Comparing Axial Behaviour of Non-Displacement and Displacement Piles using Field Load Tests

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ABSTRACT

There has been consensus on the enhanced geotechnical capacity of displacement piles as compared to non-displacement piles due the soil improvement effect of the former pile type by virtue of radial displacement along the shaft and pre-loading at the tip of the pile. Many researchers have developed different models over time to estimate the enhancement of geotechnical capacity of partial and full displacement piles. The results of such research have always been reported in terms of revised values for the empirical coefficients used in already existing pile design methods. However, there is a basic lack of evidence in comparison of geotechnical capacities of displacement and non-displacement piles as both types are seldom used on same project and at best the comparison has been made for similar soil conditions in the nearby areas or projects. The objective of present research is thus to compare the geotechnical capacities of various pile types in same soils. Four axial compression load tests were conducted on heavily instrumented piles, one each of drilled cast-in-place (CIP), continuous flight auger (CFA), drill displacement steel pile (DDSP) and drilled displacement (DD) type of same length and similar dimensions installed at one cohesive site in Alberta to eliminate the soil differing and scale effects. The paper evaluates the axial capacities and the load-transfer mechanisms of all four types of piles during axial load tests to assess the increase in geotechnical capacity of the displacement piles in comparison with non-displacement pile and evaluates the existing design methods adopted for these types of piles.

RÉSUMÉ

Il existe un consensus sur la capacité géotechnique renforcée des piles de déplacement par rapport aux Piles de non-déplacement, grâce à l'effet d'amélioration du seuil en vertu d'un déplacement radial le long de l'arbre et un pré-chargement à la base du pieu. Plusieurs chercheurs ont développé différents modèles pour estimer le renforcement de la capacité géotechnique des pieux partiellement et complétement enfoncés. Les résultats de ce type de recherches ont été présentés en termes de valeurs empiriques révisées pour les méthodes déjà existantes de conception de piles. Cependant, il y a un déficit d'information sur le sujet des comparaisons entre les capacités géotechniques des piles de déplacement et non enfoncés comme tous les deux sont rarement utilisés dans le même projet et les meilleures comparaisons ont été faites à partir de conditions de seuils semblables dans des régions voisines. L'objectif de la recherche actuelle est de comparer les capacités géotechniques de différents types de piles de mêmes sols. Quatre essais de charge de compression axiale ont été réalisées sur pilotis fortement instrumentés, un chacun percé coulé en place (CIP), tarière continue (CFA), forage pieux d'acier de déplacement (DDSP) et forés déplacement (DD) type de même longueur et dimensions similaires installées sur un site cohérent en Alberta pour éliminer les sols différents et effets d'échelle. Cet article scientifique mesure les capacités axiales et les mécanismes de transfert de charge des quatre types de pieux lors des essais de chargement axial, afin d'évaluer l'augmentation de la capacité géotechnique des pieux enfoncés. Par ailleurs, on fait aussi une comparaison entre les méthodes de conception existantes, adoptées pour ces types de piles.

1 INTRODUCTION

There has been continuous research to enhance the knowledge and understanding of factors governing the axial capacities of piles. In general, axial capacities depend upon soil type, pile type, pile surface roughness, and method of installation and so on. Displacement or densification effects of pile installation can significantly affect the pile capacities and thus classify piles into non-displacement, partial displacement, and full displacement piles. Examples of non-displacement piles are drilled cast-in-place or bored pile that is constructed by drilling an open shaft and pouring the concrete into it upon completion. The stabilization of open pile shaft sometimes required a temporary casing or slurry to support the wall of the shaft and prevent its sloughing which led the industry to invent the continuous flight auger (CFA) or auger cast-in-place (ACIP) piles. The CFA or ACIP piles are partial displacement piles as some degree of densification is achieved during drilling by displacing part of the excavated soil radially and some due to pouring of concrete under pressure during auger withdrawal, causing the pile shaft to be bigger than the nominal auger diameter. The full displacement piles can be divided into two categories: concrete and steel displacement piles. The drilling tool for concrete displacement piles (or DD) pile typically consists of a) soil displacement body, b) a helical, partial-flight auger segment and c) a specially designed sacrificial tip attached to the bottom of the tool. The partial auger assists in penetration, the displacement body provides the densification of the soil and the sacrificial tip is released once the drilling is complete. The concrete or grout is placed as the drilling tool is withdrawn and the piles will develop the pile axial capacity after the curing of the concrete or grout (Salgado 2008). Steel displacement piles, also known as drilled displacement steel pile (DDSP, Shah and Deng 2015) is a recent addition to DD piles. DDSP are made of a specially design conical tip welded to the tip of steel tube, which has some cutting teeth and a partial helix to drill and transport the soil upward where the steel tube pushes the soil radially to complete the displacement process.

