In-situ Axial Load Tests of Drilled Displacement Steel Piles

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ABSTRACT
Drilled Displacement Steel Pile (DDSP) is an innovative pile type that fully displaces the soil during installation. DDSP consists of a conical tip with steel blades and a steel tubular pipe. Compared to conventional drilled displacement piles, this pile type has many advantages that include rapid installation, immediate usability, and greater pile capacities. Consequently, DDSP has recently been widely used in varieties of engineering applications in Western Canada. However, the axial behaviour of DDSP has not been well understood due to the lack of research, and a design guideline has not been developed. A series of in-situ load tests of full-scale piles were conducted at two testing sites in Alberta. Three axial compression and two tensile tests were conducted on heavily instrumented piles. The paper evaluates the axial capacities and the load-transfer mechanisms during axial loading tests and compares the existing design methods adopted for other pile types.

RÉSUMÉ :
Le pieu foré en acier (Drilled Displacement Steel Pile, DDSP) est un type novateur de pieux qui déplace complètement le sol lors de son installation. Le DDSP est formé d'une pointe conique avec des lames et un tube en acier. Par rapport aux pieux forés conventionnels, ce type de pieu possède de nombreux avantages incluant une installation rapide, une exploitabilité immédiate et de plus grandes capacités. En conséquence, le DDSP a récemment été largement utilisé dans diverses applications de l'ingénierie dans l'ouest du Canada. Cependant, le comportement axial du DDSP n'a pas été bien compris vue le manque de recherche et vue qu'aucun guide de conception n'a été établi. Un ensemble d'essais de chargement en place a été réalisé à l'échelle réelle en deux sites en Alberta. Trois essais en compression et deux essais en tension ont été réalisés sur des pieux largement instrumentés. L'étude évalue les capacités axiales et le mécanisme de transfert de charge pendant des essais de chargement axial et compare les méthodes de conception existantes adoptées pour d'autres types de pieux.

1. INTRODUCTION

The use of piles to transmit the structural loads to underlying soils is a conventional method in foundation engineering. There has been continuous improvement in the pile design and construction with regards to the material, methods of installation and design procedure. Pile foundations can be classified into three categories: non-displacement piles, partial displacement piles and displacement piles (Salgado 2008). Drilled displacement piles are one of the most commonly used partial displacement piles. Basu and Prezzi (2009) summarized existing drilled displacement piles available in the industry, such as the Auger Pressure-Grouted Displacement piles (Brettmann and NeSmith 2005), Atlas (Bottiau 2006), De Waal (Huybrechts and Whenham 2003), Fundex (Van Impe 2004), Olivier (Holeyman and Charue 2003), and Omega piles (Bottiau et al. 1998). Typically, the drilling tool of conventional drilled displacement piles contains: 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).

The growing economy of Western Canada requires pile foundations that can be installed and loaded in a short period. Drilled displacement steel piles (DDSP) are an innovative pile type that has recently been used in North America in the past decade. DDSP is composed of a conical tip and steel tubular pipe shaft. The conical tip is welded with blades and cutter teeth to drill and transport the soils from the pile tip during installation and thus reduces the installation resistance. Unlike conventional drilled displacement piles, DDSP are made of a specially design conical tip welded to the tip of steel tube, which drills and transports the soil and the steel tube pushes the soil radially to complete the displacement process. DDSP provides a viable alternative to conventional drilled displacement concrete piles, especially in the cases where lateral capacity is required such as in case of seismic loading (Tuladhar et al. 2008). DDSP has higher structural capacity by virtue of being made of steel pipe and better quality assurance as the quality of the end product is not highly dependent upon concrete equipment operator (Brown 2005).

Upon completion of the drilling, the steel shaft of DDSP will remain in the ground to avoid the re-shearing of the soil surrounding the steel shaft during the removal or withdrawal process. The innovative pile type may have many advantages over conventional drilled displacement concrete piles, especially in Western Canada's soils. The advantages include the rapid installation, large axial and lateral capacity, and immediate usability after installation.

