A design approach for piles in langley clay based on testing

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ABSTRACT
A series of PDA tests were performed on four 610 mm diameter steel pipe piles installed to depths of about 65 m in a normally consolidated to lightly overconsolidated cohesive soil profile in the Lower Mainland of British Columbia. The axial capacity of the piles was assessed based on in situ and laboratory tests using the API RP2A (2000) method for determining axial capacity in compression. The piles were installed over a one week period and monitored with a PDA system. Restrikes at setup periods ranging from 5 days to 30 days were used to predict the increase in axial capacity with time after initial driving. The results obtained form the basis for the suggested design approach.
RESUMEN
Mediciones con un sistema PDA fueron realizadas para 4 pilotes installados hasta 65 m de profundidad en una arcilla ligeramente sobreconsolidada. La capacidad axial de los pilotes fue determinada basada el el metodo API RP2A (2000). Se installaron los pilotes durante una semana y fueron hincado de nuevo a diferente periodos de espera de 5 a 30 dias para evaluar el incremento de capacidad axial con el tiempo. Se presenta un metodo de diseno para pilotes basado en los resultados

1 INTRODUCTION

As part of a design-build project in the Langley area of British Columbia, it was decided to install test piles to monitor the development of axial capacity with time in order to optimize the foundation design for an elevated overpass structure supported on steel pipe piles. A pile testing program based on PDA® measurements was defined in order that the piles could be designed with a factor of safety of 2 on axial capacity, as permitted in the design-build contract. Prior to commencing construction, four test piles were driven at the location of one of the support bents to confirm the axial capacity of the piles. PDA® measurements were made during initial driving and at various times after installation. The results of the PDA® testing provided the basis for determining the axial capacity set-up of the piles and for finalizing the pile design. The PDA® results provide confirmation that the ultimate axial capacity can be conservatively estimated using the best estimate soil strength profile and the API RP2A (2000) calculation method. The requirement to correct for the L/D ratio was also assessed as part of the study.

The main overpass structure was to be founded on 610 mm (24 inch) diameter steel pipe piles with a wall thickness of 12.7 mm (0.5 inch). The pile test program was conducted on the 610 mm pipe piles.

The pile test program was designed so that the end-of-driving results could be used to predict the fully-setup ultimate axial pile capacity, thus removing the need to test the installed piles at times of 30 days, or more, after initial pile installation. The necessity to test the piles after complete setup (30 days or more) would have had impacted the construction schedule.

2 SOIL CONDITIONS AT TEST PILE SITE

The results of field and laboratory testing were used to determine the appropriate design parameters for a soil profile considered to be sensibly homogeneous and representative of the ground conditions along the overpass structure. The representative soil profile consists of an overconsolidated crust followed by soft clay that becomes firm and then stiff with depth (Figure 1).

On the south side of the site, the clay layer extends to depths of more than 100 m. On the north side of the site, the clay is underlain by a very dense clayey gravel layer at a depth of about 85 m.

Profiles of total unit weight, plasticity index and overconsolidation ratio are also indicated on Figure 1.

In situ vane tests, CPT data and laboratory test results (Torvane, Penetrometer, Miniature Vane and Triaxial UU) have been used to define the best estimate variation with depth of the undrained shear strength (Figure 2). Lower and upper bounds on the undrained strength profile are also indicated on Figure 2. Data from remolded vane and triaxial UU tests have been used to define the remolded strength profile. The sensitivity of the clay based on the laboratory and field data (FVT and CPT) is plotted on Figure 3.

One-dimensional incremental load consolidation (oedometer) tests provide the basis for the interpreted overconsolidation ratios presented on Figure 1. The variation in OCR with depth was also estimated based on the CPT data.