Research projects in the literature devoted to the DDSP or to comparing displacement piles performance with other conventional pile types in the same soil profile have been fairly limited. This leads to in a lack of knowledge about the axial load-transfer of these piles and related soil improvement. To bridge the knowledge gap, the present paper reports the field load test program of a DDSP, a CIP, DD, and CFA pile that have the same length and similar dimensions installed at one cohesive site in Alberta. All piles are instrumented with multiple strain gauges stations that provided internal shaft load and the average skin adhesion. The main objectives are to 1) compare the pile axial capacities and load-transfer mechanisms, 2) assess the effects of pile installation process, and 3) evaluate the current design methods in predicting the axial capacities.

2 SUBSURFACE CONDITIONS

The pile load tests were carried out at a site located in Acheson, Alberta, within the Greater Edmonton Area as shown in Figure 1.

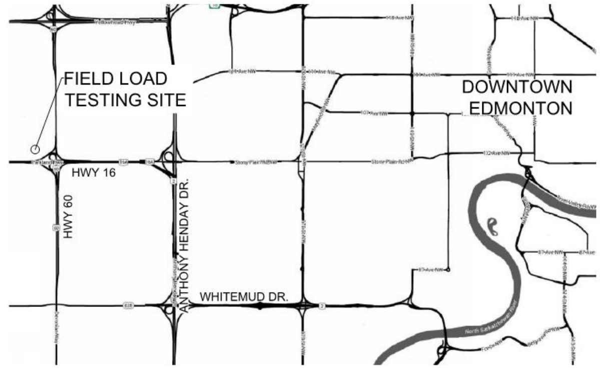

Figure 1. Location of field load testing site

The regional geology of Greater Edmonton Area where the pile load tests were carried out has been generally elaborated by Kathol and McPherson (1975). The basic stratigraphy in test site consists of successive glacial deposits overlying sand formation underlain by bedrock of clay shale. Subsurface geotechnical investigations were carried out using the standard penetration tests (SPT) and laboratory tests of disturbed samples to characterize the soil types and geotechnical properties of subsurface soils illustrated in Figure 2.

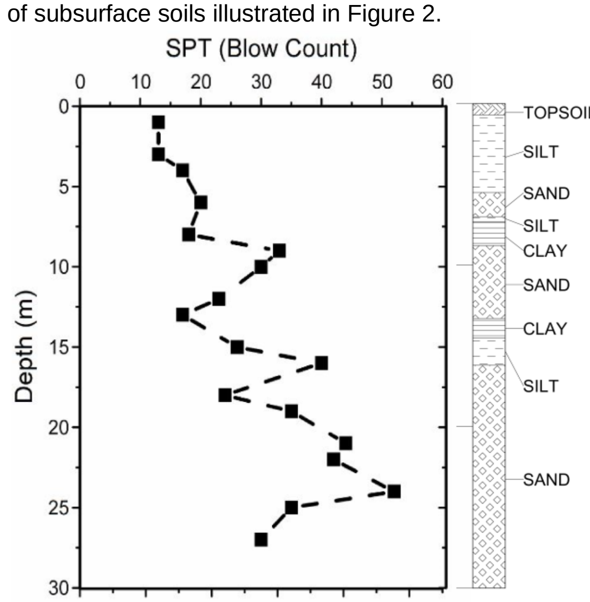

Figure 2. Soil stratigraphy at test site

It is observed that subsurface soils consist of topsoil of 0.7 m thickness overlying a 4.8 m thick silt layer underlain by interbedded layers of clay, sand, and silt. The silt underlying the topsoil was clayey with trace of fines and interbedded sand, low plastic, moist and compact, extending to a depth of 5.5 m below the ground surface (BGS). The interbedded layers of silt, sand and clay were found from 5.5 m to 8.8 m BGS. There was a

4.5 m thick sand layer below the interbedded silt, sand and clay which was fine grained, damp to moist and compact to dense. A 1.2 m thick, moist, stiff clay layer and a 1.5 m thick, clayey, wet silt layer were encountered below the sand layer. Sand layer was encountered at a depth of 16 m below grade which extended to the end of the borehole. The sand was silty with thin clay and silt lenses, fine grained, wet and dense

3 FIELD TEST PROGRAM

Four test piles designated as TP1 to TP4 of selected types (Figure 3) and reaction piles were installed in January 2016 as per layout shown in Figure 4, field load tests were conducted three weeks after the installation to enable the soil setup. Test pile configuration is summarized in Table 1. The DDSP (TP1) is composed of steel pipe filled with concrete; it has the nominal outer diameter of 324 mm and wall thickness of 9.53 mm. All other test piles (TP 2 to TP4) are reinforced concrete pile having a nominal diameter of 406 mm.

