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Drilled Displacement Piles in Southeastern North Dakota

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ABSTRACT

INTRODUCTION

A heavily loaded structure as part of an industrial development was to be built in North Dakota. The subsurface conditions encountered at the site were generally unfavorable to the construction of the structure, as it consisted of loose granular soils with a very shallow groundwater surface. Smaller structures on the site were capable of being supported by spread footings with local soil corrections or stone columns. For the heaviest structures, ultimately Drilled Displacement Piles were selected as the preferred support alternative.

REGIONAL SETTING

The project area is located in south eastern North Dakota, near the outer edge of the Glacial Lake Agassiz basin. The region generally consists of a thick mantle of shore deposits that generally consist of unconsolidated, relatively fine-grained sand deposits. These deposits apparently were deposited nearer the shoreline and they are succinctly different than the heavy clay deposits in the Red River valley.

Underlying the unconsolidated granular deposits, there is glacial till commonly at 75 to 100 feet deep. The glacial till may have been deposited during several glacial advances and can consist of clayey and sandy deposits.

The topography was relatively flat, varying by less than about two feet across the site and region. With the flat terrain, granular soils, and little drainage, groundwater is commonly observed in the upper four feet of the soil profile.

SITE CHARACTERIZATION

The project site was investigated with a combination of hollow stem and mud rotary drilling with Standard Penetration Tests (SPT) at 2.5 to 5 foot intervals and Cone Penetrometer Test (CPT) soundings. Total depths ranged between 20 and 100 feet for the SPT borings and. At least one boring or CPT sounding was performed at each major structure. Figure 1 is a composite plot of N-values and equivalent N-values from borings and CPTs in the area of interest for this paper.

