An assessment on reliability of SPT data in determining the bearing capacity of pile installed into cohesive soil (a case study)

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ABSTRACT: Nowadays, in situ test has played an important role in geotechnical engineering and subground technology. Beside lab tests conducted on undisturbed soil samples, many different kinds of in-situ tests were used and proved to be more efficient in foundation design such as pressionometer PMT, cone penetration test CPT, stantdard SPT etc. Among them, standard penetration test (SPT for short) is a very common and easy to carry out at site. For decades, it has proved to be reliable to sandy soil, but many view points and opinions argued that the test was not appropriately applicable to cohesive soil because of scattered and dispersed data of SPT blow counts through different layers. This paper firstly studies how much SPT data are predictable for physical properties and cohesive soil strength for comparison with other kinds of soil (i.e. sand, silty sand); secondly, this paper quantifies and assesses how confident the reliability of N SPT values is, particularly in predicting bearing capacity of pile. By analysing data from 40 boreholes located in 18 projects in Ho Chi Minh City, South VietNam, many significant correlations between SPT numbers and physical and mechanical properties of cohesive soil was found. Finally, the results from analytical approaches of predicting pile bearing capacity were compared to those of finite element program Plaxis 3D and static load test at site. Correlation between the capacity computed by using corrected N-values in lieu of soil strength and results of static load test has proved to be well suitable in evaluating bearing capacity of driven and jack-in piles.

Keywords: Standard Penetration Test (SPT), cohesive soil, statistical correlation, physical and mechanical properties of soil , bearing capacity of pile.

1. Introduction

Bearing capacity of a pile installed in soil foundation can be predicted by several different methods. Some methods used directly soil properties, i.e. shear strength parameters (cohesion and internal friction angle) obtained by lab test on undisturbed soil samples; other *indirect methods* used data from in situ tests to infer or soil strength, then analytical formula was applied to estimate bearing capacity of pile with such converted properties. SPT is an in-situ test that in decades hitherto was proposed to be reliable to sandy soil.

For sandy soil, there is a real significant correlation between internal friction angle and N value (Hatanaka and Uchida, 1996[1]). SPT can be used for predicting bearing capacity of shallow footings in which, bearing capacity factor for depth of footings N_q, for friction N_γ were calculated from regression equations founded by different authors (Burland and Burbidge, 1984) [2]. In the research, energy ratio is 60 percent, corrected SPT value N_{60} can be used for estimating the allowable contact pressure with respect to a 30 % probability of exceeding a settlement of 25 mm.

For clayey and cohesive soil in general condition, it is rather complicate to ensure practicability of SPT for predicting soil properties. Due to scattered data in correlation between SPT values and the increase of pore water pressure as the rod plunged into clay layers. Moreover, low permeability of soil may lead to appear a temporary resistance for driving, higher number of blows will be obtained. It might have generally been proposed that SPT is rather unreliable to this kind of fine grained soil. Nevertheless, SPT has still been a main in-situ test in soil investigation reports, even for large scale projects. Mostafa Abdou Abdel Naiem Mahmoud (2013) studied reliability of using SPT in predicting geoengineering properties of silty clay and sandy soil. The results indicated not physical properties but shear strength parameters such as cohesion and internal friction angle had significant correlation to corrected N SPT number, even silty clay [3].

Variability of soil can be described clearly by coefficient of variation (COV). This coefficient varied to a very wide range, depending on soil type. Samples from large data set at different location had the COV (ratio of standard deviation divided by mean) from more than 40 % to 150 % for some conventional properties (Phoon, K.K. and Kulhawy, F.H., 1999) [4]

Table 1. Wide range of coefficient of variation for clay soil [4]

Clay Layer	Depths (m) parallel to surface	Depths (m) Horizonal plans	Statistics	N-SPT
Layer 1	2 – 4	1 – 3	Mean St. Dev COV (%)	2.6 3.9 150
Layer 2	9 – 11	8 – 10	Mean St. Dev COV (%)	8.2 10.4 127
Layer 3	28 – 42	28 – 42	Mean St. Dev COV (%)	8.9 4.1 47

On the other hand, soil has inherent uncertainties due to spatial variability and its response with respect to load and effects. Response of sand is quite different as compared to that of clay. Kulhawy and Traumann (1996) pointed out 4 categories of uncertainties, relating to soil type; they are: saturation status, devices and techniques of testing and other uncontrollable factors (borehole diameter, efficient energy delivered to hammer, robs and type of drilling equipment…etc.). There are more than 27 sources of uncertainties relating to soil characteristics (5 factors), water table (2 factors), equipment (7 or more factors) and more than 10 factors of working condition at sites (Zekkos, D.P, Bray J.D. and Der Kiureghian A., 2004) [4]. Therefore, regarding to SPT, it should take into account uncertainties in the performance of the test. There were many research works for this content (Kulhawy and May, 1990; Schmertmann, 1975; Barton, 1990; Youd et al., 2001, etc.). Both ASTM D-1586, ASTM D6066-96 also prescribed recommendation for this feature as well.

In sandy soil, N is largely affected by overburden pressure. The more value of this pressure is, the correction factors will be higher, approaximately proportioning to square of relative density D_r (Meyerhoff, 1957) as expressed in following formula :

$$N = D_r^2 (a + b.p') \qquad (1)$$

where *a* and *b* are factors of material dependence and p' is the mean effective stress (Kudmetha, K.K., Dey, A., 2012) [6]. Then the correction factor due to overburden pressure, namely C_N, is

$$C_N = \frac{N_{correct}}{N} = \frac{D_r^2(a+b.100)}{D_r^2(a+b.p')} = \frac{a+b.100}{a+b.p'} = \frac{a/b + 100}{a/b + p'} \qquad (2)$$

The correction factor C_N by Skempton (1986) defined in Eq. (2) is

$$C_N = \frac{n}{1 + 0.01\sigma'_{vo}} \qquad (3)$$

where σ'_{vo} is overburden pressure in kPa, *n* = 2 for loose sand and *n* = 3 for dense sand.