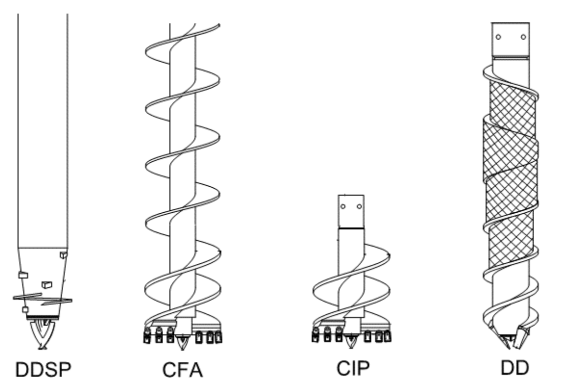

Figure 3. Drilling tools for selected types of test piles

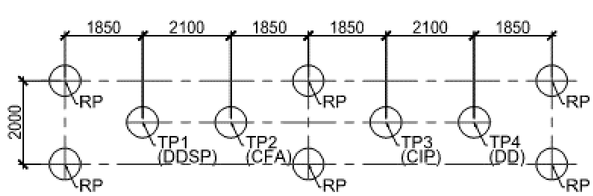

Figure 4. Test pile layout (RP = reaction pile)

Table 1. Configuration of Test Piles

Code	Pile Type	Length (m)	Diam. (mm)
TP1	DDSP	12.0	324
TP2	CFA	12.0	406
TP3	CIP	12.0	406
TP4	DD	12.0	406

3.1 Instrumentation

In order to measure the skin friction at different levels along the pile shaft and end bearing at the tip of the pile, vibrating wire strain gauges were attached to a Dywidag bar of 36 mm nominal diameter at selected levels (Figures 5). The bar was placed inside the pile shaft and backfilled with concrete.

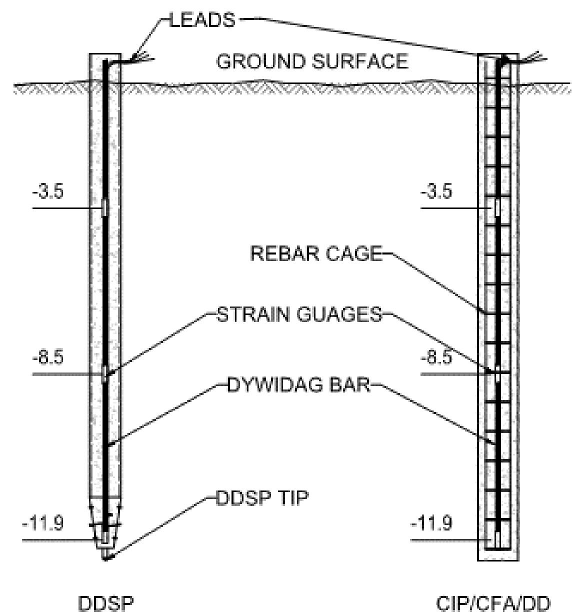

Figure 5. Schematic of instrumentation (dimension: m)

The axial strain of the pile shaft during axial loading tests will be transferred to the bar and measured by the strain gauges. The measurement of shaft strains will be used to estimate the load transfer mechanism of the piles. In addition to the strain gauges a load cell was used to record the load at the pile head and two displacement transducers were used to measure the pile head movement.

3.2 Test setup and procedures

Field load tests were conducted in accordance with ASTM D1143M-07 (ASTM 2007) quick test procedures. Compressive loads were applied to the test piles using a

4500-kN capacity hydraulic jack reacting against the underside of the reaction beam shown schematically in Figure 6.

