A Study on Pile Capacity Setup of Driven Steel Pipe in Edmonton Till

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ABSTRACT
As one of the major deep foundation types, driven steel pile (DSP) is widely used in all construction projects in Canada. Especially in rural northern Alberta areas where concrete supply is not accessible in a cost-effective manner, DSP foundation is highly preferred by heavy industrial development such as oil and gas related facilities.

For driven steel pile set in the fine-grained soils, significant pile-soil setup (pile capacity gain) is expected due to excessive pore water pressure dissipation after the pile installations. In the field, pile appeared to have a much lower capacity at the end of the installation compared to long-term performance. In a fast-paced construction environment, the time cost to wait and verify the pile long-term capacity is not desirable. To proceed the upper structure construction without any delay, a reasonable prediction of DSP setup is required. Extensive research has been conducted to explain the mechanism and magnitude of the pile-soil setup effect. However, very limited study has been done on the rate / time of pore water pressure dissipation in clayey soils.

This study is aimed to provide a case study of the pile setup effect of DSP set in Edmonton clay till by using dynamic load testing, and wave equation analysis methods. A finite element numerical model is built to illustrate the pore water pressure dissipation and increase in radial effective stress, and allow geotechnical engineer to assess the pile setup behavior with available soil testing results and reasonable assumptions.

RÉSUMÉ
En tant que l'un des principaux types de fondations profondes, les pieux battus en acier (DSP) sont largement utilisés dans tous les projets de construction au Canada. Surtout dans les régions rurales du nord de l'Alberta où l'approvisionnement en béton n'est pas accessible de manière rentable, la fondation DSP est hautement préférée par le développement industriel lourd comme les installations liées au pétrole et au gaz.

Pour les pieux battus en acier enfoncés dans les sols à grains fins, une configuration significative du sol des pieux (gain de capacité des pieux) est attendue en raison de la dissipation excessive de la pression interstitielle après l'installation des pieux. Sur le terrain, le pieu semble avoir une capacité bien inférieure à la fin de l'installation par rapport aux performances à long terme. Dans un environnement de construction en évolution rapide, le temps nécessaire pour attendre et vérifier la capacité à long terme du pieu n'est pas souhaitable. Pour procéder à la construction de la structure supérieure sans aucun délai, une prédiction raisonnable de la configuration du DSP est requise. Des recherches approfondies ont été menées pour expliquer le mécanisme et l'ampleur de l'effet de mise en place du tas de terre. Cependant, une étude très limitée a été réalisée sur le taux / temps de dissipation de la pression interstitielle dans les sols argileux.

Cette étude vise à fournir une étude de cas de l'effet de la mise en place des pieux de DSP mis en place dans le till d'argile d'Edmonton en utilisant des tests de charge dynamique et des méthodes d'analyse d'équation des vagues. Un modèle numérique par éléments finis est construit pour illustrer la dissipation de la pression interstitielle et l'augmentation de la contrainte radiale efficace, et permettre aux ingénieurs géotechniciens d'évaluer le comportement de mise en place des pieux avec les résultats d'analyses de sol disponibles et des hypothèses raisonnables.

1 INTRODUCTION

1.1 Background

It appears that driven steel piles (DSP) installed in fine-grained soil would gain a significant amount of resistance after the initial installation and this behavior is described as pile-soil setup effect. Typically, the percentage of this setup typically ranged from 10% to 100% (Randolph, 1979) depending on the soil type and moisture condition. A main reason of this setup effect is that DSP installation would induce a radial displacement which would generate excessive pore water pressures (PWP). This excessive PWP would be significant in fine-grained soils which have relatively low coefficients of permeability compared to coarse-grained soils. Based on the Theory of Effective Stress (Terzaghi, 1925), the effective strength of the soil around the pile shaft would decrease temporarily due to increase of pore water pressure, but it will gradually get back to the original in-situ condition with the dissipation of excessive PWP.

