Formation of soil plugs within H-Piles driven in stiff clayey Port Stanley till near London, Ontario

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ABSTRACT
The formation of soil-setup soil plugs within H-Piles is demonstrated using the results of Pile Driver Analyzer (PDA) tests. Sixteen (16) PDA tests were carried out; six (6) on H-Piles driven continuously to 20 m and ten (10) driven after a 24 hours pause to accommodate pile splicing. Test results on piles continuously driven to 20 m demonstrated shaft resistances that agreed with the pre-construction static analysis and showed virtually no scatter in the measured pile resistances. Test results on all piles driven to further depths after welding demonstrated a severe loss of shaft resistance within the top 20 m indicating that a 24 hour delay provided sufficient soil-setup time for the formation of long soil plugs between the flanges which were subsequently dragged down leaving the upper portion of the pile with no reliable soil contact. This paper also demonstrates the use of PDA equipment to develop site specific soil setup functions for predicting long term resistances of friction piles and to investigate unanticipated pile-soil responses.

RÉSUMÉ
La formation d'un bouchon de sols au sein de pieux en H est démontrée en utilisant les résultats obtenus par l'analyseur de battage de pieux (PDA). Un total de seize (16) essais PDA a été conduit, dont six (6) sur des pieux H avancés en continu jusqu'à 20 m de profondeur et dix (10) sur des pieux reposés de 24 h afin de permettre les montages. Les résultats sur les pieux avancés en continu jusqu'à 20 m ont montré des résistances latérales en accord avec les résultats des analyses statiques pré-construction, avec quasiment aucune dispersion dans les mesures des capacités des pieux. Les résultats sur l'ensemble des pieux avancés plus en profondeur après assemblage, ont montrés une perte importante en résistance latérale dans les premiers 20 m, indiquant que ce délai était suffisant pour permettre la formation d'un bouchon de sols entre les ailes des pieux. Ce bouchon a été entrainé en profondeur lors du battage, laissant la section supérieure des pieux sans contact véritable avec les sols. La présente étude démontre aussi que l'utilisation du PDA permet de définir la variation de la capacité statique des pieux en fonction du site donc de prédire la capacité à long terme des pieux en friction et d'examiner toute interaction inattendue entre pieux et sols.

1 INTRODUCTION

1.1 General

During the process of deep foundation design and construction monitoring near London, Ontario an opportunity presented itself to compare design and measured resistances of frictional piles driven in very stiff to hard clayey silt till. The project site is indicated in Figure 1. The project consisted of the construction of a new interchange that included a new Highway 401 overpass at the proposed future extension of Wonderland Road near the southwestern boundary of the City of London, Ontario. The proposed overpass included two single-span, semi-integral abutment bridges with precast girders. Each bridge would have a span of 39 m and a skew of 35°.

The site is located within the western limits of the Westminster Moraine found within the Mount Elgin Ridge physiographic region (Chapman and Putnam 1984). The geological information of the site indicates that the general soil conditions of the project site consist of Port Stanley silty clay / clayey silt till with localized lacustrine deposits. The soil stratigraphy includes an approximate three to four metres thick surficial stratum of very stiff to hard clayey silt overlying an extensive deposit of firm to hard clayey silt to silty clay deposit. Bedrock was deeper than 50 m below existing ground surface.

