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Axial Bearing Capacity of Driven Piles in Accordance with API and DNV

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Abstract
Piles are fundamental part of most offshore structures. Thus assessment of pile capacity is critical for the design and installation of offshore structures. Currently, there are various codes that provide guideline on the pile capacity assessment. The most common ones are API and DNV. Each code provides slightly different design approach and utilises different safety factors. Thus, it is often not easy to directly compare the pile designs of different code. Furthermore, most appropriate design methodology is often chosen based on the available input parameters. i.e. geotechnical properties or CPT results. For a single design case, adapting different codes can result in different pile length requirements. This difference in pile length requirement is purely due to different methodology and associated safety factors used in codes. This paper aims to provide an overview of all common pile design methodologies from different codes and present a comparison of design pile lengths.

1. Introduction

Assessment of axial bearing capacity of pile varies in different codes in terms of methodology and safety factors. As a result, the outcome of the pile length assessment differs from one code to another. Nevertheless, axial bearing capacity of the pile is a single value perhaps has an offset from the results obtained from bearing capacity assessment based on various methods outlined in different codes. The objective of this paper is to present the variation of pile length for a single compressive load based on methodologies presented in the codes above.

2. Methodology

API and DNV codes describe slightly different approaches to assess the axial bearing capacity of a pile. These codes provide guidline for the calculation of pile length in common soil conditions such as clay (cohesive) or sand (cohesionless). The assessment also depends on the type of soil information available i.e. laboratory test results showing soil properties such as undrained shear strength and friction angle or the *in situ* Cone Penetration Test (CPT) data from the field tests. Thus, the suitable design approach is chosen based on the available soil data as shown in Figure 1.

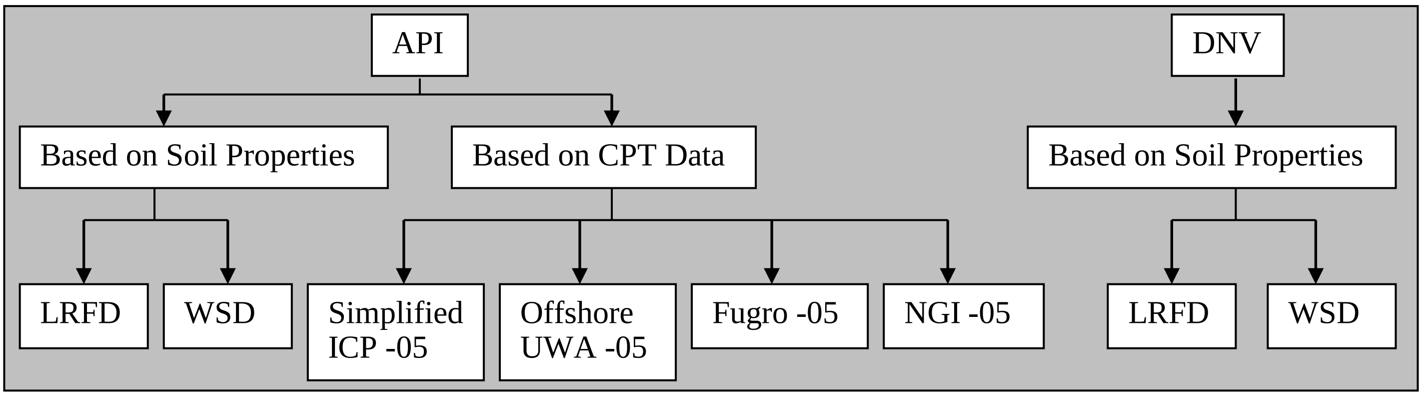


Figure 1 Design approaches accoding to API and DNV codes

2.1 API:

2.1.1 Working Stress Design (WSD) Method

Upon availability of soil properties such as undrained shear strength (S_u) or friction angle (ϕ'), API 2A-WSD, Ref [1], presents the following methodology for pile capacity assessment:

In cohesive soils, unit skin friction (f) can be assessed by $f = \alpha c$. Where α is a dimensionless factor and c is the undrained shear strength of the soil at the point in question.

α can be computed by

$$\alpha = 0.5\psi^{-0.5} \qquad \psi \le 1.0$$

$$\alpha = 0.5\psi^{-0.25} \qquad \psi > 1.0$$

With the constraint that, $\alpha < 1.0$, where $\psi = c / p_0'$ for the point in question

p_0' is effective overburden pressure at the point in question.

In cohesionless soil, unit skin friction (f) can be computed by $f = \beta p_0'$. Where β is dimensionless skin friction factor and p_0' is the effective overburden pressure at the depth in question. β values for open-ended piles driven unplugged are given in Table 1.

