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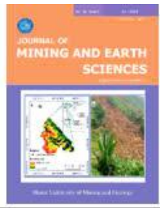

The initial research results on the improvement of offshore axial pile capacity calculation using CPTu data at block A, offshore Vietnam

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ABSTRACT

This paper presents the use of CPTu data in calculation of offshore axial pile capacity based on the CPT-based methods recommended in Appendix C of API RP 2GEO (2014) which are the Simplified ICP-05 Method (Method 1); Offshore UWA-05 Method (Method 2); Fugro-05 Method (Method 3); and NGI-05 Method (Method 4). Formerly, the strength values of cohesionless layers were usually taken from onshore laboratory results or if there were none done, the values were generally assumed based on recommendations of API RP 2A-WSD (2000) standard which often resulted in unconservative axial pile capacity. With the availability of CPTu data which better reflects the in-situ soil conditions, the selection of design parameters for cohesionless layers is improved and consequently, the improved axial capacity calculation both in tension and compression. The improvement in predicted axial pile capacity allows better pile design (for example, choice of pile length, optimization of pile diameter) and hence the economical aspects of the final design at a later design stage. Results from the analyses by CPT-based methods indicate that Method 1, 2 and 3 produce comparable results of capacities in both compression and tension, whereas Method 4 shows somewhat a deviation from the other 3 methods. Hence, a conservative approach to using the capacities calculated by method 4 and especially the API RP 2A-WSD (2000) method should be exercised properly by applying appropriate safety factors.

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1. Introduction

The prediction of axial pile capacity is a critical component in offshore foundation design, particularly for structures such as wind turbines, oil platforms, and other marine infrastructure. Offshore soil conditions can vary significantly due to the complex interaction of geological history, climate change, and human activities, making accurate prediction methods essential for both design and economic efficiency. Traditionally, methods such as the API RP 2A-WSD (2000) have been used to estimate axial capacity. However, many studies have shown that these methods often overestimate capacities, leading to unconservative pile designs (Kolk & Der Velde, 1996). These discrepancies have driven the adoption of more advanced, data-driven methods.

In recent years, the use of the Cone Penetration Test with pore pressure measurements (CPTu) has gained prominence, providing better in-situ soil characterization, especially for cohesionless soils, than older laboratory-based methods. Studies by Mayne (2007) have demonstrated that CPT data offers more accurate predictions of pile behavior under both tensile and compressive loads, resulting in improved design parameters for axial pile capacity calculations. CPT-based methods such as those recommended by API RP 2GEO (2014) have been widely accepted in the offshore engineering community. These methods provide more accurate capacity estimates, reducing the need for conservative safety factors, as they are based on direct correlations of soil resistance with CPT measurements.

Recent advancements in machine learning have also contributed to improving pile capacity predictions. Nguyen et al. (2024) demonstrated that machine learning techniques could be used to enhance the prediction of base resistance in long piles, particularly in soft soils, by leveraging extensive datasets from field tests. Their study shows how settlement influences the base resistance of long piles and provides more accurate assessments of pile behavior compared to conventional empirical methods. This study applies CPT-based methods to Block A offshore Vietnam, integrating data from both boreholes and CPTu measurements to derive more accurate design parameters. By leveraging these advanced methods, the study aims to improve the prediction of axial pile capacity for offshore structures, offering potential optimizations in terms of pile length and diameter, thus reducing construction costs without compromising safety.

2. Project description and theory of pile capacity's calculation method

2.1. Background information

In 2014, a geotechnical site investigation was performed to extract soil conditions for the development of the proposed ST-PIP (Song Tinh - Production Injection Platform) and ST-LQ (Song Tinh - Living Quarter) platforms. The fieldwork at the location comprised one 140.0-m Sample borehole and one 140.0-m CPT hole, designated as ST-LQ and ST-PIP respectively. These boreholes are about 148 m apart. The data obtained from both holes are combined in order to generate hypothetical soil parameters for the purpose of pile foundation design. The hypothetically combined location would be referred to as ST-LQ/PIP Location. The engineering analyses are for 60-in. dia. pipe piles.

The water depth of the final investigated locations is extracted from the survey and presented in Table 1 below.

