Comparative study on methods of predicting the axial bearing capacity of driven steel pipe piles – Case study

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ABSTRACT
Driven steel pipe piles are widely used in different projects around the world to transfer the superstructures loads onto the pile shaft resistance or a competent dense soil layer or bedrock. However, determination of the axial bearing capacity and settlement of the piles is a complex procedure that involves different approaches, such as analytical methods based on geotechnical properties of soils obtained from the laboratory or in-situ tests, static load tests on a driven pile, and dynamics methods which are based on the dynamics of pile driving or wave distribution.
A case study is presented on the Pile Dynamic Analysis (PDA) tests performed on 51 driven steel pipe piles at a mining site located in the North of British Columbia, Canada. Data from PDA tests have been used to determine the axial bearing capacity including the tip and the shaft resistances along the pile length.
This paper compares the axial bearing capacity of piles determined from the analytical and semi-empirical method with the PDA test results. Outcome of the study showed that the calculated axial bearing capacity derived from the API method is in good agreement with the estimated axial bearing capacity by PDA tests.

RÉSUMÉ
Les pieux de tubes d'acier battus sont largement utilisés dans différents projets à travers le monde pour transférer les charges des superstructures sur la résistance de l'arbre du pieu ou une couche de sol dense compétente ou un substrat rocheux. Cependant, la détermination de la capacité portante axiale et du tassement des pieux est une procédure complexe qui implique différentes approches, telles que des méthodes analytiques basées sur les propriétés géotechniques des sols obtenues en laboratoire ou des essais in situ, des essais de charge statique sur un pieu battu, et des méthodes dynamiques basées sur la dynamique du battage de pieux ou de la distribution des vagues.
Une étude de cas est présentée sur les tests analyse dynamique de battage de pieux (PDA) effectués sur 51 pieux de tubes d'acier battus sur un site minier situé dans le nord de la Colombie-Britannique, au Canada. Les données des essais PDA ont été utilisées pour déterminer la capacité portante axiale, y compris les résistances de la pointe et de l'arbre sur toute la longueur du pieu.
Cet article compare la capacité portante axiale des pieux déterminés à partir de la méthode analytique et semi-empirique avec les résultats des essais PDA. Les résultats de l'étude ont montré que la capacité portante axiale calculée dérivée de la méthode API est en bon accord avec la capacité portante axiale estimée par les tests PDA.

1 INTRODUCTION

Piles are the structural elements in a foundation that have the function of transferring a combination of vertical and horizontal loads from the superstructure to lower levels in the soil mass, by distributing the load along the pile shaft or by applying the load directly on a competent dense soil or bedrock.

Piles can be constructed and installed in a variety of diverse ways (large displacement piles, small-displacement piles, and replacement piles) depending on their type and the ground's conditions. The selection of the appropriate pile type depends on the type of structure, load conditions, the subsurface conditions, and the expected lifetime of the project. For structures on land, any of the construction types can be considered for piling, while displacement piles are the first choice for a marine structure.

While materials for piles, installation, and fabrication can be precisely specified, the calculation of their capacity is a complex procedure that currently is based on theoretical concepts derived from classical soil and rock mechanics relations, but mainly on empirical or semi-empirical methods. Each of these equations and methods has its limitations and assumptions, resulting in differences in the prediction of the actual pile bearing capacity. Furthermore, varying soil conditions and uncertainties of soil parameters cause additional errors, demonstrating the importance of in-situ pile load tests at the beginning of the pile installation. Pile load testing can be divided into three main categories, as shown below:

1. Static pile load testing;
2. Dynamic load testing by Pile Driving Analyzer (PDA);
3. Pile Integrity Testing (PIT).

Static pile load tests are the most accurate means of determining pile capacity (FHWA 1992, 2010); however, they are expensive and may not be practicable due to accessibility and existing structures issues on some sites. Alternatively, the Pile Driving Analyzer (PDA) is the most popular tool for determining the axial pile capacity.

This paper presents the comparison between the measured axial bearing capacity of 51 driven steel pipes using the PDA test, and the bearing capacity calculated using the API (2010) method.

