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Effective stress analysis and set-up for shaft capacity of piles in clay

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ABSTRACT

A case history of repeated dynamic and static loading tests in Alberta on two pipe piles during dissipation of driving-induced pore pressures is presented together with three reanalyzed published case histories involving similar records. The four case histories demonstrate that, for each case, the same effective-stress proportionality coefficients, beta-coefficients, fit the capacities at different degrees of dissipation of excess pore pressures. For two of the test sites, the beta-coefficients back-calculated from the tests differed considerable from the values determined from the soil plasticity relation, while for two, the agreement is good. For one case, the back-calculated shaft resistance agreed well with the values of vane shear strength, while a less good agreement was found for the other tests. Neither case showed good agreement was found for methods combining undrained shear strength and effective overburden stress. Capacity calculations for two cases employing methods based on CPT soundings gave excellent agreement with one test and a poor agreement with the other. The increase of capacity due to aging after dissipation of excess pore pressures did not agree with cited recommendation for calculations of aging effect.

INTRODUCTION

It is well-accepted in engineering practice that load-transfer for piles in sand is proportional to effective overburden stress. Since more than 40 years, investigations on instrumented piles in clay (e.g., Bjerrum et al. 1965; 1969, Endo et al. 1969, Burland 1973, Konrad and Roy 1987, Bond and Jardine 1995) have established that this applies also to load transfer in clay. Yet, many practioners still consider the unit shaft resistance to be a function of the undrained shear strength of the clay and calculate the unit shaft resistance along a pile from the undrained shear strength values determined from in-situ tests such as vane-shear tests, converted from results of CPT-soundings, or obtained from laboratory tests on recovered samples using various types of undrained tests. The enduring popularity of total stress analysis is partially due to the ease and low cost of field-vane testing and of obtaining representative samples in cohesive soil for laboratory testing. When data are available that can be combined with various theoretical solutions for shear forces based on bearing capacity formulae or more elaborate approaches, it is

understandably tempting to correlate these values to pile unit shaft resistance. Inasmuch as this approach is based on and adjusted to observations of actually measured pile shaft resistance, the results can often be used to predict closely the shaft resistance for similar piles in soils of similar geologic background and subjected to similar loading conditions—experience is the paramount tool for geotechnical engineers. However, in the author's opinion, such empirically correlated undrained values are little more than index quantities, useful for the designer working in a known soil, but the values do not correlate consistently to pile unit shaft resistance in general.

The universality of effective stress analysis in clays, as opposed to total stress analysis, is demonstrated in a case history from Alberta involving full-scale static and dynamic testing of two pipe piles, with correlation to field vane, CPTU soundings, and measured pore pressures. The results are put into context by results from reanalysis of three other case histories from full-scale tests in Bangkok Thailand, Vancouver, and Quebec. The case data results are first correlated to the effective overburden stress and then discussed in direct reference to undrained shear strength and to methods combining undrained shear strength and effective stress.

CASE HISTORIES

Bangkok, Thailand

Results of short-term measurements of load-transfer were presented by Indraratna et al. (1992) on a 20 m long, 400 mm diameter concrete pile driven in highly plastic, soft to firm, marine clay in Bangkok, Thailand (Fig. 1). The water content of the clay is about 90 % and the plasticity index is about 50. The undrained shear strength determined by field vane tests ranges from about 10 KPa at 4 m depth increasing progressively to about 40 KPa at 18 m depth. The groundwater table lies at a depth of about 2 m and the pore pressure distribution is hydrostatic within the upper 20 m depth. Interrupting the pile installation of the pile, a series of uplift loading tests were performed one week after driving at depths of 8 m, 12 m, 16 m, and 20 m. The author has back-calculated the distribution using effective stress analysis to approximately match the pile capacities for each depth. For all four tests, a good fit was obtained for a beta-coefficient of 0.25, constant along the full pile lengths.

Vancouver, BC, Canada

Davies (1987) and Robertson et al. (1988) reported the results of a series of static loading tests performed in Fraser River deltaic soils at Lulu Island, Vancouver, BC. The soil profile consists of a surficial 2 m thick sand fill placed on 13 m of very soft clay with silt and sand lenses that overlies a 13 m thick layer of compact micaceous sand followed by soft clay and silty clay to large depth and containing numerous silt and sand lenses. Results from vane shear tests are not available. The water content of the clay at 2 m through 15 m depth is about 30 % and the plasticity index is about 15. The upper 2 m to 3 m and the lower about 2 m thick portions of the sand layer are interspersed with clay and silt lenses. The CPTU cone stress, q_t , ranges from about 8 MPa through about 10 MPa. The groundwater table lies at a depth of

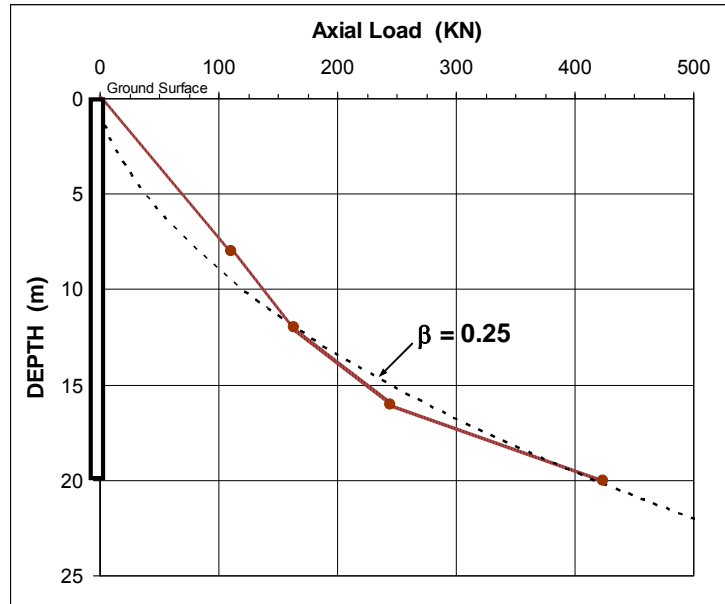


Fig. 1 Distribution of capacity in tension (uplift) tests at different depths
(Data from Indraratna et al. 1992)

about 1 m and the pressure distribution is hydrostatic within the embedment depth of the piles. The soils are similar to those at the adjacent Annacis Island, described by Bazett and McCammon (1986).