2.2. Soil properties and stratigraphy

The soil conditions as revealed at the ST-PIP/LQ Location indicate that they consist of alternating granular and cohesive materials from mudline to the final investigated depth of 140 m.

All the soil samples collected offshore were transferred to an onshore laboratory for testing soil properties such as grain size analyses, Atterberg limit analyses, undrained unconsolidated triaxial tests, consolidation tests,

Table 1. Water depths of referenced locations.

Borehole Designation	Observed Water Depth [m]	Final Investigated /Pile Driven Depth [m]
BH ST-LQ (Sample)	55.0	140
CPTU ST-PIP	55.2	140

carbonate content tests and consolidated drained triaxial tests.

From the results of onshore laboratory tests, the clays are generally of low plasticity with Liquid Limits (w_L's) of 23 to 46% and stiff to very stiff in consistency.

The sand/silty sand layers are generally inferred to be medium dense to dense in relative density based on CPT data at various depths. Silt/silt with sand/sandy silt layers are encountered from about 40.6 to 43.6 m, 56.8 to 64.0 m and 90 to 93.6 m. Coralline gravel and siliceous carbonate sand layers are observed from about 9.6 to 13.4 m and 13.4 to 15.2 m respectively with carbonate content ranging between 71% and 93%. A silty gravel with sand layer is present from 37.6 to 40.6 m.

The strength data together with all the other available laboratory classification test data are used to determine the hypothetical soil stratigraphy. The hypothetical stratigraphy at the location is presented in Table 2.

Table 2. Hypothetical Soil Stratigraphy – ST-LQ/PIP Location (Field report at Block A, offshore Vietnam).

No.	Depth (m)	Inferred Description
1	0÷1.6	Loose silty fine SAND
2	1.6÷3.7	Medium dense to dense silty fine SAND
3	3.7÷6	Very dense silty fine SAND
4	6÷7	Medium dense silty fine SAND
5	7÷7.6	Stiff lean CLAY
6	7.6÷9.6	Medium dense to dense silty fine SAND
7	9.6÷13.4	Silty CORRALLINE GRAVEL with sand
8	13.4÷15.2	Silty siliceous carbonate fine SAND
9	15.2÷24	Stiff to very stiff lean CLAY
10	24÷28.4	Medium dense silty fine SAND
11	28.4÷37.6	Very stiff to hard lean CLAY
12	37.6÷40.6	Medium dense silty GRAVEL with sand
13	40.6÷43.6	SILT with sand
14	43.6÷47.5	Medium dense to dense fine SAND
15	47.5÷50.8	Very stiff CLAY
16	50.8÷56.8	Very stiff to hard lean CLAY
17	56.8÷64	Sandy SILT
18	64÷73.8	Stratified silty fine SAND and very stiff CLAY
19	73.8÷90	Very stiff lean CLAY
20	90÷93.6	SILT with sand
21	93.6÷99	Very stiff lean CLAY
22	99÷120.4	Dense silty fine to medium SAND
23	120.4÷122	Stratified very stiff lean CLAY and silty SAND
24	122÷125	Medium dense to dense silty fine to medium SAND
25	125÷130	Hard lean CLAY
26	130÷133	Medium dense calcareous silty fine SAND
27	133÷135.5	Very stiff CLAY
28	135.5÷138.2	Medium dense silty fine SAND
29	138.2÷140	Stratified silty fine SAND and very stiff to hard CLAY

2.3. Interpretation of piezocone penetration test (CPTu) data

Cone penetrometer test results were used to:

- Interpret material properties;
- Determine stratigraphy and soil conditions;
- Select appropriate parameters for granular soils and inferred undrained shear strength for clays.

The cone resistance and pore pressure data obtained during this study were used to interpret undrained shear strength of cohesive soils and to estimate the relative densities of granular soils. Sleeve friction data, also presented as friction ratio (defined as the ratio of sleeve friction to point resistance expressed as a percent), as well as the measured excess pore pressure, were used to assess soil characteristics.

2.3.1. Data for interpretation of CPTu data

The ratio of sleeve to cone tip resistance and the excess pore pressure readings generally supports the clay and sand/silt classification, and readily identifies the stratified layers of sand and silt soils.