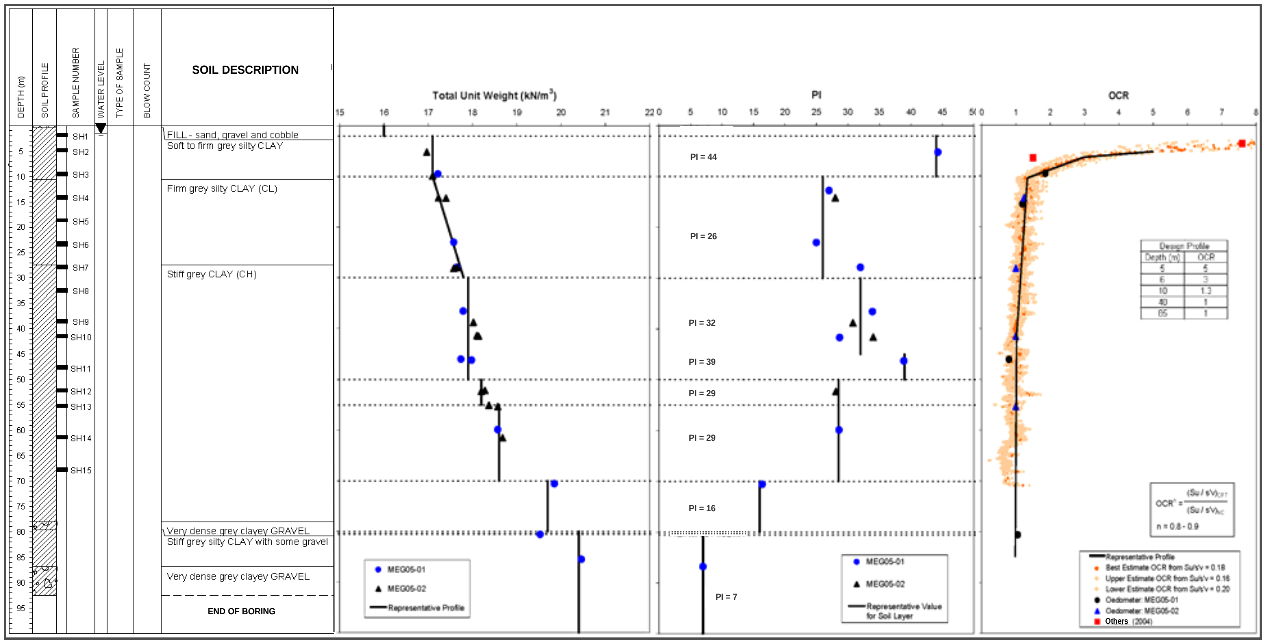

Figure 1. Geotechnical profile and soil properties.

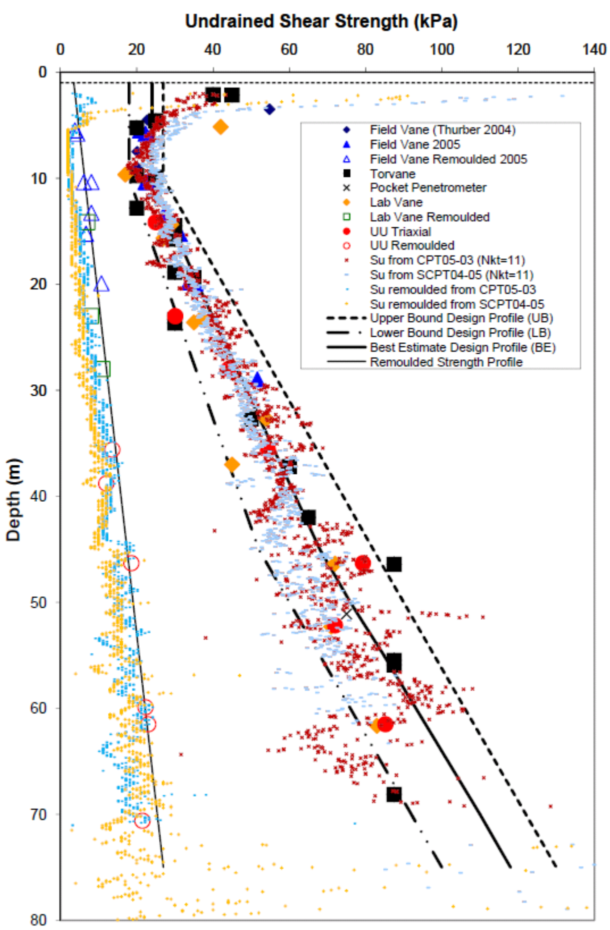

Figure 2. Undrained shear strength profile based on field and laboratory data.