Although DDSP have been widely used in practice, there is a lack of the research or general design guideline for this pile type. To investigate the bearing capacity and load-transfer mechanisms of DDSP, a series of in-situ load tests of full-scale piles were conducted on heavily instrumented piles installed at two testing sites in Alberta. The soil subsurface conditions based on the standard penetration tests (SPT) were presented. The present paper analyzes the axial load vs. displacement curves of testing piles and evaluates the load-transfer mechanism. The paper evaluates the axial capacities using two existing design methods.

2. SUBSURFACE CONDITIONS

Pile load tests were carried out at two sites located in Alberta. The first site is located in Acheson industrial area near Edmonton and the second is on the campus of the University of Alberta. Figure 1 shows the locations of the testing sites. The regional geology for Edmonton where the pile load tests were carried out has been generally elaborated by Kathol and McPherson (1975). However, site specific geotechnical investigations were carried out at both sites using the standard penetration tests and laboratory tests of disturbed samples to characterize the soil types and mechanical properties of subsurface soils.

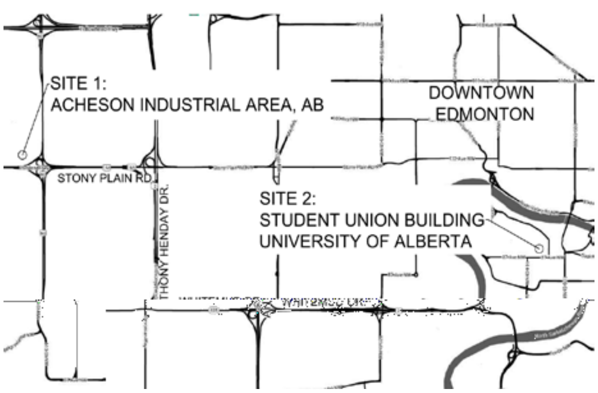

Figure 1. Location map of two test sites

The SPT blow count and the soil types at Site 1 are shown in Figure 2. It was observed that Site 1 consists of topsoil that is 0.7 m thick and contains clay and silt with organics 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 grade. The interbedded layers of silt, sand and clay were found from 5.5 m to 8.8 m below grade. 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.