2.3.2. Estimate of shear strength in cohesive soils

Cone penetrometer results can be used to estimate shear strength for cohesive materials. CPTu results were correlated with the undrained shear strength measured in laboratory tests using the equation given as follows Schmertmann (1975):

$$c_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{vo}}{N_{kt}} \quad (1)$$

Where: c_u - undrained shear strength; q_t - corrected CPTu tip resistance; q_{net} - net cone resistance; σ_{vo} - total overburden pressure, (including hydrostatic); and N_{kt} = cone factor.

As discussed by Schmertmann (1975), the value of N_{kt} depends on many variables such as:

- Method of determining the reference undrained shear strength;
- In-situ soil stress condition;
- Stress history;
- Soil structure;
- Sensitivity;
- Plasticity characteristics;
- Type of penetrometer cone;
- Mode of CPTu operation and rate of penetration.

The N_{kt} values of 15 and 20 are recommended to be used to a depth of 24 m. For the cohesive materials below 24 m, the N_{kt} values adopted are 20 and 25.

3. Axial pile capacity calculation

3.1. Analysis method for axial pile capacity

Table 3 summarizes the analysis methods for axial pile capacity used in this article.

Table 3. Analysis methods for axial pile capacity.

Method	Cohesive Soil Model	Frictional Model
-	Kolk & Van der Velde (1996)	CPT-based Methods*

* CPT-based Methods are based on the Simplified ICP-05, Offshore UWA-05, Fugro-05 and NGI-05 methods as per API RP 2GEO (2014) Annex C. The hypothetical design soil parameters are tabulated in Table 4.

3.2. Ultimate axial pile capacity of driven piles

Analyses of axial pile capacity were performed using the procedures described in the Kolk & Der Velde (1996) Method for the cohesive layers.

Table 4. Parameters for axial pile capacity model – CPT-based methods.

Depth from to [m]	Ground unit name	Ground unit behaviour	UW [kN/m³]	q_c [MPa]	c_u [kPa]	Delta [deg]
0.0 1.6	Sand	Frictional	20.3 20.3	0.8 1.2	-	27.0
1.6 3.7	Sand	Frictional	20.5 20.5	4.0 4.0	-	28.8
3.7 6.0	Sand	Frictional	19.5 19.5	12.0 12.0	-	28.8
6.0 7.0	Sand	Frictional	19.2 19.2	4.0 4.0	-	28.8
7.0 7.6	Clay	Cohesive	19.4 19.4	-	50.0 50.0	-
7.6 8.3	Sand	Frictional	19.7 19.7	18.0 18.0	-	28.8
8.3 9.6	Sand	Frictional	19.7 19.7	10.5 10.5	-	28.8
9.6 13.4	Coral	Frictional	8.8 18.8	6.0 6.0	-	23.3
13.4 15.2	Sand	Frictional	19.1 19.1	4.0 4.0	-	28.8
15.2 17.3	Clay	Cohesive	19.3 19.3	-	120.0 80.0	-
17.3 20.0	Clay	Cohesive	19.3 19.3	-	80.0 120.0	-
20.0 22.8	Clay	Cohesive	19.3 19.3	-	120.0 50.0	-
22.8 24.0	Clay	Cohesive	19.3 19.3	-	50.0 140.0	-
24.0 28.4	Sand	Frictional	21.0 21.0	8.0 8.0	-	28.8
28.4 34.2	Clay	Cohesive	20.0 20.0	-	110.0 110.0	-
34.2 37.6	Clay	Cohesive	20.0 20.0	-	110.0 200.0	-
37.6 40.6	Gravel	Frictional	19.9 19.9	8.0 8.0	-	22.8
40.6 43.6	Silt	Frictional	20.2 20.2	5.5 5.5	-	28.8
43.6 47.5	Sand	Frictional	20.0 20.0	13.0 13.0	-	28.8
47.5 50.8	Clay	Cohesive	19.4 19.4	-	105.0 105.0	-
50.8 56.8	Clay	Cohesive	20.0 20.0	-	120.0 170.0	-
56.8 64.0	Silt	Frictional	20.4 20.4	12.0 12.0	-	28.8
64.0 73.8	Sand	Frictional	19.9 19.9	6.0 6.0	-	28.8
73.8 90.0	Clay	Cohesive	19.7 19.7	-	125.0 125.0	-
90.0 93.6	Silt	Frictional	19.0 19.0	10.0 10.0	-	28.8
93.6	Clay	Cohesive	19.8	-	130.0	-