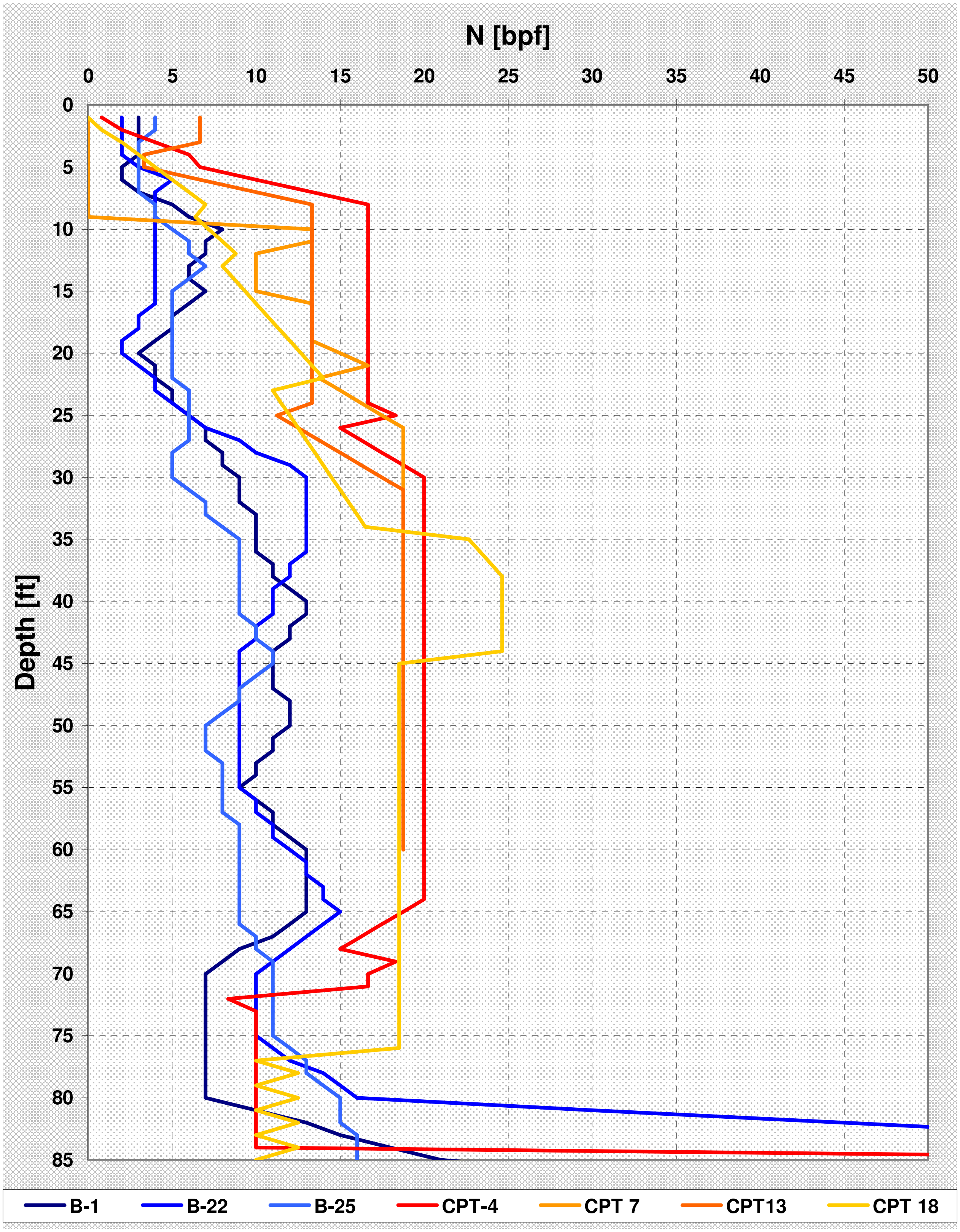


FIGURE 1: Composite Plot of SPT Results

The unconsolidated granular soils were observed to depths on the order of 70 to 80 feet. SPT tests in the upper 25 feet were generally in the range of 2 to 8 blows per foot, with CPT soundings ranging between 20 and 50 tsf. SPT tests between 25 and 75 feet were generally between 9 and 15 blows per foot, with CPT soundings ranging between 75 and 100 tsf. Once the borings penetrated the unconsolidated granular soils, they encountered a glacial till with clayey and granular strata. SPT tests generally reached refusal conditions after penetrating the upper 10 to 15 feet of this material.

Groundwater was observed at shallow depths, generally between one-half and two feet deep. Laboratory tests were generally limited to water content tests with select particle size tests. Water contents ranged between 20 and 30 percent. Fines contents on select samples of the granular soil ranged between approximately 25 and 50 percent. There are no known significant seismic activities in the project vicinity.

STRUCTURAL REQUIREMENTS

The supported structures consist of two silos each with a dead load on the order of 50,000 kips when full. Each silo load is supported on a mat foundation with contact pressure of approximately 7,000 pounds per square foot. In between the silos there is a below grade pit that extends up to 15 feet below the floor elevation. Frost depth was estimated to be up to 6 feet below the exterior grade. The finished floor elevation of the silo is approximately two feet above the existing grade. Design compressive loads on individual piles ranged from 80 tons to 120 tons.

POSSIBLE FOUNDATION SOLUTIONS

As the subgrade was clearly unsuitable for support of the mat foundations, various deep foundations and ground improvements were considered. Stone columns were used at other locations on the site, up to bearing capacities of up to 6,000 psf, However, the required capacity approached or exceeded maximum values for stone columns and there were concerns about the effects of the changes in the subsurface stress field resulting from the excavation and dewatering necessary for the below grade structures.

Concrete filled steel pipe piles are commonly used to support heavy structures in the region. Driving depths for sufficient support was estimated at 90 to 100 feet, which would be expensive. Driven piles, additionally, may have significantly disturbed the subgrade, especially in the upper 25 feet, which were quite susceptible to liquefaction induced by dynamic driving stresses. This would have made access for and construction of the subsequent piles difficult to achieve.

Traditional drilled shafts were also considered, but would have required the use of casing or slurry, making their installation difficult and costly.

In light of these considerations, it was considered desirable to use an augered or drilled cast-in-place foundation system such as augered cast-in-placed (ACIP) piles. ACIP piles are constructed without the use of casing and are generally economical as compared to driven steel piles or drilled shafts where skin friction is the predominate means of support. ACIP piles, however, would likely be very difficult to successfully place in the difficult subsurface conditions on the site for the following reasons.