Three 324-mm diameter closed-toe pipe piles driven to embedment depths of 13.7 m, 16.8 m, and 31.1 m were subjected to static loading tests at 197 days, 85 days, and 38 days after initial driving, respectively. The toe of Pile 13.7 m was about 1.3 m (about 4 pile diameters) above the sand. Pile 16.9 m was driven about 1.9 m into the weaker, clay and silt interspersed upper portion of sand layer. Pile 31.1 m was driven about 3 m below the bottom boundary of the sand layer, i.e., about 10 pile diameters below the sand/clay boundary.

The load-movement curves of the static loading tests are presented in Fig. 2. What capacity value to evaluate from the tests is a matter of judgment and personal preference. The sloping dashed line in the figure rising from the abscissa is the offset elastic line whose intersection with the curves defines the offset-limit load, commonly considered to be a lower-bound capacity value. The increase after the offset-limit load is considered to be mostly due to increase of toe resistance. While full dissipation of driving-induced pore pressures might not have occurred for the 38-day test on Pile 31.1 m, they have probably dissipated for the 85-day test on Pile 16.8 m and for the 197-day test on Pile 13.7 m.

Two methods for calculating the pile capacity were applied to the CPTU sounding: the CPT-based LCPC method (Bustamante and Gianeselli 1982) and the CPTU-based

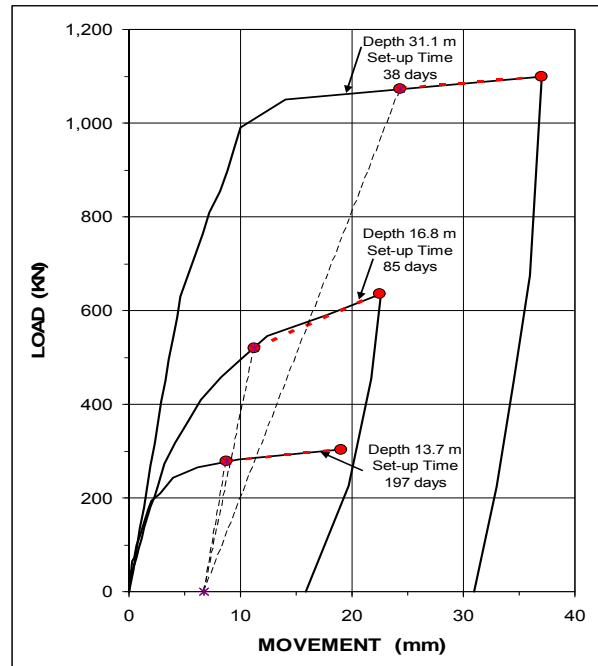


Fig. 2 Load-movement curves from static loading tests on three 324 mm diameter, closed-toe, driven pipe piles in Vancouver, BC. Data from Davies (1987).

Eslami-Fellenius method (Eslami and Fellenius 1997). Fig. 3A presents the distribution of the total shaft resistances according to the two methods. The shaft resistance distributions are very similar, which is a coincidence because the methods differ appreciably: The CPT-method uses the cone stress uncorrected for pore pressure on the cone shoulder, disregards the sleeve friction, and includes cut-off limits of shaft resistance based on ranges of cone stress and either of two main soil types, clay and sand, as input by the user. In contrast, the CPTU-method uses corrected cone stress values and determines the soil type directly from the sounding, separating the profile on six soil types for every measurement point. Sleeve friction is relied on for the determination of soil type and is referenced in determining the toe resistance.

The fit of the loading test results to an effective stress analysis was made by dividing the shaft resistance values determined by the CPTU-method by the effective overburden stresses at corresponding depths, resulting in equivalent values of β -coefficients. The average values in the three main soil layers were 0.20, 0.30, 0.20, and 0.15, respectively. These values are low, which probably is due to the presence of mica flakes in the soil. As shown by Gilboy (1928), micaceous soils are more compressible and weaker as opposed to non-micaceous soils. The average shaft resistances were used to determine the distribution of total capacity. The calculated resistance distributions are shown in Fig. 3B together with the resistances of each test shown as pairs of square dots, with the first indicating the estimated shaft resistance and the second the total resistance of the test. The average of the CPT and the CPTU methods is indicated in the figure as "effective stress". The beta-coefficients and toe coefficients employed are also indicated in Fig. 3.

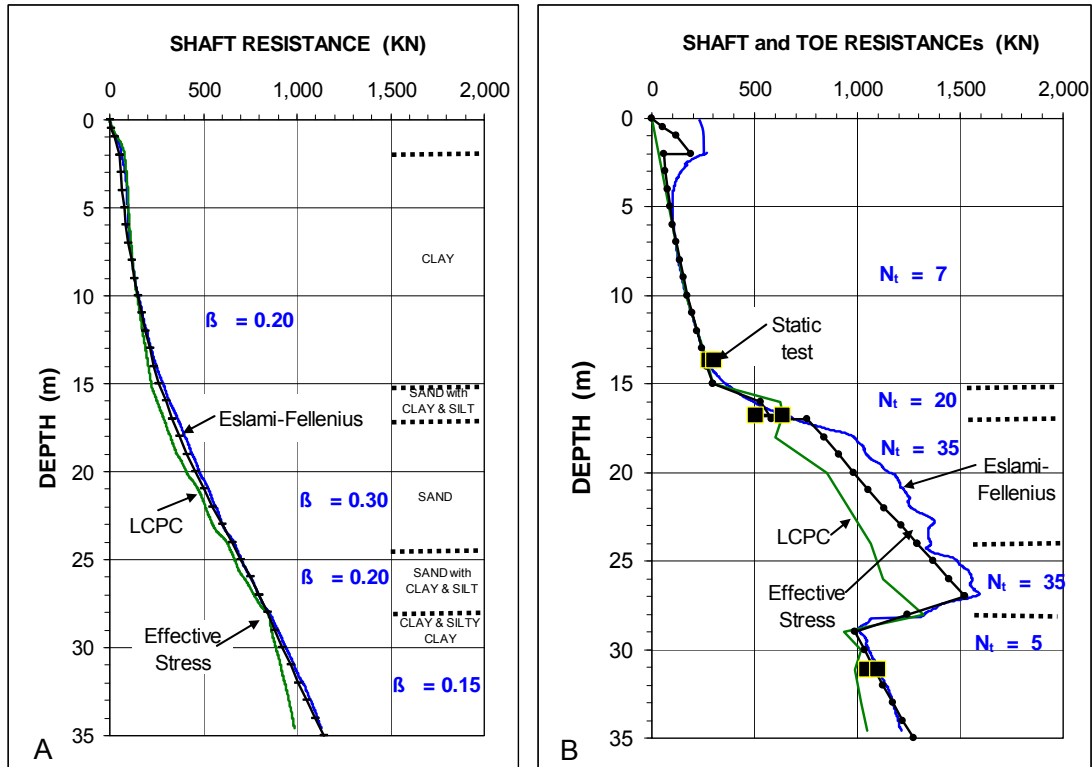


Fig. 3 Distributions of shaft resistance (A) and total resistance (B) from effective stress calculations fitted to the distributions calculated from the CPT-LCPC and the CPTU Eslami-Fellenius methods and to capacity from the static loading tests (Bustamante and Gianceselli 1982; Eslami and Fellenius 1997).