Unit end bearing (q) is assessed by $q = 9c$ for cohesive soils and it is assessed by $q = N_q p_0'$ in cohesionless soils. Where, N_q is dimensionless bearing capacity factor (From Table 1) and p_0' is effective overburden pressure at the depth in question.

Table 1 Design parameters for cohesionless soil, API, Ref [1]

Relative density	Soil description	Shaft friction factor β	Limiting shaft friction values (kPa)	End bearing factor N_q	Limiting end bearing values (kPa)
Very Loose Loose Loose Medium Dense Dense	Sand Sand Sand-Silt Silt Silt	Not Applicable	Not Applicable	Not Applicable	Not Applicable
Medium Dense	Sand-Silt	0.29	67	12	3000
Medium Dense Dense	Sand Sand-Silt	0.37	81	20	5000
Dense Very Dense	Sand Sand-Silt	0.46	96	40	10000
Very Dense	Sand	0.56	115	50	12000

The ultimate pile capacity is assessed by adding total skin friction and total end bearing as show below:

$$Q_D = Q_f + Q_P = fA_S + qA_P$$

Where: Q_f – Skin friction resistance, in force units

Q_P – Total end bearing, in force units

f – Unit skin friction capacity, in stress units

A_S – Side surface area of pile

q – Unit end bearing capacity, in stress units

A_P – Gross end area of pile

Table 2 Safety factor for allowable pile capacity, API, Ref [1]

Load condition	Factor of Safety
Design environmental conditions with appropriate drilling loads	1.5
Operating environmental conditions during drilling operations	2.0
Design environmental conditions with appropriate producing loads	1.5
Operating environmental conditions during producing operations	2.0
Design environmental conditions with minimum loads (for pullout)	1.5

The allowable pile capacity is determined by dividing the ultimate pile capacity by safety factor relevant to loading type on pile. Safety factors corresponding to various loading types are presented in Table 2.

2.1.2 Load Resistance Factor Design (LRFD) Method

Unlike WSD method, safety factors are used to account for uncertainty in loading in pile resistance in LRFD method. According to API RP 2A- LRFD, Ref [2], the axial pile resistance should satisfy the following conditions:

$$P_{DE} < \phi_{PE} Q_D$$

$$P_{DO} < \phi_{PO} Q_D$$

Where:

P_{DE} (or P_{DO}) – Axial pile load for extreme (or operational) environmental conditions determined from a coupled linear structure and nonlinear foundation model using factored loads.

ϕ_{PE} –Pile resistance factor for extreme environmental conditions (= 0.8)
ϕ_{PO} –Pile resistance factor for operating environmental conditions (= 0.7)
Q_D – Ultimate axial pile capacity, which is determined by adding total skin friction and total end bearing as shown in WSD method.

Load facors on gravity loads:
$Q = 1.3D_1 + 1.3D_2 + 1.5L_1 + 1.5L_2$
D_1 – Self weight of the structure.
D_2 – Dead load imposed on the platform by weight of equipments and other objects.
L_1 – Live load 1 includes the weight of consumable supplies and fluids in pipes.
L_2 – Short duration force exerted on the structure from operations like lifting, drilling.

Load factors on wind, wave and current loads:
Under extreme condition the following factors are used:
$Q = 1.1D_1 + 1.1D_2 + 1.1L_1 + 1.35(W_e + 1.25D_n)$
W_e – The force applied to the structure due to the combined action of the extreme wave (typically 100 year return period) and associated current and wind.

Under operating condition the following factors are used:
$Q = 1.3D_1 + 1.3D_2 + 1.5L_1 + 1.5L_2 + 1.2(W_e + 1.25D_n)$

2.1.1 CPT based Methods

The CPT based methods are based on direct correlations of pile unit friction and end bearing data with cone tip resistance (q_c) values from cone pentration tests. According to API RP 2A-WSD, Ref [1], the CPT based methods are preferred to the methods based on soil parameters as these methods have shown statistically closer predictions of pile load test results.

The four recommended CPT-based methods considered here for cohesionless soil are:

1. Simplified ICP-05
2. Offshore UWA-05
3. Fugro-05
4. NGI-05

Details of the pile capacity assessment based on these methods are given in detail in API RP 2A-WSD, Ref [1] and hence not reproduced in this paper. These methodologies have been followed for the pile capacity assessment based on CPT data.

2.2 DNV

Similar to API, DNV also provides WSD and LRFD methods for pile capacity assessment but with different safety factors.