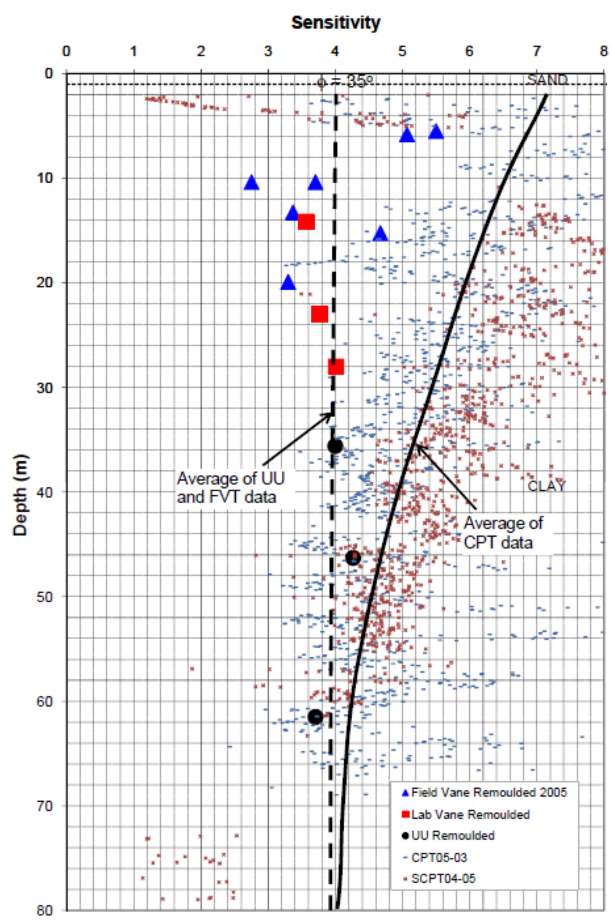

Figure 3. Sensitivity for the undrained shear strength of Langley Clay.

Table 1. Summary of average geotechnical parameters for soils at test pile site.

Thickness (m)	Soil Description	w (%)	LL (%)	PI (%)	G_s	γ_t (kN/m^3)	e_o	ϕ	S_u (kPa)	OCR	μ	E_s (MPa)	C_c	C_r	C_α
0.5-1	FILL: Sand, Gravel and Cobble	-	-	-	2.65	16.0	-	35°	-	-	-	-	-	-	-
10-11	Soft to firm grey silty CLAY	45	72	44	2.70	17.1	1.22	-	18 – 27	>4.0	0.49	8	0.56	0.050	0.015
17-22	Firm grey silty CLAY	42	48	26	2.70	17.5	1.13	-	28 – 43	1.3	0.39	9	0.40	0.035	0.010
49-51	Stiff grey CLAY	40	60	35	2.71	18.3	1.08	-	68 – 98	1.0	0.38	18	0.54	0.047	0.014
>20	Stiff grey CLAY with some gravel	20	20	7	2.71	19.7	0.87	-	>100	1.0	0.33	30	0.54	0.047	0.014
>10	Dense clayey GRAVEL	20	-	-	2.68	20.0	0.54	40°	-	1.0	-	-	-	-	-

The water table at the site was encountered at depths of between 1 m and 2 m below ground level.

A summary of the geotechnical parameters for the individual soil layers used in the foundation analyses is presented in Table 1 (w = Natural Water Content; LL = Liquid Limit; PI = Plasticity Index; Gs = Specific Gravity; γt = Total Unit Weight; e_o = Void Ratio; ϕ = Internal Friction Angle; S_u = Undrained Shear Strength; OCR = Overconsolidation Ratio; μ = Poisson's ratio; Es = Stress-Strain modulus; C_c = Compression Index; C_r = Recompression Index; $C\alpha$= Coefficient of Secondary Compression).