Quebec, Canada

Konrad and Roy (1987) and Roy et al. (1981) reported static loading tests on two 7.6 m long, 219 mm diameter closed-toe pipe piles jacked into Champlain clay at the St Alban test site, west of Quebec City. The soil profile consists of thin weathered clay crust on 8 m of soft silty marine clay. The plasticity index of the silty clay is about 20 and the water content is appreciably higher than the liquid limit. The pore pressure distribution is hydrostatic from a groundwater table located at a depth of 0.4 m. The deposit is overconsolidated with an OCR of about 2.2 developed due to secondary consolidation or aging of the clay. The undrained shear strength measured by in-situ vane tests varies linearly from 10 KPa at 2 m depth to a maximum of 30 KPa at the bottom of the soft silty clay layer. The values are the average determined in 17 soundings with the envelope of the low and high boundaries deviating about $\pm 30\%$ from the average.

The piles were instrumented to separate shaft and toe resistances, and piezometers were installed at different depths next to the piles. The ultimate shaft resistances determined at the end of the jacking installation and at several static loading tests performed at 4 days, 8 days, 20 days, and two years after the installation are shown in Fig. 4. The values are shown as a ratio (%) between ultimate resistance at the

specific test over the resistance (77 kN) of the two-year test, and ratio (%) of the excess pore pressures measured on the specific test occasion over the excess pore pressures immediately after the pile installation.

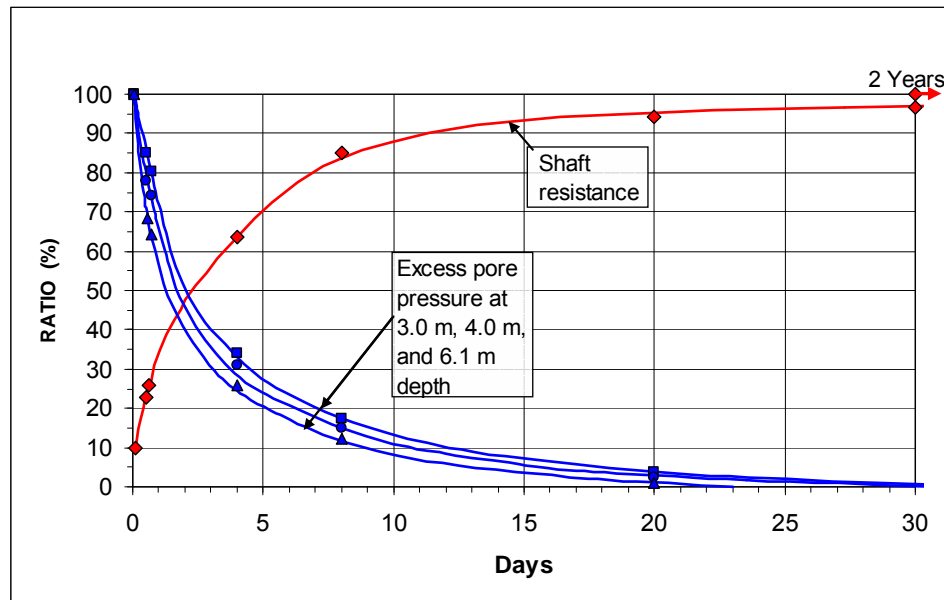


Fig. 4 Shaft resistance and excess pore pressure measurements at installation and at several static loading tests during up to two years after the pile was jacked into the soil at Quebec. (Data from Konrad and Roy 1987).

The two-year ultimate shaft resistance corresponds to a beta-coefficient of 0.6 (ratio of shaft shear to effective overburden stress). When the analysis is repeated for the effective stress adjusted for the excess pore pressures measured at 4 days, 8 days, and 20 days after the installation, a fit between the actual effective stresses and the measured ultimate resistances is achieved with beta-coefficients of 0.7, 0.6, and 0.6, respectively. That is, essentially the same beta-coefficient applies to the shaft resistances measured for all three of the mentioned testing occasions during the ongoing dissipation of the excess pore pressures.

North of Edmonton, Alberta, Canada

In 1990, Alberta Department of Transportation, undertook a long-term study of the development of capacity with time for two 324 mm diameter, closed-toe, concrete-filled, pipe piles at a highway bridge site located about 180 Km Northwest of Edmonton, Alberta, for which project the author served as a consultant. The site conditions are presented by Dyaljee and Pariti (2002).

The two test piles were driven on October 30, 1990, about 2.4 m apart in line with three previously installed anchor piles to depths of 16 m and 20 m. Static loading tests were performed 15 days, 30 days, and 4 years (1,485 days) after EOID. Restrike tests were carried out one day after end-of-driving and at the 30-day tests the day before the static loading test. The 30-day restrike consisted of 5 to 8 blows to a total penetration of about 50 mm.

The clay is an eroded, transported, and re-deposited glacial till clay interspersed at 5 m depth by an about 0.5 m thick layer of silty sand. The groundwater table lies at a depth of 1.5 m and the pore pressures are hydrostatically distributed. The soil profile consists of silty clay with a water content of about 35 % through 70 %, a Liquid Limit of about 60 % through 70 %, a Plastic Limit of about 15 % through 40 %, and a Plasticity Index of about 25 %. Three field vane tests were performed at 8 m, 11 m, and 17 m depth, showing almost linearly increasing values from 45 KPa through 85 KPa. The Janbu modulus number ranges from 12 through 20 and the recompression modulus is about fifteen times larger. The OCR is about 3. Triaxial consolidated and undrained tests and direct shear testing on the clay indicated a strain-softening soil having a friction angle ranging from 21° through 25° with a residual (post peak) value of about 21°. A small cohesion intercept was found in the range of 10 KPa through 25 KPa.

At a depth of 7.5 m lies a second 0.5 m thick layer of silty sand. The sand is followed by soft silty sandy gravelly ablation clay till that continues to the end of the borehole at a depth of about 26 m. The ranges of water content and indices of the clay till are about the same as those of the upper clay layer. Consolidation tests on samples from the clay till show the Janbu modulus number of the clay to range from 19 through 21 and the recompression modulus to be about ten times larger. The OCR is about 2.