Depth from to [m]	Ground unit name	Ground unit behaviour	UW [kN/m³]	q_c [MPa]	c_u [kPa]	Delta [deg]
96.8			19.8		130.0	
96.8 99.0	Clay	Cohesive	19.8 19.8	-	130.0 170.0	-
99.0 101.7	Sand	Frictional	20.1 20.1	37.0 37.0	-	25.6
101.7 103.5	Sand	Frictional	20.1 20.1	12.0 12.0	-	25.6
103.5 105.3	Sand	Frictional	20.1 20.1	37.0 37.0	-	25.6
105.3 120.4	Sand	Frictional	20.1 20.1	42.0 42.0	-	25.6
120.4 122.0	Sand	Frictional	20.0 20.0	14.0 14.0	-	26.6
122.0 123.0	Sand	Frictional	20.8 20.8	30.0 30.0	-	26.6
123.0 125.0	Sand	Frictional	20.8 20.8	36.0 36.0	-	26.6
125.0 127.4	Clay	Cohesive	19.6 19.6	-	200.0 200.0	-
127.4 130.0	Clay	Cohesive	19.6 19.6	-	200.0 240.0	-
130.0 133.0	Sand	Frictional	20.7 20.7	21.0 21.0	-	28.8
133.0 135.5	Clay	Cohesive	20.5 20.5	-	150.0 150.0	-
135.5 137.0	Sand	Frictional	20.4 20.4	11.0 11.0	-	28.8
137.0 138.2	Sand	Frictional	20.4 20.4	26.0 26.0	-	28.8
138.2 140.0	Sand	Frictional	19.8 19.8	8.0 8.0	-	28.8

The CPT-based Methods are used to compute axial capacities in the frictional layers.

The four recommended CPT-based methods in API RP 2GEO (2014) Annex C are listed as follows:

- Simplified ICP-05 Method (Method 1);
- Offshore UWA-05 Method (Method 2);
- Fugro-05 Method (Method 3);
- NGI-05 Method (Method 4).

Ultimate axial pile capacity curves of the proposed 60-in. dia. pipe pile under compression and tensile loading generated are presented in Figures 2 and 3.

As per the Client's instruction, the target depths 60-in. dia. pile are 110 m. The pile weight has not been taken into account in pile capacity curves.

Friction and end-bearing contributions to pile capacity are assumed to be uncoupled. Hence, for all methods, the ultimate axial pile capacity in compression, Q_c, and in tension, Q_t, of plugged open-ended piles is determined by Equations (2) and (3).

$$Q_c = Q_{f,c} + Q_p = \pi D \int f_c(z)dz + qA_p \quad (2)$$

$$Q_t = Q_{f,t} = \pi D \int f_t(z)dz \quad (3)$$

Where: Q_c - the axial pile ultimate capacity in compression, in force units; Q_t - the axial pile ultimate capacity in tension, in force units; $Q_{f,c}$ - the shaft friction capacity in compression, in force units; $Q_{f,t}$ - the shaft friction capacity in tension, in force units; Q_p - the end bearing capacity, in force units; $f_{c(z)}$ - the unit shaft friction in compression, in stress units, which is a function of depth, geometry and soil conditions; $f_t(z)$ - the unit shaft friction in tension, in stress units, which is a function of depth, geometry and soil conditions; z - the depth below the original seafloor; q - the unit end bearing at the pile tip, in stress units; D - the pile outside diameter; A_p - the gross end area of the pile, $A_p = \pi D^2/4$.