2 METHODS OF CALCULATING PILE AXIAL BEARING CAPACITY

Many guidelines, codes, and scientific publications provide a variety of analytical and empirical correlations to calculate the pile's ultimate bearing capacity based on soil characteristics. However, for a given geotechnical condition, estimations of these correlations are quite different. CFEM (2006), OCDI (2009), AASHTO (2002), API (2010), Das B.M. (2014), and Tomlinson M. & Woodward J. (2015) are examples of these references.

In addition, there are several methods for estimating pile axial bearing capacity based on resistance to driving or restriking the pile. FHWA-Modified Gates equation, the Washington State Department of Transportation formula (WSDOT), and the Pile Driving Analyzer (PDA) are three examples of methods that use driving resistance to calculate pile capacity. The first two methods estimate pile capacity using dynamic formula while the PDA method requires a detailed measurement of the temporal variation of pile force and velocity during driving.

The PDA test involves generating an impulse using the pile hammer (for driven piles) or a drop weight (for bored piles), then determining pile integrity and pile capacity using signal matching. By using the downward wave as input, the signal matching method iteratively adjusts a numerical soil-pile interface model until a comparison can be made between the computed reflected upward force wave and the measured reflected upward force wave. The most common software used for signal matching is Case Pile Wave Analysis Program (CAPWAP).

3 CASE STUDY

The project is an expansion of a mine site mill building located northwest of British Columbia in Canada. A geotechnical investigation was conducted to review in-situ subsurface conditions and design the foundation elements for the support of the new building, equipment, and tanks.

3.1 Summary of subsurface stratigraphy and soil parameters

Based on the results of the site investigation program, the encountered subsurface soils at the site generally consist of the three main stratigraphic units as presented below. The soil geotechnical parameters of each layer are presented in Table 1.

1. Sand and Gravel (Fill/Colluvium): A sand and gravel, non-plastic, moist to wet stratigraphic unit were encountered in the upper depths of the ground. A layer of stiff to very stiff sandy silt materials is interbedded within the sand and gravel strata. The SPT $(N_1)_{60}$ values indicate a relatively dense soil layer.
2. Sandy Silt / Silt: A glacial outwash consisting of sand, sandy silt and silt, with low plasticity and firm to stiff consistency is underlying the sand and gravel layer.
3. Glacial Till: A glacial till layer consists of gravelly, silty sand to sand and gravel, non-plastic, with a relative density of very dense to hard. The upper depth of the glacial till was identified as a weathered till and consists of silty gravel material.

The project site area has been divided into three different zones (zone 1 to 3) for the calculation of the pile bearing capacity to consider the variation of the soil layers' thickness within the project site. It should be noted that seismic evaluation using simplified site response analyses showed the project site is not liquefiable.

Table 1. Soil geotechnical parameters

Material	Depth range (m.B.G.L)	Thickness range (m)	γ' *(kN/m³)*	φ' (deg.)	S_u *(kPa)*
Sand and Gravel	0 -8.5	5.5-8.5	20	33	-
Silt / Clay	8.5 -10.5	5-7	18	-	52
Glacial Till	12 -15 <	-	21	40	-

3.2 Pile Axial Resistance

The tubular steel piles (pipe piles) in two (2) different sizes (324 and 406 mm) were considered for the foundation of the new building.

The method of analysis was based on the API RP 2A-2010 guidelines. Furthermore, for axial loading considerations, based on AASHTO recommendation, piles were spaced at three (3) pile diameters (center to center) to act as single piles, with no group interaction effects with regards to axial resistance. In addition, a geotechnical resistance factor of 0.5 was applied to the ultimate axial resistance value assuming that the PDA testing will be performed during pile installation.

Figures 1 to 3 present the variation of calculated ultimate and factored static axial resistance with depth for two (2) sizes of pipe piles in three different zones of the project area. The axial pile resistance is a combination of shaft friction and end bearing. Considering the capacity needed for each equipment and element in the building, as well as the limitations of the space, pile diameter, number, and length are chosen for each foundation. Table 2 presents the estimated pipe lengths necessary in different zones for each pile size. Also, Table 3 summarizes the minimum factored structural demand for each pile size in all three zones.