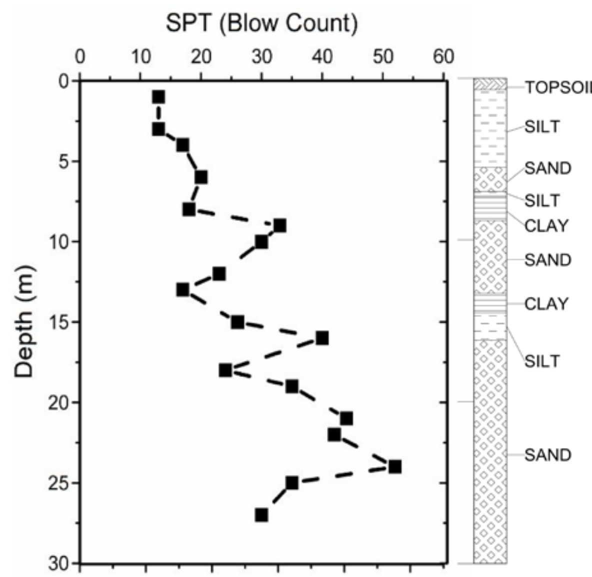

Figure 2. Soil stratigraphy at Site 1

The SPT blow count and the soil types at Site 2 are shown in Figure 3. The stratigraphy of Site 2 consists of top soil approximately 3 m thick fill made up of alternating layers of sand/gravel and clay fill varying in thickness from 0.2 m to 1.1 m which is underlain by a 0.75 m thick layer of brown, silty, fine grained sand having inclusions of trace oxides overlying a 1.5 m thick layer of firm brown clay with dark brown silt inclusions and oxides. Under the clay layer, there was 4.6 m thick sand having compact and dense composition. The sand layer was found to have occasional thin lenses of coal, silt and clays. Another 0.40 m thick clay layer was found under the sand having dark brown color and very stiff consistency with some coal chips, trace oxides and fine sand inclusions overlying a 3.60 m thick layer of dark brown, dense, compact and highly dilatant fine sandy silt. Below the silt layer was silty, sandy clay till extending below the tip of the test pile having very stiff to very hard consistency and dark brown in color having trace pebbles, coal chips, gypsum and oxides.

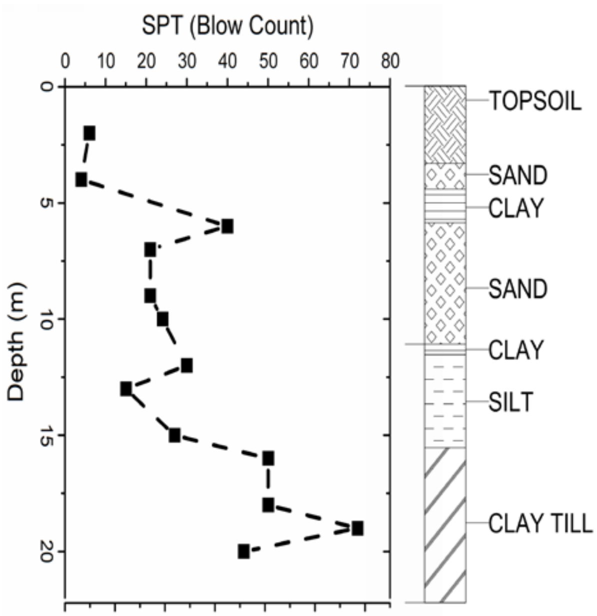

Figure 3. Soil stratigraphy at Site 2

3. IN-SITU TESTING PROGRAM

At Site 1, four test piles designated as P1, P2, P3 and P4 along with required reaction piles were installed in 2011. Two of the test piles (P1 and P3) were advanced to 24.4 m below grade and the remainder two of them (P2 and P4) to 12.2 m. At Site 2, one test pile designated as TP and four reaction piles were installed in 2014.

3.1 Test Piles

The test piles configuration and the types of load test for both sites are summarized in Table 1. Drilled Displacement Steel Pile is made of steel tubular pipes, which after installation will be filled with concrete and reinforced with partially embedded steel cage for connection to the pile caps or super structure. All test piles have the outer diameter of 324 mm and the wall thickness of 9.53 mm. The steel pipe is fitted with a specially designed closed-end conical tip welded to the bottom of the pipe shown in Figure 4. The conical tip includes a single steel-helix plate welded onto the tip along with various blades and cutting teeth. The teeth disturb the soils and the helix transports the disturbed soils upward that are radially displaced by the tubular pipe. The pile is screwed into the ground to the desired depth using a high-capacity rotary drill rig under small vertical compressive load.

Table 1. Configuration of test piles

Code	Site	Length (m)	Wall thick (mm)	O.D. (mm)	Test Type
P1	1	24.4	9.53	324	Comp.
P2	1	12.2	9.53	324	Comp.
P3	1	24.4	9.53	324	Tens.
P4	1	12.2	9.53	324	Tens.
TP	2	17	9.53	324	Comp.

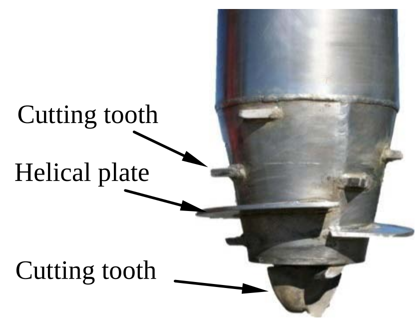

Figure 4. Typical design of conical tip of DDSP

3.2 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 later placed inside the hollow pile shaft. The pile annulus space was filled with concrete that will connect the Dywidag bar to the pile shaft in a rigid manner to the pipe. 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.

