

BEARING CAPACITY OF LATERITIC SOILS IN CHANTHABURI

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Abstract

The objective of this research was to study the bearing capacity of lateritic soils in Chanthaburi for foundation design. The research methodologies were including comparing the plate bearing capacity with the theoretical Terzaghi's bearing capacity equation taking soil friction angle and cohesions from the correlation graph using standard penetration test values (SPT-N values) and from the laboratory test; the direct shear test; finding the correlation graph with the SPT-N value to determine the empirical equation. The plate bearing tests were conducted at the eleven sites around Chanthaburi. The plate bearing capacity values were then compared to various theoretical approaches. The results showed that the bearing capacity from the plate bearing test yielded the maximum values following by the bearing capacities calculated from Terzaghi's equation with the soil parameters (c and ϕ) obtained from direct shear tests and the one calculated from SPT test, respectively. Finally, correlation equation between average ultimate bearing capacity (q_u) and SPT-N value can be expressed as $q_u = 8.88N \text{ t/m}^2$ for $N \leq 20$.

Keywords: Foundation design, lateritic soils, Chanthaburi, bearing capacity

Introduction

The population growth in Chanthaburi is rising continuously resulting in a high demand for housing. Besides, tourist attractions and various infrastructures have developed significantly due to the government policies in the eastern economic corridor project and border area economic development zones (Office of the

National Economic and Social Development Board, 2016, 2017). It is necessary to construct a lot of large buildings on lateritic soils due to Chanthaburi's geology. For foundation design, it is required SPT-N values as a factor for calculating the bearing capacity of the soils. However, the calculated bearing capacities are

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usually less than ones on field because of the underlying of rock or hard soil on site. Thus, the foundation designs based on the calculation are always over design and costly. Furthermore, obstacles of the hard soils or variations of subsoil strata could obstruct pile driving at the desired depth (Geminai and Associate Co., Ltd., 2013; Srisaket Taweepol Construction Co., Ltd., 2016). These problems may cause wasting of a large number of resources when doing the construction projects in lateritic soil areas. The lateritic soil has physical properties like granular soil such as sand and gravel, and plastic properties like cohesive soil such as clay and silt. Moreover, the studies on lateritic soil in Thailand mostly focused on its engineering properties for highway construction (Thienphat, 2002; Rakpra, 2004; Ngenprom *et al.*, 2007; Horpibulsuk *et al.*, 2013). Therefore, it was important to study the properties of lateritic soils for foundation design in Chanthaburi province to prevent and mitigate the problems, economically utilizing resources and saving construction costs.

Subsoil Conditions

Generally, the topography of Chanthaburi could be divided into three zones including coastal plain and hills in the south, river basin and hills in the central, and hills and mountains in the north and east (Department of Mineral Resources, 2011; Boonyong and Ngamlai, 2018).

In this study, testing locations were in the hill zones. For example, soil profiles of the industrial technology faculty building in Rambhai Barni Rajabhat University situated on hill terrain are illustrated in Figure 1. The Figureure shows that apart from the first few meters, rock strata changes significantly which agrees well with the report by the subcommittee of geotechnical engineering (2002). The report stated that subsoil condition was relatively non-uniform in Chanthaburi. However, boring logs from the twelve various locations in the hill zones throughout Chanthaburi conducted by the Department of

Public Work and Town & Country Planning showed that variation of depths of surface to firm strata were from 2 m to 10 m (Thepa and Ruang-leu, 2009). It can be seen that the subsoil strata was consisted of four strata with different thicknesses including a weathered crust layer, the low plasticity clay layer, the clayey sand or gravel layer and the firm strata, respectively. The liquid limits of the soils were less than 50%. Their unit weights were between 1.5 and 2.2 t/m³.