2.2.1 WSD Method

DNV OS C201 code, Ref.[4] reports structural design of offshore units according to WSD method. But this code does not provide a specific foundation design method. Instead, it states that the foundation design shall be carried out according to either LRFD method (as descibed in section 2.2.2) or in accordance with DNV CN 30.4, Ref. [5], or other acceptable standards. In this paper, in order to compare the resulting pile legnths from each design method, the axial pile capacity assessment in accordance to WSD method has been carried out based on DNV CN 30.4. According to DNV CN 30.4, Ref. [5], the compression capacity of pile is sum of cuumulated skin friction and end resistance as in API WSD method presented in section 2.1.1.

Method to assess unit skin friction and end resistance for cohesion soil is exactly same as in API WSD method with same coefficients and limits. Similarly, the method to assess unit skin friction and end bearing in cohesionless soil is same as in API method, except unit skin friction, which is defined as $f_s = Kp_o' \tan\delta \leq f_1$.
Where K is the lateral earth pressure, taken as 0.8 and δ is the soil-pile interface friction angle.

Also, the factors and limits are slightly different from API WSD method presented in Table 1. These are shown in Table 3. It must be emphasised that the DNV CN 30.4 is based on API RP 2A (1987) and still in its first version published 1992 and it is still referred in DNV OS C201, Ref. [4] for foundation design.

The unit end resistance of plugged piles in cohesion-less soil, q_p, may be taken as $q_p = p_o' N_q \leq q_l$
Where p_o' is the effective overburden pressure at the pile tip elevation, N_q is bearing capacity factor and q_l is limiting end bearing as given in Table 3. In pile capacity assessment, safety factor of 1.5 (as in Table 1) was used to compare required pile length according to DNV WSD method with API WSD.

Table 3 Design parameters for cohesionless soil, DNV CN 30.4, Ref [5]

Relative density	**Soil description**	**δ (degrees)**	**f_1 (kPa)**	**End bearing factor N_q**	**Limiting end bearing values (MPa)**
Very Loose Loose Medium	Silt Sand-Silt Silt	15	48	8	1.9
Loose Medium Dense	Sand Sand-silt Silt	20	67	12	2.9
Medium Dense	Sand Sand-Silt	25	81	20	4.8
Dense Very Dense	Sand Sand-Silt	30	96	40	9.6
Dense Very Dense	Gravel Sand	35	115	50	12.0

2.2.2 LRFD Method

Pile capacity design guidelines according to LRRD method is described in DNV OS C101, Ref [3].

For determination of design soil resistance against axial pile loads in ULS design, a material coefficient γ_m=1.3 shall be applied to all characteristic values of soil resistance, e.g. to skin friction and tip resistance.

For pile foundations of structures where there are no or small possibilities for redistribution of loads from one pile to another, or from one group of piles to another group of piles, larger material coefficients than those given above shall be used. This may for example apply to pile foundations for tension leg platforms or to deep draught floaters. In such cases the material coefficient shall not be taken less than γ_m=1.7 for ULS design.

For calculation of design lateral resistance, the following material coefficients shall be applied to characteristic soil shear strength parameters for Ultimate limit state (ULS) design:

γ_m = 1.2 for effective stress analysis

γ_m = 1.3 for total stress analysis

For accidental limit state (ALS) and serviceability limit state (SLS) design, the material coefficient γ_m may be taken equal to 1.0.

Load factors:

For analysis of ULS, two sets of load combinations shall be used when combining design loads as defined in Table 4 below. The combinations denoted (a) and (b) shall be considered in both operating and temporary conditions. The load factors are generally applicable for all types of structures, but other values may be specified in the respective object standards.

Table 4 Load factors for different combinations, DNV OS C101, Ref [3]

Combination of design loads	Load categories			
	G	Q	E	D
(a)	1.3	1.3	0.7	1.0
(b)	1.0	1.0	1.3	1.0

Load categories are:
G – permanent load
Q – variable functional load
E – Environmental load
D – deformation load

The code further states the following aspects when considering load factor:

1. When permanent loads (G) and variable functional loads (Q) are well defined, e.g. hydrostatic pressure, a load factor of 1.2 may be used in combination (a) for these load categories.

2. If a load factor γ_f = 1.0 on G and Q loads in combination (a) results in higher design load effect, the load factor of 1.0 shall be used.

3. Based on a safety assessment considering the risk for both human life and the environment, the load factor γ_f for environmental loads may be reduced to 1.15 in combination (b) if the structure is unmanned during extreme environmental conditions.

3. Pile Capacity Assessments

In order to demostrate the differences in resulting penetration depth requirement from different design-codes considered in this paper, required pile penetration for a given design senario was assessed in accodance with the codes. Outcome of the design methods of codes were compared in terms of pile length required to carry an unfactored axial load of 2000kN which comprises dead load of 1000kN, live load of 600kN and environmental load of 400kN. Since the aim is to focus on the pile design methods in codes, scour around the pile and other secondary aspects have not been considered in the assessment.