In the construction field, pile installations are controlled and monitored by recording blow counts. In order to evaluate the blow count records, preliminary driving criteria should be developed (e.g. Wave Equation Analysis Program (WEAP)). However, in the WEAP analysis, the assumption of pile-soil setup percentage (typically 30% for clayey soil based on software manual) is totally based on engineer's experiences and judgement. If actual pile setup percentage is much higher than that in the WEAP assumption, the final pile capacity after the setup would be underestimated based on the blow count records collected at the end of installation. Meanwhile, the most economical way to verify the final pile capacity probably is to re-tap the same pile and obtain additional blow count after a waiting period. Certainly, this method will still cause some delay in the foundation production since the primary installation equipment (hammer rig) will be used to verify the final pile capacity. With a better understanding of the pile setup percentage, setup up rate, and the relationship with soil parameters, the engineers could better assess the pile long-term performance and select the optimum pile re-tap time frame to speed up the foundation production.

1.2 Objectives of Study

The objectives of this study are as follows:

- Review the existing soil information including geotechnical investigation and soil laboratory testing results.
- Provide summary of field observations including pile installation records, dynamic pile load testing (PDA) results, and estimate pile capacity based on re-tap results.
- Perform numerical modeling on pore water pressure dissipation or consolidation after an induced displacement of a soil mass.
- Compare and summarize the estimated pile setup gain from the field observations and numerical results.
- Propose an optimum time frame for final pile capacity verification of driven steel piles installed in Edmonton clay till.

2 REVIEW OF SOIL INFORMATION

2.1 Site Location and Local Geology

The clay / clay till deposits studied were found near Fort Saskatchewan, Alberta in a proposed petroleum industrial plant. The local clayey soils are considered as a common soil type in Edmonton area. This site is within Sturgeon County and surrounded by industrial plants and storage yards. The overall topography of this site was relatively flat.

The information gathered through the field investigations at this site are consistent with the records of the Alberta Geological Survey. The upper sand that is near the surface, is part of the aeolian sand dunes with minor loess typical of the local area. The underlying clay is consistent with Glacial Lake Edmonton deposits. The upper variation of clay till is consistent with Cooking Lake till, while the lower till appears to conform to Lamont glacial till. The larger layers of sand and silt in between the tills are consistent with Ministik Lake stratified sediments. The sand and gravel encountered at the terminating depths of boreholes in the investigation were similar to the Empress formation in a buried pre-glacial river valley. Deeper deposits would be the sandstone and siltstone bedrock of the Belly River formation (Andriashek, 1988).

2.2 Soil Properties

The table below provides a summary of in-situ bulk hydraulic conductivity testing (slug test) results.

Table 1. Summary of Bulk Hydraulic Conductivity

In-situ Slug Test Depth	Average Bulk Hydraulic Conductivity (cm/s)
6 m	8.1×10^{-10}
11 m	5.8×10^{-10}
28 m	6.9×10^{-10}

The table below provides a summary of soil properties based on laboratory testing results for soil samples collected at different depth during field investigation.

Table 2. Characteristics of Soil Samples

Characteristics	6 m	11 m	16 m
Liquid limit	63	39	33
Plastic limit	31	18	15
Sand (%)	9.7	3.1	36.2
Silt (%)	30.1	50.6	18.2
Clay (%)	60.2	46.3	45.2
In-situ Void Ratio e_i	1.0	0.9	0.5
Compression Index C_C	0.3	0.3	0.1
Compressibility m_v(m^2/MN)	0.15	0.10	0.11
Φ angle (degree)	-	27	-
Stiffness E50 (MPa)	-	16	-

3 DRIVEN STEEL PILE CAPACITY EVALUATION

3.1 Pile Information

The foundation of the proposed petroleum plant consists of approximately 800 piles which were installed in a 300 x 400 m area. Piles consist of straight shaft open ended steel pipes with diameters and wall thicknesses of 219x9.5, 324x12.7, 406x12.7 and 508x12.7 mm sections. The pile embedment depths ranged from 12 m to 22 m. Since 508x12.7 mm piles had a more consistent embedment depth of about 20 m, this pile size will be the focus of this study. The pile spacings are so far apart that interference between piles is negligible during field testing.