Materials and Methods

Eleven locations throughout Chanthaburi province except Leam-sing district whose terrain is flat plain, were selected for the in-situ testing and sampling as shown in Figure 2. The names of locations and coordinates are tabulated

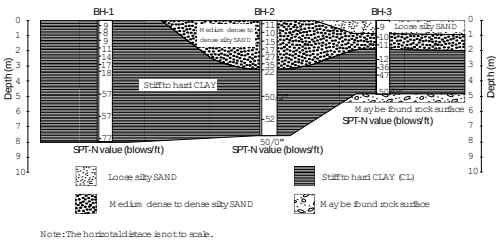


Figure 1. Soil profiles of the industrial technology faculty building

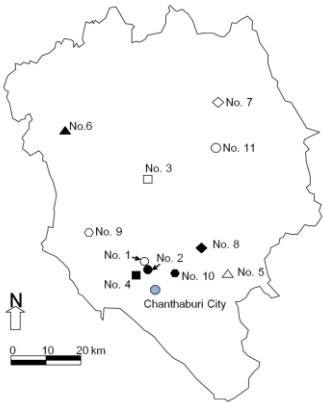


Figure 2. Sampling locations in Chanthaburi province map

in Table 1. The investigation program included in situ testing, sampling and laboratory testing on these samples. The plate bearing and SPT tests were conducted on site. In order to obtain the engineering properties of soils, sieve analysis, Atterberg's limits, modified compaction test, and direct shear test were performed in the laboratory. The testing standards are in Table 2. The brief details of each test are as follows.

The plate bearing test was performed at the selected locations and at 0.5 m deep by placing a 30 cm circular steel plate on the testing soil. Then, pressed the plate by hydraulic jack with dead load reaction normally using backhoe or truck, and recording the settlements corresponding to each load increment. The test load was gradually increased till the plate started to settle at a rapid rate. The total value of the load on the plate was divided by the area of the steel plate which gave the value of the ultimate bearing capacity of the soil.

The SPT was performed at the selected locations at 0.5 m deep below the ground surface. The test was conducted by the standard split spoon sampler. By means of a drop hammer of 63.5 kg mass falling from

a height of 75 cm at the rate of 30 blows per minute, the sampler was driven into the soil. The number of blows of hammer required to drive a depth of 15 cm was counted. Then it was driven by another 15 cm and the blows were counted. Similarly, the sampler was once again driven by another 15 cm and the number of blows recorded. The number of blows recorded for the first 15 cm were not taken into consideration. The number of blows recorded for the last two 15 cm intervals were added to give the standard penetration number (N).

Sieve Analysis was performed by arranging a nest of sieves including sieves No.4, 10, 40, 100, 200, and pan, placing the stack of sieves in the mechanical sieves shaker and sieve for 5 to 10 min, then determined percent finer by weight for each sieve and plotted in the graph.

The Atterberg's limits including liquid limit and plastic limit were conducted by using soil samples passing sieve No. 40. The liquid limit was the water content at which the soil behaved practically like liquid, but had small shear strength. It flowed to close the groove in just 25 blows in Casagrande's cup. The plastic limit of soil was the water content at which soil

Table 1. Locations of testing samples

Sample Nos.	Locations	N	E
1	T.Tha-Chang, A.Muang	12.666260	102.096656
2	T.Tha-Chang, A.Muang	12.660466	102.101329
3	T.Takien-tong, A.Kitchakoot	12.876838	102.094383
4	T.Ploy-Wien, A.Thamai	12.627548	102.063525
5	T.Tokphom, A.Klung	12.622848	102.312266
6	T.Kaeng-Hangmaew, A.Kaeng-Hangmaew	12.998303	101.870222
7	T.Saikao, A.Soidao	13.077991	102.282354
8	T.Ang-Kiri, A.Ma-Kam	12.640074	102.240756
9	T.Wang-mai, A.Na-Yai-Arm	12.730970	101.938473
10	T.Pubpla, A.Muang	12.618369	102.168308
11	T.Tub-Sai, A.Pong-Nam-Ron	12.960187	102.280033

Table 2. Testing standards

Testing	Testing Standard
In situ Test	
Standard Penetration Test	ASTM D-1586
Plate Bearing Capacity Test	ASTM D-1194
Laboratory Test	
Sieve Analysis	ASTM D-422
Atterberg's Limits	ASTM D-4318
Compaction Test (Modified Proctor Test)	ASTM D-1557
Direct Shear Test	ASTM D-3080

began to behave as a plastic material. At this water content, the soil crumbled when rolled into threads of 3.2 mm in diameter.