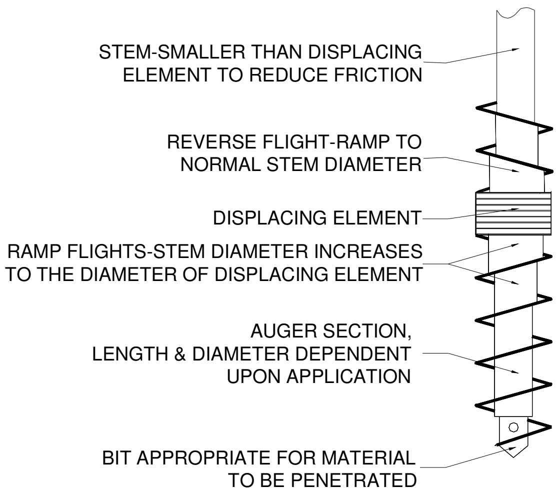
The installation of conventional augered, cast-in-place (ACIP) piles is a “negative” displacement process. Although the auger flights, and the soil they contain, provide some support for the adjacent soil, stress relief adjacent to the pile area is inevitable, even with good construction procedures. The soil in the vicinity of the pile location moves toward an “active” (Ko) condition and this process results in the adjacent soils being weaker than they were before the pile installation process was began. This process is most pronounced in granular materials below the groundwater level. In fine-grained soils that are self-supporting, the impact of the installation of ACIP piles may not be significant. At this site, any rotation of the augers in excess of the minimum necessary could rapidly draw the fine silty sand laterally, and significantly weakening the lateral stress field of the subgrade, especially on previously constructed piles.

In contrast, the installation of Drilled Displacement (DD) piles produces the opposite effect. The materials that occupy the pile area are displaced laterally, and in many soils, this process results in an increase in the strength and stiffness of the materials adjacent to the pile, which leads to higher shaft and toe components. In addition to the technical benefit of increased in shaft and toe resistance, the amount of spoils produced is much less than that produced during ACIP pile installation. In general, only the material contained in the auger section of the displacement tool is brought to the surface.

For these reasons, DD piles were specifically selected as the preferred design alternative. The Berkel Displacement Pile tool is shown in Figure 2, and the installation platform is shown in Figure 3. Currently, tools ranging from 12 inches to 18 inches in diameter are available. The auger section is typically about 3 feet in length, but may vary depending upon application. The installation platform includes a vertical mast with an attached turntable capable of producing 180,000 ft-lbs of torque, and a system of cabling that allows a downward force (crowd) of 80,000 lbs to be placed on the tools.

DESIGN METHODOLOGY

Initial analysis for skin friction was performed using the O’Neil and Reese Method in the Federal Highway Administration (FHWA) Design Shaft Manual. This method is an effective stress method, where the skin friction is proportional to the effective stress and a factor (B) that decreases with depth. A major underlying assumption of the O’Neil and Reese method is that dense soils will loosen and loose soils will slightly densify; consequently, the capacity is independent of the in-situ relative density or strength. A summary of the results of this analysis is presented in Figure 4.