Soil data from an offshore platform location has been used in the pile capacity assessments. Samples were taken from the site and the required soil properties were obtained from onshore laboratory tests. These soil data is then used for LRFD and WSD methods defined in API and DNV codes. CPT data from offshore survey in the same location has been used for CPT based methods recommended by API. These soil data and the CPT data are shown in Table 5 and Figure 2 respectively.

Table 5 Soil properties obtained from laboratory tests

Depth (m)	Soil	Submerged unit weight (KN/m^3)	Friction angle (degrees)	Undrained shear strength (kPa)
0-2.3	Sand	9.5	30	
2.3-3.2	Clay	10		150
3.2-15	Sand	10	32.5	
15-25	Clay	11.3		328

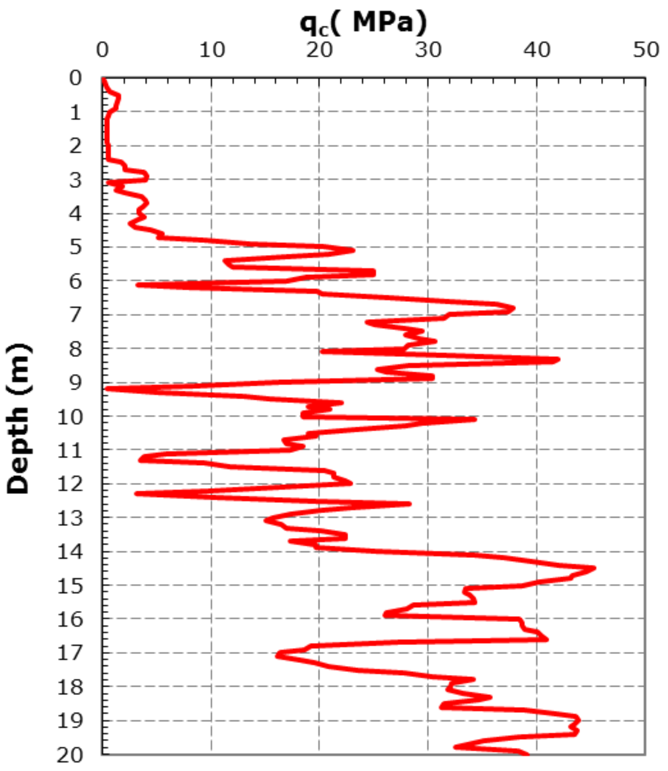


Figure 2 Cone resistance from CPT data

4. Results

Open end pile with outer diameter of 24” (610mm) and wall thickness of 19mm was considered in the analysis. It has been assumed that the pile can be installed to the desired penetration depth without refusal or any fatigue issues.

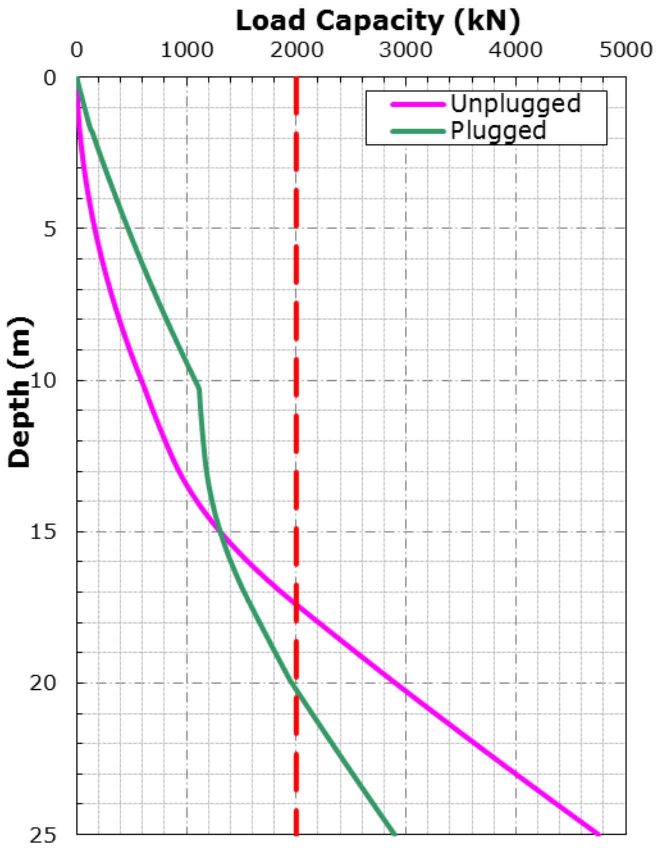
Pile capacity was assessed from the design methods based on soil properties and the results are presented in Figure 3. Both plugged and unplugged states of the pile have been shown by separate curves where appropriate. External skin friction and the end bearing of the total pile cross section were summed to evaluate the ultimate capacity of the pile in plugged condition. In unplugged state, internal and external skin frictions were added to the end bearing of the pile annulus area to calculate the ultimate capacity of the pile.