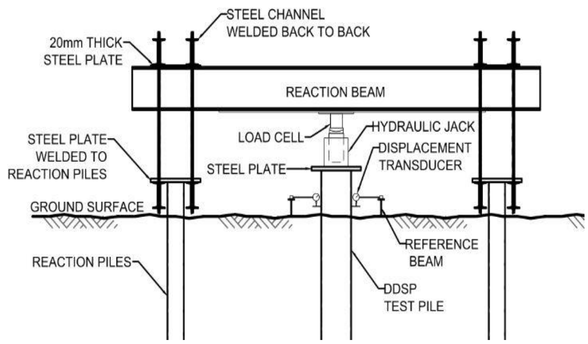

Figure 6. Compression load test set-up

Load vs. displacement curves at the pile head were recorded manually and automatically using the instruments. In addition to applied load and pile head settlement, the data logger was recording the micro strain on all strain gauges. The tangent modulus analytical method (Fellenius 1989, 2001) was adopted to convert the measured strain into the axial load at each gauge level to understand the load transfer mechanism and shaft resistances at various depth. The mobilized end bearing was interpreted from the load distribution diagram by assuming a similar value of shaft resistance was mobilized along the bottom portion of the pile shaft (below the deepest strain gauge), the remaining portion of the applied load was assumed to be generated as end bearing.

4. PILE LOAD TEST RESULTS

All four piles were loaded until geotechnical failure characterized by plunging behavior when continuous or progressive increase in pile head settlement was observed without an increase in applied load. The first test pile designated TPI carried a load of 1650 kN at limit state as shown in Figure 7. It was observed from the strain gauge records (Figure 8 and 9) that the DDSP experienced an average shaft resistance at limit state q_{sL} ranging from 30 to 200 kPa along the shaft; the end bearing resistance at limit state q_{bL} was calculated to be 4000 kPa.

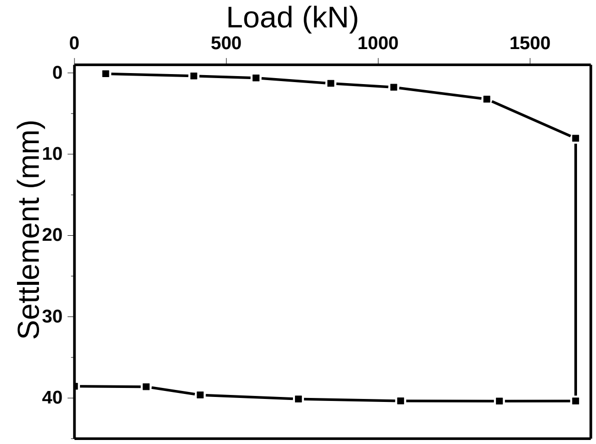

Figure 7. Load vs settlement curve for TP1 (DDSP)

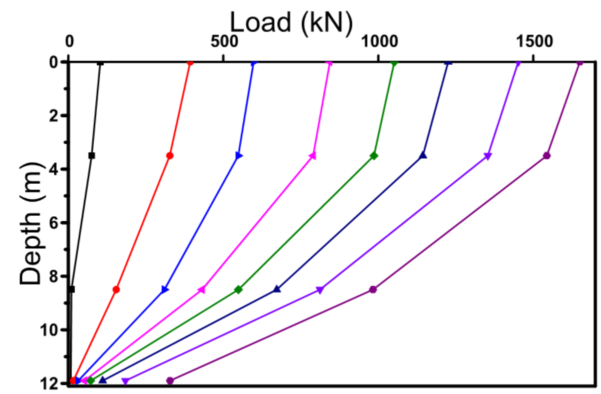

Figure 8. Development of internal load distribution for TP1 (DDSP)

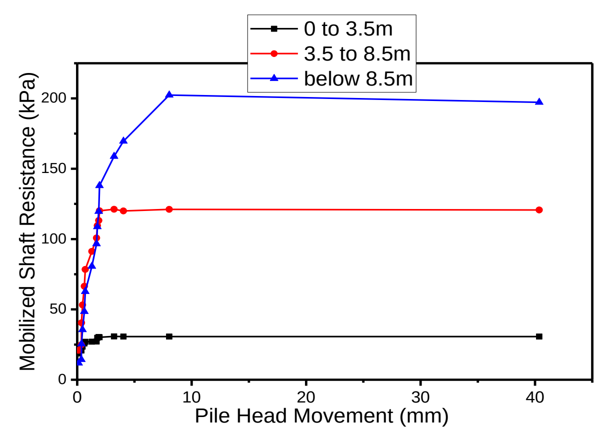

Figure 9. Mobilized shaft resistance for TP1 (DDSP)

The field test results of TP2 (CFA pile) are shown in Figures 10 to 12. The total bearing capacity at limit state Q_L 2046 kN (Figure 10). The strain gauges' data (Figure 11 & 12) showed that the pile exhibited average q_{sL} ranging from 25 kPa to 180 kPa and an end bearing value of 3500 kPa.