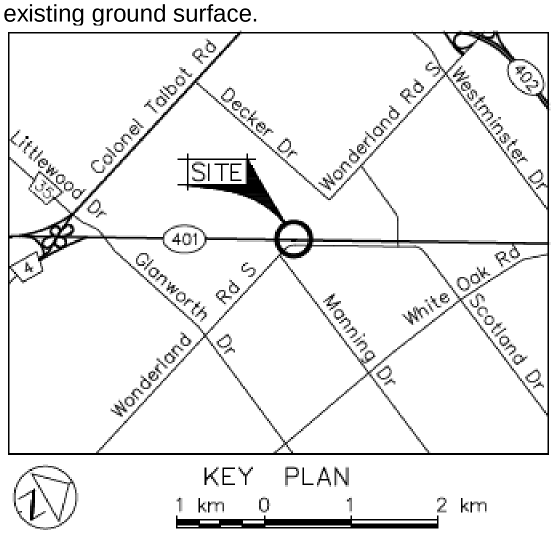

Figure 1. Site location

1.2 Soil Plug Literature Review

In general, there is paucity of documented experience regarding the soil plugs, especially in conjunction with piles having H sections. The formation of clay plugs during pile driving activities, between the flanges and web of driven H-piles, is discussed by Tomlinson and Woodward (1994). In this work, the clay plug is considered to form during the pile driving operation when the total shaft resistance developed along the web and the flanges exceeds the bearing resistance of the "box area" formed by the outer dimension of the H-pile section. Beyond this point, a soil plug is formed and carried down by the pile into the lower clay layer (refer to Figure 2). Tomlinson and Woodward (1994) do not discuss soil setup as a contributing factor to the formation of a soil plug.

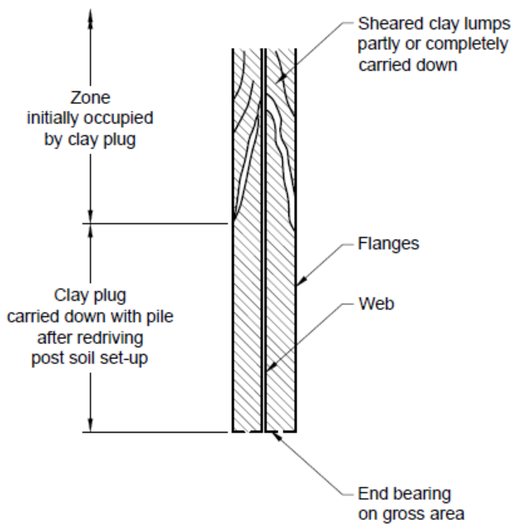

Figure 2. Formation of clay plug in an H-pile (modified from Tomlinson and Woodward, 1994).

2 PRELIMINARY PILE RESISTANCE EVALUATION

The project was awarded based on a design-build contract. During the bid process, available preliminary geotechnical investigation results were provided which formed part of the bid documents. Subsequently, preliminary pile resistances were calculated during the bid process. At the outset, a frictional pile foundation system consisting of H-piles presented itself as the best possible foundation alternative given the site geology (*depth to bedrock not known but likely very deep*), the anticipated semi-integral abutment bridge configuration and the experience of the client (*Ontario Ministry of Transportation, MTO*).

Preliminary pile resistances were developed based on the method of American Petroleum Institute (API 2000) for computing shaft friction. According to this method, the unit shaft friction for cohesive soils is estimated as the product of the undrained shear strength (s_u) and a coefficient, α, as a function of s_u and is recommended by API RP2A. Soil parameter selection was also compared with the Canadian Foundation Engineering Manual (CGS 2006) guidelines.

3 DETAILED INVESTIGATION AND DESIGN

Subsequent to the contract award, a detailed foundation investigation was carried out at the critical locations of the proposed overpass to validate and/or improve the preliminary design. The investigation included Standard Penetration Test (SPT), field vane tests and cone penetration testing (CPT) at select locations as well as laboratory tests.

The undrained shear strength of the cohesive soil was estimated by a combination of correlation with SPT blow count (N-values) based on local experience, field vane tests and the CPT results. The soil profile model developed for the site is provided in Figure 3. In general, the detailed investigation results are consistent with preliminary results.

HP310x110 steel piles were selected as the preferred deep foundation option for the project. The predicted pile resistances are provided in Figure 4. The required geotechnical resistance (factored) of the piles at the Ultimate Limit State (ULS) was 1300 kN which called for an approximate pile length of 36 m.

Due to the anticipated variability in the geotechnical axial resistance of driven steel H-Piles within the stiff silty clay deposit, a Pile Driving Analyzer (PDA) test was recommended to be performed on production piles to help in design validation, pile length optimization and to provide quality control for the installed piles. A pile-soil setup period of two weeks was recommended prior to carrying out the PDA testing on the piles. The design recommendation called for PDA testing on at least four piles per bridge (two per abutment).

4 PILE DRIVING AND PDA TESTING

4.1 Soil setup investigation

Soil setup is the increase in the static pile resistance resulting from time dependent increase in soil shear strength. This typically occurs in piles driven in saturated cohesive soils as the excess pore water pressure dissipates gradually. In the absence of site-specific pore water pressure dissipation data, it is generally specified that load testing of piles driven in cohesive deposits be delayed at least two weeks after end of initial driving (EOID).