The direct shear test was done by preparing at least three samples of soil passing sieve No. 4 and compacting the soil samples in the mold with various densities and water contents as ones on the selected sites conditions instead of using undisturbed samples due to the difficulty in collecting brittle and hard samples at the weathered crust layer to obtain shear strength parameters consisting of soil friction angle and cohesion.

The compaction test (modified Proctor test) was performed by taking air-dried soil passing sieve No. 4 and mixing it with various quantities of water to determine the optimum moisture content and the maximum dry density at the specific energy.

Results and Discussion

Results of the plate bearing capacity and SPT test are illustrated in Table 3. It is showed that the plate bearing capacities were between 71 t/m² and 150 t/m². It should be noted that for three samples including sample Nos. 5 to 7, the testing values could not reach the ultimate bearing capacity because of the insufficient counterweight. The SPT-N ranged from 7 to 19 blows/ft with an average value of 12 blows/ft except for the sample No. 7 with the SPT-N of 35 blows/ft. The soil densities of the samples were 1.44 to 1.78 t/m³ with an average value of 1.64 t/m³. Finally, the water content of the

samples were 8.2% to 20.2% with an average value of 13.0%. Mostly the liquid limit of lateritic soils ranged from 20% to 50% except for the sample No. 8 that yielded the highest liquid limit of 64.8%. The average of liquid limit was 40.6%. Additionally, their plastic limits were between 6% and 29%. Their Atterberg's limits are presented in the plasticity chart as shown in Figure 3. The sieve analysis results are illustrated in Figure 4. It was found that the six samples (sample Nos. 2 to 7) were classified as sand and the remaining samples were classified as clay or silt. For the modified Proctor compaction test, it can be seen that the maximum dry densities were between 1.78 and 2.07 t/m³ with the optimum moisture contents ranging from 10% to 17% as shown in Figure 5. The direct shear tests done on eleven samples showed that the friction angles were 22 to 38 degrees with cohesion of 1.0 to 3.0 t/m² as tabulated in Table 4. Consequently, the soil samples were classified by Unified soil classification and ASSHTO systems as shown in Table 5. As per the unified system, the samples were classified as clayey sand for sample Nos. 2 to 4, silty sand for sample Nos. 5 to 7, low plasticity clay for samples Nos. 1, 9, and 11, low plasticity silt for sample No. 10 and high plasticity silt for sample No. 8. As per the ASSHTO system, they were classified as A-4 for sample No. 4, A-6 for sample Nos. 1, 3, and 11, A-7-5 for sample No. 10 and A-7-6 for sample Nos. 2 and 5 to 9. Moreover, the previous studies showed that SPT-N value significantly correlated with friction angle for granular soils

Table 3. Testing results

Sample No.	SPT-N (blows/ft)	Unit Weight (t/m ³)	Water Content (%)	Plate Bearing Test, q_u (t/m ²)	Friction Angle, ϕ (degree)	Cohesion, c (t/m ²)
1	15	1.48	12.1	85.3	25	3.0
2	7	1.66	13.9	85.3	31	1.0
3	13	1.68	12.0	142.2	32	1.0
4	8	1.78	8.2	71.1	30	2.0
5	19	1.73	14.9	>149.3*	30	2.4
6	13	1.62	11.7	>71.1*	22	2.0
7	35	1.78	10.2	>97.4*	38	1.0
8	12	1.54	15.1	127.9	30	2.0
9	9	1.60	14.8	142.9	33	2.0
10	11	1.44	20.2	71.1	29	1.5
11	18	1.78	10.2	142.2	35	1.5

Note: * denotes counterweight was not sufficient.