The unit shaft friction formulae for open-ended steel pipe piles for CPT-based methods 1, 2, and 3 can all be considered as special cases of the general formula:

$$f(z) = uq_c(z)\left(\frac{p'_o(z)}{p_a}\right)^a \times A_r{}^b\left[max\left(\frac{L-z}{D}, v\right)\right]^{-c} \times [tan\delta_v]^d \times \left[min\left(\frac{L-z}{D} \times \frac{1}{v}, 1\right)\right]^e \quad (4)$$

Where: $f(z)$ - the unit shaft friction, in stress units, which is a function of depth, geometry and soil conditions; $q_c(z)$ - the CPT cone-tip resistance at depth, z, in stress units; $p'_o(z)$ - the effective vertical stress at depth z; p_a - the atmospheric pressure, in stress units, (e.g. p_a = 100 kPa); A_r - the pile displacement ratio; L - the embedded length of the pile below the original seafloor; δ_{cv} - the sand constant volume friction angle at the interface between the sand and the pile wall.

The values of a, b, c, d, e, u and v are unit shaft friction parameter values for driven open-ended steel piles as described in API RP 2GEO (2014) Annex C.

Ultimate unit shaft friction values for tension, $f_t(z)$, and compression, $f_c(z)$, for driven open-ended steel pipe piles in Method 4 are as below:

$$f_t(z) = (z/L)p_a F_{sig} F_{Dr} \quad (5)$$

$$f_c(z) = 1.3(z/L)p_a F_{sig} F_{Dr} \quad (6)$$

Where: $F_{sig} = (p'_o(z)/p_a)^{0.25}$; $F_{Dr} = 2.1(D_r - 0.1)^{1.7}$.

The formulae for calculation of unit end bearing are as presented in (7), (8), (9) and (10).

For method 1:

$$q = q_{c,av,1.5D}\left[0.5 - 0.25\, log_{10}\frac{D}{D_{CPT}}\right] \quad (7)$$

For method 2:

$$q = q_{c,av,1.5D}(0.15 + 0.45A_r) \quad (8)$$

For method 3:

$$q = 8.5p_a\left(\frac{q_{c,av,1.5D}}{p_a}\right)^{0.5} {A_r}^{0.25} \quad (9)$$

For method 4:

$$q = \frac{0.7q_{c,av,1.5D}}{1 + 3{D_r}^2} \quad (10)$$

Where: $q_{c,av,1,5D}$ - the average value of $q_c(z)$ between 1.5D above the pile tip and 1.5D below the pile tip; D_{CPT} - the diameter of the CPT tool = 36 mm for a standard cone with a base area of 10 cm^2. D_r - the relative density of the sand ($0 \leq D_r \leq 1.0$).

3.3. Selection of target penetration depth

It is recommended that pile penetrations be selected using appropriate factors of safety or pile resistance factors. For working stress design (WSD), API RP 2A (2000) recommends that pile penetrations be selected to provide a factor of safety of at least 2.0 with respect to normal operating loads and at least 1.5 with respect to maximum storm loads. These factors of safety should be applied to the design of compressive and tensile loads.

3.4. Charts & graphs

Cone resistance (q_c) versus depth below mudline is presented in Figure 1.

Pile capacity curves calculated from different methods are presented in Figure 2 and Figure 3.

3.5. Results of analyses and discussion

The results of the analyses for axial pile capacity in compression/tension of 60in. dia. pile from different methods are presented in Table 5 below.

The calculated capacities in compression from the NGI-05 method and API RP 2A-WSD (2000) are larger than those of the Simplified ICP-05 Method, Offshore UWA-05 Method and Fugro-05 Method.

In addition, the calculated capacity from the NGI-05 method is smaller than that of API RP 2A-WSD (2000) and should be used with caution by