During the field investigation, eight CPTU dissipation tests were performed. The horizontal coefficient of consolidation (C_h) was calculated based on Robertson et al. (1992) using a rigidity index of 100. The results are presented in Table 2.

Table 2. Horizontal coefficient of consolidation from CPTU

CPT Sounding	Test Depth (m)	t_{50} (s)	C_h (cm^2/min)
CPT05-01	10.00	2897	0.25
	20.00	2393	0.30
	29.97	2609	0.27
	40.00	1799	0.40
CPT05-02	10.00	3885	0.18
	20.00	1888	0.38
CPT05-03	10.00	3347	0.21
	20.00	4000	0.18

The time for 50% of dissipation (t_{50}) is calculated based on the initial and equilibrium pore pressure values.

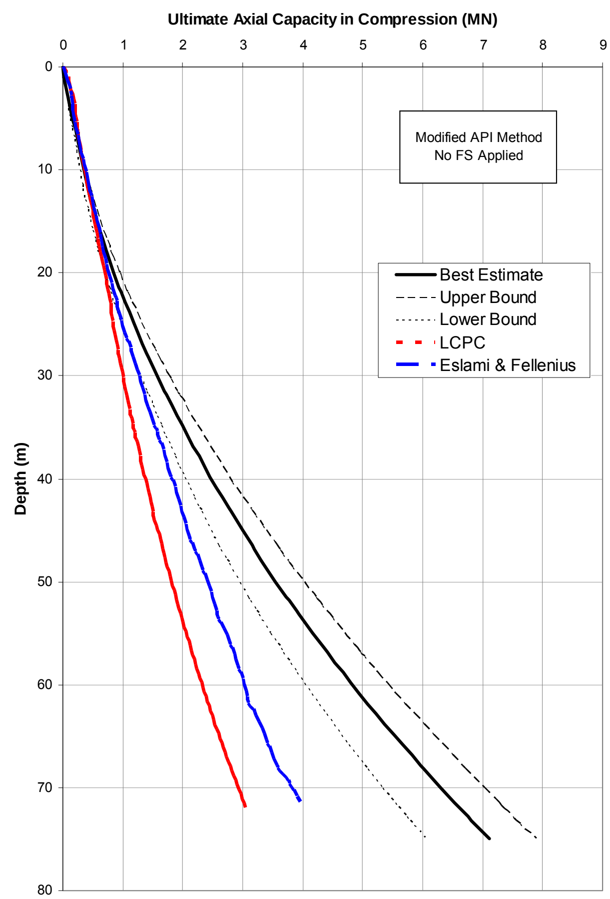

Figure 4. Ultimate axial capacity for 610-mm open-ended steel pile.

The ultimate axial capacity for the 610 mm diameter open-ended steel piles was calculated based on the working stress design methodology provided in the American Petroleum Institute recommend practice (API RP2A 2000). The three design undrained shear strength profiles indicated on Figure 2 were used to determine the best estimate, lower and upper bound ultimate axial capacity profiles for the 610 mm pipe piles (Figure 4).

Also indicated on Figure 4 are the axial capacity profiles based on direct calculations using the CPT data, based on the LCPC (Bustamante and Gianeselli 1982) and Eslami and Fellenius (1997) methods.

Good general agreement exists between the best estimate ultimate profiles of axial capacity on Figure 4 for the three methods up to depths of just over 20 m. Thereafter, the three methods diverge considerably with the direct CPT methods giving much lower calculated axial capacities compared to the API method. This may reflect the conditions used for developing the empirical pile capacity calculation method, whereas the API method is generally applicable to long slender piles (L/D>20).