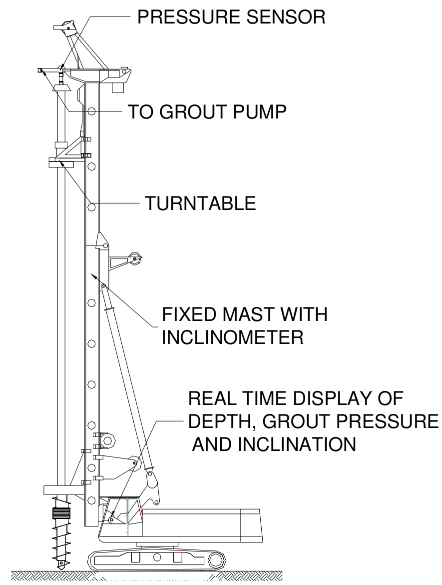


FIGURE 2: Berkel Displacement Tool **FIGURE 3: Installation Platform**

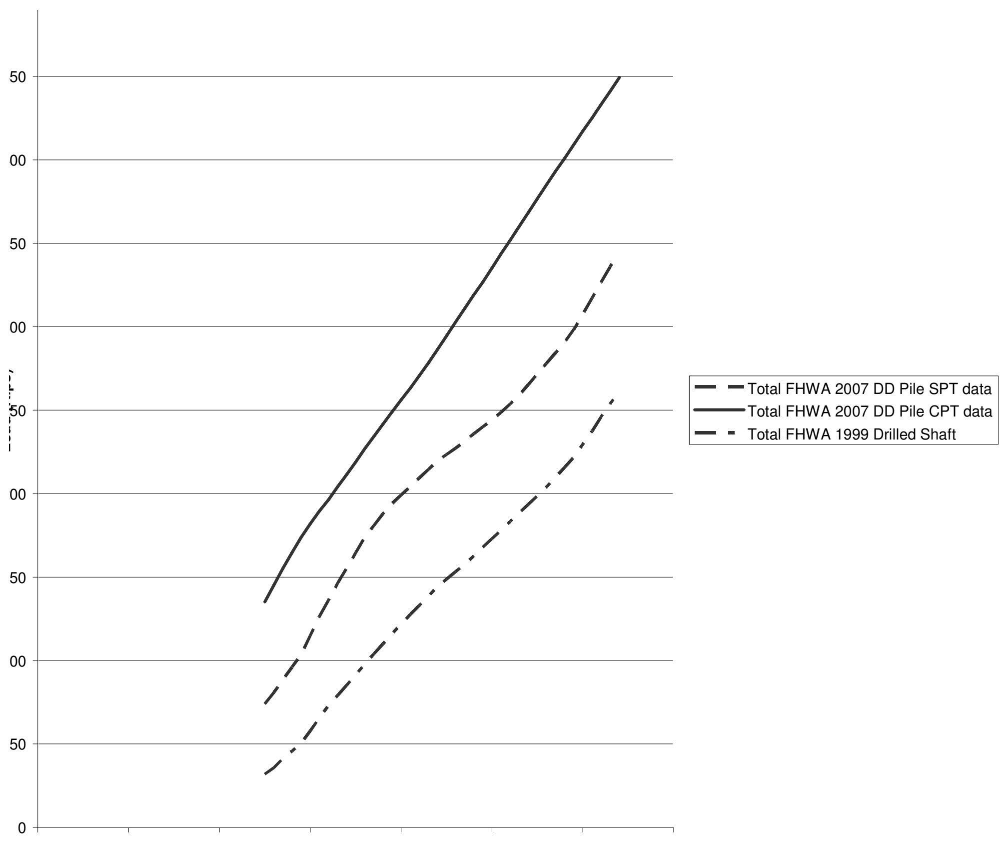


FIGURE 4: Estimate of Ultimate Load for 18" Dia. Piles – FHWA Method

Given the different installation methodology, it is evident that DD piles could significantly alter the stress field as compared to traditional drilled shafts, similar to driven displacement piles. Subsequent analysis was performed according to the method originally published in the following reference and later incorporated into the FHWA CFA Manual published April 2007:

NeSmith, W.M., 2002. "Static Capacity Analysis of Augered, Pressure-Injected Displacement Piles", ASCE Geotechnical Special Publication No. 116, Proceedings from DEEP FOUNDATIONS 2002, Orlando, FL, pp. 1174-118

Both shaft and toe resistances were calculated based on CPT results with modifiers appropriate for the characteristics of the granular material penetrated. The calculation methodology is summarized in Tables 1 and 2.

SHAFT RESISTANCE	**TOE RESISTANCE**
$f_{s,cpt} = 0.01 * q_c + w_s$	$q_{t,cp\,t} = 0.4 * q_{cm} + w_t$
$f_{s,cpt}$ = unit shaft resistance [tsf]; limited to 1.7 for dirty, rounded, uniform particles; limited to 2.2 for clean, angular, well-graded particles	$q_{t,cpt}$ = unit toe resistance [tsf]; limited to 75 for dirty, rounded, uniform particles; limited to 90 for clean, angular, well-graded particles
q_c = CPT tip resistance [tsf]; limited to $q_c \leq 200$ tsf	q_{cm} = CPT tip resistance [tsf] from 4 diameters above to 4 diameters below the pile toe; limited to $q_c \leq 200$ tsf
w_s = modifier for fines, angularity and uniformity	w_t = modifier for fines, angularity and uniformity