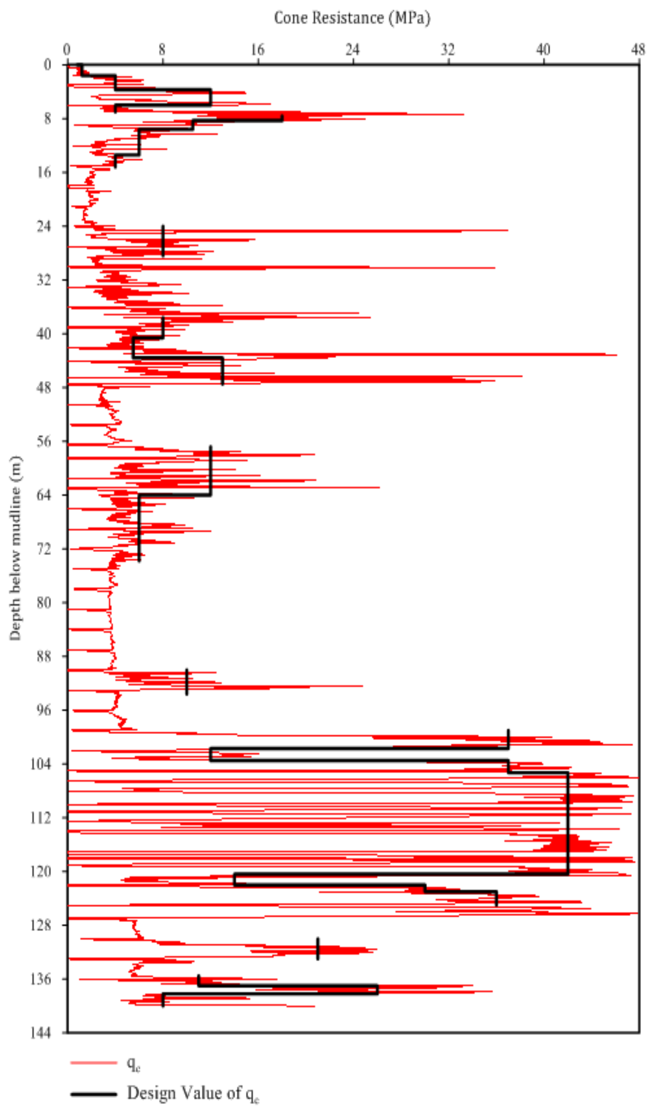

Figure 1. Cone resistance versus depth at survey location.

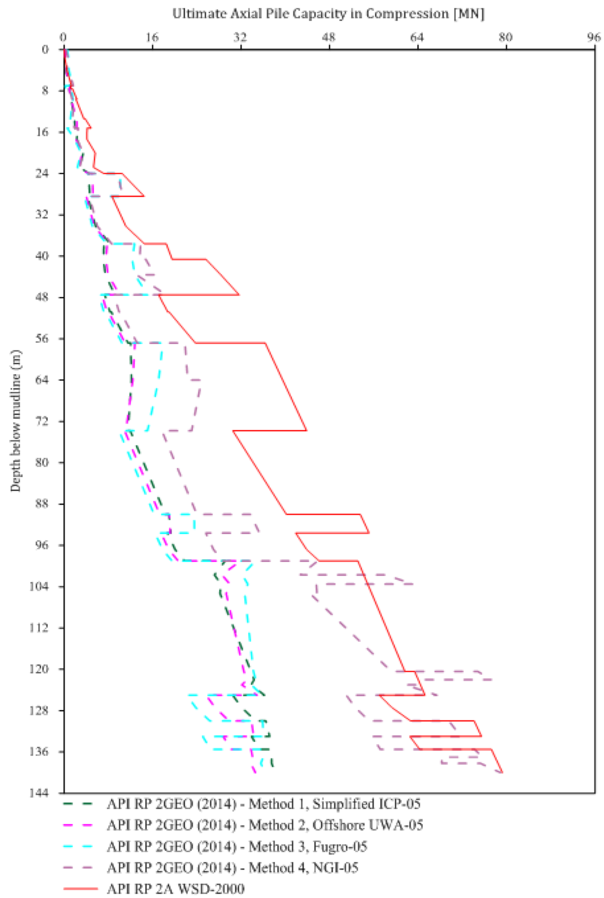

Figure 2. Ultimate axial pile capacity in compression from different methods.

