification System).

[12] American Society for Testing and Materials. (ASTM D1194-94). Standard Practice for Bearing Capacity of Soil for Static Load and Spread Footings. West Conshohocken, Pennsylvania, 2000.

[13] European Committee for Standardization. Basis of Design and Actions on Structures, Eurocode 1, Brussels, Belgium, 1994.

[14] Das B.M., Principles of Foundation Engineering, 8th ed., USA, PWS, 2014.