Punch through efffect due to presence of weaker soil layers was considered with the depth of influence zone of 2.5 times pile diameter in soil properties based methods.

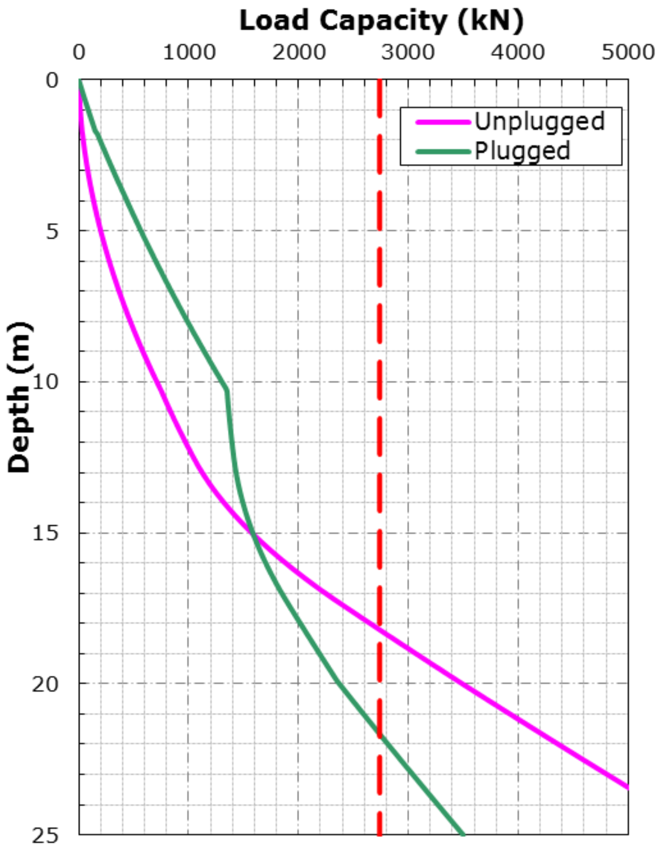
Pile capacity results from CPT data based methods are presented in Figure 4. Both unplugged and plugged pile capacity curves are plotted only for NGI-05 method. However, pipe piles are generally plugged as stated in API, Ref [1].

The required load capacity is shown by a vertical red dotted line in all cases. This load requirement is either factored or un-factored depending on design method. Safety factor of 1.5 has been used for all the CPT based design methods to evaluate the allowable pile capacity.

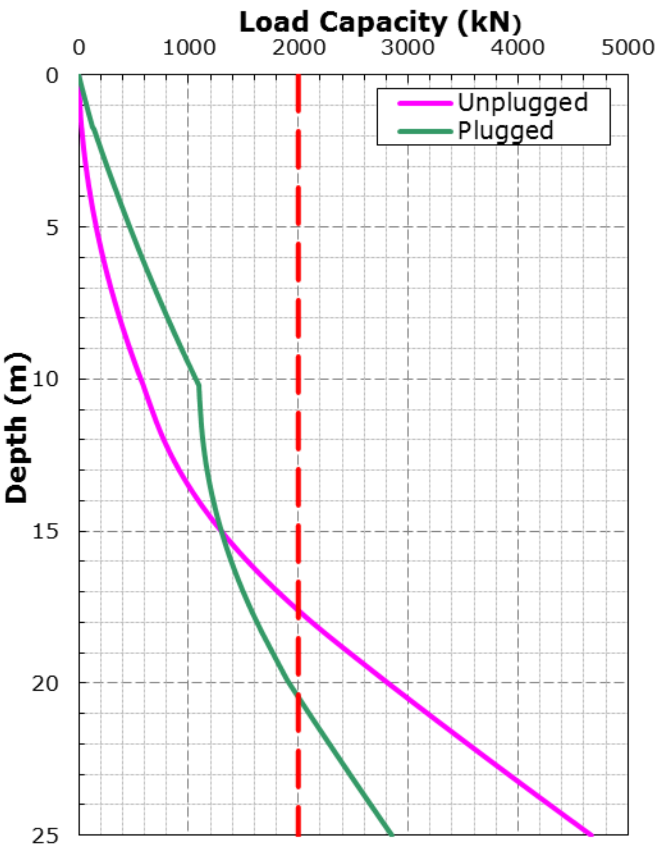
In WSD methods, safety factor of 1.5 has been used in calculations to derive the allowable pile capacities which are then compared with working load of 2000kN. In API LRFD method, load factors of 1.3, 1.5 and 1.35 were used for dead load, live load and environmental load respectively and material resistance factor of 0.8 has been used in line with API, Ref [2]. In DNV LRFD method, load factor of 1.3 was used for both dead load and live load. Load factor of 0.7 was used for environmental load along with material safety factor of 1.3 which is based on DNV guidelines, Ref [4]. Material safety factors in LRFD methods have been applied on load capacity of the pile.