3.2 Pile Blow Count Evaluation

Wave Equation Analysis Program (WEAP) is the most common method for pile capacity evaluation of majority production piles. WEAP requires the modelling of hammer, driving system, pile, and soil as the inputs for wave equation analysis. A mathematical model using the one-dimensional wave equation was developed by Smith (1960). In WEAP analysis, pile capacity can be estimated based on equation below.

$$\frac{1}{B_c} = u_{toe} - q_t = \quad u_{toe} - \frac{\sum[R(m,t)q_x]}{R_t} \qquad (1)$$

where:

B_c = blow count, blow/m
u_{toe} = toe displacement, m
q_t = average toe quake (spring length), m
q_x = individual toe quake for each pile segment, m
R (m, t) = maximum individual soil resistance of each pile segment at time t, kN
R_t = total soil resistance, kN

For each pile segment, R(m,t) is the maximum individual soil resistance for each soil segment. The equation above indicate that the estimated blow count is proportional to the total soil resistance which means the higher soil resistance will require higher blow count.

3.3 Dynamic Pile Capacity Testing (PDA and CAPWAP)

In actual pile installations, soil condition variation, cold weather condition, switching hammer helmet, cushion, outdated hammer calibration, or possible alignment offset will all affect the blow count record. Because all these field variations are not considered in the WEAP analysis, the pile capacity estimation based on WEAP could be questionable. Therefore, dynamic pile load testing or Pile Dynamic Analysis (PDA) in accordance to ASTM D4945-17 (2017) is introduced to assess the in-situ pile resistances (Linkins, 2004) and verify the WEAP assumptions.

Strain / Force and acceleration / velocity curves of pile during a single hammer blow were measured by portable Pile Driving Analyzer (PAX) equipment. Case method (Rausche, 1985) was then applied to the blow record to obtain the pile static capacity by using following equation.

$$R_t = \frac{1}{2}\left[F_{tm} + F_{t_m+\frac{2L}{c}}\right] + \frac{1}{2}\left[v_{t_m} - v_{t_m+\frac{2L}{c}}\right]\frac{EA}{c} \qquad (2)$$

$$R_s = R_t - R_d \qquad (3)$$

$$R_d = J_c v_t \qquad (4)$$

$$v_t = 2v_{t_m} - \frac{c}{EA}R_t \qquad (5)$$

Combining equations (2) to (5) yields:

$$R_s = \frac{1}{2}\left\{(1-J_c)\left[F(t_m) + \frac{EA}{c}v_t(t_m)\right] + \ (1+J_c)\ \left[F\left(t_m + \frac{2L}{c}\right) - \frac{EA}{c}v_t(t_m + \frac{2L}{c})\right]\right\} \qquad (6)$$

where:

R_t = total pile resistance, kN
R_s = static pile resistance, kN
R_d = dynamic pile resistance, kN
t_m = time when maximum force was transferred to the pile, s
F(t_m) = force measured at time t_m
V(t_m) = velocity measured at time t_m
2L/c = time for the compressive wave travel from the pile top to the pile toe and back to the pile top
c = wave speed of steel, 5123 m/s
J_c = case damping factor, s/m

A typical PDA measurement of a single hammer blow is provided in Figure 1 below.

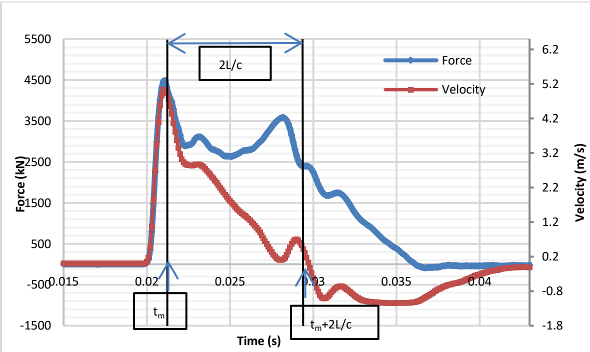

Figure 1. Force and Velocity Measurements from a Hammer Blow

It should be noted that the Case method above does not consider the soil profile and resistance distribution (layered system) nor the toe damping and quake. A signal matching computer program was developed by Professor Goble in the 1970s. It is a computer program to verify the pile static resistance with more accurate numerical estimation of the soil resistance distribution and dynamic soil parameters along the depth. This program called Case Pile Wave Analysis Program (CAPWAP) which applies a signal matching method.