The method of Skov and Denver (1988) was used to predict soil set-up:

$$Q_t = Q_o\,[A \log (t/t_o) + 1] \quad [1]$$

where:
Q_t = axial resistance at time t after driving,
Q_o = axial resistance at time t_o,
A = a constant, depending on soil type, and
t_o = an empirical value (initial time) measured in days
t = an empirical value (rest time) measured in days

For the pile setup analysis it was necessary to establish the appropriate time t and constant A. The time t_o corresponding to Q_o is generally in the order of one day (Komurka et al. 2003). The PDA tests on the first four installed piles was carried out 14 days after EOID; the corresponding pile resistances are designated as Q_{14}. As part of the testing process, the piles were driven an additional one metre beyond its beginning of restrike (BOR) conditions to model EOID conditions; the corresponding strengths are referred to as Q_o. The constant A was determined to be 0.265 based on the Q_o and Q_{14} values of the four tested piles.

To validate the appropriateness of the selected value for A, MTO static pile load test database (MTO 1993) was used for piles installed within close proximity of the site at locations having similar soil conditions. This database consisted of measured pile resistances for piles tested two days after initial drive to greater than one year after initial drive. Review of the database indicated that for H-piles installed in similar soil conditions, the strength gain after one year was negligible. In fact, most of the piles in the database achieved close to 100% of their resistances approximately 300 days after EOID. Subsequently, the long-term (100%) pile resistances of the piles installed at this site were considered to be achieved 300 days after EOID; the corresponding pile resistance will be designated as Q_{300}. Table 1 summarizes the PDA test results for the first four tested piles. The results are indicated in Figure 4 as "PDA Site Visit 1 + Setup".

Table 1. PDA test results from first four piles

Pile[1]	Length (m)	Ultimate Pile Resistances[2] (kN)		
		Q_o	Q_{14}	Q_{300}
P1 (P1)	30.2	1053	1359	1641
P2 (P3)	30.0	946	1276	1544
P3 (P5)	24.0	953	912	1133
P4 (P7)	24.5	622	800	995

[1]production pile designation in parentheses
[2]measured PDA result based on case method estimated capacity (CMEC) and or CAPWAP analysis result

4.2 Discussion of the PDA test results

As indicated above, the four PDA tests were carried out to (1) assess if the required pile resistances were achieved (i.e., *basically perform QC test*), and (2) to optimize embedment length (*i.e., to determine if shorter piles than initially anticipated can be used*). The design pile resistance was 1300 kN at Ultimate Limit State (ULS) per pile. Using a resistance factor of 0.5, as per the Canadian Highway Bridge Design Code (CSA 2006), an unfactored ultimate axial geotechnical resistance in compression of 2600 kN was required per pile. However, as indicated in Table 1, the predicted maximum long-term pile resistance (Q_{300}) for the four piles tested after 14 days was only 1641 kN (for a pile length of 30 m), significantly less than the required ultimate resistance of 2600 kN.

4.3 CAPWAP® analysis and additional PDA tests

To better understand the less than expected pile resistances obtained from the four tested piles, a CAPWAP® (CAse Pile Wave Analysis Program) was performed on the PDA test results. CAPWAP is a Signal Matching Program that uses as input the force and velocity data obtained from the Pile Driving Analyzer® (PDA) system. In this project, CAPWAP was used to estimate the total static soil resistance as well as the resistance distribution along the pile shaft and the static toe resistance.

The PDA test results from Piles P1 and P2 were used for further interpretation using CAPWAP. The CAPWAP analysis indicated that very little adhesion was occurring within the upper approximately 20 m of the pile (*approximately 2 to 6% of the undrained shear strength of the soil*). According to the Canadian Foundation Engineering Manual (CGS 2006) the estimated adhesion against the pile for this type of soil is 40 to 45% of the undrained shear strength.