For decades, numerous research works tried to determine the reliability of SPT data in predicting bearing capacity of pile installed into *cohesive soil*. For analysis of reliability, it is necessary to quantify uncertainties by statistical parameters (i.e. standard deviation, mean and the law of distribution). Some authors proposed a scheme of reliability based design in geotechnical engineering (Honjo Y., 2011 [7]).

For examining correlations between properties in soil foundation, scale of fluctuation should be determined. That is a distance in soil foundation within which there is an autocorrelation for a specific property (Vanmarke, 1983) [8]. According to Duong Tan Tai (2017) [9], Vanmarcke's theories can be used to quatify the spatial variability of soil properties in estimating the allowable bearing capacity of bored pile. The result is a regression formula as below:

$$Q_a = 4346.3 - 5202.1\rho_{12} + 34.9\theta + 353.3COVc' + 357.7COV\varphi' - 565.1COVE - 2.9\beta + 0.14Q_{gh} \qquad (3)$$

where ρ_{12} is the coefficient of correlation between the two zones 1 and 2 along the pile shaft and θ is scale of fluctuation in vertical direction. COV(c) COV(*φ*) is coefficient of variation of cohesion and internal friction angle, respectively; β is reliability index and P_u is the ultimate bearing capacity of single pile [9].

In this paper, SPT data were corrected to use indirectly in predicting bearing capacity of a driven pile. By using regression analysis over large amount of data, correlation between SPT and soil properties such as water content, depth of testing, modulus of elasticity, Atterberg limit etc. of samples taking from 40 boreholes of 18 projects in Ho Chi Minh City were found, soil strength parameters converted from corrected SPT numbers together with physical properties of soil was also determined. At least three approaches of computing bearing capacity of pile installed into foundation were used in which layer of cohesive soil was dominant. Results obtained were compared to that of static load test.

2. Method

2.1. Literature reviews on N SPT data

Undisturbed soil samples were brought to laboratory to identify physical and mechanical properties. Conventional tests, e.g. Atterberg Limits test, direct shear test, unconfined compression test etc. were sufficient for supplying materials for computing bearing capacity of footings and single pile; N data were in-situ test and they were raw data recorded at site without any correction. Depths of in situ testing were noted together with N blow counts.

Bazaraa's formula [10] can be chosen to correct the raw N numbers. The first correction is due to the overburden pressure and the second is only for silty and fine grained soil. There was no other correction for borehole diameter or for energy delivered to the rod penetrating into soil layers. Therefore, the first uncertainty was the *energy percentage* delivered to the penetrator. Besides, there was no evidence that the site test had the same efficiency at every size of boreholes; and how about the rod length affects… In general, the abovementioned issues could be taken into account by the formula as below:

$$N_{60} = N_{field}.C_E.C_B.C_S.C_R \qquad (4)$$

where N_{field} are the blow counts recorded at site; C_E is the factor of energy correction, C_B _ factor of boreholes, C_S _ factor of soil sample size, and C_R is the factor of rod length. Many different authors had pointed out the different levels of energy ratio ER (that was defined to be a ratio of measured energy divided by theoretical energy). Aoki and De'Alencar (1975) [11] indicated ER = 70 %; Shioi and Fukui (1985) suggested ER=55 % , Meyerhoff (1976) ER=55 %. A number ER= 55 % was proposed to be appropriate to use in pile bearing capacity for projects in Viet Nam (Hoang T.Q and Tam, N.M, 2016) [11].

Table 2. Bearing capacity of pile using SPT numbers [11]

Formula by	Skin friction	Point bearing
Aoki and De'Alencar (1975)	$Q_s = \frac{a.k}{3.5} N_s$ a =14, k =1 (sand);	$Q_p = \frac{k}{1.75} \overline{N_b}$ a = 60,k = 0.2 (clay)
Shioi and Fukui (1985)	Sand $Q_s = 2N_s$ Clay $Q_s = 10N_s$	Sand $Q_p = (1+0.04\frac{D_b}{B})N_b$ Clay $Q_p = 0.06\frac{D_b}{B})N_b$
Meyerhoff (1976)	$n_s = 1$ $Q_s = n_S N_s$ Non-displacement pile; $n_s = 2$ Displacement pile	$Q_p = 0.4 N_b C_1 C_2$ C_1C_2 factors dependent to ratio D/B (i.e. diameter to pile length)
Bazaraa and Kurkur (1986)	$Q_s = n_S N_s$ n_s=2~4	$Q_b = n_b N_b$ n_b=0.06~0.2 N_b average of N taken 1B above and 3,75B below pile tip

2.2. Correction Factors

Bazaraa (1967) [10] proposed the following corrections to obtain the actual count N, based on the overburden pressure (N modified into N') :

For $p_o \leq 75$ kPa

$$N' = \frac{4N}{(1+0.04 p_o)} \tag{5a}$$

For $p_o > 75$ kPa

$$N' = \frac{4N}{(3.25+0.01 p_o)} \tag{5b}$$

where N' is corrected N value; N is observed N-value;
p_o is overburden pressure, (kPa) = γD;
D is depth of penetration (m);
γ is unit weight of soil at the time of testing.

