International Journal of GEOMATE, Dec., 2019 Vol.17, Issue 64, pp. 108 - 114
ISSN: 2186-2982 (P), 2186-2990 (O), Japan, DOI: https://doi.org/10.21660/2019.64.61042
Geotechnique, Construction Materials and Environment

VARIATION OF BEARING CAPACITY PREDICTION FOR SHALLOW FOUNDATIONS BY SPT AND LABORATORY TESTS

*Omer Muhie Eldeen Taha

Department of Civil Engineering, Kirkuk University, Kirkuk, Iraq

*Corresponding Author, Received: 10 July 2019, Revised: 08 Aug. 2019, Accepted: 28 Aug. 2019

ABSTRACT: This study presents the results of the bearing capacity variation for the shallow foundation from field tests (SPT test) and laboratory tests. Site investigation and geotechnical evaluation for soils Nasiriyah, Iraq. The main purpose of this study is to determine the surface and subsurface conditions with the physical, mechanical and chemical properties of the encountered materials in order to provide recommendations for design and construction of the proposed project foundations. Two types of samples were taken (disturbed and undisturbed samples) for laboratory testing. Laboratory tests were performed on the recovered samples in order to identify the physical, mechanical and chemical properties of the encountered materials. Classification and index tests: moisture content, Atterberg limits, particle size distribution, and specific gravity are carried out to classify the soils. Strength tests: Triaxial (Unconsolidated Undrained UU) and unconfined compression strength (UCS) test to evaluate the cohesion of the soil, c. In addition, chemical tests: pH, sulfate, chloride, TDS, EC and organic matter are carried out. The results showed that the measured bearing capacity depending on the UCS test and Triaxial UU tests data is more reliable than the measured bearing capacity depending on the SPT test date. The estimated value for cohesion, c by eq.2 is very high and does not represent the actual value especially in clayey soils. The increase in fine material (clay and silt) content decreases the N value. While the increase in the course material (sand and gravel) content increases the N value. Moreover, the increase in the soil sample water content decreases the N value.

Keywords: SPT, Soil Bearing Capacity, Triaxial UU, Unconfined Compressive Strength

1. INTRODUCTION

Estimating of soil bearing capacity from the field tests are easier and more economic. In particular, the Standard Penetration Test (SPT test) could provide enough information about soil strength in some cases [1]. In the designing of foundations and earth structural buildings, the Reliability of the Bearing-Capacity data will play an important role [2]. Therefore, most of the designers are preferred to calculate the soil bearing capacity by many methods and preferred laboratory-measured soil parameters. However, the field tests like SPT test are more common in most of site investigation works. The data taken from the SPT tests are varied widely depending on the soil type [3]. Many researchers previously have correlated a number of blows (N value) of SPT test for estimating bearing capacity, relative density and friction angle of the soil [4-6]. The conventional method of calculating soil bearing capacity is mainly depending on the soil parameters (i.e. friction angle and cohesion). In addition to the conventional method, soil bearing capacity can be calculated directly from field tests which called in-situ method [7]. Many correlations between N value and mechanical properties of the soil are suggested and developed by many types of research [8-10]. This paper presents a verification about the correlation suggested by Terzaghi, Peck, et al. [11] between the soil cohesion, c, and the corrected N value by calculating the soil bearing capacity.

2. SITE LOCATION AND DESCRIPTION

The soil samples were taken from a flat plain area located in Nasiriyah city southeastern Iraq. In this study, eighteen (18) boreholes (BH-01 through BH-18) were drilled. The boreholes were drilled to approximate depths ranging from 6.0 to 15.0m below the existing ground surface with a total drilled depth of 167.0 m. Drilling was executed with the aid of one rig, using the rotary air flush drilling technique. After logging and sampling were completed, the boreholes were backfilled and compacted to ground level using the excavated soil. Samples were obtained from the boreholes for laboratory testing and geological description purposes.