3.4 Pile Setup Assessment

By using blow count evaluation (WEAP) and dynamic pile load testing (PDA and CAPWAP) methods, the pile capacity setup percentage can be obtained by using following correlation:

$$Pile\ Setup\ Percentage = \frac{R_{BOR} - R_{EOID}}{R_{EOID}} \text{ x } 100\% \qquad (7)$$

where:

R_{EOID} = estimated pile resistance at the end of initial drive, kN

R_{BOR} = estimated pile resistance at the beginning of restrike, kN

Based on the above correlation, pile setup percentage for 508 mm piles are provided in Figure 2 below.

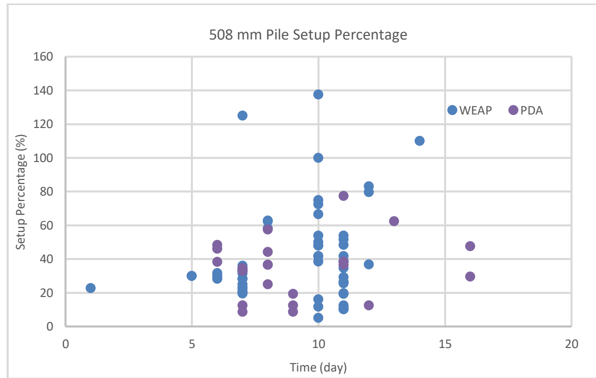

Figure 2. Pile Setup Percentage for 508-mm Diameter Piles

It can be seen that the setup percentage ranged from 18 to 165 percent for 508 mm piles at this site. Due to the cost of PDA testing is much higher than WEAP study of blow count records, most of the pile setup estimations were conducted using WEAP method based on blow count records.

4 FINITE ELEMENT MODELING (FEM) OF PWP DISSIPATION

4.1 Model Input and Soil Parameters

Based on information and results provided in Section 3.1, it is understood that piles with 508-mm diameter had the most field data and relatively similar embedment depths. Therefore, in this numerical modelling, 508-mm piles were modelled. The assumptions of this ABAQUS model are provided below:

- It is assumed that the 508-mm piles are not fully plugged during the installation, so the induced soil displacement due to pile driving will be the steel pipe thickness of 12.7 mm which can be treated as cavity expansion problem. The analysis of cavity expansion was conducted using ABAQUS for this soil displacement.

- Given the induced displacement of 12.7 mm which is about 2.5% of strain considering the diameter of the pile is 508 mm. It is assumed this deformation of the clayey soil is still within the elastic region.

Detailed soil parameters are provided in Table 3.

Table 3. Soil Parameters for Cavity Expansion Model

Soil Parameters	FEM Model
Radius of Cavity (m)	0.25
Elastic Modulus (Pa)	1×10^{7}
Poisson's Ratio	0.35
Void Ratio	0.40
Permeability (cm/s)	2×10^{-9} to 2×10^{-10}

4.2 Finite Element Type and Boundary Conditions

An 8-nodes axisymmetric quadrilateral, biquadratic displacement, bilinear pore pressure, reduced integration (CAX8RP) element type was selected for this model. A total of 100 elements and 341 nodes were included in this model. The applied boundary conditions and simulation steps are provided as follows:

- BC-1: Fixed displacement at X and Y directions for the bottom and right sides at initial step.

- BC-2: Fixed displacement at X and Y directions for the left side below the pile toe at initial step.
- BC-3: Applied 20 mm of displacement along the cavity in positive X direction at Step 1.
- BC-4: Zero pressure along the pile shaft area at Step 2.
- Allow pore water pressure dissipation in Step 2 for up to 100 days after the induced displacement in Step 1.

The length of the cavity expansion is about 20 m deep. The initial in-situ pore water pressure distribution along the depth (in pascal). The model dimensions and mesh are shown in Figure 3.

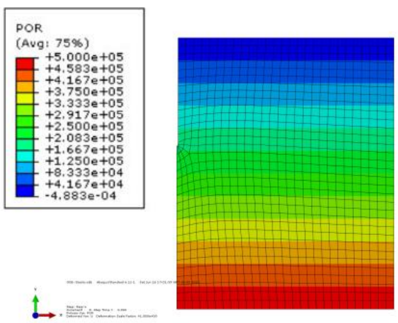

Figure 3. Initial In-situ Pore Water Pressure Distribution Prior to Cavity Expansion

The PWP increases along the pile length after the installation of driven pile or the cavity expansion are shown in Figure 4 below.