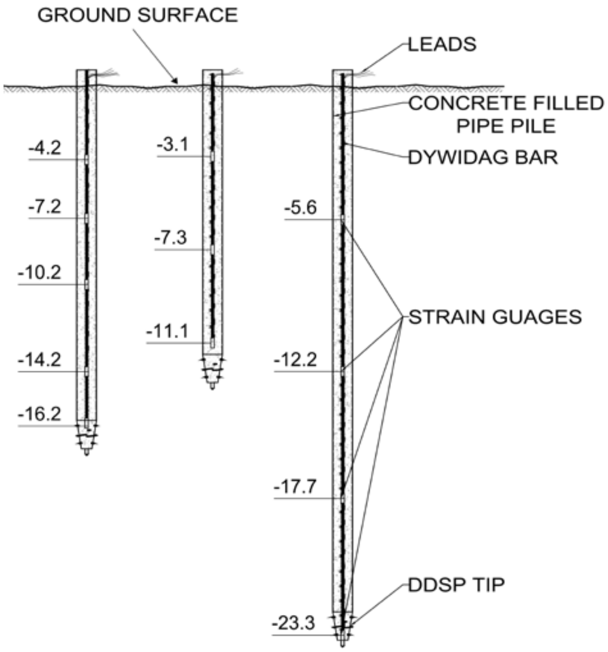

Figure 5. Schematic of instrumentation (dimension: m)

3.3 Test setup and procedures

The loading procedure was conducted in accordance with ASTM D1143M-07 (ASTM 2013) for compressive loading and ASTM D3689-07 (ASTM 2013) for tensile loading.

Compressive loads were applied to the test piles using a 4500-kN hydraulic jack reacting against the underside of the reaction beam while the required uplift resistance was provided by the reaction piles as shown in Figure 6.

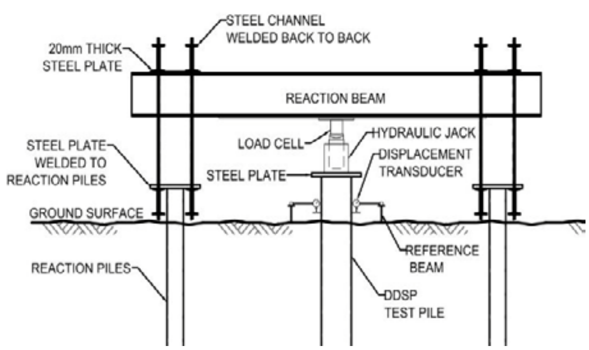

Figure 6. Compression load test set-up

Tensile loads were applied to test piles using the hydraulic jack reacting against the top of the reaction beam and transmitted to the test pile by the combination of steel plates and 36-mm-diameter Dywidag bars while the reaction piles provided the necessary seating compression to the test setup as shown in Figure 7.