The results of a CPTU sounding at the pile testing site indicate a relatively homogeneous cone stress in the clay and the clay till with a cone stress, q_t , ranging from about 1 MPa through 2 MPa. According to Kulhawy and Mayne (1990), the undrained shear strength can be determined by subtracting the overburden total stress from the corrected cone stress and dividing the balance with a factor " N_{kt} " ranging from 10 through 20. A N_{kt} -factor of 10 makes a good fit between all three vane shear values and the q_t -values at corresponding depths.

Prior to the pile driving, six piezometers were installed on the center of the test pile group to depths of 3 m, 6 m, 9 m, 11 m, 16 m, and 18 m, respectively. As indicated in Fig. 5, the piezometer measurements show that the pile driving produced considerable excess pore pressures that dissipated with time. The actual time for full dissipation was not measured. However, the extrapolation in the logarithmic plot, Fig. 4B, indicates that the dissipation was completed 70 days after the driving.

In Fig. 6, the vertical distribution of pore pressures are shown as measured before the pile driving, 15-days, 30 days, and 4 years after the pile driving. (The 4-year measurements were obtained in a new set of piezometers also installed near the test piles and to the same depths as the original piezometers). The measurements show that significant excess pressures still remained at the time of the 30-day static loading tests.

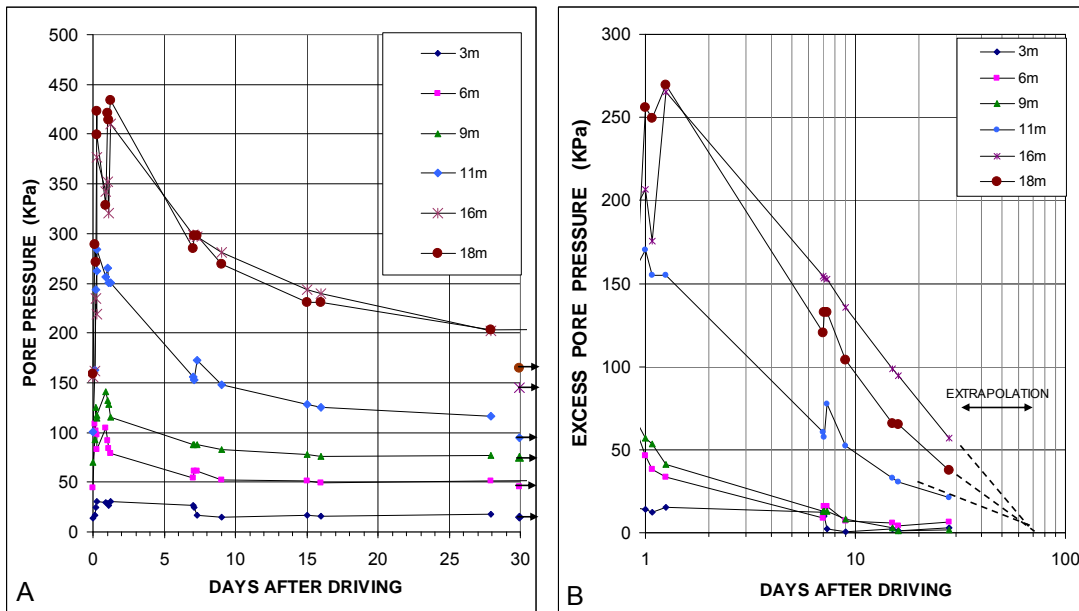


Fig. 5 Dissipation of measured pore water pressures with time (days) after driving. A. 0 - 30 days, Linear scale. B. 1 - 100 days, Logarithmic scale

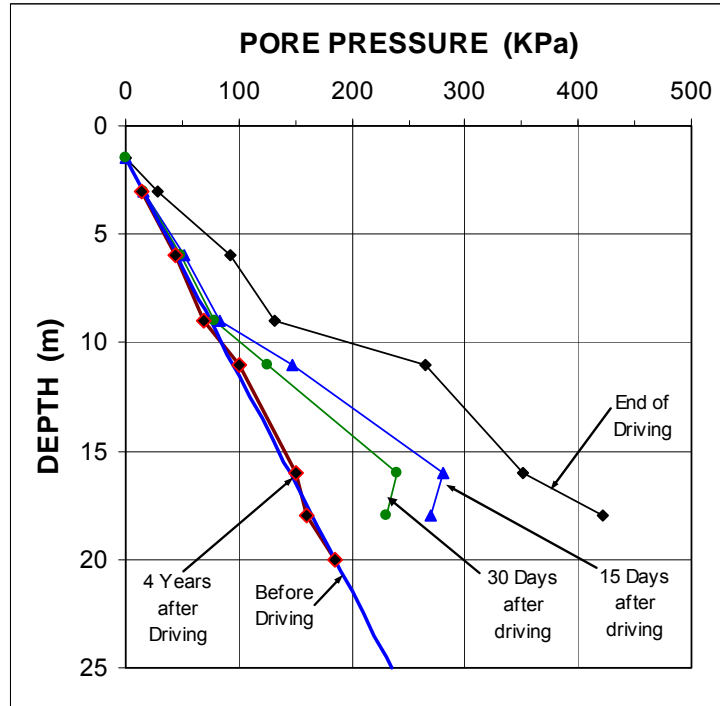


Fig. 6 Distribution of measured pore water pressures with depth.

Fig. 7 presents the load-movement curves of the static loading tests on Piles 16 m and 20 m, respectively. The 15-day and 30-day tests on Pile 16 m were carried to movements in excess of 100 mm. After a movement of about 12 mm, the applied load reached a peak, which is taken as the ultimate resistance (capacity) of the test. Thereafter, the load required to move the pile diminished. This is an indication that the toe resistance of both piles is very small for all tests. When evaluating change of pile capacity with time by means of repeated compression tests, so-called "stage-testing", it is essential that the toe resistance is small. Otherwise, the movement-dependent increase of toe resistance from one test to the next could falsely indicate a capacity increase with time (Fellenius 2002).