Table 1: CPT-based Capacity Analysis Method

Soil Description	w_s	w_t
Rounded; ≥40% passing #200 sieve; Uniform (GP, SP)	0.0	0
Increasingly cleaner, more angular and more well graded	interpolate	interpolate
Angular; ≤10% passing #200 sieve; Well Graded (GW, SW)	0.5	15

Table 2: Values for Capacity Modifiers

It should be noted, for the purpose of comparison, the design method is similar to LPC method for continuous flight auger (CFA) piles which are similar to the ACIP piles described previously. The coefficients, however, for side shear are approximately twice as high for DD piles as for CFA piles, allowing for a significant reduction in required length.

It should also be noted that drilled shaft group efficiencies in granular soils is less than 1.0 for center to center pile spacing of less than 6 diameters. Group efficiencies for displacement

piles, in contrast, are typically greater than 1.0. For conservatism, a value of 1.0 was selected for design.
Based on the results of these analyses, it was estimated that pile lengths from 32 ft to 50 ft would accommodate the compressive load requirements of 80 tons to 120 tons.

CONSTRUCTION CONCERNS

An additional primary concern during pile installation was the potential effect of the increased lateral pressures on previously installed below-grade structures. The walls of these structures were designed to be supported above by a pile cap and to resist at-rest soil and hydrostatic pressures. They were not deigned to resist an unknown amount of lateral pressure due to displacement pile installation.

To accommodate the potential soil pressures, a two-inch void form was placed around the perimeter of the walls. Because the structure was supported vertically by the DD piles, only a modest compaction effort was used to place backfill around the walls to avoid collapsing the form prior to the installation and reduce the potential for compaction induced pressure of the DD piles. During construction, the on site inspector was able to observe the form collapse during construction of the DD piles.

TEST PILE PROGRAM

A total of three axial compression load tests were performed on 18-inch diameter piles. Two piles for the load test were extended to 50 feet and the other pile was extended to 40 feet. The compression load tests were performed using the "Quick Test" method with a single loading sequence in 10 or 15 ton increments. The load tests were performed to a maximum load of approximately 250% of the design compressive load. Results of one 50 foot deep pile and the 40 foot pile are shown in Figures 5 and 6. The Davisson Criterion is plotted for reference, although it was not used in the design process. It is noted that while the Davisson Offset Limit method is listed as an acceptable process for evaluating ultimate load for pile foundations in IBC 2006, the method was originally developed for driven piles and is inappropriately conservative for cast-in-place foundations. Davisson (in the following reference) recommends a modifier of between 2 and 6 when calculating the offset for evaluating a cast-in-place pile, as research on drilled piers has shown that toe deflections of 2 to 5 percent of the diameter are required to reach ultimate load, compared to less than 1% for driven piles:

Davisson, M.T. (1993). "Negative Shaft Resistance in Piles and Design Decisions". Proceedings of the Third International Conference on Case Histories in Geotechnical Engineering. St. Louis MO.

However, it is noted that historical practice has often unfortunately referenced Davisson's original approach, thus its inclusion in this paper.

As can be seen, the load tested values correlate well with the predicted ultimate values, especially with the CPT Soundings. The maximum tested value also was comparable to the

Davisson Criterion for failure. More importantly, the deflection at 250% of the allowable load was less than the project specific criterion of 1 inch.