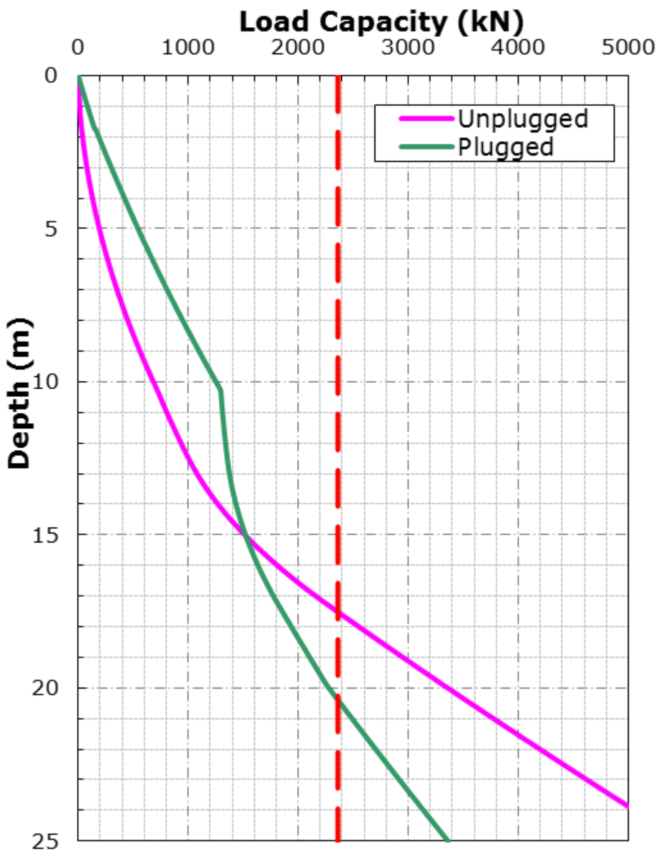
(a) API WSD



(b) API-LRFD

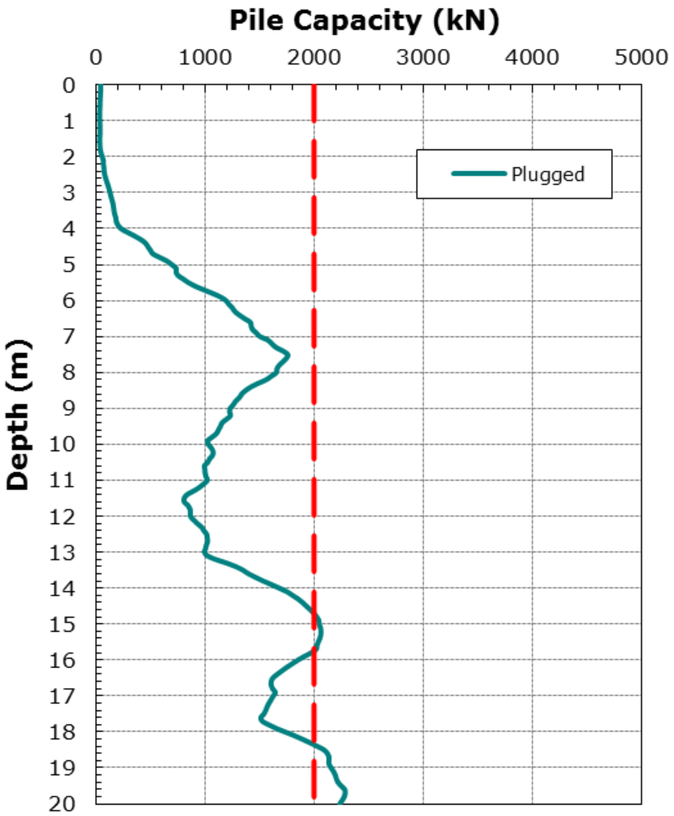


(c) DNV-WSD (DNV CN 30.4 with safety factor 1.5)