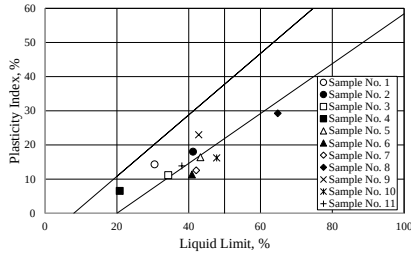


Figure 3. Plasticity chart

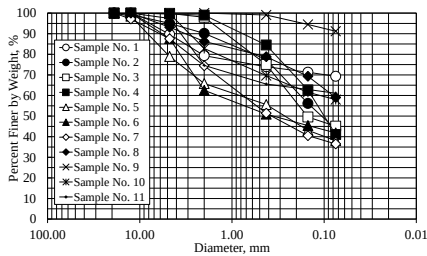


Figure 4. Grain size distribution curves

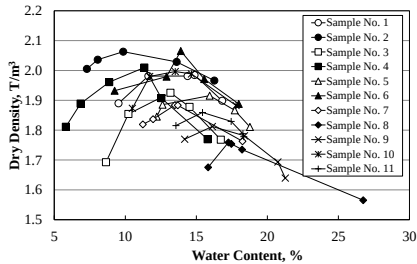


Figure 5. Modified Proctor compaction curves

(Peck *et al.*, 1974; Shioi and Fukui, 1982) and cohesion for cohesive soils (Terzaghi *et al.*, 1967; Sowers, 1979; Pitupakorn, 1982). In this study, the equation of Shioi and Fukui (1982) was selected to estimate friction angles of granular soils from the SPT-N values as shown in Equation (1). The equation of Pitupakorn (1982) was used to estimate cohesions of low plasticity cohesive soils from SPT-N values as Equation (2).

$$\phi^\circ = 0.3N + 27 \quad (1)$$

$$c = 0.52N \quad (2)$$

Table 4. Soil classifications

Sample No.	USCS	ASSHTO
1	CL	A-6(5)
2	SC	A-7-6(2)
3	SC	A-6(2)
4	SC	A-4(0)
5	SM	A-7-6(1)
6	SM	A-7-6(1)
7	SM	A-7-6(0)
8	MH	A-7-6(6)
9	CL	A-7-6(13)
10	ML	A-7-5(6)
11	CL	A-6(5)

where ϕ° = friction angle (degree);
 N = SPT -N value (blow/ft);
 c = Cohesion of soil (t/m²).

Subsequently, the soil bearing capacity can be calculated with the equation that presented firstly by Terzaghi (1943) and developed by Skempton (1951), Meyerhof (1951) and Vesic (1973). In this study, Terzaghi's bearing capacity equation (1943) as detailed in Equation (3) was selected to calculate the bearing capacity due to its simplicity and common use. In this study, the Equation (3) for circular foundation is applied to calculate ultimate bearing capacity (q_{ult}) of the plate bearing test.

$$q_{ult} = 1.3cN_c + \gamma DN_q + 0.3\gamma BN_\gamma \quad (3)$$

where γ = Soil unit weight (t/m³);
 D = Depth of foundation (m);
 B = Width of foundation (m);

N_c, N_q, N_γ = Bearing capacity factors that are non-dimensional and are only functions of the friction angle, ϕ . These factors are shown in Table 6.