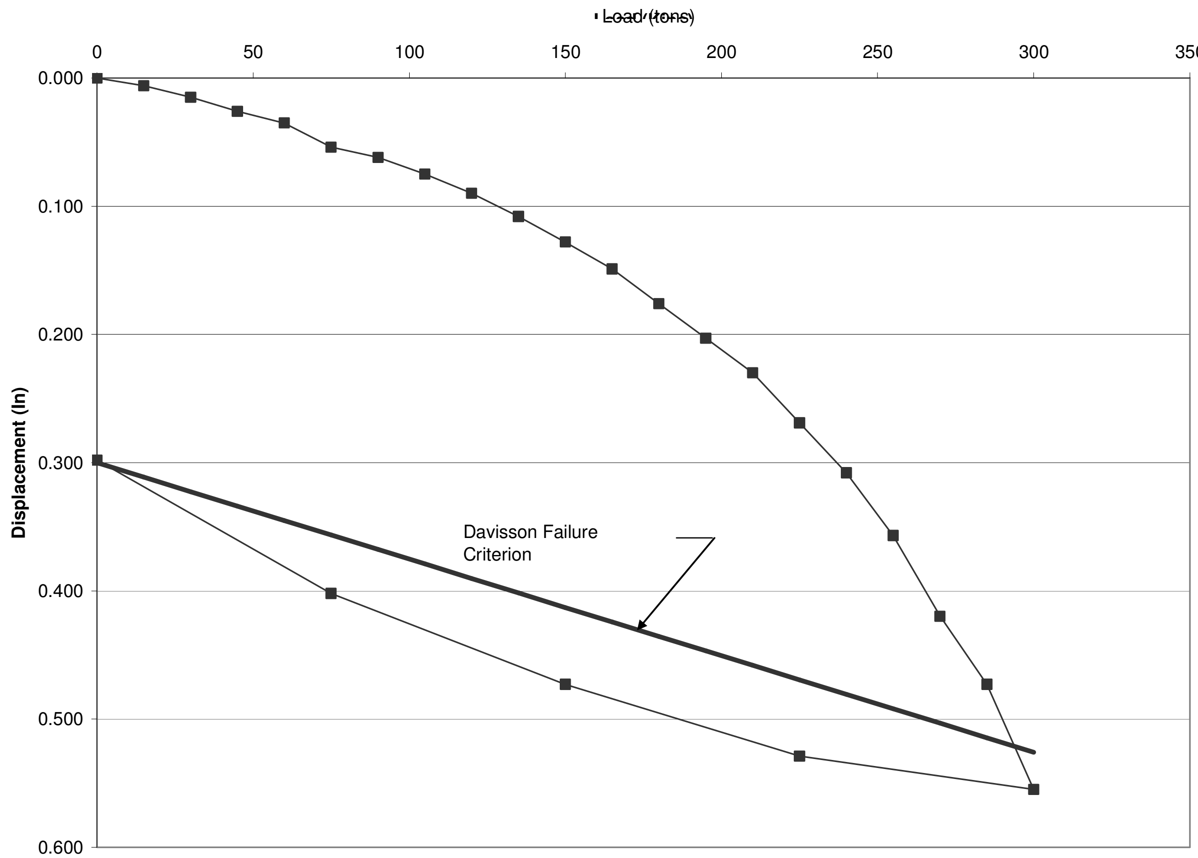


FIGURE 5: Plot of Pile-Head Deflection vs. Load – 50 ft Test Pile

QUALITY CONTROL

DD piles, like ACIP piles can not be visually inspected prior to the placement of the grout. However, the equipment used by Berkel included an on-board data acquisition system to electronically monitor several parameters during pile installation. During drilling, the pitch, tool advancement and withdrawal rate, grout flow pressure, and an estimate of torque are recorded and displayed in real time. A detailed description of the data acquisition system can be found in the following reference:

NeSmith, W. M. and NeSmith, W.M. (2006). "Anatomy of a Data Acquisition System for Drilled Displacement Piles". Proceedings of the American Society of Civil Engineers GeoCongress 2006. Atlanta GA USA. 26 February – 01 March 2006.

Grout volume was recorded by counting the number of strokes during tool withdrawal of a previously calibrated pump. Individual piles were extended when the measured torque values were observed to be less than those recorded during the installation of the test piles. Because the pile tip elevation was lower than the structural point of fixity, the minimal increase in drilling costs was only due to drilling time and grout volume; the reinforcing steel and overall production rate were not adversely affected.