3 DETAILS OF TEST PILES

The project specifications required that the allowable axial pile capacity be determined using a minimum factor of safety depending on the approach used to determine the axial capacity. For an axial capacity determined solely from CPT results, the required factor of safety is three (FS = 3). The FS reduces to 2 for a full-scale dynamic load test and to 1.8 for a static load test, provided the ultimate capacity is proven by the respective test prior to final design. In addition, a minimum safety factor of 1.1 is required for seismic loading.

To substantiate the design approach it was decided to perform a series of dynamic axial capacity measurements on piles installed at the site. The pile lengths were determined based on the API RP2A (2000) methodology using the best-estimate undrained shear strength profile.

Four test piles were installed at one of the footing locations so that the test piles could be incorporated as production piles into the final structure, provided the axial capacities proved to be as per the design assumptions. Dynamic pile tests were to be performed on the four production piles using a Pile Driving Analyzer® and subsequent CAPWAP® analyses to evaluate the axial pile capacity in compression at different times after initial driving. The test pile program was scoped to include the results of 16 PDA® tests over a period of about 30 days.

The four test/production piles for the PDA® testing were monitored during driving. Based on initial analyses and foundation loading at the test location, it was decided to drive all four piles to a final depth of 65 m below existing grade. The piles were driven and tested over a period of several weeks, commencing in January 2006 with Piles A and B. Piles C and D were started the following day. The blow counts for driving of each of the piles are presented on Figure 5. The relative pile locations are also indicated on Figure 5.

Pile driveability (wave equation) analyses were also performed using the GRLWEAP program (PDI 2010). The calculated blow count profile for setup factors of 3.5 and 4 are also presented on Figure 5.

4 PDA® MONITORING

PDA® monitoring was performed at the end of initial driving (EOID) and at varying periods after initial driving had been completed. PDA® measurements were performed for re-strikes at the following nominal times after initial driving (setup time): 1 day, 10 days, 20 days and 30 days. The setup time was defined as the number of days after end-of-initial-driving (EOID), even if interim PDA® tests had been performed subsequent to initial driving. For the re-strike program, a larger 58 kN (13,000 lb) hammer was used. The drop height of the hammer was varied as the capacity increased with setup time.

During initial driving, the PDA® measurements were made over the last meter of driving. For initial driving a 40 kN (9,000 lb) drop hammer was used. Drop heights were about 1.5 m, giving an energy level of about 60 kN.m. At the end of initial driving, the maximum blow count per meter was recorded as 180 for Pile D; the minimum blow count was 54 for Pile C (Figure 5).

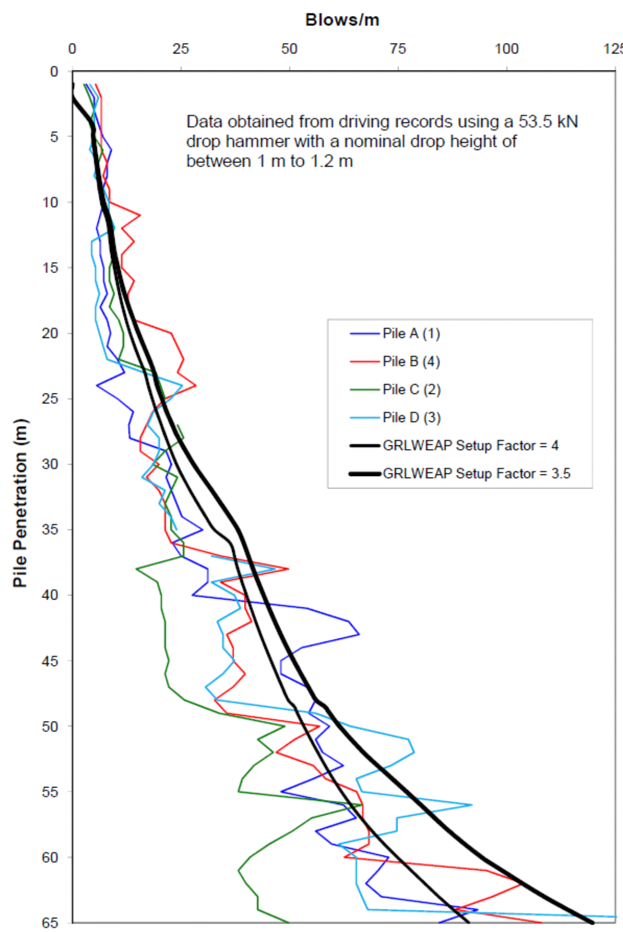

Figure 5. Blow count profile from driving records taken during the installation of the four test piles.