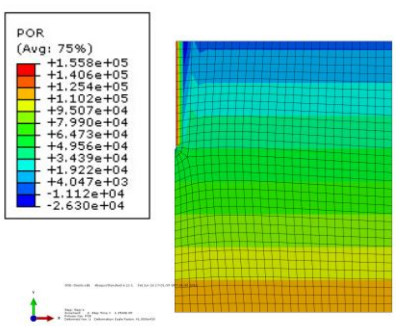

Figure 4. Pore Water Pressure during Cavity Expansion at t = 0.01 hour

4.3 FEM Results and Comparison with Field Measurements

After having studied all the relevant parameters included in this numerical model, the comparison between the field observation data and numerical analysis results have been undertaken.

Figure 5 provides the comparison between the estimated setup percentage and capacity gain from both field observation and numerical modeling for 508 mm diameter piles.

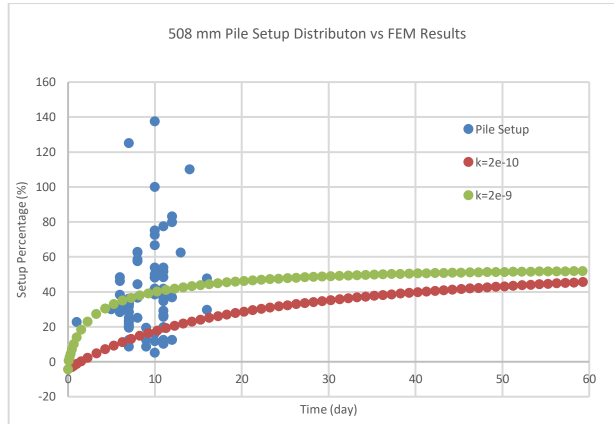

Figure 5. Pile Setup Percentage vs Time

Based on Figure 5, the setup percentage of field observation deviated more from the numerical simulation. It may be contributed to the fact that the initial pile capacities were being under estimated due to extremely low blow count readings (3 to 8 blows / 250 mm) which cannot be property estimated based on WEAP bearing graph.

In terms of the optimized pile final capacity verification period, 10 to 20 days will be the most ideal time frame. This time is highly sensitive to the permeability of the soil deposits. In sandy soils, the verification time can be reduced to 1 to 3 days. Silty soils would need about 7 days. High clay content soils may need up to 20 days of setup time.

4.4 Conclusion

This paper examines the pile capacity setup of driven steel pipe in Edmonton clay till using an integrated approach comprising dynamic load testing, wave equation analysis and finite element simulation methods. The setup time is sensitive to the soil hydraulic properties. For Edmonton clay till, 10 to 20 days may be the most ideal time frame. For a piling project with clayey subgrade soils, it is desired to have a 20-day window for final pile capacity verification. For a site with silty clay or sand clay subgrade conditions, 10 days or less should be sufficient for capacity verification purpose.

In addition, different analysis method yields different estimation or prediction on the gain in pile capacity. It is recommended that several different methods should be used to provide bounds on the gain in pipe capacity.

4.5 References

Andriashek, L. (1988). *Quaternary Stratigraphy of the Edmonton Map Area.* Terrain Sciences Department, Natural Resources Division Alberta Research Council, Open File Report #198804.

ASTM D4945-17. (2017). *Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.* West Conshohocken, PA: ASTM.

GRL-WEAP. (2010). Wave Equation Analysis Program. Developed by GRL Engineers Inc., Cleveland, Ohio.

Linkins, G. R. (2004). *Correlation of CAPWAP with Static Load Test.* Seventh International Conference on the Application of Stress Wave Theory, (pp. 153-165). Kuala Lumpur, Malaysia.

Randolph, M. (1979). *Driven piles in clay - the effects of installation and subsequent consolidation*. Geotechnique 29 No.4, 361-393.

Rausche, F. G. (1985). *Dynamic determination of pile capacity*. Journal of the Geotechnical Division, ASCE, Vol 111, Issue 3, 367-383.

Smith, E. (1960). *Pile driving analysis by the wave equation*. Journal of the Soil Mechanics and Foundations Division, ASCE, Paper No. 3306 Vol. 127 Part 1, 1145-1193.

Terzaghi, K. (1925). *Erdbaumechanik auf Bodenphysikalischer Grundlage*. Franz Deuticke,. Liepzig-Vienna.