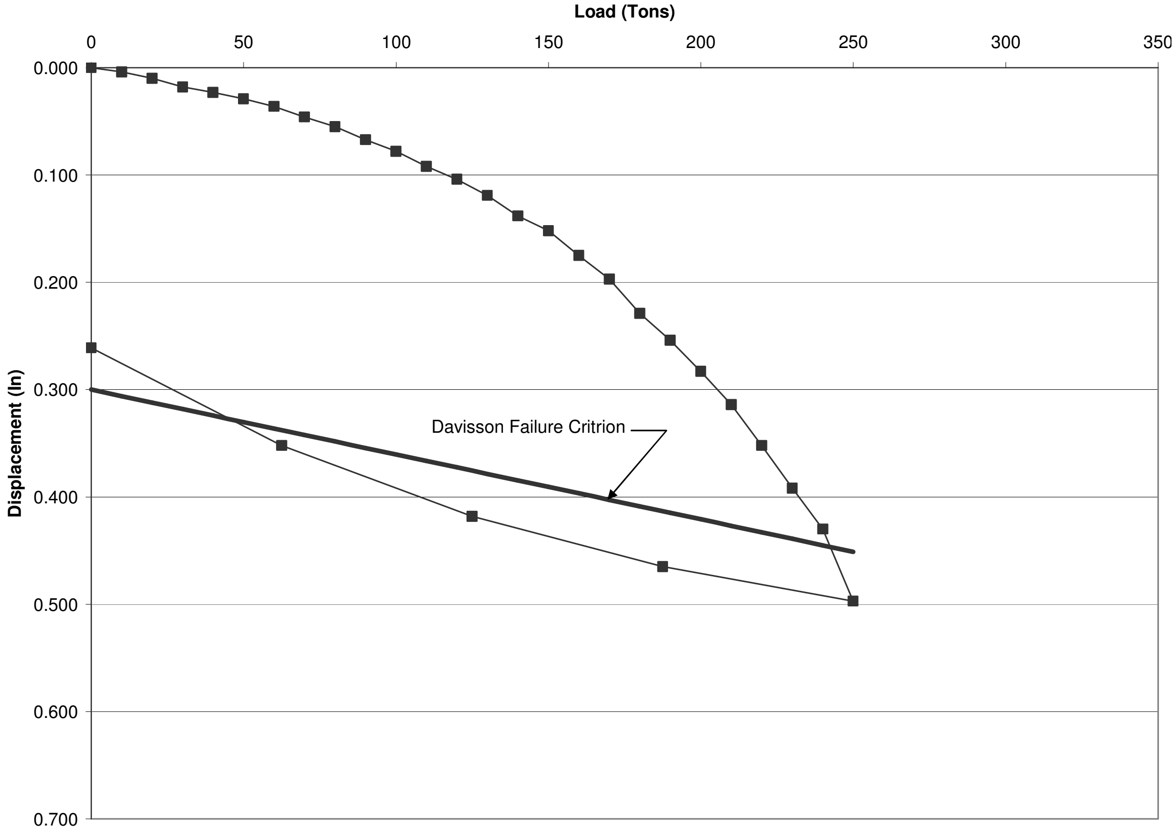


FIGURE 6: Plot of Pile-Head Deflection vs. Load – 40 ft Test Pile

SURFICIAL LIQUEFACTION

The issue during construction that did have an impact on the construction was due to some disturbance of the surface soils from general construction equipment such as fork lifts, concrete trucks, and dump trucks. As previously mentioned, the surface soils were loose and the groundwater surface was shallow, especially for locations located away from the dewatering cone of depression for the below grade structures. After rainfall, the groundwater surface was practically at the ground surface and the construction traffic locally induced pore pressure sufficient to liquefy the soil. This had the effect of disturbing the upper one to five feet of the subsurface and disturbed several freshly placed piles. The situation was remedied by controlling traffic away from freshly placed piles, chipping out the disturbed concrete (which was rather weak as it was not allowed to properly set, forming the upper portion of the effected piles, and placing fresh grout.

CONCLUSION

While still a relatively new technology in the United States, Drilled Displacement Piles appear to be well suited for support of foundations in loose granular soils. At this project site, DD Piles performed well, with the theoretical ultimate capacities aligning with the load test measured capacity. Furthermore, the technique allowed for significantly shorter piles than would have been suitable for drilled shafts or classic ACIP or CFA piles. The technology that accompanies the drilling technique helps maintain quality of the pile by continuously recording the construction activities, and accommodations to changing subsurface conditions can be made during production. While primarily a technique used in granular soils, there are also applications where the production of excavation spoils is undesirable, such as in brownfields or limited access locations like dense urban environments. It is also noted that drilled displacement piles were recently employed on an industrial project near Cohasset MN in predominantly fine-grained materials.