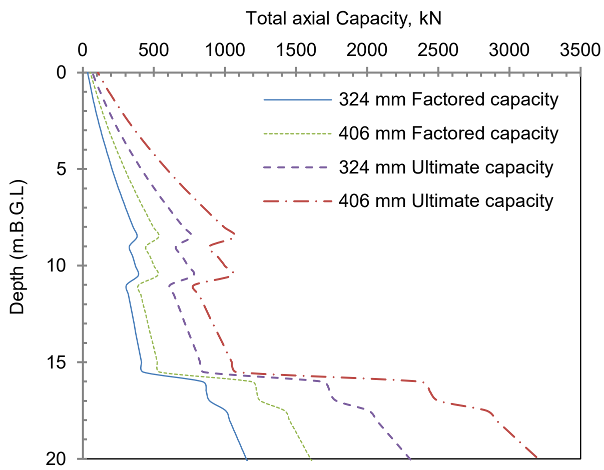

Figure 1. Variation of the ultimate and factored axial pile capacity for 343 mm and 406 mm diameter open-ended piles at zone 1 of the site

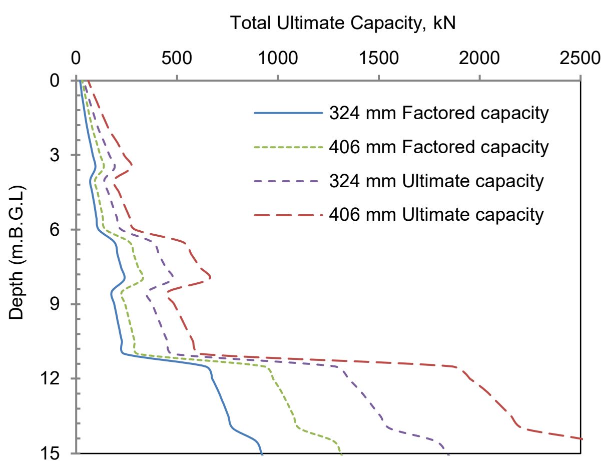

Figure 2. Variation of the ultimate and factored axial pile capacity for 343 mm and 406 mm diameter open-ended piles at zone 2 of the site

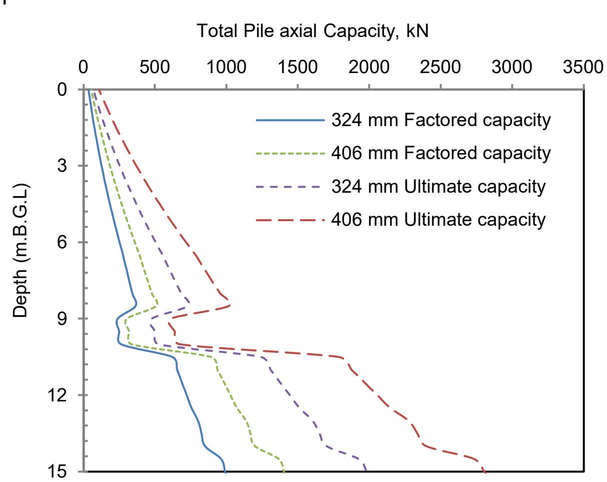

Figure 3. Variation of the ultimate and factored axial pile capacity for 343 mm and 406 mm diameter open-ended piles at zone 3 of the site

Table 2. Summary of pile axial resistance using API (2010) method

Zone	Pile dia. (mm)	Pile length (m)	Shaft resist. (kN)	End resist. (kN)	ULS (kN)	Factored ULS (kN)
Zone 1	324	16	862	823	1685	843
	406	16	1081	1297	2378	1189
Zone 2	324	12	488	791	1280	640
	406	12	612	1245	1857	928
Zone 3	324	12	535	707	1242	621
	406	12	670	1114	1784	892

Table 3. Summary of factored axial resistance for the various pile sizes

Zone	Pile diameter (mm)	Pile thickness (mm)	Factored axial load capacity (kN)
Zone 1	324	12.7	840
	406	12.7	1190
Zone 2	324	12.7	640
	406	12.7	930
Zone 3	324	12.7	620
	406	12.7	890

3.3 Pile driving termination criteria

Pile driving termination criteria can be evaluated employing wave equation analysis (first developed by Smith, E.A.L., 1960) or by using dynamic formulas. Recent work conducted to update dynamic pile driving formulas using reliability theory, as described by Paikowsky, S. G. (2004) in NCHRP Report 507, shows that this approach is generally more reliable than wave equation analysis using default parameters. Based on this theory, new dynamic pile driving formulas have been developed by Washington State Department of Transportation (Allen, T. M., 2005). The minimum required penetration resistance of piles in each zone to reach the factored axial compressive resistance was calculated by a third-party company using both methods and the results are presented in Table 4. AllWave-PDP software was used to evaluate the termination criteria using the wave equation analysis method. Finally, values of the AllWave-PDP analysis were selected for the pile driving terminating criteria as they were more conservative compared to the WSDOT values.