3. Testing Procedure

Two types of samples were taken (disturbed and undisturbed samples) for laboratory testing. Undisturbed samples of cohesive soil were retrieved from the boreholes using thin-tube 'Shelby' samplers (77mm OD & 73mm ID and 60mm length), in accordance with ASTM D 1587-

08 (Standard Practice of Thin-Walled Tube Sampling of Soils for Geotechnical Purposes). The ends of the tube were properly sealed by wax to maintain the natural moisture content. These samples were obtained to perform triaxial and unconfined compression tests in the laboratory. Disturbed samples were retrieved from the inside of split spoon samplers after each Standard Penetration Test (SPT). The recovered samples were visually described and placed in watertight plastic bags to maintain the moisture content. Solid stem auger drilling techniques (SSA) were used at intervals where no undisturbed samples or SPTs were performed. Additional disturbed samples were obtained during this process. The recovered samples were examined, visually described in accordance with appropriate standards (BS 5930; ASTM D2488). The samples were placed in waterproof plastic bags before being placed in wooden boxes to maintain their natural water content.

3.1 Laboratory Testing

Laboratory tests were performed on the recovered samples in order to identify the physical, mechanical and chemical properties of the encountered materials. The following laboratory tests were selected and performed on selected samples:

- Classification and index tests: moisture content, Atterberg limits, particle size distribution, and specific gravity.
- Strength tests: Triaxial (Unconsolidated Undrained UU) and unconfined compression strength (UCS) test.
- Chemical tests: pH, sulfate, chloride, TDS, EC and organic matter.

The tests were performed according to the relevant American Society for Testing and Materials (ASTM) Standards and/or British Standards (BS). The tests were divided into two main tests: classification & index and strength tests. Classification and index tests were performed on soil samples retrieved from different depths. The tests included moisture content, specific gravity, Atterberg limits, and grains size distribution (sieve analysis and sedimentation using hydrometer) were carried out according to ASTM D 2216-05, D 854-06, D 4318-10, and D 422-63 (2007). The results of these tests at the specified depth are as provided in Table 1. compressive strength tests were conducted on recovered cores samples. In this test, a cylindrical specimen of the soil is loaded axially, without any lateral confinement to the specimen to obtain an approximate estimation of unconfined compressive strength of cohesive specimens in accordance with ASTM D 2166-06. The maximum stress "q_{un}" measured at failure is equal to two times the undrained shear strength (S_u). Also, unconsolidated undrained (UU) triaxial compression tests were carried out on the soft soil samples in accordance with ASTM D2850-03a (2007). The purpose of this test is to determine the undrained shear strength (S_u) and stress-strain relationships of confined samples. The saturated cohesive soil specimen was sheared without drainage, since pore pressure is not allowed to dissipate, at a constant rate of axial deformation (strain controlled). The failure envelop for the total stress Mohr's circles becomes a horizontal line and therefore, the test results should be interpreted using the Φ=0 concept where the cohesive will be equal to the undrained strength c= S_u and is equal to the radius of the Mohr's circles. In this test, three confining pressures 0.25, 0.5 and 1 kg/cm2 were utilized to determine S_u with the aid of Mohr strength envelop. Moreover, to assess the corrosiveness of the ground materials to underground utilities, several chemical analyses were performed on some soil samples as shown in Table 2.

3.2 In-Situ Testing

Standard Penetration Testing (SPT) was performed in all the drilled boreholes at different in all the encountered materials to obtain approximate dynamic resistance of the ground materials. The tests were performed in accordance with ASTM D 1586-08a. The SPT equipment used in this project consisted of auto-trip hammers (63.5kg weight) and 45.0cm long split tube (5.0cm diameter) with free fall for the hammer of 76.0 cm.

4. RESULTS AND DISCUSSION

4.1 SPT Results

The SPT results are shown in Fig.1 (Logs of Boreholes). It is shown that the value of the blow count fluctuates with a depth of boreholes. Different values of blow count for each borehole was noted. The blow count from the standard penetration test is used for calculating the value of cohesion (c), from the following relation [11]:

$$\boldsymbol{c = 6N_{corrected}} \tag{2}$$

Where N corrected is the lowest average corrected value for the silty clay layer. Due to the presence of groundwater at shallow depth and possibility of punching shear, effective strength parameters shall be used. Therefore, the recommended cohesion value of c' =0.67×c, shall be considered in bearing capacity computing.