The piles used were spirally-welded so the gauge locations were important in order to obtain good results. The sensors were placed in order to minimize the effect of the weld on the measured signals. Complementary sensors were placed diametrically opposite on both sides of the pile at the quarter point of the spiral. The gauges were placed at least one-and-a-half pile diameters from the top of the pile (where impact occurs) in accordance with ASTM recommendations (ASTM D4945). For the majority of the piles tested, distances of 2 to 3 pile diameters were achieved. Good quality data were obtained using this setup.

The data were recorded on the GRL PAK Model PDA® data acquisition system owned and operated by MEG Consulting Ltd. (M+EG). The data acquisition system records the force and velocity caused by the hammer impact on the top of the pile and determines the approximate axial capacity based on a signal-matching procedure (CASE method). Subsequent analyses with the CAPWAP® software permit the soil characteristics to be included in the interpretation of the measured data and leads to a better-defined capacity assessment and distribution along the pile.

5 PDA®/CAPWAP® RESULTS

The axial pile capacities obtained from the CAPWAP® interpretation of the measured PDA® data were plotted as a function of time after initial driving. The axial capacity in clay is generally expected to increase with time as the excess pore pressures generated during initial driving gradually dissipate. The time for the pile to achieve its full capacity is partly a function of how fast the excess pore pressures dissipate. Viscous effects also affect the rate of strength increase with time. The testing program was initially designed to record the axial capacity setup over a period of 30 days. Based on our experience of pile behavior at other clay sites, it was expected that full setup would be essentially complete by the end of 30 days. There was also the expectation that setup might occur relatively quickly given that the cohesive soils in the profile, particularly at depth, have low plasticity and a high silt content.

The final testing on piles B and D was completed on March 9, 2006. The 28-day axial capacity for Pile B was mobilized using the 58 kN hammer with a drop height of 9.14 m (30 ft) and was estimated as 6,927 kN. For the test at Pile D, it was not possible to mobilize the full axial capacity since the maximum drop height attainable was only 6.55 m (21.5 ft), due to the pile stick-up. For the 29-day set up period, it was only possible to mobilize 6,200 kN of the available capacity for Pile D. Nominal 30-day tests were not possible for piles A and C.

The interpretation of the CAPWAP data was performed by M+EG with partial review of some of the results by GRL.

The pile analyses have been performed using reasonably dimensioned parameters which should lead to a slightly conservative estimate of the axial capacity for each case. A final signal matching coefficient of between 1.5 and 2 was achieved for every case analyzed.

The ultimate axial pile capacity data are plotted on Figure 6 against setup time. The data have been matched using a hyperbolic relationship which suggests an ultimate axial capacity in compression of about 6,900 kN at 30 days. This is in agreement with the 28-day re-strike on Pile B. The hyperbolic relationship is also compared against the setup calculated using radial consolidation theory and c_h values derived from the CPTU dissipation data.

Since the clay layer is over 80 m thick, the dissipation of the excess pore pressure is assumed to occur radially. The prediction for the dissipation of the excess pore pressures around the pile was estimated using consolidation theory equation with average horizontal coefficient of consolidation c_h = 0.25 cm^2/min as the average value for the clay. The prediction of pile capacity with time is included in Figure 6 and agrees very well with the measured PDA setup results.