Table 4. Minimum required penetration resistance at termination

Zone	Pile diameter (mm)	Hammer stroke (m)	Minimum required penetration resistance (blows/50 mm)	
			AllWave-PDP	WSDOT
Zone 1	324	0.6	12	11
	406	0.9	10	6
Zone 2	324	0.6	6	3
	406	0.9	5	2
Zone 3	324	0.6	8	6
	406	0.9	6	3

3.4 Pile installation and testing program

The pile driving program in this project included the installation of a total of 545 piles comprising 401 of 324 mm x 12.7 mm and 144 of 406 mm x 12.7 mm open-ended steel pipe piles. A Junttan HH55KS hydraulic impact hammer with a 49 kN ram and maximum energy of 74 kN•m has been employed for the installation. Figure 4 shows the location of piles in each zone. Please note that the larger piles (406 mm X 12.7 mm) are illustrated with solid circles in this figure.

Each pile was driven until reaching the termination criteria as presented in the previous section. Table 5 summarizes the pile installation statistics including the number of each pile size and the average embedment length of them in different zones.

High-Strain Dynamic Testing (HSDT) was conducted by a contractor to estimate the mobilized capacity of the piles, provide information for analysis of pile integrity, and assist in determining the required final penetration resistance in initial driving and in restriking. The selected piles were tested 1 to 7 days after the installation. 51 tests have been conducted in total in this project.

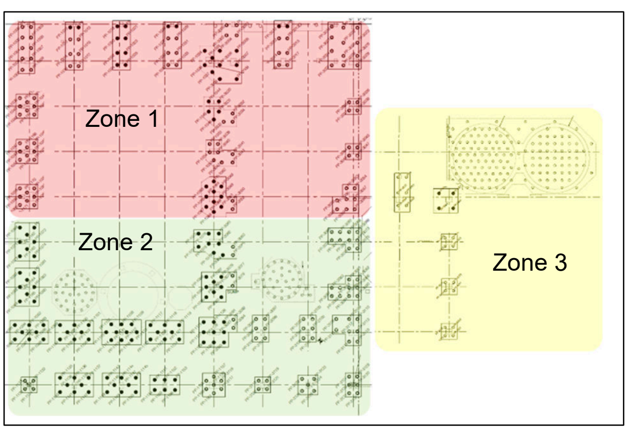

Figure 4. Three zones of the project site

Table 5. Number of the driven piles and their average embedment length

Zone	Pile diameter (mm)	Average length of driven piles (m)	Number of piles
1	324	15.2	80
	406	17	29
2	324	12.2	147
	406	14	107
3	324	12.4	174
	406	12.4	8

4 RESULT AND DISCUSSION

As presented in section 3, the ultimate bearing capacity of piles was calculated using the API (2010) method, and the results of Wave equation analysis were selected for pile driving termination criteria.

A series of pile load tests have been conducted on a selected number of driven piles in each zone and the results are presented in Figures 5 to 9. These figures show the variation of mobilized axial resistance in piles with their length. For example, Figure 5 shows that a total of six (6) tests have been conducted on piles with a diameter of 324 mm in Zone 1. Although the length of tested piles varies from 13.2 to 17.1 m, there is a minor change in their mobilized axial resistance. Table 6 shows the average and standard deviation of mobilized axial force in each zone. It can be seen that the standard deviation is up to a maximum of 20% of the average value showing the consistency of mobilized axial force in driven piles even though their embedment length varies significantly (e.g., the embedment length of the 406 mm pile tested in Zone 1 varies between 13.4 to 18.9 m - see Figure 6). This finding verifies the reliability of the pile driving termination criteria (minimum required penetration resistance - blows/50 mm) obtained from wave equation analysis or dynamic formulas.