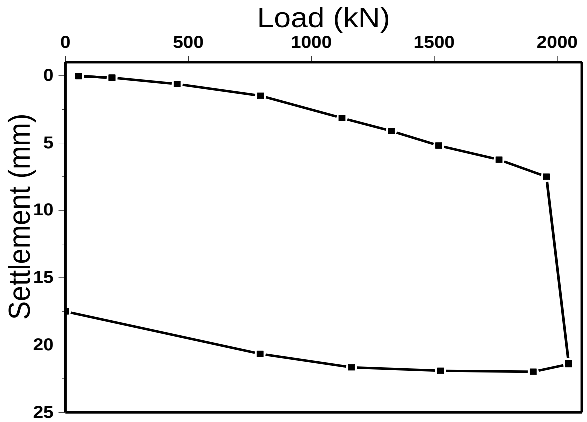

Figure 10. Load vs settlement curve for TP2 (CFA)

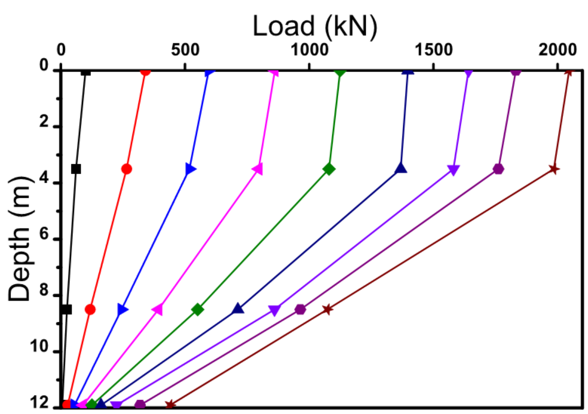

Figure 11. Development of internal load distribution for TP2 (CFA)

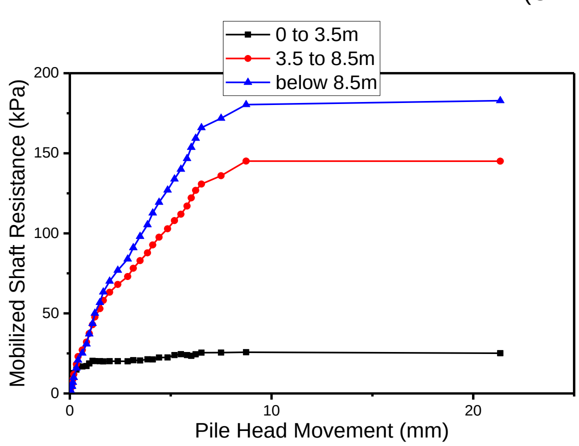

Figure 12. Mobilized shaft resistances for TP2 (CFA)

Test results of TP3 (CIP pile) are shown in Figures 13 to 15. It is seen that at the overall limit state capacity Q_L was 1360 kN (Figure 13), with q_{sL} ranging from 20 kPa to 115kPa (Figure 14 & 15) and q_{bL} of 1150 kPa.

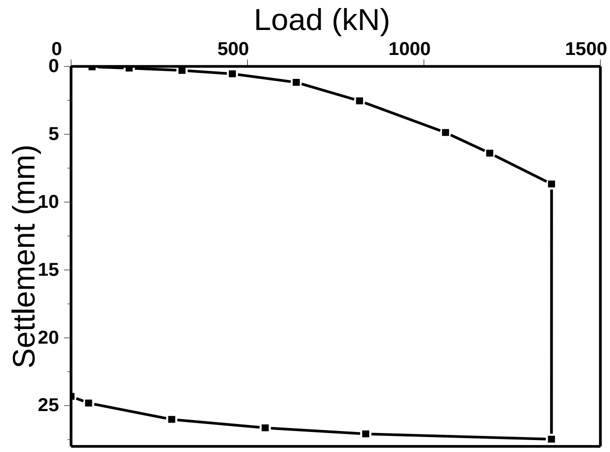

Figure 13. Load vs settlement curve for TP3 (CIP)