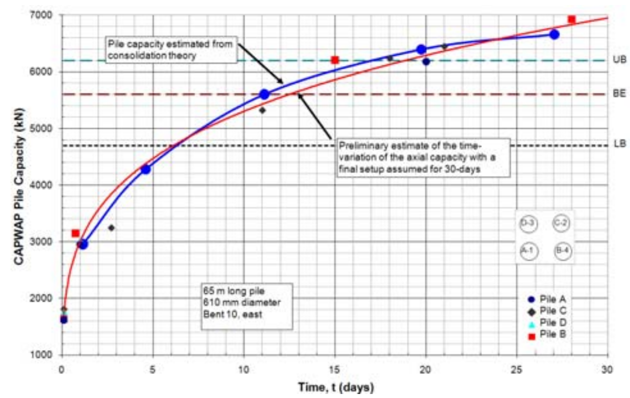

Figure 6. Variation of the ultimate axial capacity with time (setup).

6 DESIGN BASIS FOR AXIAL PILE CAPACITY

Since the axial loads on the piles across the site will be different, the results of the PDA® monitoring and CAPWAP® analyses had to be translated into a variation of ultimate axial capacity with depth.

The BE, LB and UB axial capacities for the Modified API (with L/D correction) procedure for a 65 m long, 610 mm diameter steel open-ended pile are indicated on Figure 6 for comparison with the test results. The data corresponding to the 15- to 28-day re-strikes all fall on, or above, the upper bound ultimate axial capacity estimate, which was determined based on the upper bound undrained strength profile.

The data from Figure 6 can be re-plotted in terms of normalized axial capacity. The relationship between the normalized capacity and setup time can be represented by the following equation:

$$R_t/R_{EOID} = 1.76(t)^{0.244} \qquad (R^2 = 0.97)$$

where t is the time after initial driving in days.

For t = 30 days, the best-fit line to the measured PDA® data and interpreted CAPWAP® results would suggest that the setup factor would be 4.0, i.e. that the axial pile capacity after a setup period of 30 days is four times the capacity at the end of initial driving. This is consistent with the average sensitivity of the soils that was determined to be about 4 (Figure 2). For t = 20 days, the setup factor is 3.63.

Using the measured PDA test data, the expected average error in the predicted axial pile capacity for setup times of between 10 days and 30 days is +/-190 kN.

From the CAPWAP data, the average, minimum and maximum EOID capacities are 1,705 kN, 1,620 kN and 1,810 kN. For a setup factor of 4, the average 30-day axial capacity for a 65 m long pile is calculated as 6,820 kN with minimum and maximum values of 6,480 kN and 7,240 kN, respectively. All of these capacities are in excess of the UB estimate of 6,300 kN for a 610 mm diameter pile (Figure 6) if the L/D correction is applied. The average value of 6,820 kN corresponds well with the calculated API capacity (6,800 kN) if no L/D correction is applied. The PDA results would suggest that the L/D correction to the friction capacity determined by the API RP2A method is not applicable for the Langley Clay.

7 AXIAL PILE CAPACITY DESIGN APPROACH

The results presented above suggest that the long-term axial capacity of long slender piles in Langley Clay could be based on measured soil parameters from field and laboratory testing. More specifically, the undrained shear strength from laboratory tests can be combined with the API RP2A design approach and the results of CPT soundings to deter-mine both-short term and long-term axial capacity. The proposed design approach would consist of the following steps:

i. Determine the undrained shear strength profile based on CPT, in situ shear vane and laboratory strength tests:

$$S_{u(BE)} = S_{u(DESIGN)}$$

ii. Calculate the ultimate axial capacity for the pile using the API RP2A (2000) method. If the L/D correction is applied, then the calculated capacity will be slightly conservative. In no L/D correction is applied, the axial capacity will be closer to an upper bound value.

$$(Q_{ult})_{BE} = (Q_{ult})_{API}$$

iii. Determine the remolded undrained strength profile based on in situ field vane tests or using the sleeve friction measurements from CPT.