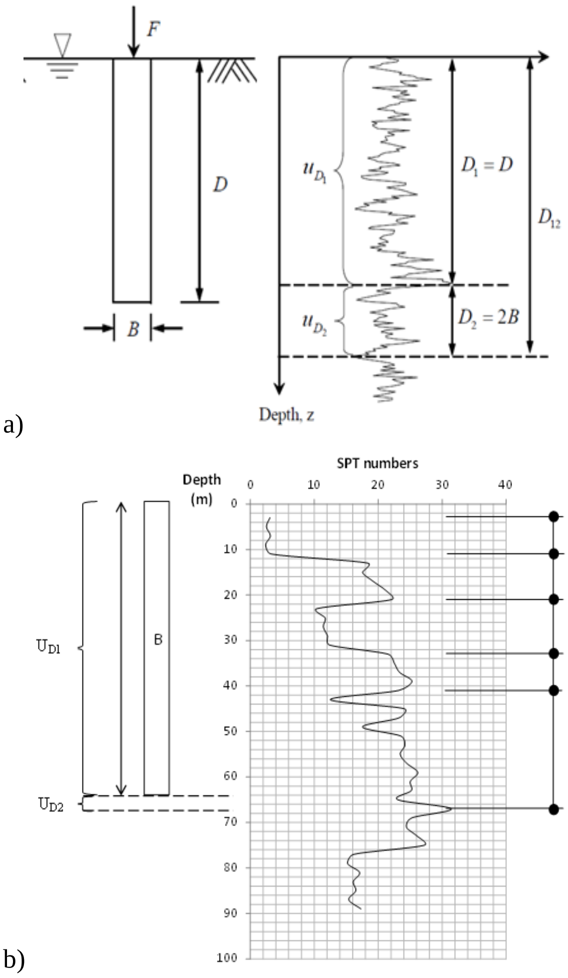
If the stratum (during testing) consists of fine sand & silt below water table, the corrected N-value (or N') has to be further corrected to get the final corrected value N" as below:

$$N'' = 15 + \frac{1}{2}(N' - 15) \tag{6}$$

As such, there are two steps of correction against N numbers for every usage for all geotechnical computations.

2.3. Spatial variability – scale of fluctuation

Pile bearing capacity includes shaft friction along the length u_{D1} and point bearing within u_{D2} as described in Fig. 2.



a)

b)

Figure 1. Diagram for computation scale of fluctuation [13]

Corrected SPT numbers were assumed to display as in Fig. 2b. Correlation between the two zone u_{D1} and u_{D2} was characterized by a correlation factor as follows:

$$\rho_{u_{D1},u_{D2}} = \frac{D_{12}^2\Gamma^2(D_{12}) - D_1^2\Gamma^2(D_1) - D_2^2\Gamma^2(D_2)}{2D_1 D_1 \Gamma(D_1)\Gamma(D_1)} \tag{7}$$

$(0 < \rho_{u_{D1},u_{D2}} < 1)$

where $\Gamma^2(\Delta z)$ denoted variance reduction factor for spatial average for interval Δz of pile shaft, determined by Vanmarcke (1983):

$$\Gamma^2(\Delta z) = \frac{1}{2}(\frac{\theta}{\Delta z})^2[\frac{2\Delta z}{\theta} - 1 + \exp(-\frac{2\Delta z}{\theta})] \tag{8}$$

Scale of fluctuation θ _ that is defined as the distance within which some specific soil property have significant correlation from point to point (Vanmarcke 1977 and 1983) [8] _ and variance reduction factor $\Gamma^2(\Delta z)$ will be applied simultaneously to calculate :

- Skin friction using average value of corrected SPT number (i.e. within scale of fluctuation);
- Point bearing using average value of corrected SPT number from 1D below pile tip and 4D above level of pile tip.
- Reliability using standard deviation, which is square root of variance multiplied by $\Gamma^2(\Delta z)$;
- Regression Formula between bearing capacity and several predictors (i.e. independent variables and parameters), somewhat like the abovementioned formula (3) postulated by Duong Tan Tai (2017) [9].

2.4. *Pile bearing capacity considering spatial variability*

As abovementioned remarks, steps for estimating bearing capacity for pile will consider two main issues: correction and spatial variability (both vertical and horizontal direction). Suitable approach will be suggested as below:

- SPT blow counts will be corrected first, two kinds of correction are obligatory: due to depth (sand) and due to fine grained soil and silty sand (clayey soil);
- Determine scale of fluctuation of SPT numbers N. According to Vanmarcke (1977) [8], scale of fluctuation θ may approximately equals to 0.8($\overline{d}$) where

$$\overline{d} = \frac{1}{n}\sum_{i=1}^{n} d_i \tag{9}$$

where d_i as shown in Fig. 2 is intersections of fluctuating property and its trend function (layers d_1, d_2… d_i is less than scale of fluctuation θ). Computation was conducted on sublayers which are smaller than θ.

- Determine characteristic length L (Cherubini, 2000 [14]) L=D+B in which D is the embedment depth and B is foundation width (i.e. pile diameter).
- Compute Γ^2(L), denoted variance reduction factor, using Vanmarcke's formula (8);
- Within scale of fluctuation, the average value of corrected N SPT number was used to compute friction component of pile bearing capacity;

Table 3. Data collection for regression analysis [15]

Project No	Depth of sampling **Z**	Water content **ω**	Dried unit weight γ_d (kN/m^3)	Initial voil ratio **e**	Plasticity index $\mathbf{I_p}$	Overburden pressure p_o (kPa)	Elastic modulus **E** (kPa)	Raw SPT N	Corrected N'	Corrected N''
2	2	0.38	13.02	1.07	0.24	35.3	22.61	2	4	10
2	4	0.39	13.03	1.06	0.25	70.8	21.49	2	3	9
2	6	0.29	14.55	0.84	0.17	88.4	24.69	5	5	10
2	6	0.33	14.19	0.90	0.19	78.6	23.26	5	5	10
…										
18	48	0.20	16.68	0.63	0.25	520	45.92	33	16	16
18	50	0.19	16.61	0.61	0.18	539	48.68	34	16	16
18	52	0.18	16.60	0.62	0.19	559	42.70	37	17	16
18	54	0.18	16.74	0.60	0.17	578	45.15	31	14	15

- Compute friction component of bearing capacity for individual segments (incremental length of pile) and point bearing component of bearing capacity of pile;
- In numerical model, layers will be divided into sub-layers that based upon scale of fluctuation within which, soil strength is indirectly determined by corrected N- values;
- If reliability index of bearing capacity is required, compute variance reduction factor (i.e. variance multiplied by variance reduction factor), standard deviation and average value (reliability index was defined as ratio between average value divided by standard deviation for a specified limit function).
- Reliability of SPT numbers will be assessed by comparing indirectly predicted value of pile bearing capacity using either Meyerhoff's formulas and finite element modeling software and that of static load test.