Table 1 Results of the Classification and Index Tests

BH-No.	Depth (m)	Water Content, %	Atterberg Limits			Plasticity	Grain Size Distribution				Expansiveness Potential
			LL (%)	PL (%)	PI (%)		G (%)	S (%)	M (%)	C (%)	
BH-01	0.0-1.0	2.60	23.1	17.3	5.8	Low	49.1	36.0	10.7	4.2	Low
	2.5-3.0	28.50	43.7	21.7	22.0	Medium	0.0	7.6	49.7	42.7	Med.
	3.0-4.0	30.48	43.4	21.4	22.0	Medium	0.0	11.1	42.8	46.1	Low
	5.5-6.0	43.71	62.7	28.8	33.9	High	0.0	1.5	31.5	67.0	Med.
BH-02	0.0-1.0	17.89	35.4	21.8	13.6	Medium	16.0	26.1	35.0	22.9	Med.
	1.0-2.5	25.84	38.2	20.5	17.8	Medium	0.0	11.6	48.7	39.7	Low
	3.5-4.0	33.08	47.4	24.0	23.5	Medium	0.0	5.4	44.2	50.4	Low
	4.5-5.0	36.82	45.3	20.6	24.7	Medium	0.0	6.0	41.3	52.7	Low
BH-03	0.0-1.0	12.69	46.4	22.0	24.5	Medium	17.5	32.2	19.2	31.2	High
	4.5-5.0	29.20	37.6	18.3	19.3	Medium	0.0	20.9	33.6	45.5	Low
	5.0-5.5	23.14	36.6	23.1	13.5	Medium	0.0	7.9	70.9	21.2	Med.
BH-04	0.0-1.0	20.62	43.6	20.4	23.2	Medium	0.6	6.1	46.3	47.0	Low
	5.5-6.0	35.46	57.6	25.2	32.2	High	0.0	2.0	36.3	61.7	Med.
BH-05	0.0-1.0	25.43	37.7	20.7	17.0	Medium	0.0	3.1	58.3	38.7	Low
	1.5-2.0	30.35	65.0	25.7	39.2	High	0.0	1.2	22.3	76.5	Med.
BH-06	8.5-9.0	36.65	54.4	24.4	30.0	High	0.0	3.5	37.6	58.9	Med.
	14.5-15	22.55	NP	NP	NP	NP	0.0	71.1	13.1	15.8	Med.
BH-07	2.5-3.0	21.30	61.7	25.3	36.4	High	0.0	1.8	29.9	68.3	Med.
BH-08	0.0-1.0	24.87	38.4	19.6	18.8	Medium	0.0	4.1	54.8	41.1	Low
	5.5-6.0	32.19	65.8	28.4	37.4	High	0.0	2.4	23.8	73.8	Med.
BH-09	0.0-0.5	22.94	48.4	23.8	24.6	Medium	1.5	2.9	50.3	45.3	Med.
	8.0-8.5	35.68	65.5	26.5	39.0	High	0.0	0.9	31.7	67.4	Med.
BH-10	0.0-1.0	3.00	34.7	18.8	15.9	Low	65.1	22.9	4.4	7.6	Low
	2.0-2.5	26.89	47.0	22.3	24.7	Medium	0.0	9.5	56.8	33.8	High
BH-11	0.0-1.0	8.52	32.3	19.5	12.7	Low	20.9	43.1	16.5	19.4	Med.
	2.5-3.0	26.45	60.7	26.3	34.4	High	0.0	1.6	22.2	76.2	Low
	5.5-6.0	35.63	53.4	26.0	27.4	High	0.0	4.0	15.3	80.0	Low
	7.5-8.0	32.28	63.7	26.0	37.7	High	0.0	0.9	16.5	82.7	Low
	9.0-9.5	33.15	45.1	22.0	23.1	Medium	0.3	9.8	45.7	44.1	Med.
BH-12	0.0-1.0	8.96	30.3	21.2	9.1	Low	35.7	44.0	8.6	11.7	Low
	4.0-5.0	22.79	38.3	24.4	14.0	Medium	0.0	2.4	31.8	65.8	Low
	8.5-9.5	23.43	51.9	26.7	25.2	High	0.0	9.7	38.5	51.9	Low
	9.5-10	35.46	62.2	24.5	37.6	High	0.0	1.3	31.1	67.6	Med.
BH-13	2.5-3.0	25.62	45.4	19.8	25.6	Medium	0.0	21.3	40.1	38.6	High
	8.0-9.5	28.54	60.7	25.3	35.3	High	0.0	1.3	21.9	76.8	Low
BH-14	0.0-1.0	8.22	35.2	18.6	16.7	Medium	36.2	38.5	8.7	16.7	Med.
	14-15	27.41	NP	NP	NP	NP	0.0	46.9	42.5	10.6	Low
	3.5-4.5	29.74	63.0	24.2	38.8	High	0.0	5.8	24.7	69.5	Med.
	7.5-8.0	33.02	60.9	27.8	33.2	High	0.0	7.4	25.1	67.5	Low
BH-15	0.0-1.0	5.71	42.2	21.1	21.1	Medium	49.8	33.9	7.3	9.0	Low
	2.5-3.0	28.46	39.4	23.5	15.9	Medium	0.0	2.4	59.5	38.1	Low
	6.5-7.0	37.71	58.0	27.4	30.6	High	0.0	3.4	43.1	53.3	Med.
BH-16	0.0-0.5	18.64	41.4	22.5	18.8	Medium	1.4	3.7	70.1	24.9	Med.
	2.0-2.5	28.87	55.2	24.8	30.4	High	0.0	7.6	44.5	47.9	High
	9.5-10	32.71	39.5	20.6	18.9	Medium	0.0	7.9	45.0	47.0	Low
BH-17	0.0-1.0	18.17	48.4	22.5	25.9	Medium	0.0	0.9	54.9	44.2	Med.
	4.5-5.5	30.59	60.7	26.3	34.4	High	0.0	1.7	31.5	66.8	Med.
BH-18	0.0-1.0	21.5	43.6	22.2	21.7	Medium	0.0	3.1	51.5	45.3	Low
	3.0-4.0	28.35	62.2	26.4	35.8	High	0.0	1.3	48.3	50.4	Very High
	8.5-9.5	35.33	50.2	22.5	27.7	High	0.0	8.4	36.9	54.7	Med.