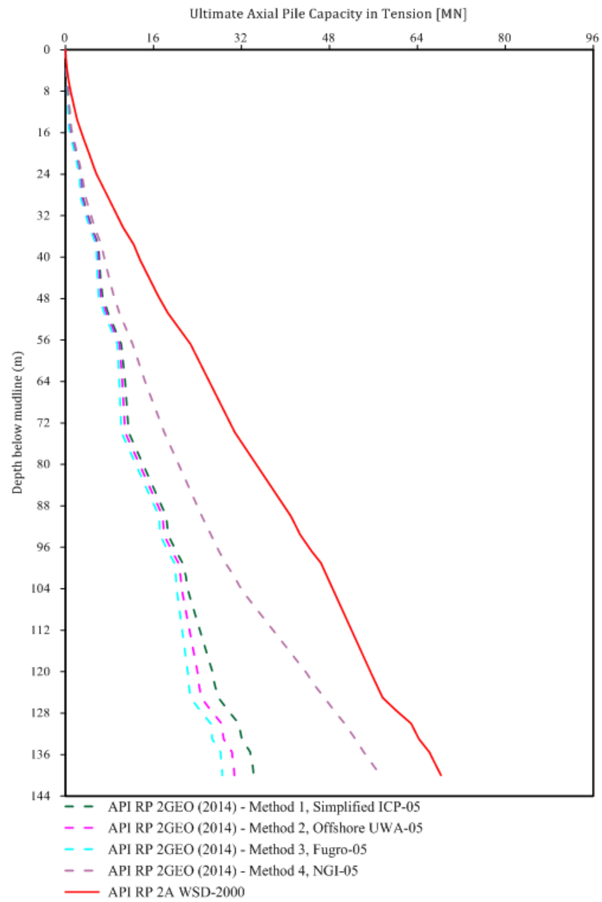

Figure 3. Ultimate axial pile capacity in tension from different methods.

Table 5. Comparison of axial pile capacity in compression / tension from different methods at target depth of 110 m below seafloor.

No.	Method	Axial capacity in compression (MN)	Axial capacity in tension (MN)
1	Simplified ICP-05 Method	29.9	23.9
2	Offshore UWA-05 Method	30.3	22.2
3	Fugro-05 Method	33.1	21.0
4	NGI-05 Method	49.0	36.3
5	API RP 2A-WSD (2000)	57.5	51.2

applying a higher safety factor for this particular method. Moreover, the API RP 2A-WSD method itself is unconservative when used to predict axial capacity in compression and tension due to the lack of in-situ tests to better derive soil parameters for calculation. Instead, soil parameters are basically derived from conventional tests performed onshore which can only replicate a portion of the whole behavior of in-situ soils.

The Simplified ICP-05 Method, Offshore UWA-05 Method and Fugro-05 Method show comparable calculated axial capacities both in compression and tension due to the fact that they all use generally the same formula (formula 4) for calculating the unit shaft friction.

4. Conclusion

This paper presents the initial research of axial pile capacity calculation by introducing the CPT-based methods recommended by API RP

2GEO (2014). It is found that the methods presented show comparable results of pile capacity both in tension and compression except for methods of NGI-05 and API RP 2A-WSD (2000). The calculated results from these two methods should be considered conservatively in real application by introducing a higher safety factor. Despite the limitations of the NGI-05 method of CPT-based methods, CPT-based methods for predicting pile capacity are recent and more reliable than API RP 2A-WSD (2000). These methods are all based on direct correlations of pile unit friction and end-bearing data with cone tip resistance values from cone penetration tests (CPT). These CPT-based methods also cover a wider range of cohesionless soils, are considered fundamentally better and have shown statistically closer predictions of pile load test results. However, offshore experience with these CPT methods is limited and hence more experience is needed before they are recommended for routine design, instead of the main text method. CPT-based methods should be applied only by qualified engineers who are experienced in the interpretation of CPT data and understand the limitations and reliability of these methods because sometimes the measured parameters such as cone tip resistance or sleeve friction may be operator-dependent and could not reflect the actual ground conditions.

For an effective improvement using these CPT-based methods, the calculated results should be combined and considered collectively among the proposed methods to derive the best-estimated capacity for the determination of an economical pile design based on the characteristics of in-situ sub-soil conditions.

The analyses of axial capacity of a 60-in diameter open pile at Block A are limited to the survey location with no available comparison to the research results from other areas and in the world. Hence, further research on this subject should be oriented towards the verification of these CPT-based methods.

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Contributions of author

Can Thanh Truong - methodology, data analysis, and drafting of Sections 1, 2, and 3, review - editing; Quyen Van Le - contributed to data interpretation, drafted Sections 3 and 4, review - editing; Long Kim Le - drafted sections 2.1 and 2.2, prepared charts/graphs, review - editing.

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