Since $\phi = 0$ for fine grain soil, replace $N_c = 5.7, N_q = 1.0, N_\gamma = 0.0$ in Equation (3);

$$q_{ult} = 1.3c(5.7) + \gamma D(1)$$

$$q_{ult} = 7.41c + \gamma D$$

Because, the tests were done at shallow depth, the term of γD became small and could be neglected. Replace $c = 0.52N$ from Equation (2) into the equation. Then,

Table 5. Terzaghi's bearing capacity factors (Terzaghi, 1943)

ϕ (degree)	N_c	N_q	N_γ	ϕ (degree)	N_c	N_q	N_γ
0	5.71	1	0	30	37.16	22.46	19.13
20	17.69	7.44	3.64	31	40.41	25.28	22.65
21	18.92	8.26	4.31	32	44.04	28.52	26.87
22	20.27	9.19	5.09	33	48.09	32.23	31.94
23	21.75	10.23	6.00	34	52.64	36.50	38.04
24	23.36	11.40	7.08	35	57.75	41.44	45.41
25	25.13	12.72	8.34	36	63.53	47.16	54.36
26	27.09	14.21	9.84	37	70.07	53.80	65.27
27	29.24	15.90	11.60	38	77.50	61.55	78.61
28	31.61	17.81	13.70	39	85.97	70.61	95.03
29	34.24	19.98	16.18	40	95.66	81.27	115.31

Table 6. Comparisons of bearing capacity from various methods

Sample No.	Ultimate bearing capacity, q_u (t/m ²) from		
	Plate bearing test	SPT-N value	Direct shear test
1	85.3	57.8	108.5
2	85.3	20.0	69.8
3	142.2	25.3	85.3
4	71.1	21.4	66.6
5	>149.3*	32.9	138.3
6	>71.1*	24.4	60.9
7	>97.4*	65.5	168.1
8	127.9	46.2	116.6
9	142.9	34.7	155.4
10	71.1	42.4	83.3
11	142.2	69.3	156.8

Note: * denotes counterweight was not sufficient.

$$q_{ult} = 7.41(0.52N) = 3.85N \text{ t/m}^2 \quad (4)$$

Since $\phi \neq 0$ and $c = 0$ for granular soil from Equation (3), then

$$q_{ult} = \gamma DN_q + 0.3\gamma BN_\gamma = \gamma[DN_q + 0.3BN_\gamma]$$

Replace $D = 0.5 \text{ m}$ and $B = 0.3 \text{ m}$ as testing condition, then

$$q_{ult} = \gamma[0.5N_q + 0.09BN_\gamma] \quad (5)$$

The ultimate bearing capacity by SPT method can be calculated as following;

1. For granular soil, calculate f of the soil from Equation (1), then find N_q , N_γ by using the ϕ in Table 6 and find the bearing capacity with Equation (5);

2. For fine grain soil, replace N in Equation (4) to find the bearing capacity directly.

The ultimate bearing capacity by the c and f from direct shear test method can be calculated as following;

1. From the direct shear test, the c and ϕ of the testing soil are obtained;

2. then, find N_c , N_q , N_γ by using the ϕ in Table 6 and determine the bearing capacity with Equation (3).

Consequently, the bearing capacities calculated from the above two methods were compared with the results from plate bearing test at the sites as shown in Figure 6.

Correlation Graph Between the SPT-N Value and the Bearing Capacity

The correlation graph between SPT-N value and the in-situ ultimate bearing capacity from the plate bearing test at eight locations was expressed in Figure 5 with the white circle symbol. It should be noted that three locations

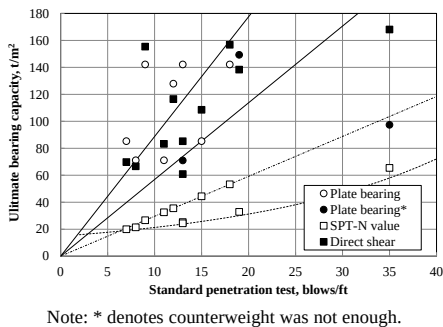


Figure 6. Relationships between SPT and ultimate bearing capacity from various methods