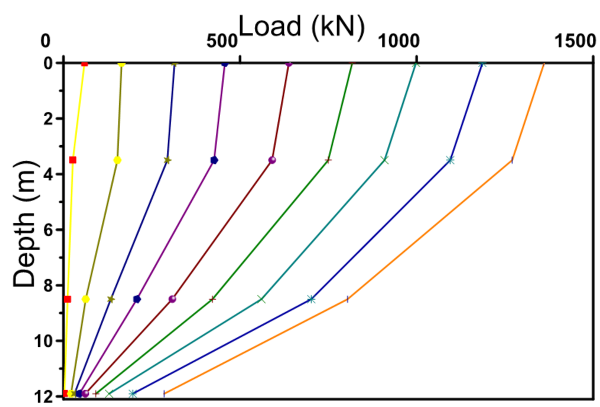

Figure 14. Development of internal load distribution for TP3 (CIP)

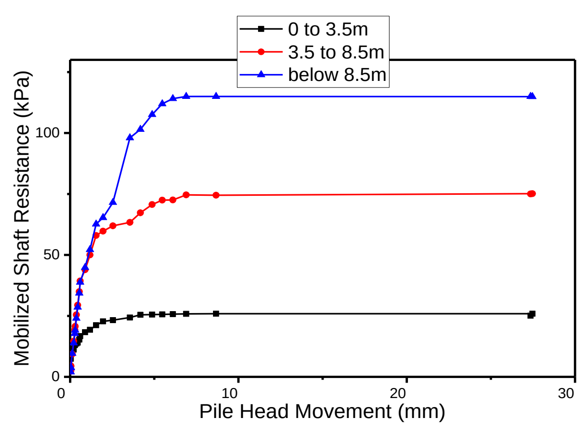

Figure 15. Mobilized shaft resistances for TP3 (CIP)

Test results of TP4 (DD pile) are shown in Figures 16 to 18. The pile experienced a Q_L of 2090 kN (Figure 16) with q_{sL} of 25 to 185 kPa (Figure 17 & 18) and q_{bL} of 3500kPa.

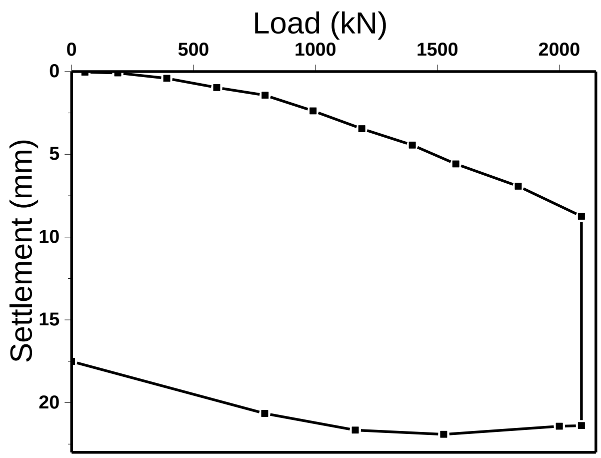

Figure 16. Load vs settlement curve for TP4 (DD)

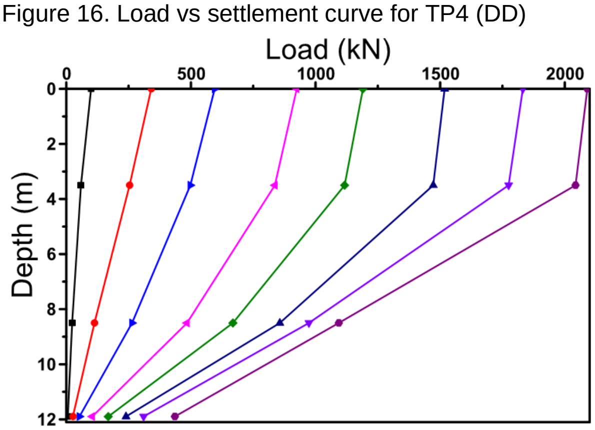

Figure 17. Development of internal load distribution for TP4 (DD)

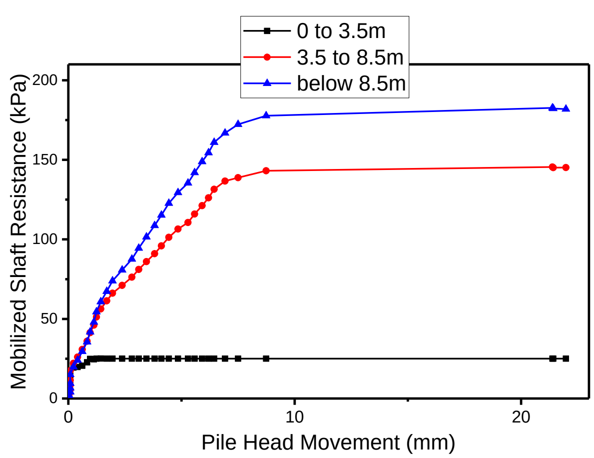

Figure 18. Mobilized shaft resistances for TP4 (DD)