In Figures 5 to 9, the average mobilized axial resistance (results of pile load testing) and the factored axial resistance (refer to Table 3) for each zone and pipe size is shown. As can be observed, the factor of safety varies between 2 and 4, which is higher than the minimum requirement. Therefore, the results from API (2010) seem to be conservative. This finding was also observed and confirmed by Chow (1996) and Tomlinson (2001).

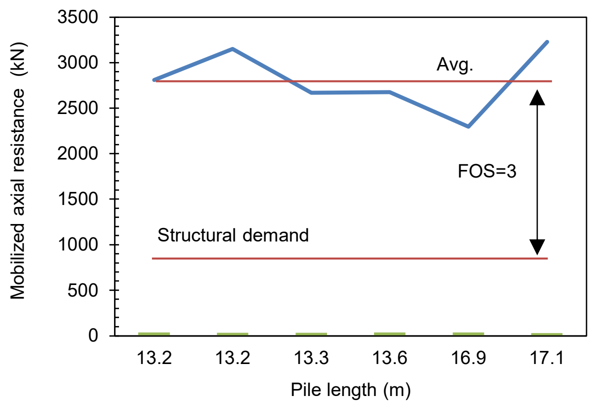

Figure 5. PDA test results on piles in zone 1 with a diameter of 324 mm (number of tests = 6)

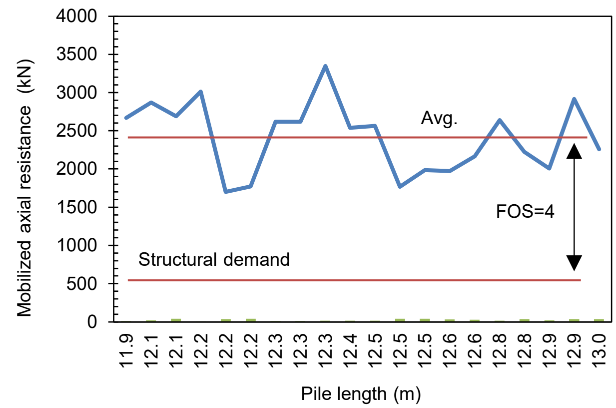

Figure 8. PDA test results on piles in zone 3 with a diameter of 324 mm (number of tests = 20)

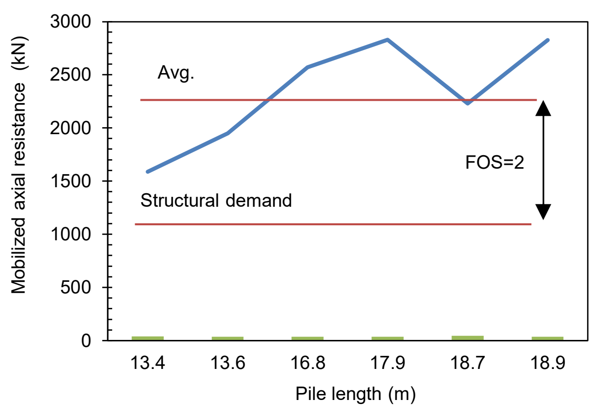

Figure 6. PDA test results on piles in zone 1 with a diameter of 406 mm (number of tests = 6)

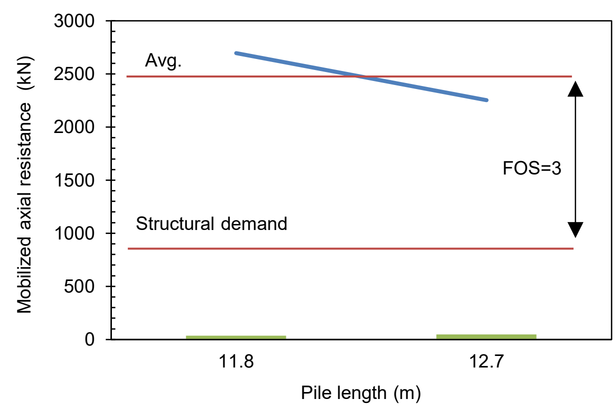

Figure 9. PDA test results on piles in zone 3 with a diameter of 406 mm (number of tests = 2)