of the plate bearing test represented by the black circle symbol could not reach the ultimate capacity because of the insufficient counter weight. From the graph, the white circles can be divided into two groups: the points with medium bearing capacity having q_u between 70 and 85 t/m^2 with N values of 7 to 15 blows/ft for sample Nos. 1, 2, and 4, and the points with the high bearing capacity having q_u between 120 and 140 t/m^2 with N -values of 9-18 blows/ft at the remaining areas. It was observed that the samples Nos. 4 and 9 had N values of 8 and 9, respectively. However, they yielded quite different values of 71.1 and 142.9 t/m^2 , respectively for the field bearing capacities. This may be due to the soil layer of the sample No. 9 was underlying by rock stratum or hard soil layer. The Linear line in the middle of the white circle symbol group indicates a correlation graph between the average bearing capacity value and SPT based on the linear regression equation represented by $q_u = 8.88N$ t/m^2 for $N \leq 20$. The linear line below the group indicates the lower limit of correlation graph derived from the linear equation of the lowest slope point expressed by $q_u = 5.69N$ t/m^2 .

The correlation graph between N value and bearing capacity calculated from Equations (4) and (5) with SPT- N values for cohesive and cohesionless soils are illustrated by a white square symbol. The linear dotted line expresses

relation in equation (4) and the curve dotted line represents relation in Equation (5).

The correlation graph between N value and bearing capacity derived from Equation (3) with ϕ and c in the direct shear test is illustrated by a black square symbol. It was noticed that the calculated bearing capacity values from the direct shear test would be scattered over the two dotted lines calculated from the one derived from the SPT. It was because in the SPT method, lateritic soils were classified as either cohesive or cohesionless soil. Therefore, either cohesive value or friction angle was replaced in bearing capacity equation instead of both parameters as a result the calculated bearing capacity would be less than the calculated one obtained by direct shear test when both f and c of the soil samples would be determined from laboratory testing. Additionally, the sample preparation played an important role in having high or low friction angle and cohesion values. When testing, if the failure path passed through granular particles in the samples, it would give higher friction angle than one when the failure path passed through fine grain particles. Therefore, some testing results showed some inconsistent values.

In summaries, the correlating graph showed that plate bearing test yields the maximum bearing capacity, followed by the bearing capacity calculated from equation using the soil parameters (ϕ and c) from the direct shear test and the one from SPT- N values, respectively. The bearing capacity of the plate bearing test would yield the highest value because soil particles in the field were cemented by iron oxide (Krinitsky, 1976) and sometimes the testing soil layers were underlying of rock or hard soil on the sites.

Conclusions

Based on the field and laboratory testing results in this study, it can be concluded as follows.

1. Lateritic soils selected from hill slope in Chanthaburi were classified by Unified Soil

Classification System as Group CL, ML, MH, SC and SM and by ASSHTO as Group A-4, A-6, A-7-5 and A-7-6.

2. Unit weight of natural lateritic soils ranged from 1.44 to 1.78 t/m³ with an average of 1.64 t/m³. The water contents were 8.2% to 20.2% with an average of 13.0%. The liquid limits were 20.8% to 64.8% with an average of 40.6%. The plastic limits were 14.3% to 35.5% with an average of 24.9%. The plastic indexes varied from 6.5% to 29.3% with an average plastic index of 15.7%. For modified Proctor's compaction test, the maximum dry densities were between 1.76 t/m³ and 2.07 t/m³ and the optimum moisture contents were 9.9% to 17.3%.

3. From the direct shear test, the friction angles were 22° to 38° and cohesions ranged from 1.0 to 3.0 t/m³. All samples had the fine content more than 20%, which could not be passed the specifications for using as base or subbase in roadwork.

4. For in-situ test, the most of SPT-N values were 7 to 19 blows/ft with an average N-value of 12 blows/ft and bearing capacities were 70 to 150 t/m².

5. In comparison of the bearing capacity with various methods, the one obtained from in-situ test yielded the highest values following by the one calculated from parameters (c and f) obtained from the direct shear test and from the SPT test, respectively.

6. For $N \leq 20$, correlation equations between bearing capacity and SPT-N value were $q_u = 8.88N \text{ t/m}^2$ for the average value and $q_u = 5.69N \text{ t/m}^2$ for the lower limit value, which could be used for preliminary foundation design.

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