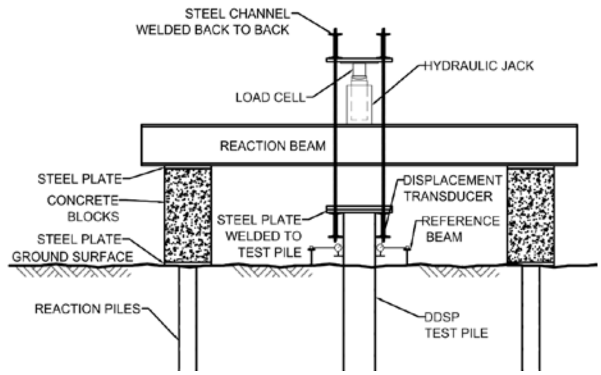

Figure 7. Tension load test set-up

4. PILE LOAD TEST RESULTS

At Site 1, four piles were loaded until failure. Piles P1 and P3 (pile length of 24.4 m) experienced structural failure in the pile shaft and piles P2 and P4 (pile length of 12.2 m) experienced geotechnical failure. Loading on pile P1 was terminated due to buckling of the pipe at the pile head. Loading on the pile P3 was terminated due to the tensile failure of the pipe at the pile head. The cause of the structural failure in P1 was that the top 600 mm pile shaft was filled with the grout which should have been the concrete instead. It was observed that under large axial loading, the uncured grout was not able to carry out the vertical loading and thus the pile shaft buckled. The failure mode of piles P2 and P4 was observed to be bearing capacity failure, when continuous and progressive settlement was observed at the pile head without increasing the axial load.

The tangent modulus analytical method (Fellenius 1989, 2001) was adopted to convert the measured strain into the axial load at each gauge level. The determination of the composite modulus of concrete and steel is complicated due to the uncertainty of the concrete modulus (Hayes and Simmonds 2002). The modulus of steel is constant; however, the modulus of concrete varies and is a function of the imposed load. As a result, the pile composite modulus is not a constant but a linear function of the imposed strain (Fellenius et al. 2000).

It was observed from the strain gauge records that at Site 1, the pile P1 experienced shaft resistance ranging from 50 to 190 kPa. Pile P2 experienced shaft resistance ranging from 50 to 215 kPa. Pile P3 experienced shaft resistance ranging from 50 to 250 kPa and pile P4 experienced shaft resistance ranging from 50 to 175 kPa along the embedded pile depth. At Site 2, the test pile TP experienced shaft resistance ranging from 25 to 190 kPa. The mobilized shaft frictions at the limit state are shown in Figure 8.

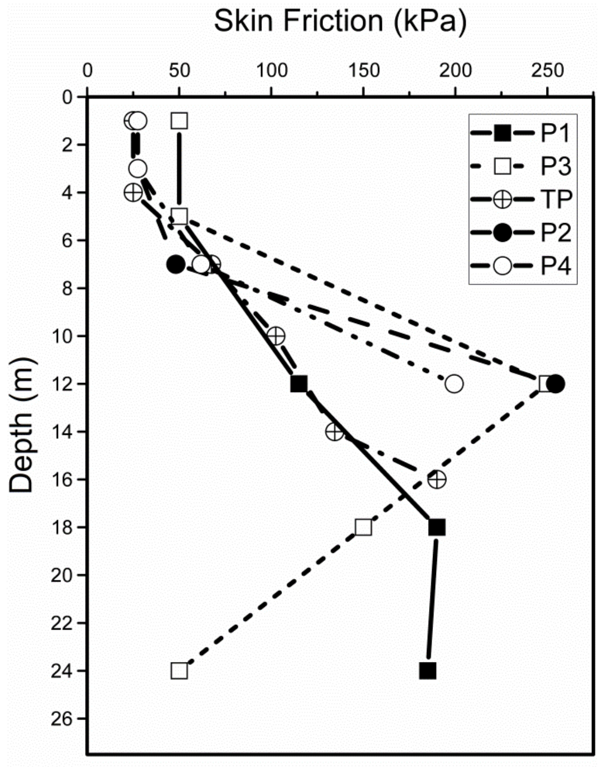

Figure 8. Mobilized Shaft Resistance at the limit state

The mobilized end bearing for compression piles can also be interpreted from the load distribution diagram. Assuming a similar value of shaft friction 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. The pile P2 showed a limit state end bearing of 4124 kPa. Pile P1 developed a negligible amount of end bearing due to the fact that the pile failed in the shaft at small settlement that was not sufficient to mobilize the limit state end bearing capacity. The pile TP developed a limit state end bearing of 3250 kPa. The load vs displacement graphs of five tests are shown in Fig. 9 to 13. Davisson (1972) offset method was used to find the ultimate limit state capacity of each pile. It was observed from the load vs settlement curves shown in Figures 9 to 13 that all test piles exhibited plunging failure when there was continuous and progressive movement of the pile head without the increase in the applied load. It was also observed that the pile P1 and P3 at Site 1 failed structurally and the full geotechnical capacity was not mobilized. The results showed enhancement in the geotechnical capacity of the soil. However the improvement in the upper layers and end bearing is not fully realized which may be due to the lack of overburden resulting in global heaving and jeopardizing of the preloading by drilling.