3. *Results and discussions*

3.1. *Data collection and regression analysis*

In order to assess the reliability of SPT numbers in predicting the physical properties and strength of cohesive soil, the procedure is:

- Classify data into 4 groups: medium sand, silty sand, clayey sand and sandy clay.
- Soil data of 40 boreholes taken from 18 projects were tabulated as in table 2, in which soil was again classified into three groups: non-cohesive soil (sand), fine soil (clay) and cohesive soil (both clayey sand and sandy clay) for clearly physical properties.
- Because SPT data must be corrected by transforming N into N' for sand and N' into N" for clay, depth of sampling was taken into account in all regression equations.

Table 4. Collected Data of 18 projects [15]

Type	Data		
	Project Number	Number of samples	State of soil
Sand	2,6,10,17, 18 (BH1)	20	Medium density
Clayey sand, sandy clay	1, 2, 3, 4, 6, 7, 8, 9, 10, 11, 12, 13, 16, 17, 18 (BH1)	185	Mainly plastic
Clay	1,2,3,4,5,6,7 9,10,11,12, 16,17,18 (BH2)	233	Semi solid to stiff, low plasticity

In order to find out the correlation between SPT numbers and soil physical properties such as unit weight γ, moisture ω, void ratio *e*, modulus of elasticity E, plasiticity index I_p and shear strength (c_u, φ), data were tabulated as described in table 3 for each soil group.

Because SPT numbers were not only related to one specific property, multi variate linear regression analysis is applied via Data Analysis tool of Excel.

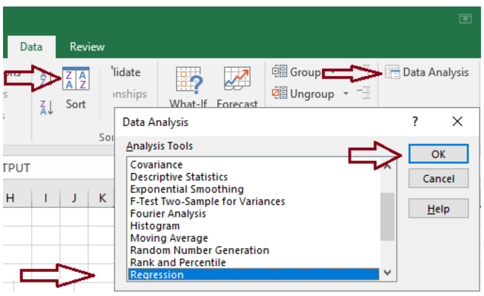


Figure 2. Data analysis tool in Excel [15]

With level of confidence is 95%, corrected N is chosen to be dependent variable Y and independent variables Xs.

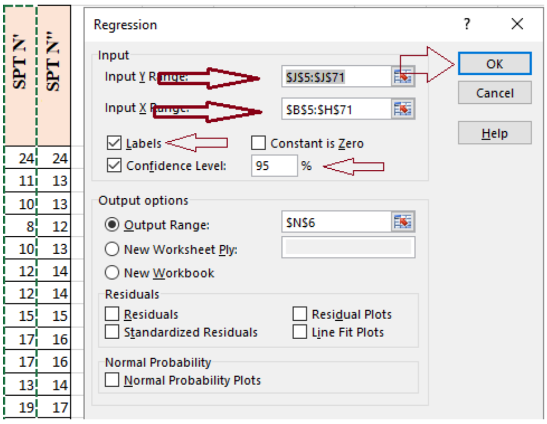


Figure 3. Regression analysis window for correlation between SPT and other properties

N' will be correction value for sand and N" for both clayey sand and sandy clay (cohesive soil in general).

This paper will assess the reliability of SPT in predicting the physical and mechanical properties of cohesive soil. With soil strength parameters (cohesion and internal friction angle) and physical properties related to SPT number, regression equation will be obtained and used in evaluating bearing capacity of soil foundation or driven pile.

Comparisons between three values :

a) of soil properties (physical parameters and soil strength) in terms of SPT numbers (this study).
b) of conventionally obtained lab tests.
c) of previous studies about correlations between soil properties (physical, mechanical properties and compressibility).

will be studied in order to choose the most appropriate values to use in foundation engineering.

3.2. Correlation equations

Some results were described in tables 4, 5 and equations of multi-variable regression are shown below:

Table 5. Regression statistics and variance analysis ANOVA [15]

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.4346
R Square	0.1889
Adjusted R Square	0.171
Standard Error	2.8338
Observations	233

ANOVA	*df*	*SS*	*MS*	*F*	*Significance F*
Regression	5	424.449	84.890	10.571	3.85969E-09
Residual	227	1822.933	8.031		
Total	232	2247.382			

	Coefficien	*andard Err*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*ower 95.0%*	*Upper 95.0%*
Intercept	78.629	14.587	5.390	1.8E-07	49.885	107.373	49.88504	107.3728407
Depth Z	-0.044	0.016	-2.659	0.00839	-0.076	-0.011	-0.07633	-0.011353392
Water content ω	-82.652	14.928	-5.537	8.5E-08	-112.066	-53.237	-112.066	-53.23732043
γk (kN/m3)	-2.810	0.761	-3.690	0.00028	-4.311	-1.310	-4.31058	-1.309613702
Plasticity index Ip	7.310	6.338	1.153	0.25002	-5.180	19.799	-5.17963	19.79866403
E	0.048	0.014	3.484	0.00059	0.021	0.075	0.020672	0.074502705

Results of multi variable linear regression for different kinds of soil (Sand, Clay, Sandy Clay and Clayed Sand) are tabulated in Table 5.