iv. Use the remolded soil strength from both CPT (S_{ur}=f_s) values and S_{ur} from UU and FVT to calculate the remolded axial capacity. The average of the two calculations can be used to define the remolded capacity which can be used for evaluating resistance to driving. The resistance should be multiplied by appropriate factors to estimate the required hammer size for pile installation. A factor of 1.2 to 1.5 on the remolded axial capacity (based on rate effects) would be suggested to assess the resistance to driving. The axial capacity calculated in this way would provide the conditions at the end of initial driving (EOID).

$$(Q_{ult})_{REM} = AVG[(Q_{ult})_{BE} *(S_t + S_f)]$$

v. Use the soil sensitivity (S_t) from the peak and remolded undrained strength measurements from the UU and FVT results to calculate a pile capacity setup factor (S_t=S_f). The ratio between the peak and remolded strengths determined from the CPT tip resistance (q_t) and sleeve friction (f_s) can also be used, where $S_f = S_u(q_t)/S_{ur}(f_s)$. q_t and f_s values should be averaged over 500 mm windows at the depths where laboratory or field strength tests have been performed.

vi. The ratio between the remolded axial capacity in (iv) and the full peak axial capacity in (ii) can be related to, and should be similar to, the S_f profile from (iv). S_f from the UU and FVT tests will provide a lower bound value; the $S_u(q_t)/S_{ur}(f_s)$ ratio will provide an upper bound.

vii. The relationship between the normalized capacity and setup time can be used to calculate the pile capacity at any time after initial installation. If necessary, PDA testing can be performed on the pile at any time t after initial pile installation and the final capacity determined using the R_t/R_{EOID} relationship presented above.

viii. Alternatively, the prediction of the pile capacity at any time after driving can be obtained from consolidation theory with the horizontal coefficient of consolidation determined from in situ CPTU dissipation tests.

By the above method, two independent estimates of the best-estimate ultimate axial pile capacity can be obtained. PDA testing will allow a third assessment to be made. Lower and upper bound estimates can also be derived. This should increase the reliability of the pile design method and provide cost-effective evaluations of required pile lengths based solely on axial capacity considerations.

8 CONCLUDING COMMENTS

The results of a series of PDA® measurements and CAPWAP® analyses have demonstrated that the 20-day (or more) axial capacity of four piles installed to a depth of 65 m below existing grade is slightly greater than the capacity calculated using the recommendations contained in API RP2A (2000). The 30-day axial capacity was determined to be even higher.

The expected axial capacity for a 610 mm diameter pile can be predicted either from the setup relationship with the end-of-driving PDA® result (using the relationship derived from Figure 6) or by comparing a PDA® result for a specific setup time with the setup curve presented on Figure 6. For the latter case, a minimum setup time of about 10 days would appear to be acceptable in order to reduce errors associated with the initial steep section of the setup curve. The expected error associated with the above approach is estimated to be +/-200 kN. Where the setup curve of Figure 6 is not available, consolidation theory based on CPTU-derived c_h values from dissipation tests can be used to evaluate the setup with time.

The results of the PDA tests and analyses indicate that the ultimate capacity can be taken as the upper bound curve derived from the Modified API RP2A methodology (which includes an L/D correction). However, the PDA result gives better agreement with the API calculation when the L/D correction is not applied.

As suggested by the data plotted on Figure 4, the API RP2A (2000) predicted axial capacities were significantly higher than those estimated based on the direct CPT methods (Bustamante and Gianeselli 1982; Eslami and Fellenius 1997). This would suggest that the direct CPT methods for determining the ultimate axial capacity of piles are conservative and may lead to significantly longer piles. Given that direct CPT methods usually require a minimum factor of safety of 3, PDA testing should be performed to allow the upgrading of CPT calculated axial capacities.

The results presented have been obtained for a specific site with the characteristic clay parameters described in this paper. The results are only applicable to this site and any application to other sites will require careful consideration of the assumptions involved. However, with good sampling procedures and quality laboratory testing, coupled with PDA testing, the method should be generally applicable to other standard normally consolidated to lightly overconsolidated clay sites.

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