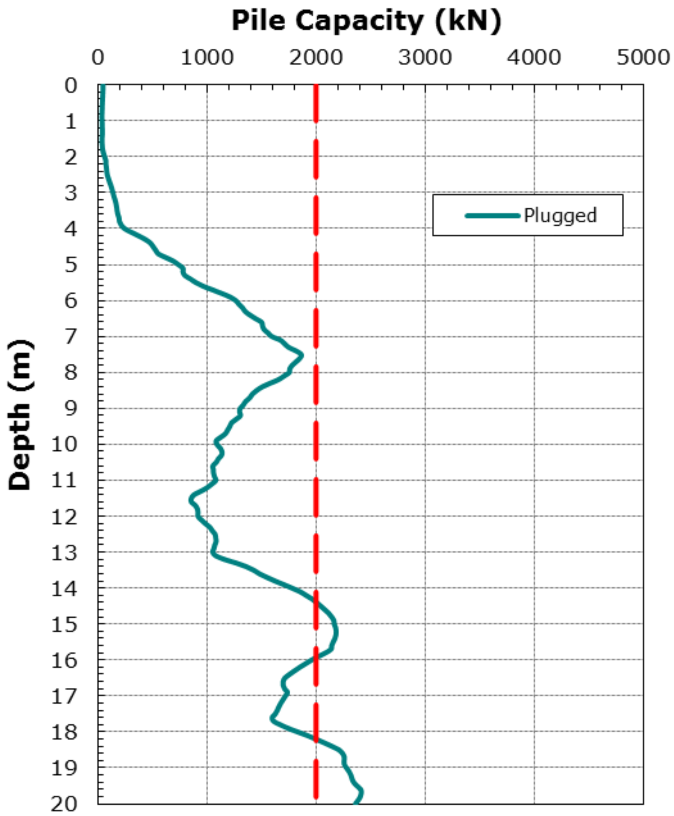


(d) DNV-LRFD

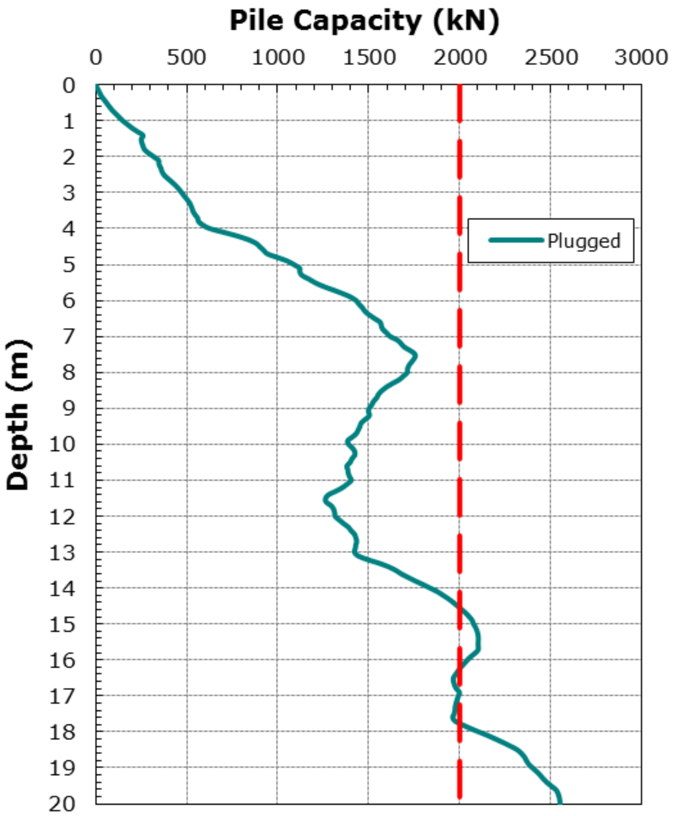
Figure 3 Pile capacity results based on soil properties (a) API WSD (b) API LRFD (c) DNV WSD (d) DNV LRFD