Below a depth of approximately 21 m, the average adhesion at 14 days was approximately 35% of the undrained shear strength. Considering the setup function discussed previously, the long-term adhesion in the lower portion of the pile will be in the range of 45% of the undrained shear strength. At this point, little was known as to why little adhesion was developing in the upper portion of the pile except for the fact that the unspliced pile length was approximately 20 m. It was originally hypothesized that a small annular space may have been created along the surface area of the shaft (*possibly due to the shape of the driving shoe*) or that the clay deposit may have been behaving similar to a carbonate soil and not expanding to press against the pile surfaces. The PDA test results indicate that within the deeper portion of the pile, a 'plug' appears to have formed between the flanges and is moving with the pile; suggesting a non-displacement pile behaviour above the plug and a displacement pile behaviour below the plug.

4.4 Additional PDA tests

To better understand the cause of the unexpectedly low pile resistances for the four piles driven at the project sites, PDA testing was testing was completed on seven additional piles. At this point, pile No. 1 had already been driven to a length of 43 m and achieved the target ultimate resistance of 2600 kN (Figure 4, result depicted as "EOID Site Visit 1 + Setup"). In reflecting on the outcome of the first set of PDA tests, a hypothesis was established that a soil plug may have formed during pile setup while the piles were being spliced and that the plug so formed may have been driven forward after re-initiation of driving, Thus, the pulling down of the soil 'block' above the plug may have created a disturbed zone. This interpretation was supported by minimal shaft resistance above the splice level as discussed above.

Six of the seven additional piles were driven to approximately 20 m prior to testing. PDA testing was carried out on these piles to assess their resistances prior to re-initiation of driving pile splicing. The purpose of this testing was to confirm if the reduced resistances were due to a soil plug advancing with the pile. One pile was driven to approximately 21 m without the driving shoe to test the hypothesis that the welded plate may be leaving a gap at the outside of the pile during driving and therefore reducing the overall soil to steel contact area. The results are indicated in Figure 4 as “Non-Spliced Piles”.

Table 2 summarizes the PDA test results of the additional seven piles tested.

Subsequent to testing of the non-spliced piles, PDA testing was also carried out on five piles driven after splicing to depths ranging from approximately 24.5 to 43.0 m. The results of PDA tests on these piles are summarized in Table 3.

Table 2. PDA test results from non-spiced piles

Pile[1]	Length (m)	Ultimate Pile Resistances (kN)		
		Q_o	Q_{14}	Q_{300}
P5 (P2)	21.0	965	990	1623
P6 (P9)	20.2	838	1005	1488
P7 (P11)	20.2	893	1020	1535
P8 (P13)	20.0	701	1010	1382
P9 (P15)	20.2	772	1015	1424
P10 (P17)	20.2	781	1040	1454
P11 (P21)	20.0	767	1014	1421

[1]production pile designation in parentheses

Table 3. PDA test results from spiced piles

Pile[1]	Length (m)	Ultimate Pile Resistances (kN)		
		Q_o	Q_{14}	Q_{300}
P1 (P1)	43.0	1725	2013	3008
P5 (P2)	30.0	695	717	1173
P2 (P3)	31.4	979	1401	1809
P4 (P7)	24.5	815	1160	1500
P6 (P9)	25.6	735	745	1227

[1]production pile designation in parentheses

Based on the results PDA test results summarized in Tables 2 and 3 as well as Figure 4, the following observations can be made:

- Piles driven from the ground surface to the first stop required for splicing have substantially greater shaft resistance within 20 m from ground surface compared to those driven deeper after splicing. This confirms that soil plugging is occurring only after the delays associated with the splicing of the piles.
- For the piles tested prior to the re-initiation of driving post-splicing the calculated long-term resistances, using the PDA tests results, and the established setup function for this site were comparable which form a precise cluster that is very close to the design line of pile resistance versus embedment depth plot (Figure 4). This suggests that the site soils are not very variable and that the soil strength parameters selected for the project were appropriate.
- One pile (designated as “Pile 2” in Figure 4) was driven without a pile shoe. At a depth of 21 m (prior to recommencing of pile driving), the calculated long-term resistance based on the PDA test and setup function followed the same trend as the piles driven with a shoe. It is concluded that the presence of the welded shoe did not noticeably impact the adhesion developed between the soil and pile within the upper 20 m (above the splice level).
- Where testing was carried out on piles driving after splicing, the estimated long-term resistances using the PDA test results and the setup function are significantly less than the design values (left of the design line in Figure 4) used for the project. The drop in the pile resistances is not directly related to pile embedment. In addition, CAPWAP analysis shows that piles penetrating the deeper soils lost their adhesion within the upper portion of the pile. The calculated resistances show a great degree of scatter compared to the pile resistances obtained for the unspliced piles. This scatter is consistent with the randomness of the disturbance of the soils above the plug (above splice level) as it is pulled down by the underlying soils moving down with the pile.
- The randomness of the amount of disturbance is exemplified by the variable proportion of loss in pile resistance prior to splicing (Figure 5). For example, referring to Figure 5, two lines are shown indicating the 50% and 100% loss in resistance with respect to the pre-splicing resistance.
- The relatively low to negligible adhesion between the pile and soil within 20 m of the ground surface observed from the piles tested after splicing is attributable to the development of a soil plug between the flanges.
- Figure 5 indicates that to get the target resistance at ULS, the piles would have to be driven to elevation of 219 m, suggesting approximately 10 m in excess of the originally proposed length. This would be significant increase in the cost of the project.

The formation of clay plugs within H-piles during driving has not been previously reported in Ontario and as such this situation was not anticipated for this project. As discussed above, the PDA test results indicate that piles driven without stoppage did not form any soil plugs and that the plugs formed during stoppage for splicing was sufficiently strong enough to be carried down for some part of the driven embedment length. The following section discusses the remedial options considered.

4.5 Remedial options considered

The following options were explored:

- Drive piles to embedment length of 46 m (tip elevation of 219 m)
- Drive less number of piles to embedment depth of 51 m (elevation of 214 m)

- Switch to opened end pipe piles which would likely be less prone to disturbance outside the pile surface area
- Splice the piles prior to driving and drive to the target tip depths (elevation) in a single driving operation
- Use the same pile, avoid splicing and increase the number of piles to make up for the reduced pile length to avoid splicing.

Given the unpredictability of the proportion of resistance loss after splicing, project cost and schedule, the preferred alternative was to drive the piles to approximately 238 m (pile length in the order of 20 to 21 m), it was decided that piles be driven to approximate lengths of 20 m to avoid splicing. The number of piles were subsequently increased to match the target pile resistances.

CONCLUSION

Based on the experience gained during the design and construction monitoring of driven frictional piles in this project, the following conclusions can be drawn:

1. PDA test results demonstrated that for the clay this site, highly repeatable resistances can be achieved for unspliced piles driven in a single operation.
2. PDA test results are consistent with predicted resistances for unspliced piles.
3. Clay plugs developed, after a 24-hour soil set-up rest period, at this site resulted in substantial resistance reduction upon reinitiating of pile driving. The amount of this loss in pile resistance with respect to the predicted pile resistance was measured to vary from 50% to 100% (i.e., in the absence of splicing).
4. There is limited literature discussion regarding the formation of clay plugs in H-piles. Even this limited discussion focussed on plug formation during pile driving. The current study discussed the case where soil plugs form during a 24-hour set-up period and that further driving results in the loss of the resistance within the upper portion of the pile.
5. PDA testing provided a useful and relatively inexpensive means to investigate construction problems associated with frictional pile resistances prediction when coupled with CAPWAP analysis and setup function.