Table 6. Significance of analysis of variance (ANOVA) for sand, sandy clay/clayey sand, clay [15]

Type	**Regression Statistics**		**Analysis of variance (ANOVA)**	
	R^2	Adjusted R^2	F	Significance F
Sand (20 samples)	0.623	**0.456**	6.194	0.003
Clayey sand, sandy clay (185 samples)	0.363	**0.341**	16.905	2.07E-15
Clay (233 samples)	0.189	**0.171**	10.571	3.86E-9

Regression equation for sandy soil

$$\mathbf{N' = 224.9 - 0.13\,Z + 0.022\,\omega - 7.89\,\gamma_d - 139.67e + 0.02E} \quad (7)$$

where Z is the depth of sampling, other symbols are given in Table 2.

In multi variable linear regression analysis, R^2 adjusted is used instead of (R Squared). **R^2 adjusted = 0.456** (or R = 67.5%) indicated that the predictors or a few independent variables as prescribed only explained nearly 46 % dependent variables N'. It meant that more than 54 % was due to others uncertainties in measurement, errors in lab tests etc.

These properties of sand were **rather** reliable in predicting N'.

As for *silty sand*, results from observed 70 samples showed no correlation found.

Regression formula for clayey sand and sandy clay

$$\mathbf{N'' = 9.955 + 0.036Z - 0.026\,\omega + 0.813\gamma_d + 0.098I_p + 0.009E} \quad (8)$$

R^2 (**R Squared adjusted**) = 0.341 (slightly smaller as that of Sand with R Squared adjusted = 0.456) indicated that the predictors or a few independent variables as prescribed only explained nearly 34 % dependent variables N'. It meant that more than 66 % was due to others uncertainties in measurement, errors in lab tests etc. These properties were **relatively** reliable in predicting N".

Regression formula for clay

$$\mathbf{N'' = 78.629 - 0.044Z - 82.65\,\omega - 2.81\,\gamma_d + 7.31I_p + 0.048\,E} \quad (9)$$

R^2 (**R Squared adjusted**) = 0.171 indicated that the predictors or a few independent variables as prescribed only explained nearly 17 % dependent variables N'. It meant that more than 83 % was due to others uncertainties in measurement, errors in lab tests etc.

These properties were **weakly** reliable in predicting N". Based on soil data and corrected numbers of SPT data, by conductiong multi-variable regression analysis, some results are:

- Sand : relatively usable
- Sandy Clay/ Clayey Sand**:** tentatively usable

- Clay: tentatively usable with remarkable caution.
- Based on R^2 adjusted (R^2 adjusted = 0.456 for sand and = 0.341 for clayey sand and sandy clay; for clay, R^2 adjusted = 0.171) SPT is reliable for sand and clayey sand, and *weakly reliable for clay* in a multi variable regression model.
- Although the R squared was relatively small, but strongly related to each other, expressed in very small value Significant F in the most right column of Table 7.

Table 7. Regression statistics and variance analysis ANOVA [15]

Regression Statistics	
Multiple R	0.626757
R Square	0.392824
Adjusted R Square	0.294892
Standard Error	3.958149
Observations	37

ANOVA					
	df	*SS*	*MS*	*F*	*Significan*
Regression	5	314.2166	62.84331	4.0112[illegible]4	0.006372
Residual	31	485.6753	15.66695		
Total	36	799.8919			

- For sand moisture, dried density, initial void ration and overburden pressure (depth of samplings) affected most to corrected SPT numbers instead of recording data without correction.
- For clay, depth of sampling, plasticity index did not affect SPT both N' and N". Cohesion and modulus of deformation had a slight effects on N". This might be unclear.
- For cohesive soil, depth of sampling affected most significant the corrected SPT numbers. Hence, correction was obviously necessary.
- Corrected SPT numbers were applicable for both sand (N') and clay (N"), with different coefficient of determination.

3.3. Single-variable regression analysis for shear strength parameters of cohesive soil

At 95% confidence level: there were weak correlation between corrected SPT number N" and many different soil parameters altogether but it was a significant correlation between N" (or N') and E, C, φ.

Regression formulas for main parameters of soil strength and compressibility for different soil are in Table 7.

Table 8. Regression formulas for converting N-values to soil properties (units in SI, i.e. kPa and degree) [15]

Soil type		Confidence level	
		85 %	95 %
Sand	E	E=111.5+1.826N'	E=118.8+2.431N'
	φ	φ =32.6°-0.01N'	φ =32.93°-.02°N'
	E	E=57.67+2.38N''	E=61.63+2.37N''
Clayey sand/ Sandy Clay	C	C=0.134+0.001N''	C=0.141+0.002N''
	φ	φ =19.81+0.52N''	φ =20.33+0.56N''
	E	E=52.6+1.7N''	E=56.7+1.939 N''
Clay	C	C=0.367+0.02N''	C=0.407+0.022N''
	φ	φ =15.89+0.36N''	φ =16.5+0.39N''
	E	E=52.6+1.7N''	E=56.7+1.939N''

Results obtained from abovementioned regression analysis can be compared to those of previous works conducted by Mostafa Abdou Abdel Naiem Mahmoud (2013) [3] in which shear strength of silty clay with sand soil can be calculated as following equations:

φ (in degree) = 0.209N" + 19,68

c (in kG force/cm^2) = 0.014 N" – 0.18

where N" is the corrected SPT numbers, E in kPa.

4. Approach for determining bearing capacity as per local code of practice TCVN 10304:2014 [16]

Mean values of corrected SPT N-values will be estimated in sub-layers which should be smaller than scale of fluctuation θ (Fig. 5). Soil properties within scale of fluctuation are chosen by using converted values from corrected SPT numbers.