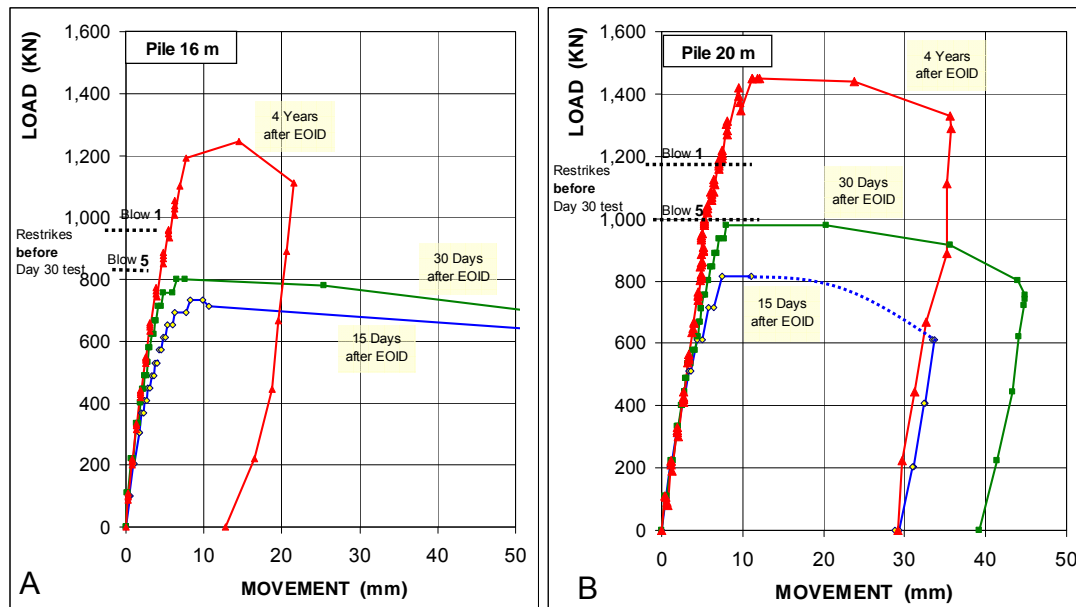


Fig. 7 Load-Movement curves from tests on Pile 16 m.

A: Pile 16 m. B: Pile 20 m.

For both test piles, the capacity increased from one test to the next. This phenomenon is called "set-up". With regard to shaft resistance, set-up is usually credited to dissipation of excess pore pressures and reconsolidation of the disturbed clay. The piles were restruck the day before the 30-day static loading tests. Dynamic analysis (CAPWAP) on the restrike data show that the restrike reduced the capacity from blow to blow and, therefore, also the capacity in the subsequent static loading test. That is, the 30-day capacities would have been higher had the restrike not been performed.

The CPTU analyses of resistance distribution show a gradually increasing unit shaft resistance with depth and a small toe resistance, no more than about 50 kN to 100 kN. However, the CPT methods considerably underestimate the capacity of the test piles. Fig. 8 shows a comparison between the capacities determined in the test and calculated from five cone methods: Eslami and Fellenius (1997), Tumay (Tumay and Fakhroo 1981), Schmertmann (1975), Dutch (DeRuiter and Beringen

1979), and LCPC (Bustamante and Gianceselli 1982). The full height of the columns is the capacity and the lowest part of each column is the toe resistance. Moreover, unlike the Vancouver case, the Eslami-Fellenius and the LCPC methods give very different results. The reason lies in the somewhat arbitrary cut-off limits of shaft resistance imposed in the LCPC method and its use of uncorrected cone stress values. This reason explains both the agreement between the two methods found in the Vancouver case and the disagreement found in the Alberta case.

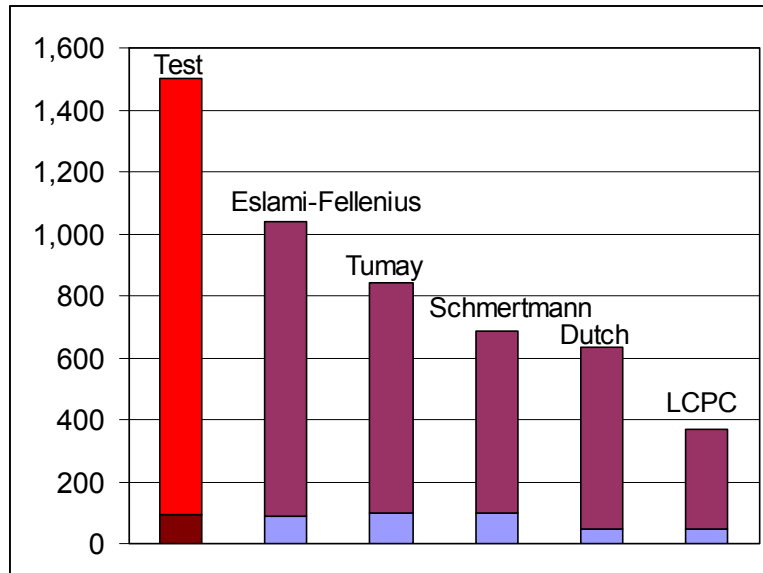


Fig. 8 Capacities determined in the static loading test on the 20 m long pile compared to capacities calculated by five CPT methods for the Alberta case.

The mentioned gradual increase of unit shaft resistance with depth supports that the resistance is controlled by effective stress. Although the pore pressure distribution is well established, the distribution of total stress along the piles to be less well defined than desirable: while the site is generally horizontal, some ten metre beyond the piles, the natural ground slopes gently down toward a river located at a distance of about 20 m away from the piles, and a few metre away from the test pile location is a two metre high road embankment. Calculations using upper bound ground surface contours (no embankment but slope toward the river and vice versa) versus horizontal ground surface show the uncertainty in the amount of effective stress to be about $\pm 15\%$. Therefore, an acceptably representative effective stress distribution for the long-term condition can be determined on the assumption of horizontal ground surface in the vicinity of the test piles. For this condition, a beta-coefficient of 0.5 and a toe-bearing coefficient (N_t) of 6, result in a fit to the capacity of the 4-year static loading tests for both piles.

The difference of effective stress for the 15-day and 30-day tests on the piles in relation to the 4-year tests is known because it is equal to the difference of pore pressure illustrated in Fig. 6. By multiplying the pore pressure differences to the hydrostatic line with the beta-coefficient and subtracting the values from the 4-year

distributions of resistance, the resistance distributions are determined for the 15-day and 30-day tests. The so-determined distributions are shown in Fig. 9 for the three static loading tests on Piles 16 m and 20 m, respectively.

The calculated capacity for the 30-day test is larger than the capacity of the static loading test. As indicated in the foregoing, the 30-day static loading test is affected by the preceding restrrike. The calculated 30-day capacities should indeed come out larger than the values found in the static loading tests. That the agreement between the calculated capacities is as good as shown in Figs. 9A and 9B is probably somewhat fortuitous. The main finding is that a reasonable agreement between the calculations for the three pairs of static loading tests, performed at different extents of pore pressure dissipation, is obtained by assigning the soil-pile response to be proportional to the effective overburden stress.