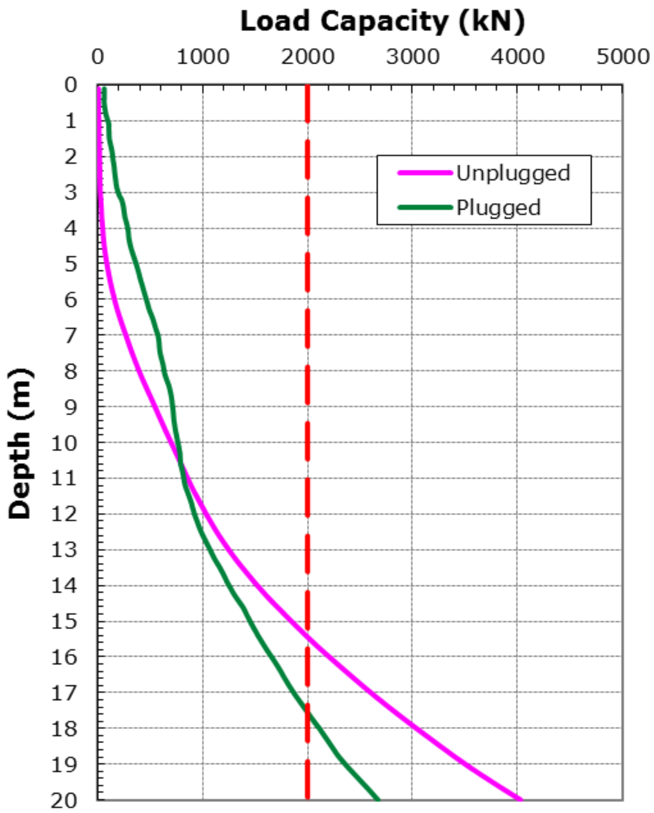
(a)Simplified ICP-05



(b) Offshore UWA-05



(c) Fugro-05



(d) NGI-05

Figure 4 Pile capacity results based on soil properties (a) Simplified ICP-05 (b) Offshore UWA-05 (c) Fugro-05 (d) NGI-05

5. Discussion

The primary difference between WSD and LRFD methods is on how the uncertainty of loading is considered in the design. In LRFD, a partial safety factor is incorporated with each type of loading to account for uncertainty in the loading and partial safety factors are used to account for material uncertainties. On the other hand, no safety factor is considered for loading in WSD method; instead, a combined safety factor of larger value is considered to evaluate the allowable capacity. Thus, different loading types will not make any difference in the required pile capacity in WSD method as only the total load on the pile is considered in the assessment.

The minimum required penetration depth in accordance with each design method are summarised in Table 6. According to this table and as shown in figure 4 and 5, there is a slight difference between the outcome of the assessment in accordance with different methods presented in API and DNV codes. However the minimum required penetration depth for a given load at a particular site would be single value. In other words, pile driven to a certain depth has compression capacity of a single value. Therefore the difference noticed in the results above is purely due to the differences in design guidelines such as the adopted safety factors adopted and empirical coefficients. Some design methods incorporate conservatism in design method to overcome uncertainty in load and soil properties. Nevertheless it is difficult to point out where the conservatism is in each design approach without measurement of pile capacity from field tests, which is beyond the scope of this paper.

Among the results from methods based on soil properties, WSD methods shows slightly less penetration requirement compared to LRFD methods. As the assessment of ultimate pile capacity being the same among these two methods, the difference in required pile penetration depth is due to the difference in safety factors used in these methods.

Table 6 Summary of minimum required penetration depth results

Design method	Required penetration depth (m)
API – WSD	20.3
API – LRFD	21.6
DNV – WSD	20.3
DNV – LRFD	20.4
CPT based - Simplified ICP-05	18.4
CPT based -Offshore UWA-05	18.2
CPT based -Fugro-05	17.8
CPT based -NGI-05	17.6

When the pile capacity assessment methods based on soil properties and CPT data are compared, it is clearly evident that longer pile penetration is required if assessment is carried out based on soil properties compared to design methods based on CPT data. The primary reason behind the difference between these methods can be associated with the limits enforced on unit skin frction and end bearing in methods based on soil properties. Though, these limits have been provided to ensure safe design, they can be conservative compared to other design methods. Even though CPT based design methods result in comparatively smaller penetration depths, design codes like API emphasise that the methods based on CPT data must be used only by experieneced engineers. This caution can be due with several aspects. The first thing is variation in soil properties at a particular site is not captured in CPT data. This will lead to failure to account for soil strength variation in pile design. Another importatnt aspect is that the CPT based design methods are relatively new and calibration from field test is limited for these methods. On the other hand, methods based on soil prpoerties in accordance to both API and DNV has been in use for many years and there is vast amount of field data to support its performance.

6. Summary and Conclusion

Eight difference methods of evaluating pile penetration length requirement for a driven pile have been presented and discussed. Among the WSD and LRFD methods based on both API and DNV, LRFD method shows slightly longer penetration requirement than WSD method. This is associated with influence of safety factors used in both methods. According to the results, CPT based methods have shown much smaller penetration requirement comprared to design methods based on soil properties. However, these methods must be used only by expeienced engineers as some of these methods are relatively new and the field data calibration for these methods are limited, particularly for offshore piles.

7. References

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