Table 2 Some Chemical Test Results for the Soil Samples

BH-No	Depth (m)	Sulfate SO3 (%)	Water Soluble Sulfate SO4 (%)	Chloride Cl (%)	Organic Matter (%)	pH Value
BH-01	0.0-1.5	0.742	0.890	0.397	3.26	7.8
	5.5-6.0	0.851	1.021	0.263	1.98	8.1
BH-02	0.0-1.0	0.951	1.141	0.263	3.52	8
	1.0-2.5	0.922	1.106	0.702	3.31	8
BH-03	2.5-3.0	0.849	1.019	0.191	1.99	7.8
BH-06	8.5-9.0	0.682	0.818	0.688	2.86	8.5
BH-07	2.5-3.5	0.846	1.015	1.085	0.00	7.0
BH-10	0.0-1.0	0.678	0.814	0.212	1.24	7.6
BH-11	0.0-1.0	0.912	1.094	0.142	2.31	8.3
	5.5-6.0	0.889	1.067	0.603	4.11	8.1
BH-13	2.5-3.5	0.792	0.950	0.397	3.39	8.0
BH-14	6.0-6.5	0.925	1.110	0.567	3.58	7.9
	7.5-8.0	0.632	0.758	0.603	3.04	8.1
BH-15	0.0-1.0	0.954	1.145	0.687	0.00	8.7
BH-16	2.0-2.5	0.998	1.198	0.588	2.51	7.9
BH-17	0.0-1.0	0.921	1.105	0.681	1.88	7.8
BH-18	3.0-4.0	0.675	0.810	0.638	3.09	7.8