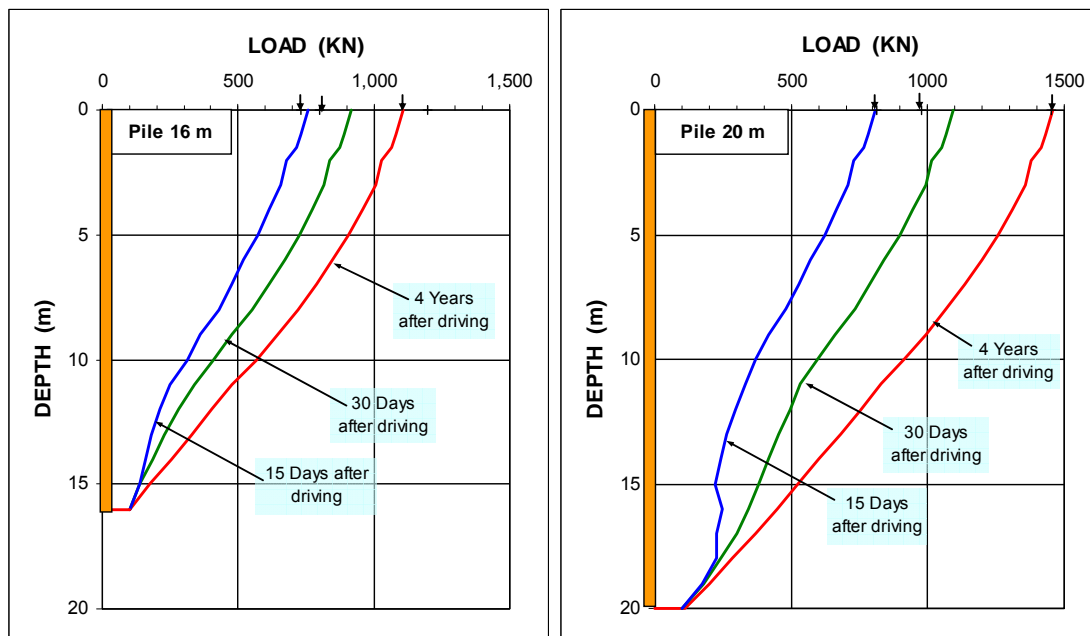


Fig. 9 Load distribution for the 15 day and 30 day loading test capacities calculated by the same coefficient values as used for the 4 year test and with adjustment to the measured excess pore pressures. The small arrows on the abscissa indicate capacities determined in the loading tests. A. Pile 16 m; B. Pile 20m.

DISCUSSION

Shaft Resistance

In principle, the shaft resistance is a friction coefficient times the normal effective stress acting against the pile shaft. The friction coefficient is usually considered to be equal to the $\tan \delta'$, where δ' is the interface friction angle. Bozozuk (1972), incorporated a M-coefficient equal to $\tan \delta' / \tan \phi'$ and suggested that this ratio is

about 0.8. The coefficient, K_s , applied to the overburden stress, is usually set equal to the earth stress coefficient at rest, K_0 , which for normally consolidated clays is taken to be equal to $1 - \sin \phi'$. In overconsolidated clays, Burland (1973) suggested that the beta-coefficient for overconsolidated clays should increase in proportion to the square root of the OCR, also suggested by Esrig and Kirby (1979). Thus, the beta-coefficient is a function of several parameters: the ratio between $\tan \delta'/\tan \phi'$, or $\tan \phi'$ if the shearing is considered to occur a distance out from the pile shaft, the pile surface material and roughness, the earth stress coefficient, the OCR, as shown in Eq. 1.

$$(1) \quad \beta = M \tan \phi' (1 - \sin \phi') (OCR)^{0.5}$$

where

- M = $\tan \delta'/\tan \phi'$
- ϕ' = effective soil friction angle
- δ' = effective interface friction angle;
a function of pile surface texture and material
- OCR = overconsolidation ratio

Jardine et al. (2005) proposed a relation for the β -coefficient according to Eq. 2, obtained from fitting measured pile capacity to a series of soil parameters.

$$(2) \quad \beta = 0.8 \{2.2 + 0.016(OCR) - 0.871 \lg(S_t)\} (OCR)^{0.42} (2h/b)^{-0.2} \sigma'_z \tan \delta'$$

where

- OCR = apparent overconsolidation ratio
- S_t = sensitivity, related to "relative void ratio index" from oedometer tests
- h = height of point above pile toe; $h/b \leq 4$
- b = pile diameter
- δ' = interface friction angle

Regarding Eq. 1, not all of the three parameters are determined for a routine design case and, when they are, they vary considerably depending on natural variation and method of determining them. Also influencing is the method of pile installation, and the soil gradation, mineralogical composition, density, and depositional history, etc. In fact, once the potential ranges of each parameter are considered, for any specific case, the beta-coefficient calculated per Eq. 1 will range from the very small to the very large. In the author's opinion, chances are good that an experience-based judgment call for the beta-coefficient directly would be considerably closer to the actual value than any value determined from the mentioned parameters, however well researched in separate tests.

As to Eq. 2, fitting a series of precise two-digit constants and exponents to approximate indices, such as sensitivity (determined in an oedometer test, no less) and OCR values is hardly applicable to engineering design or analysis. The author finds it difficult to take such a relation seriously.

It should be recognized that the beta-method (as the effective stress method often is called) is a simplistic method. It disregards several factors, such as that the imposed movement of the pile surface in a static loading test causes rotation of principal stresses along the pile surface, can result in strain softening or strain hardening of the soil, and generates small additional pore pressures which reduce the effective stress against the pile surface. Still, as demonstrated in the above four case histories, actual shaft resistance is proportional to the effective overburden stress, and, therefore, the results of a test can be correlated through the beta-coefficient, and the test results be applied to the design of other similar piles at the particular site.

The question of importance to persons active in design of pile foundations is to what extent the beta-coefficient can be determined from soil parameters established in a routine site investigation at a site and applied to a design without first being "calibrated" in a loading test.

Karlsrud et al. (2005) compiled results from shaft resistances determined in tests on pile in clay and presented a relation between the beta-coefficient and the plasticity index, I_p , shown in Fig. 10. The beta-coefficients determined from the cases in Thailand, Vancouver, Quebec, and Alberta are added to the original data. Of the latter, the coefficients from Thailand and Vancouver cases lie close to the mean curve of the original plot. However, the coefficients from Alberta and Quebec deviate considerably from the Karlsrud et al. (2005) curve and data. That the first two cases are from normally consolidated clay, while the latter two are from overconsolidated clay may have had an influence. That is, according to Eq. 1, the beta-coefficients for the Alberta and Quebec would be about 50 % larger than for a normally consolidated clay. However, the values are larger than so and about twice to three times larger than indicated by the mean curve shown in Fig. 10. In the author's opinion, the scatter of data and the fact that two of the four cases deviate from the proposed relation make the plasticity index not directly useful for a specific design case.