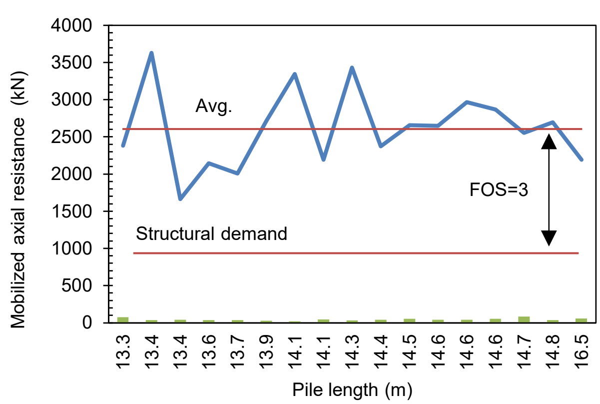

Figure 7. PDA test results on piles in zone 2 with a diameter of 406 mm (number of tests = 17)

Table 6. Statistics of PDA tests

Zone	Pile dia. (mm)	No. of PDA tests	Range of tested pile length (m)	Avg. of mobilized axial resist. (kN)	Standard deviation of mobilized resistance (kN)	FoS (Mobilized resistance / structural demand)
1	324	6	13.2 -17.1	2805	314	3
	406	6	13.4-18.9	2332	457	2
2	324	-	-	-	-	-
	406	17	13.3 -16.5	2615	510	3
3	324	20	11.9- 13	2417	447	4
	406	2	11.8-12.7	2475	221	3

Another important point is the contribution of end and frictional resistance in pile bearing capacity for the API (2010) method and pile load testing results, as illustrated in Table 7. The end resistance ratio is higher than the frictional resistance ratio in API's prediction since it was decided to extend the pile length up to the glacial till layer (layer 3) and place the pile end on a stiff layer. This ratio is higher in Zones 2 and 3 compared to Zone 1 since the till layer is in a deeper depth in Zone 1 and, therefore, the pile is longer in this zone (i.e., higher frictional resistance and higher ratio).

The ratio of end resistance is also higher than the frictional resistance ratio in PDA test results which are similar to API's predictions. However, the value of the end resistance ratio is higher in the PDA test compared to API for a pile with diameter of 324 mm, while this value is lower for 406 mm pile (larger pile). For example, the end resistance ratio for 324 mm in Zone 3 is expected to be 57% by API, but this ratio is increased to 75% in the PDA test. On the other hand, the end resistance ratio is predicted to be 62% for 406 mm in Zone 3, while it is decreased to 52% in PDA tests. These outcomes show that although API's prediction of the pile total bearing capacity is acceptable, these correlations are underestimating the end bearing for smaller piles or overestimating the end bearing for larger piles, and the API's prediction of each element of the resistance (i.e., end and frictional) is not accurate as of its prediction for total bearing capacity.

Table 7. Comparison between the end and frictional resistance ratio for each pile size in different zones based on the outcome of API (2010) design method and pile load tests

Zone	Pile diameter (mm)	Method	Frictional resistance ratio (%)	End resistance ratio (%)
1	324	API (2010)	50	50
		PDA	33	67
	406	API (2010)	45	55
		PDA	38	62
2	324	API (2010)	38	62
		PDA	-	-
	406	API (2010)	33	67
		PDA	44	56
3	324	API (2010)	43	57
		PDA	25	75
	406	API (2010)	38	62
		PDA	48	52

5 CONCLUSION

The results of 51 pile load tests performed on steel pipe piles in a project in British Columbia, Canada were presented. Steel piles with 324 and 406-mm diameters and different lengths were studied. The ultimate bearing capacity of piles at different zones was calculated based on the API (2010) approach. In addition, the pile driving termination criteria was calculated using two different methods (wave equation analysis and WSDOT) and the first one was selected for this project.

A pile design based on API's recommendations proved to be a conservative method when compared with load test results. In addition, the ratio of end bearing resistance and frictional resistance based on API was found to have a similar pattern as the load test data (end bearing higher than frictional resistance); however, its accuracy in predicting the contribution of bearing elements (end and frictional) is low.

Finally, although the API's procedure was verified to be suitable for the prediction of steel pipe piles and the results were confirmed with the pile load test, these findings are highly dependent on soil conditions and load test results. Therefore, all results and data presented in this paper must be viewed in the context of the current case study and further research should be conducted.

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