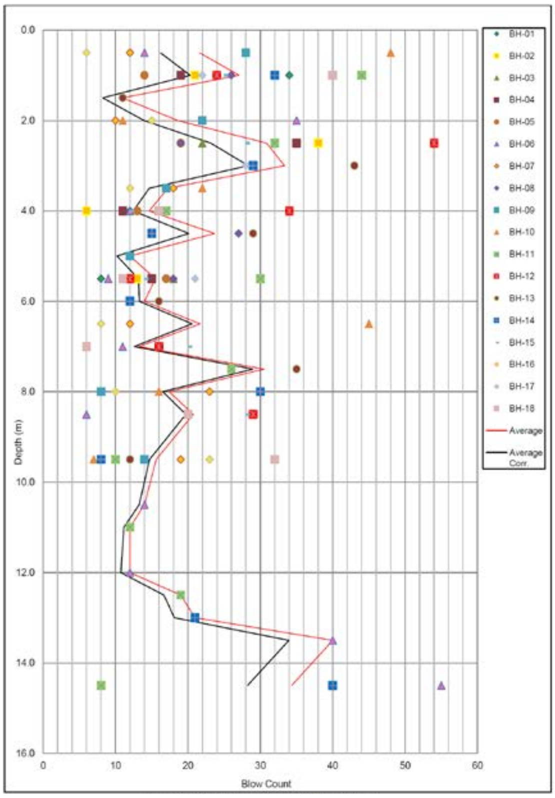

Fig.1 Graphical Illustration of SPT Results

4.2 Unconfined Compressive Strength

The following equation was used to calculate the cohesion values from the unconfined compressive strength test results [12]:

$$c = S_u = \frac{q_{un}}{2} \quad (3)$$

Where c is the cohesion, which is equal to the undrained shear strength (Su), qun is the unconfined compressive strength, q_{un} = 75 kPa (Average value). The unconfined compressive strength test results of some soil samples are shown in table 3. The maximum stress "q_{un}" measured at failure is equal to two times the undrained shear strength (S_u). The reliability of this test decreases with respect to increasing sampling depth because the sample tends to swell after sampling resulting in greater particle separation and reduce shear strength.

4.3 Triaxial (UU) Test Results

In this test, three confining pressures 0.25, 0.5 and 1 kg/cm2 were utilized to determine S_u with the aid of Mohr strength envelop are as summarized in Table 4.

5. BEARING CAPACITY

The ultimate bearing capacity was calculated based on the results of the lab unconfined compressive strength tests, using modified Terzaghi's bearing capacity equation (modifications provided by Vesic) which is presented as follows [13]:

$$q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma \quad (4)$$

$$q_{netult} = q_{ult} - q \quad (5)$$

Where q_{ult} and q_{netult} are ultimate bearing capacities and net ultimate bearing capacities, respectively. c is the cohesion. N_c, N_q & N_γ are bearing capacity factors, S_c, S_q & S_γ are shape factors, d_c, d_q & d_γ are depth factors, B is the foundation width (m), $\bar{q}$ is the effective overburden pressure, from the relation: $\bar{q} = \gamma * D$, γ is bulk density of the overburden, D is height of overburden. For quick failure (Φ = 0); the following values should be used:

- Bearing capacity factor, which is equal to:

$$N_c = 5.14, N_\gamma = 2(N_q + 1)\tan\phi = 0, N_q = 1 \quad (6)$$

- Shape factor, which is equal to:

$$S_c = 1.0 + \left(N_q/N_c\right) \times \left(B/L\right), S_\gamma = 1 - 0.4\left(B/L\right), S_q = 1 \quad (7)$$