5. EVALUATION OF DESIGN METHODS

The total stress method as recommended in both Federal Highway Administration Authority (FHWA 2007) and Canadian Foundations Engineering Manual (CFEM 2006) suggested Equation [1] and [2] to be used for estimating the capacity of non-displacement or cast –in-place (TP3) and partial displacement piles or continuous flight auger pile (TP2):

$$q_s = \alpha\, S_u \qquad [1]$$
$$q_b = N_t\, S_u \qquad [2]$$

where q_s is the limit state shaft resistance, q_b is the limit state end bearing, Su is the undrained shear strength of the soil and α & N_t is a dimensionless factor.

The Brettmann and NeSmith (2005) SPT-based design method for drilled displacement (DD) piles that is adopted by Federal Highway Administration Authority (FHWA 2007) was used for estimation of both drilled displacement steel pile (TP1) and drilled displacement concrete pile (TP4). The limit state shaft resistance and end bearing values are given by Equations [3] and [4] respectively.

$$q_s = 5N + W_s \qquad [3]$$
$$q_p = 190N_{60} + W_T \qquad [4]$$

where q_s is shaft resistance and q_p is end bearing, N is the average SPT value along the pile shaft for shaft resistance and N_{60} is the average SPT blow count at pile toe for estimation of end bearing, and W_s & W_T are empirical constants in Brettmann and NeSmith (2005).

Furthermore, the SPT based formulae for estimation of limit state shaft resistance and end bearing for full displacement piles reported in Canadian Foundation Engineering Manual (CFEM 2006), shown in Equations [5] and [6] respectively were also used.

$$q_s = \alpha(2.8N_{60} + 10) \qquad [5]$$
$$q_b = K_b N_b \qquad [6]$$

where α is an empirical constant equal to unity for displacement piles, N_{60} is the average SPT blow count (normalized to 60 % of energy efficiency) along the pile shaft for shaft resistance and N_b is the average SPT blow count in the vicinity of pile toe and Kb is another dimensionless empirical constant.

The comparison of estimated values using the above referenced methods and measured values is summarized in Table 2 which shows the measured results for non-displacement pile (TP3) are lower than the estimated values for the top layer and higher for the lower layers as well as end bearing. The measured shaft resistances for continuous flight auger pile (TP2) exhibit the same trend of being lower than estimated for the top layer and higher than estimated foe the lower layers but with a much wider gap and the measured end bearing is slightly lower than estimated value.

The measured shaft resistance for the two displacement piles (TP1 and TP4) are also lower than both estimated values for the top layer but higher for the lower layers. On the other hand, the estimated values calculated using the CFEM method are very conservative and may lead to erroneous design. The end bearing values also reveal the same trend.

Table 3. Comparison of Measured and Estimated Geotechnical Parameters

Pile code	Depth (m)	Shaft Resistance FHWA Method (kPa)	Shaft Resistances CFEM Method (kPa)	Shaft Resistances Measured (kPa)	End Bearing FHWA Method (kPa)	End Bearing CFEM Method (kPa)	End Bearing Measured (kPa)
TP1 (DDSP)	0 to 3.5	65	46.4	30			-
	3.5 to 8.5	91.66	61.33	120	-		-
	8.5 to 12.0	143.33	90.26	200	3800	3300	4000
TP2 (CFA)	0 to 3.5	42.5	42.5	25			-
	3.5 to 8.5	60.5	60.5	145			-
	8.5 to 12.0	94.6	94.6	180	3800	1080	3500
TP3 (CIP)	0 to 3.5	42.9	42.5	20			-
	3.5 to 8.5	60.5	60.5	75			-
	8.5 to 12.0	94.6	94.6	114	1080	1200	1150
TP4 (DD)	0 to 3.5	65	46.4	25			-
	3.5 to 8.5	91.66	61.33	145			-
	8.5 to 12.0	143.33	90.26	185	3800	3300	3500

In order to evaluate the densification effect of partial and full displacement piles, a comparison was carried out with using the non-displacement pile (TP3) as a bench mark which is presented in table 4. The results show a similar enhancement effect for both types of full displacement piles (TP1 and TP4) and nearly same enhancement effect for the partial displacement piles (TP2) installed using the continuous flight auger (CFA) technique.