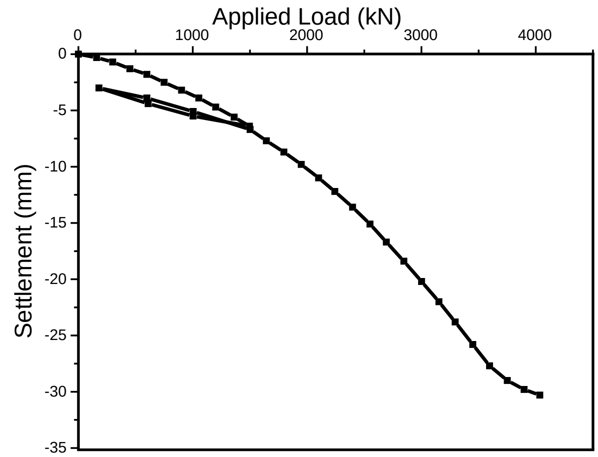

Figure 9. Load vs Settlement Curve for test pile P1

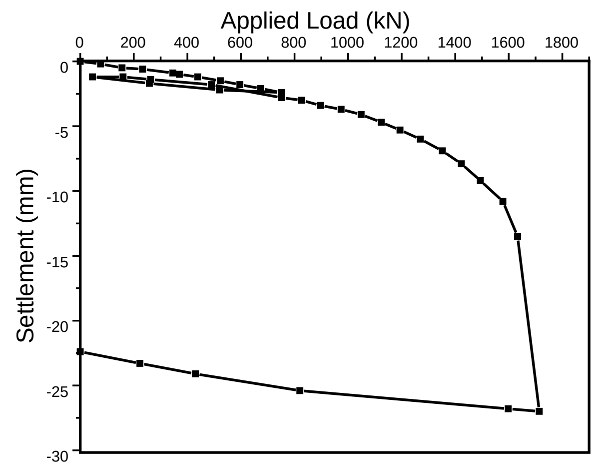

Figure 10. Load vs Settlement Curve for test pile P2

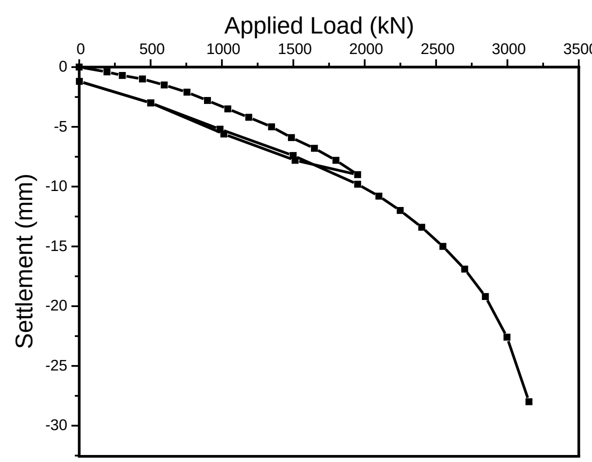

Figure 11. Load vs Settlement Curve for test pile P3

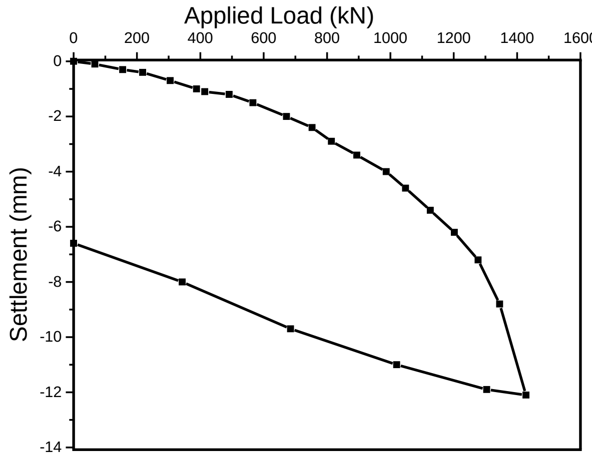

Figure 12. Load vs Settlement Curve for test pile P4

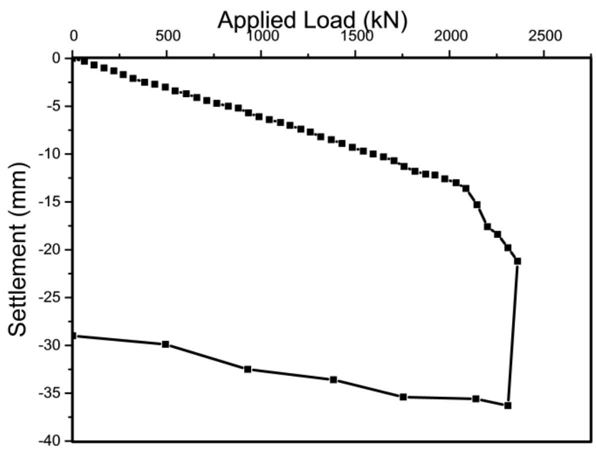

Figure 13. Load vs Settlement Curve for test pile TP

5. EVALUATION OF DESIGN METHODS

Brettmann and NeSmith (2005) developed a SPT-based design method for conventional drilled displacement concrete piles in cohesive soils. The method was adopted by Federal Highway Administration (FHWA 2007) for drilled displacement piles in cohesive soils. The ultimate limit state skin friction and end bearing values are given by Equations [1] and [2] respectively.

q_s(kPa)=5N+W_s [1]

q_p(kPa)=190N_{60}+W_T [2]

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

The empirical relations based on SPT results for ultimate state skin friction and end bearing for drilled displacement piles adopted in Canadian Foundation Engineering Manual (CGS 2006) are shown in Equations [3] and [4] respectively.

q_s (kPa)=α(2.8N_{60}+10) [3]

q_b (kPa)=$K_b N_b$ [4]

where α is an empirical constant equal to unity for displacement piles, N_{60} is the average SPT value (normalized to 60 % of energy efficiency) along the pile shaft for skin friction and N_b is the average SPT value in the vicinity of pile toe. K_b is another empirical constant (CGS 2006). Figure 14 and 15 shows a comparison of the measured soils parameters with the estimated values using the FHWA and CFEM empirical correlations for Site1 and Site 2 respectively.

The comparison shows the measured values are in close agreement with FHWA adopted empirical correlation of the SPT values. However, the empirically estimated values are higher for upper layers which can be attributed to global heaving instead true radial displacement of soil in those shallow layers where the overburden is not heavy enough to stop heaving. On the other hand the empirical values calculated using CFEM correlation are very conservative and may lead to erroneous design. The end bearing values shows somehow an irregular relationship with both empirical estimations and may require the engineering judgment based on sound local experience.