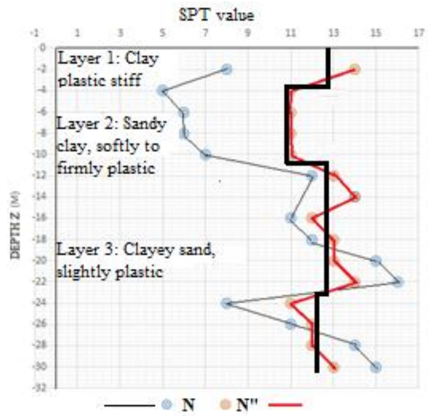
4.1. Bearing capacity

4.1.1. Soil profile

Soil profile are shown in Fig 5. A designed pile will be installed through 5 soil layers. Clayey soil is dominant. Average depth is from formula (9),

$$\bar{d} = \frac{5.7+1.9+7.5+6+8}{5} = 5.82m`$$

Scale of fluctuation was taken θ = 0.8$\bar{d}$ =4.65 m. Pile bearing capacity can be calculated appropriately by using soil properties within a distance $d_i < \theta$.



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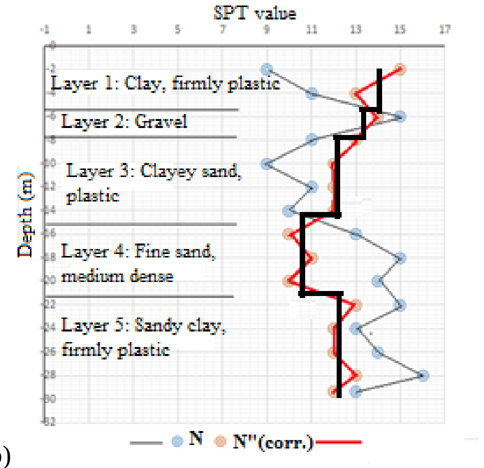

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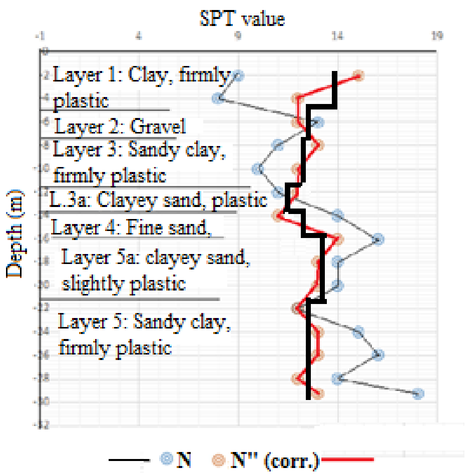

c)

Figure 4. Inconsistent trend of corrected SPT. a) Borehole BH1 (all N");b) Borehole BH1 (some N', other N"); c) Borehole BH2, all corrected N" [15]

In general, ultimate bearing capacity is calculated by formula as below:

$$Q_s = q_b A_b + u \sum (f_{c,i} l_{c,i} + f_{s,i} l_{s,i}) \qquad (10)$$

where, q_b is point bearing resistance in kPa, A_b u are cross section of pile tip and perimeter of pile section, respectively. $f_{c.i}$ and $l_{c.i}$ are skin friction and length of ith pile segment penetrating in clay, respectivley; and the second terms in paratheses of (10) is for sand ith layers.

4.1.2. Meyerhoff's formula

This formula has still been used popularly in Vietnam as an alternative for comparison purpose [16].

Soil strength will be used to compute shaft pile friction and point bearing.

For pile installing into sandy soil,

$$f_{s,i} = K_s . \overline{\sigma_{vo}}' . \tan \delta \qquad (11a)$$

$$q_b = \overline{\sigma_v}' N_q + c.N_c \qquad (11b)$$

where N_q N_c are bearing capacity factor, dependent on friction angle of soil; K_s is coefficient of lateral pressure, K_s =(1~1.2)(1-sinφ') for driven pile; tan(δ) is coefficient of friction between soil and pile shaft.

Appendix G of TCVN 10304:2014 [16] also recommended a Meyerhoff's formula of skin friction and point bearing resistance using SPT N-values, as below:

$$f_{s,i} = \frac{10.N_{s,i}}{3} \qquad (11c)$$

$$q_b = 300.N_p \qquad (11d)$$

For pile installing into cohesive soil

$$f_{c,i} = \alpha_p . f_L . c_{u,i} \qquad (12a)$$

$$q_b = 9.c_u \qquad (12b)$$

where α_p is factor applied for driven pile, depended on the ratio of undrained strength to average effective stress (in spreadsheet denoted a_p); f_L is a factor considering slenderness length h/diameter D of pile.

These abovementioned formulas were applied to driven or jacked-in pile. For considering spatial variability, each soil layer will be chosen as d_i as in Fig. 2. If scale of fluctuation is taken into account, friction and point bearing will be computed within that scale of fluctuation.

Table 9. Illustrated spreadsheet for computing abutment B pile bearing capacity using Meyerhoff's formula and corrected N', borehole BH-1 [15]

WITH CORRECTED N VALUES											8.4	m	
									8.4	BH-1	0.35	m	(D=0.35m)
No.	Soil layers	thick ness	γ	σ'$_{vo}$	ϕ	c	N-SPT	a_P	Cu,i	k_2	fi	Axq	Qsi
		m	T/m3	T/m2	(^{o})	T/m^2			KPa		T/m 2	m2	Tons (T)
1	Firmly plastic clay	2	1.98	1.98	16.8	0.24	7.75	1.00	8.39	2	0.84	2.8	2.35
2	Firmly plastic clay	2	1.98	5.94	16.8	0.24	13.97	1.00	20.31	2	2.03	2.8	5.69
3	Firmly plastic clay	1.7	1.98	9.60	16.8	0.24	13.44	1.00	31.33	2	3.13	2.4	7.46
3	Granular soil, dense	1.9	2.02	13.21	31.3	0	13.64	0.63	80.29	2	5.05	2.7	13.42
4	Plastic clayey sand	0.8	1.05	15.54	25.2	0.11	12.99	0.68	74.24	2	5.02	1.1	5.62
											Friction component (tons)		**34.53**
Point bearing component			N_p	k_1	q_b			A_b	Q_b				
					kPa			m2	tons				
			13.2	400	5260			0.12	64.44				**64.44**
											Total bearing capacity (tons)		**98.97**