Table 4. Comparison of Measured Geotechnical Parameters for Displacement Piles with the Non-displacement Pile (TP3)

Item	TP3 (CIP)	TP1 (DDSP)	Increase (%)	TP2 (CFA)	Increase (%)	TP4 (DD)	Increase(%)
Average q_{sL} 0m to 3.5m (kPa)	20	30	50	25	25	25	25
Average q_{sL} 3.5m to 8.5m (kPa)	75	120	60	145	93	145	93
Average q_{sL} 8.5m to 12.0m (kPa)	115	200	73	180	56	185	60
q_{bL} (kPa)	1150	4000	250	3500	205	3500	205
Total Capacity -Q_L (kN)	1360	1650*	55*	2046	50	2090	53

* Calculated for 12m long pile of 406mm diameter

6. CONCLUSIONS

Field load tests of four pile types were carried at a cohesive site in Alberta, with the objective of comparing the axial behaviour of the different displacement pile types to non-displacement pile of similar dimensions. Following conclusions can be drawn:

1. DD and DDSP have shown enhanced geotechnical resistance in both shaft resistance and end bearing due to the soil improvement effects obtained as a result of the soil densification caused by the radial displacement of the excavated materials and the preloading of the soil at the pile tip.

2. The shaft resistance in upper layers (0 to 3.5 m) is not much enhanced which may be due to lack of overburden to help the densification by radial displacement and end bearing should be selected conservatively for reduction in pile tip dimension resembling the drilling tip.

3. Testing results are predicted well by the SPT-based design method for drilled displacement concrete piles which is adopted by FHWA.

4. The Continuous flight auger should be treated as partial displacement piles and historical concrete overage in similar soils should be taken into effect to predict their capacity.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

American Society for Testing and Materials (ASTM). 2013. *ASTM D1143 / D1143M – 07, Standard Test Methods for Deep Foundations under Static Axial Compressive Loads.*

Basu P., Prezzi, M., and Basu, D. 2010. Drilled displacement piles – current practice and design, *DFI Journal,* 4(1): 3-20.

Basu, P., and Prezzi, M. 2009. Design and applications of drilled displacement piles, Final Report, FHWA/IN/JTRP-2009/28, *Joint Transp. Res. Program, Indiana Dept. of Transp. and. Purdue University*, West Lafayette, Indiana, USA.

Brettmann, T., and NeSmith, W. 2005. Advances in auger pressure grouted piles, design, construction and testing. *Geotechnical Special Publication No. 129,* ASCE, 262 – 274.

Brown, D.A., and Drew, C. 2000. Axial capacity of augured displacement piles at Auburn University, Geotechnical Special Publication *No. 100, Eds. N. D. Dennis, R. Castelli and M. W. O'Neill, ASCE,* 397-403.

Brown, D. A. 2005. Practical considerations in the selection and use of continuous flight auger and drilled displacement piles. Geotechnical Special Publication No. 129, ASCE, 251-261.

Canadian Geotechnical Society. 2006. *Canadian Foundation Engineering Manual*, 4th Edition, Richmond, B.C., Canada.

Federal Highways Authority (FHWA). 2007. Design and Construction of Continuous Flight Auger Piles, *Geotechnical Circular No. 8*.

Fellenius, B.H. 1989. Tangent modulus of piles determined from strain data. *The American Society of Civil Engineers, ASCE, Geotechnical Engineering Division, Foundation Congress,* 500-510.

Fellenius, B. H., Brusey, W. G., & Pepe, F. 2000. Soil set-up, variable concrete modulus, and residual load for tapered instrumented piles in sand. *Geotechnical Special Publication*, *ASCE*: 98-114.

Fellenius, B. H. 2001. From strain measurements to load in an instrumented pile. *Geotechnical News Magazine,* 19(1): 35-38.

Kathol, C.P., and McPherson, R.A. 1975. Urban Geology of Edmonton. *Alberta Research Council,* Bulletin 32.

Salgado, R. 2008. *The Engineering of Foundations.* McGraw Hill Inc. New York, NY, USA.

Shah, F. and Deng, L. 2015. In-situ Axial Load Tests of Drilled Displacement Steel Piles. CGS Annual Conference, September 2015 Quebec, QC.