- Depth factor, which is equal to:

$$d_c = 1 + 0.4\left(D/B\right) for\ D/B \leq 1\ \& = 1 + 0.4\tan^{-1}\left(D/B\right)\ for\ D/B > 1, d_\gamma = 1, d_q = 1 \quad (8)$$

In this case, the net ultimate bearing capacity is:

$$q_{netult} = 5.14\ c\ S_c d_c \quad (9)$$

Figure 2 shows the value of bearing capacity calculated by SPT tests, unconfined compression tests, and triaxial unconsolidated undrained tests. The results show that the measured bearing capacity by unconfined compression test and triaxial unconsolidated undrained tests in some cases are close to each other. While the measured bearing capacity by SPT tests is far away from the measured bearing capacity by unconfined compression tests, and triaxial unconsolidated undrained tests. In general, the values of the measured bearing capacity by SPT tests are very high, fluctuated and uncertain [14].

Table 3 Unconfined Compressive Strength Test Results

BH-No.	Depth (m)	Soil Type	Water Content (%)	Bulk Density (g/cm3)	Unconfined Compressive Strength q_{un} (kPa)	Undrained Shear Strength S_u (kPa)
BH-03	5.0-5.5	Silty Clay	35.33	1.88	26	13
BH-05	2.0-2.5	Silty Clay	25.62	1.97	56	28
BH-06	3.5-4.0	Silty Clay	33.02	1.9	74	37
BH-08	4.0-4.5	Silty Clay	24.39	2.16	107	53.5
BH-09	6.5-7.0	Silty Clay	38.2	1.86	27	13.5
BH-10	5.0-5.5	Silty Clay	40	2.02	14	7
BH-13	2.5-3.0	Silty Clay	38.58	1.85	33	16.5
BH-14	7.5-8.0	Silty Clay	3421	1.87	175	87.5
BH-16	5.0-5.5	Silty Clay	26.92	2.06	44	22
BH-17	5.0-5.5	Silty Clay	23	2	152	76

Table 4 Undrained Strength Results Obtained from Some Triaxial (UU) Test

BH-No.	Depth (m)	Soil Type	Undrained Shear Strength S_u (kPa)
BH-08	4.0-4.5	Silty Clay	17.06
BH-10	5.0-5.5	Silty Clay	9.7
BH-09	6.5-7.0	Silty Clay	42.43
BH-14	7.5-8.0	Silty Clay	40.48

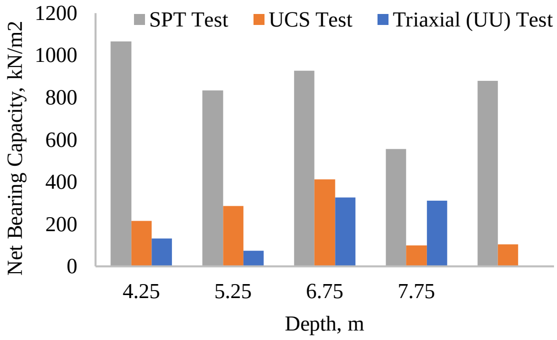

Fig.2 Soil bearing capacity vs. depth calculated using SPT, UCS, and Triaxial (UU) tests

The variation or the difference in between the calculated bearing capacity from the laboratory data and the field or site data can be interpreted as shown in the subsequent paragraphs.

The results showed that the increase in the clay content decreases the N value significantly (Fig.3). Also, similar behavior was noted when increasing the silt content, the N value decreased (Fig.4). While increasing the sand and gravel content causes to increase the N value (Figs. 5 and 6). Similar behavior was noted by Liang, Cao et al. [15].