For uncorrected N values, bearing capacity equals approximately to 85.86 tons with

- Friction component: 28.26 tons (-18.1 % as compared to that of using corrected N)
- Point bearing component: 57.28 tons (-11.1 % as compared to that of using corrected N)

4.1.3. Architectural Institute of Japan (AIJ, 1988)

Appendix G of National Standard TCVN 10304: 2014 [16] described steps to apply main contents of Recommendations for Design of Building Foundation (Architectural Institute of Japan issued in 1988, hereinafter denoted AIJ for short) in predicting bearing capacity of pile, both driven and bored piles. SPT data were used to indirectly compute friction $f_{s,i}$ and point bearing resistance q_b.

$$f_{s,i} = \frac{10.N_{s,i}}{3} \tag{13a}$$

$$q_b = 300.N_p \tag{13b}$$

where, $N_{s.i}$ is average number of SPT in i^{th} soil layer; N_p is average value of SPT blow counts taken within a zone 1D below pile tip level and 4D above level of pile tip. An Excel spreadsheet for computing bearing capacity was displayed as in Table 9 below:

Table 10. Illustrated spreadsheet for computing abutment B pile bearing capacity using recommendation of AIJ and corrected N', borehole BH-1[15]

WITH CORRECTED N VALUES									8.4	m	
							8.4	BH-1	0.35	m	(D=0.35m)
MST	Soil layers	thickness	γ	σ'_{vo}	SPT	a_P	f_L	$C_{u,i}$	fi	Axq	Qsi
		m	T/m^3	T/m^2				KPa	T/m^2	m^2	Tons
1	Firmly plastic clay	2	1.98	1.98	7.8	0.5	1	48.44	2.42	2.8	6.78
2	Firmly plastic clay	2	1.98	5.94	14.0	0.5	1	87.29	4.36	2.8	12.22
3	Firmly plastic clay	1.7	1.98	9.603	13.4	0.5	1	83.99	4.20	2.38	10.00
3	Granular soil, dense	1.9	2.02	13.21	13.6	0.7	1	85.26	0.45	2.66	1.21
4	Plastic clayey sand	0.8	1.05	15.54	13.0	0.8	1	81.21	6.56	1.12	7.35
Friction component (tons)											**37.56**
Point bearing component		Np	k1	q_b	A_b	Q_b					
				kPa	m2	tons					
		13.2	400	5260	0.12	8.82					**8.82**
Total bearing capacity (tons)											**46.38**

For uncorrected N blow counts, total bearing capacity equals approximately to 37.8 tons with

- Friction component: 30.5 tons (-18.8 % as compared to that of using corrected N)
- Point bearing component: 7.3 tons (-17.2 % as compared to that of using corrected N)

4.2. Numerical model (Plaxis 3D)

For comparison purpose, a finite element model using Plaxis 3D was studied. Mohr Coulomb (MC) soil behavior model was chosen because of its relevancy to the bearing capacity problem, partly because of limited data from soil reports (without results from triaxial compression tests). Data of soil properties input into software were converted from corrected SPT N-values, as abovementioned regression equations of table 7. Calibration for model was disregarded for accepting a linear proportion factor between measured bearing capacity and computed one.

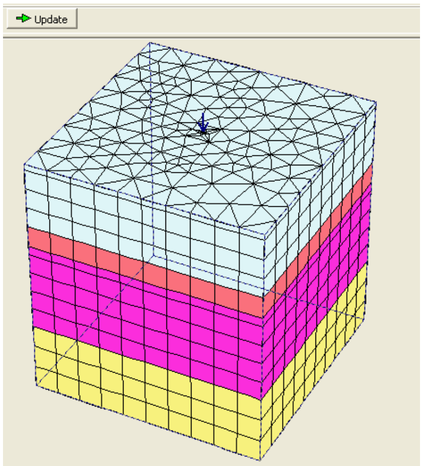

Figure 5. Plaxis model for determining bearing capacity of pile [15]

At-site determination of ultimate bearing capacity of pile was complied with item 7.3.2 of TCVN 9393: 2012 „Pile Foundation – Code for design building foundation and construction works“ [16] that admitted a settlement at failure as below:

$$S = \xi S_{gh} \tag{14}$$

where, S_{gh} is settlement at ultimate condition, taken as 40mm (item 7.3.2); ξ = 0.2. As such, ultimate bearing capacity will be the load at which pile settlement equals to 8 mm.

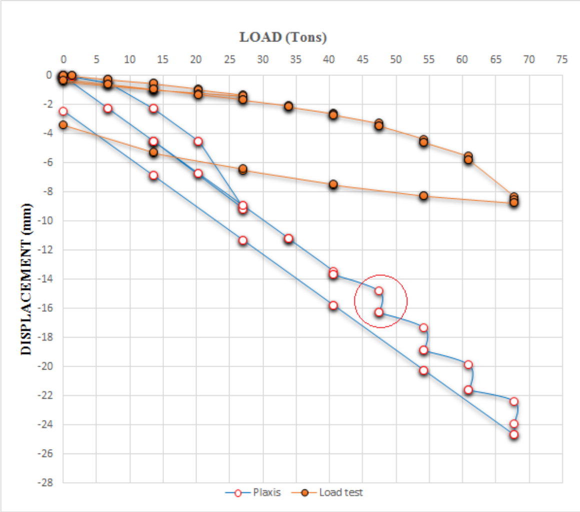

Figure 6. Ultimate bearing capacity of 12” square pile from static load test and Plaxis model [15]

Ultimate bearing capacity by static load test is 67.68 tons, while this value determined by yield point (big displacement at a constantly kept load) at P=47.37 tons (solid circle line in Fig. 6).