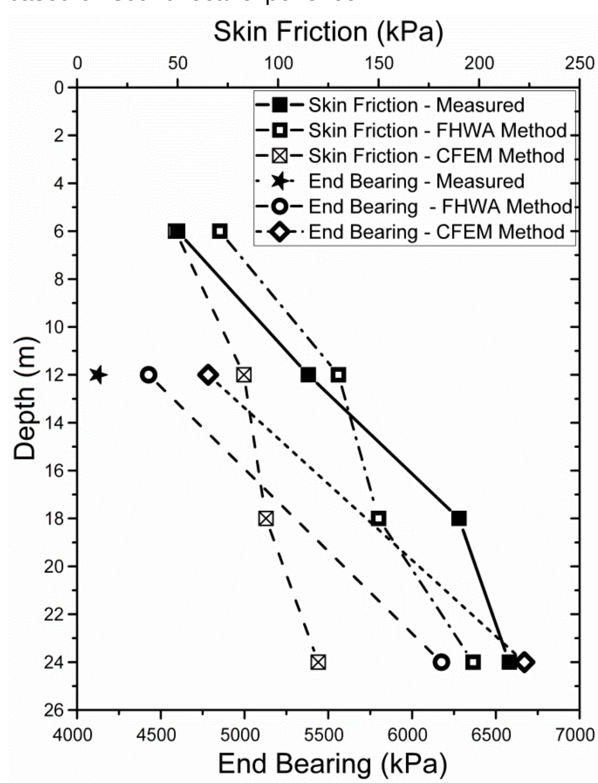

Figure 14. Comparison of the measured and empirically estimated values for Site 1

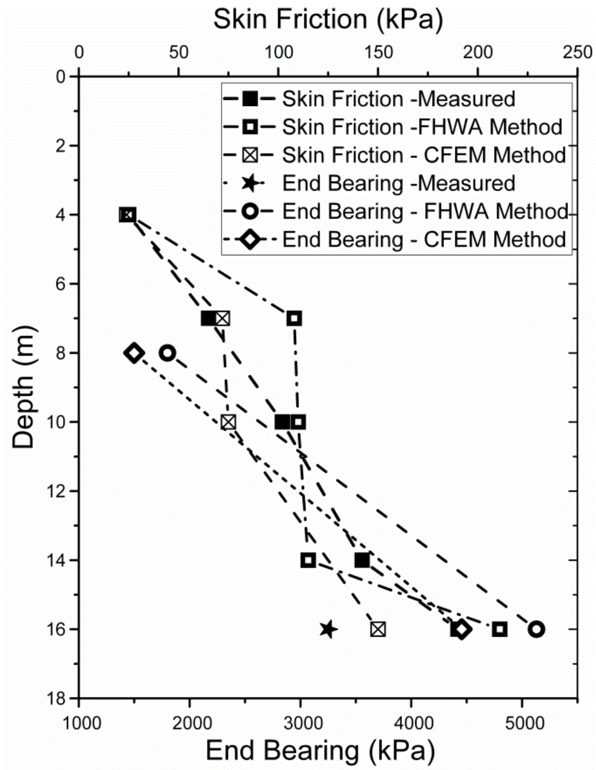

Figure 15. Comparison of the measured and empirically estimated values for Site 2

6. CONCLUSIONS

Drilled displacement steel piles are an innovative pile type being used during the last decade in North America. The pile has several advantages compared with conventional drilled displacement concrete piles. Thus far, DDSP design method has not been well developed and therefore *ad hoc* in-situ tests should be conducted on DDSP for each project. A series of axial loading tests of DDSP were carried out at two sites in Western Canada. The objectives of the research program are to investigate the axial load vs. displacement behaviour, load transfer mechanism, and provide the testing results to assist in the development of the design method of DDSP. The following conclusions may be drawn from the testing results.

1. DDSP has shown enhanced geotechnical resistance in both skin friction 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 skin in upper layers (3 to 5 m) is not much enhanced which may be due to lack of overburden to help the densification by radial displacement.

3. The observed end bearing values are scattered but are generally on lower bound of the estimated values which might be due to the drilling effect undermining the pre-loading to some extent.

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

DDSP have their own limitations such as requirement of heavy drill rigs for installation, made of steel which is an expensive construction material and premature refusal in case of very dense soils or shallow presence of intact bedrock. However, DDSP provides a viable and cost effective alternative to conventional drilled displacement concrete piles if installed in the right soil types and where minimal or no spoils are desired such as contaminated sites.

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