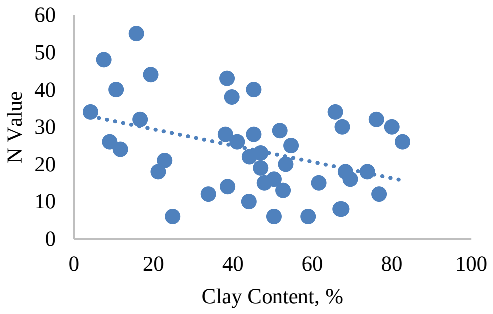

Fig.3 SPT value vs. clay content

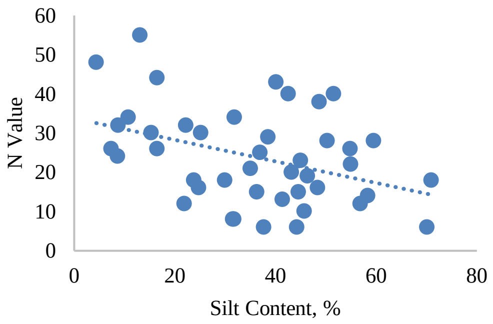

Fig.4 SPT value vs. silt content

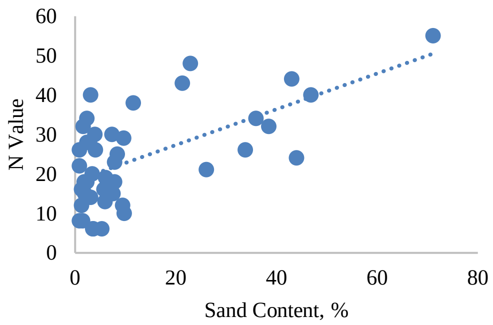

Fig.5 SPT value vs. sand content

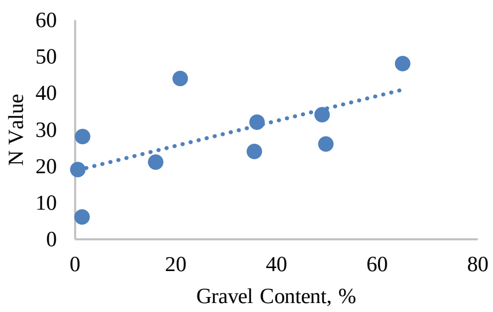

Fig.6 SPT value vs. gravel content

This behavior indicates that the N value gives valid information in soils content high quantity of silt or clay due to the lower friction values between the clayey or silt and the SPT tube in which the values of N decreased as well. On the other hand, the high-water content of soil samples decreases the friction between the SPT tube and the clayey or silty soil samples more. Which in turn decrease the N values (Fig. 7). Therefore, the previously mentioned correlation between the soil cohesion, c and the corrected N Value in equation 2 should be adjusted to decrease the error in calculating the bearing capacity.

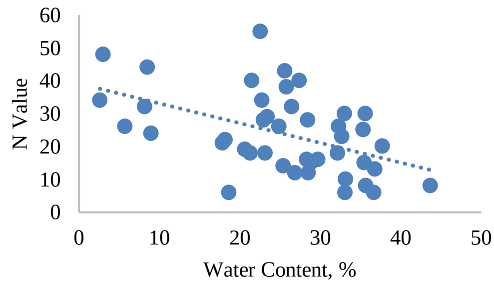

Fig.7 SPT value vs. soil samples water content

Moreover, it noted that the soil plasticity index has a slight effect on the N value. The effect of plasticity index on the SPT values is shown in figure 8. Its shown that different N values for different plasticity index values distributed randomly. This behavior possibly due to that water content of the natural soil sample was varied, which in turn leads to giving random values for N in SPT test.

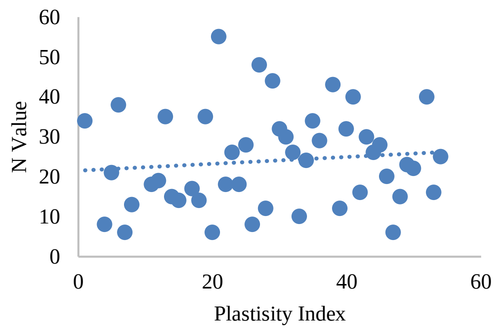

Fig.8 SPT value vs. plasticity index

3. CONCLUSION

Experimental research on the variation of the calculated bearing capacity from field test (SPT Test) and laboratory tests (UCS and Triaxial UU) were evaluated. From the analysis of experimental data, some outcomes were derived. The measured bearing capacity depending on the UCS test and Triaxial UU test is close to each other in some points. The measured bearing capacity depending on the SPT test is higher than laboratory tests. The estimated value for cohesion, c by Terzaghi, Peck, et al. 1996 (eq.2) is very high and does not represent the actual value especially in clayey soils. The increase in clay and silt content decreases the N value. While the increase in the sand and gravel content increases the N value. Moreover, the increase in the soil sample water content decreases the N value.