Three piles with different configuration were considered: For borehole BH1 with 5 layers: abutment B pile (square 35cm, L=8.4 m) and pile P58 (square 25cm pile, L=8.6 m). For borehole BH2, with 7 layers: pile P61 (square 30cm, L=12.2 m). Calculating spreadsheets are established as in Table 8, Table 9. Results are compared as in Fig. 8

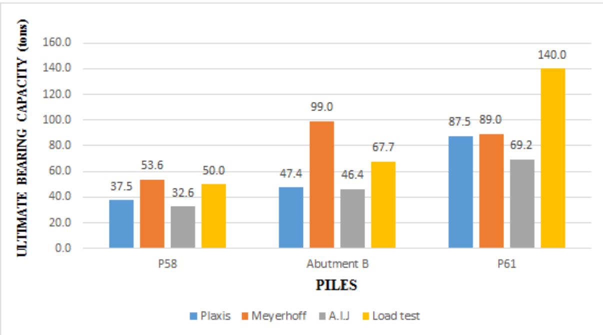

Figure 7. Comparisons between ultimate bearing capacity of pile using Meyerhoff’s formula, AIJ, Plaxis and Static Load Test [15]

4.3. Discussion

- Correlation equations in Table 5 should be studied within the scale of fluctuation for higher R^2 (adj.) instead of entire soil profile. But since thickness of soil layers was smaller than the scale of fluctuation θ, and R^2 (adj.) was very high, therefore, calculation for bearing capacity would be implemented with a sufficient and reasonable accuracy in practice.
- Regression equations for converting N-values into soil strength might have some errors. The most likely value may be computed by root mean square formula as follows:

$$C_{design} = \sqrt{(C_{labtest})^2 + (C_{convert})^2 + (C_{model})^2} \qquad (15)$$

The first term belongs to soil properties obtained by conventional lab tests; the second term refers to measurement or correlation with corrected N-values, and the third terms relates to formula of shaft friction and tip resistance (i.e. theoretical and analytical model) [12,14].

- Back analysis to calibrate the numerical model is necessarily conducted for obtaining the proper set of soil strength properties, unless a linear correlation in elastic domain is found.
- Load – displacement curve obtained by Plaxis indicated that soil foundation for the pile was still workable in elastic domain. Meanwhile, results from static load test showed a sharper trend in curvature, indicating an yielding point in bearing capacity of foundation.
- Bearing capacity computed by AIJ using directly corrected N-values proved to be close to that of static load test (Fig. 8). Furthermore, comparison on results of bearing capacity obtained by two approaches, (one from numerical finite element model _ Plaxis 3D, using converting data of soil properties from N-values_ and the other from static load test) pointed out that there was a linear correlation between them as in Fig.8 as following:

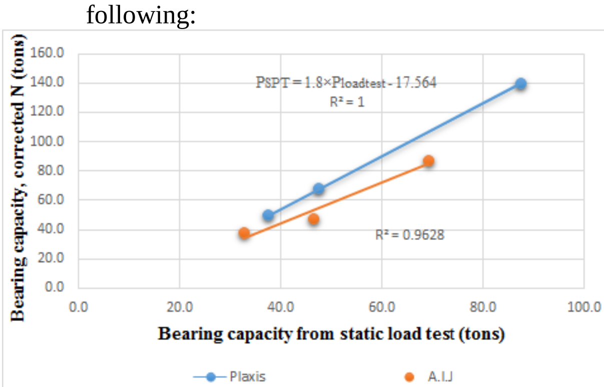

Figure 8. Results of bearing capacity obtained by Plaxis and A.I.J v/s by Static Load Test [15]

This may come to a suggestion that corrected N-values can be used tentatively in predicting bearing capacity of pile installing into cohesive soil at a specific sites; and AIJ formula using directly corrected N-values will be more predictable that other analytical approach.

- Single variable linear regression analysis at a level of confidence 95 % provides a set of converted parameters of soil strength_friction angle and cohesion_ which is possible to predict bearing capacity in a numerical model.
- For determining the reliability index of bearing capacity of a pile, it is necessary to collect much more data of load tests, both in permanent and imposed loads, in order to determine mean values, coefficient of variation, law of distribution and factors of uncertainties together with a performance function will be defined [7].

5. Conclusion

Reliability of SPT data for predicting bearing capacity of a pile can be asssessed by comparing the results from numerical model using corrected N-values to those of static load tests. Multi variable linear regression analysis showed a weak correlation between N-values and physical properties of cohesive soil and depth of testing, but single variable linear regression model showed a significant correlation of corrected N values to cohesive soil strength (i.e. friction angle and cohesion) with a level of confidence 95 %. Two locally used approaches (i.e. Meyerhoff's formula, Recommendation of Architectural Institute of Japan or A.I.J issued in 1988) were used in which soil strength were indirectly converted from corrected N-values and assigned as input data into finite element model; the results were compared to those of numerical model Plaxis 3D and of static load test. Results indicated that approach of the A.I.J using directly SPT data predicted a closer value of the bearing capacity as compared to that of static load test. Besides, the numerical model Plaxis 3D using indirect SPT data (i.e. model converted SPT data to soil strength and compressibility) pointed out a value of bearing capacity which was highly linear correlation to reliable result of static load test. These results could help practitioners in estimating bearing capacity with SPT data with a satisfactory accuracy.

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