The most common method for calculating shaft resistance is the so-called alpha method, which correlates the shaft resistance, r_s , to the undrained shear strength of the clay, τ_u , via a reduction coefficient, α (Eq. 3). For concrete and wood piles in soft clays (soft clay is defined as $\tau_u \leq 25$ KPa), α is equal to unity, while it is about 0.8 for steel piles (CFEM 1992). For undrained shear strengths of 50 KPa and 100 KPa, the α -coefficient is about 0.6 and 0.4, respectively.

$$(3) \quad r_s = \alpha \tau_u$$

where

r_s	=	unit shaft resistance along the pile
α	=	alpha coefficient
τ_u	=	undrained shear strength

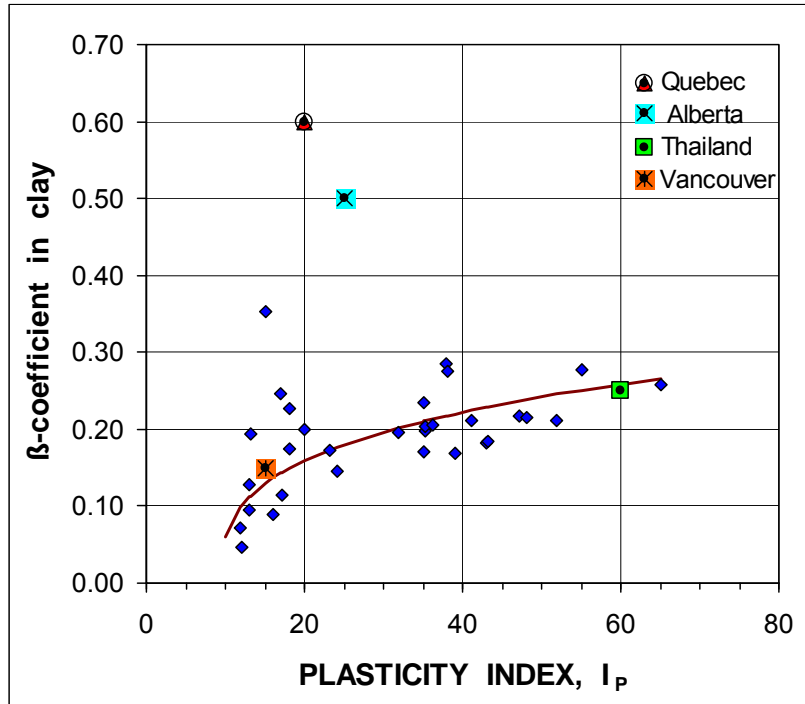


Fig. 10 Beta-coefficient versus average I_p for piles in clay (Data from Karlsrud et al., 2005). The coefficients evaluated for the cases in Quebec, Alberta, Thailand, and Vancouver are added.

Vijayvergiya and Focht (1972) combined undrained shear strength and effective stress proposing a method which equals the shaft resistance to a coefficient, λ , times the sum of the effective stress and twice the undrained shear strength, as shown in Eq. 4. The lambda-coefficient ranges from 0.5 at the ground surface to 0.1 at a depth of 60 m. The lambda method is mostly applied to long, large diameter offshore piles in the Mexican Gulf.

$$(4) \quad r_m = \lambda(\sigma'_m + 2\tau_m)$$

where

- r_m = mean unit shaft resistance along the pile
- λ = lambda coefficient
- σ'_m = mean overburden effective stress
- τ_m = mean undrained shear strength

Randolph (2003), quoting Randolph and Murphy (1985), proposed determining the unit shaft resistance from three alternative combinations of undrained shear strength and effective stress, as shown in Eqs. 5 - 7.

$$(5) \quad r_s = 0.5 (\tau_u)^{0.50} (\sigma'_z)^{0.50}$$

$$(6) \quad r_s = 0.5 (\tau_u)^{0.75} (\sigma'_z)^{0.25}$$

$$(7) \quad r_s = 0.55 (\tau_u)^{0.7} (\sigma'_z)^{0.3} \left(\frac{40}{D/b}\right)^{0.2}$$

where r_s = unit shaft resistance along the pile
 τ_u = undrained shear strength
 σ'_z = overburden effective stress
 D = embedment depth
 b = pile diameter

The case histories from Thailand, Quebec, and Alberta include records of undrained shear strength determined from field vane tests. The Thailand and Alberta only include a few values from one vane test, while the Quebec case includes a large number of vane tests. The vane shear values are used to calculate the unit shaft resistance according to Eq. 3 and Eqs. 5 - 7. The results are plotted in Figs. 11 - 13 versus the unit shaft resistance determined from the results of the static loading tests for the full dissipation of excess pore pressures. No reduction has been made to the undrained shear strength values, i.e., alpha is equal to unity and the B-figures show the actual distribution of the vane shear strength.

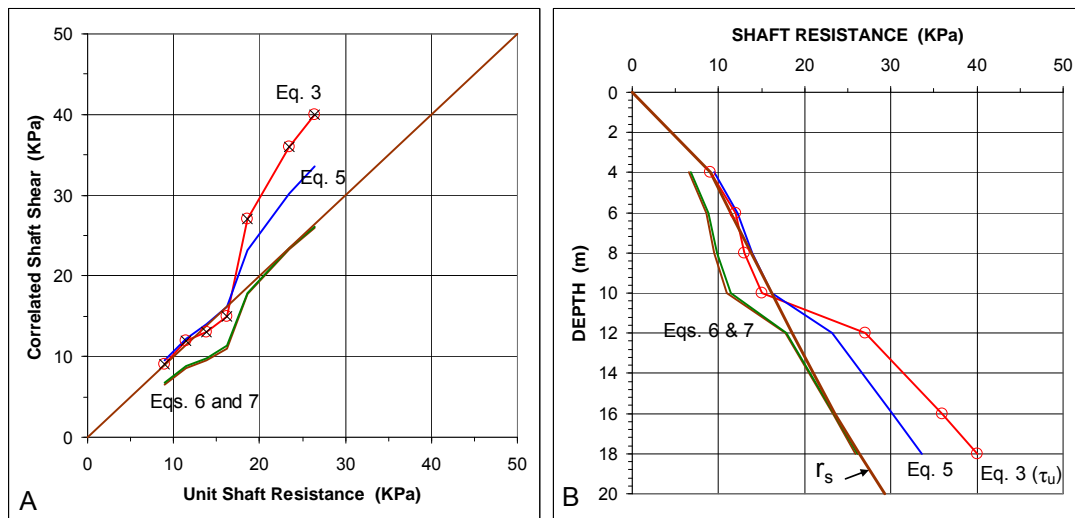


Fig. 11 Comparison between actual unit shaft resistance and shaft resistance calculated according to Eq. 3 and Eqs. 5 - 7 for the Thailand case. A: Direct correlation. B: Distribution with depth.