4. ACKNOWLEDGMENTS

This research was supported by the Arab Center for Engineering Studies (ACES) Arab Center for Engineering Studies (ACES). The author would like to thank the Arab Center for Engineering Studies (ACES) who provided insight and expertise that greatly assisted the research.

5. REFERENCES

[1] Carter, M. and S.P. Bentley, Soil Properties and Their Correlations2016: Wiley.

[2] Raheem, A.M., On the Behavior of Lateral Pipe-Soil Interaction in Ultra-Soft Clayey Soil Using Large Scale-Laboratory Tests. International Journal of Engineering and Technology Innovation, 2019. 9(2): p. 119-130.

[3] Huang, A.B. and P.W. Mayne, Geotechnical and Geophysical Site Characterization: Proceedings of the 3rd International Conference on Site Characterization (ISC'3, Taipei, Taiwan, 1-4 April 2008). BOOK Keynote papers (258 pages) + CD-ROM full papers (1508 pages)2008: CRC Press.

[4] Meyerhof, G.G., Penetration tests and bearing capacity of cohesionless soils. ASCE Journal of Geotechnical Engineering, 1956. 82(1): p. 866/1-866/19.

[5] Gibbs, H.J., W.G. Holtz, and H.J. Gibbs, Research on determining the density of sands by spoon penetration testing1956, Denver, Colo.: Commissioner's Office, Engineering Laboratories.

[6] Yoshida, Y., M. Ikemi, and T. Kokushu, Empirical formulas of SPT blow-counts for gravelly soils: Proc 1st International Symposium on Penetration Testing, ISOPT-1, Orlando,20–24 March 1988 V1, P381–387. Publ Rotterdam: A A Balkema, 1988. International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, 1990. 27(3): p. A156.

[7] John, T. and S. Thomas, Bearing capacity prediction for shallow foundations by in-situ and experimental investigation. International Journal of Scientific & Engineering Research, 2017. 8(4): p. 1375 - 1379.

[8] Ruge, J.C., et al., ANALYSIS OF THE UNDRAINED SHEAR STRENGTH THROUGH THE STANDARD PENETRATION TEST WITH TORQUE (SPT-T). International Journal of GEOMATE, 2018. 14(41): p. 102-110.

[9] Kumar, R., K. Bhargava, and D. Choudhury, Estimation of Engineering Properties of Soils from Field SPT Using Random Number Generation. INAE Letters, 2016. 1(3): p. 77-84.

[10] Akca, N., Correlation of SPT–CPT data from the United Arab Emirates. Engineering Geology, 2003. 67(3): p. 219-231.

[11] Terzaghi, K., R.B. Peck, and G. Mesri, Soil Mechanics in Engineering Practice1996: Wiley.

[12] Das, B.M., Advanced Soil Mechanics. 3 ed2008, New York: Taylor & Francis Group.

[13] Das, B.M., Principles of Foundation Engineering. 7 ed2010, Stamford: Nelson Education Limited.

[14] 14. Kudmetha, K.K. and A. Dey. Uncertainty in Predicting Bearing Capacity of Piles in Sand Using SPT Data. in Proceedings of the International Symposium on Engineering under Uncertainty: Safety Assessment and Management (ISEUSAM - 2012). 2013. India: Springer India.

[15] 15. Liang, Y., L. Cao, and J. Liu, Statistical Correlations between SPT N-Values and Soil Parameters, R.U. Department of Civil Engineering, Editor 2015, Natural Sciences and Engineering Research Council of Canada and Supported by SPL Consultants Ltd: Canada.