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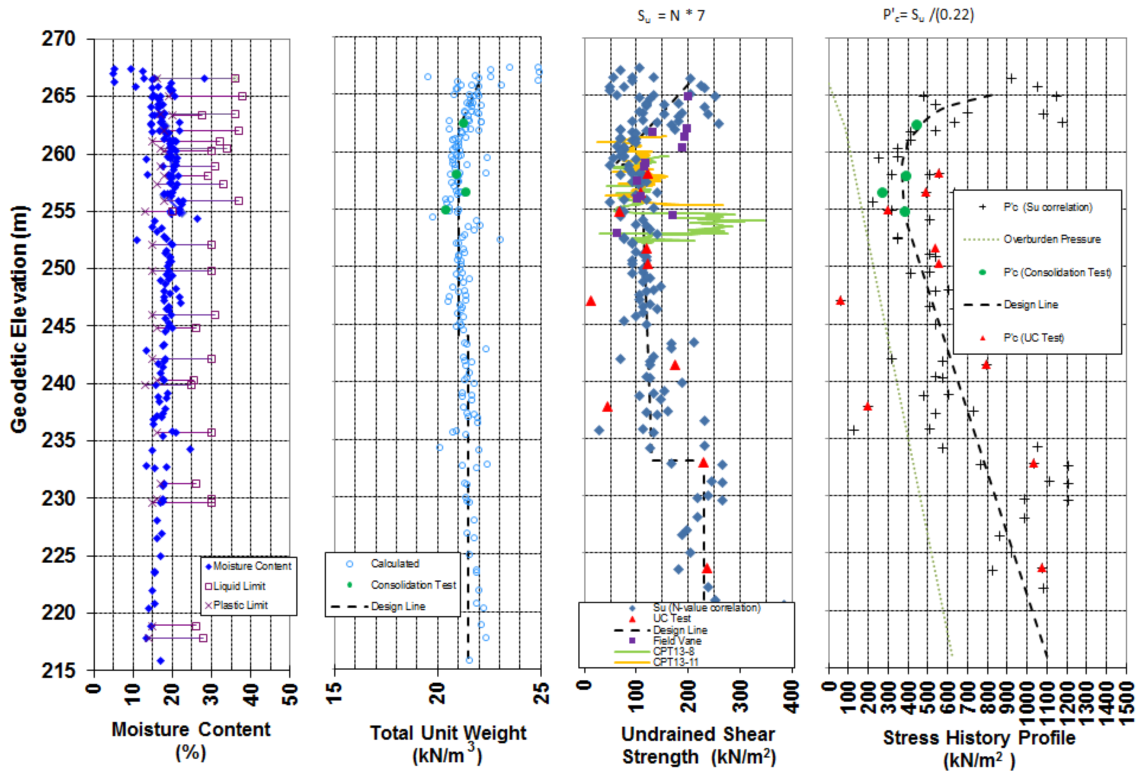

Figure 3. Soil model

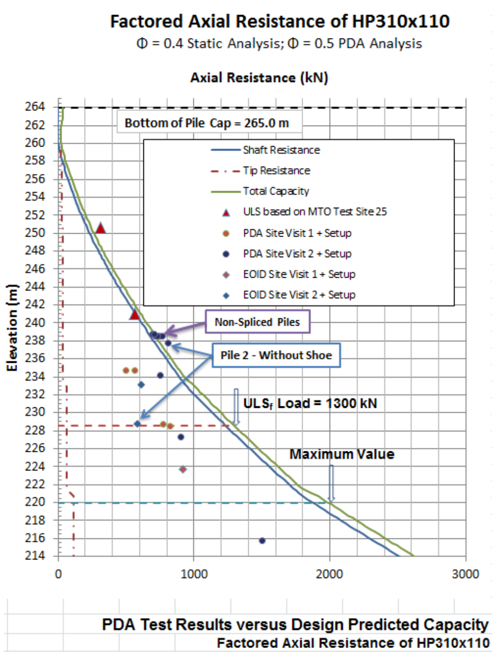

Figure 4. Predicted and measured pile resistances

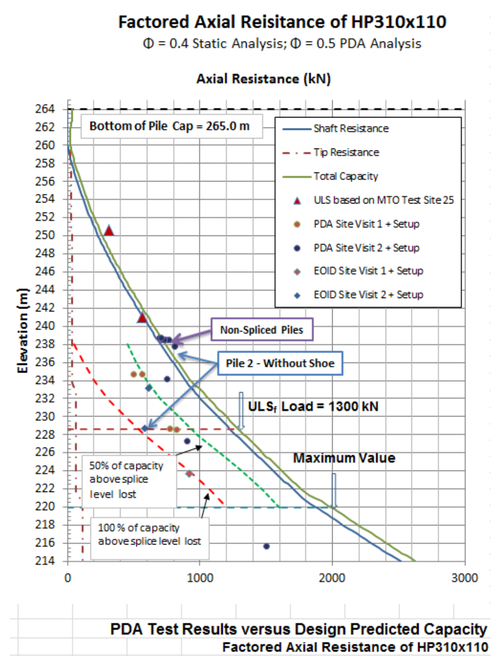

Figure 5. Pile resistance with variable loss in resistance above splice level