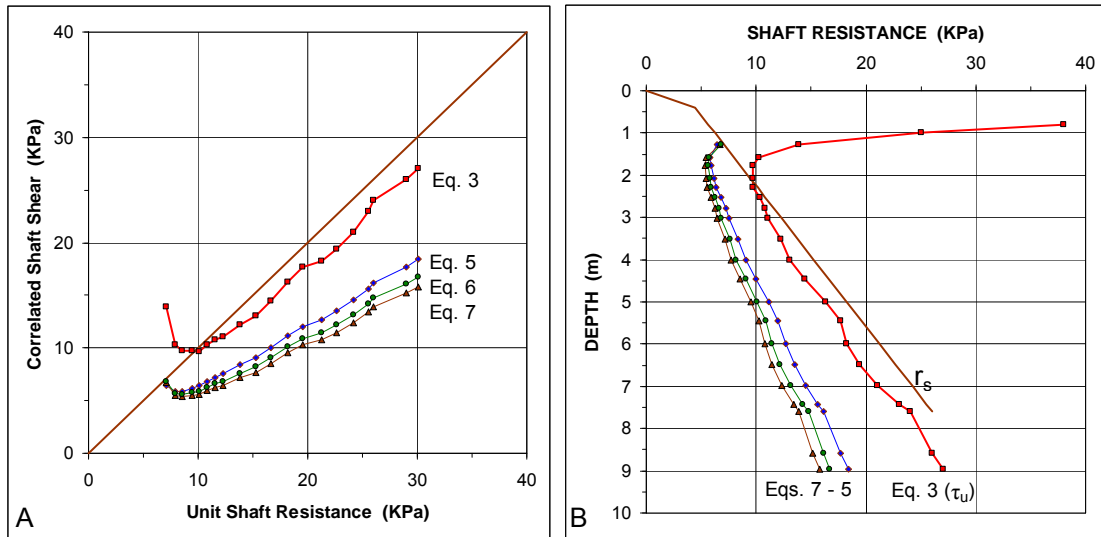


Fig. 12 Comparison between actual unit shaft resistance and shaft resistance calculated according to Eq. 3 and Eqs. 5 - 7 for the Quebec case. A: Direct correlation. B: Distribution with depth.

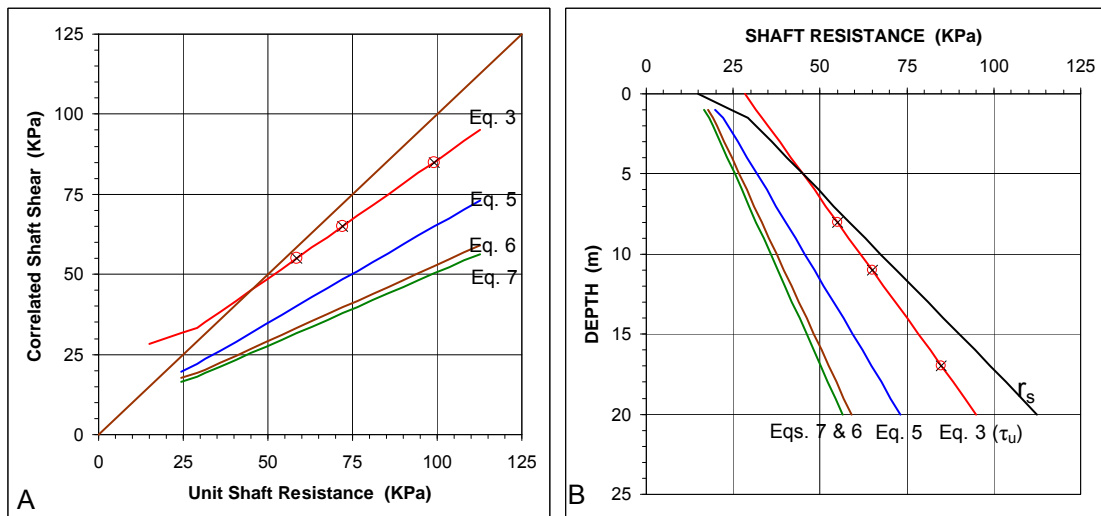


Fig. 13 Comparison between actual unit shaft resistance and shaft resistance calculated according to Eq. 3 and Eqs. 5 - 7 for the Alberta case. A: Direct correlation. B: Distribution with depth.

Of the four methods employing the undrained shear strength for determining shaft resistance, after full dissipation of excess pore pressure, the undrained shear strength values from Eq. 3 show the best correlation. For the Thailand case (Fig. 11), had the adjustment for shear strength exceeding 25 KPa been applied, the correlation would

have been next to perfect. In contrast, the same adjustment for the Alberta case would instead have increased the difference considerably. The calculated values for unit shaft resistance according to Eqs. 3, 5 - 7 differ little from each other, but deviate considerably from the measured shaft resistance.

The Vancouver case does not include any records of undrained shear strength. However, it does include good CPTU records. The two CPT methods applied, the LCPC and the Eslami-Fellenius methods, give very similar distributions of shaft resistance that agree well with the resistances calculated from the static loading tests. In contrast, when the CPT sounding is applied to the Alberta case, the results by the two methods differ considerably from each other. Both underestimate the distribution determined from the static loading test. It is interesting to notice that the capacities calculated by E-F-method for the Alberta case agree quite well with the capacities determined in the 30-day test. The piezometer measurements indicate that only about two-thirds of the excess pore pressures had dissipated at that time. However, it is normally assumed that 30 days wait is sufficient time for the excess pore pressures to dissipate, as the Quebec case also indicates. In the absence of pore pressure data, it might have been thought that the 30-day test was after completed set-up, with the belief supported by that the 30-day capacity being in good agreement with the CPTU method capacity, and the increased capacity found in the two-year test be an effect of aging. When planning a test, it is obviously prudent to install a couple of piezometers to verify that the construction-induced excess pore pressures have dissipated, and, indeed, to determine the actual distribution of also the neutral pore pressures.

Set-up due to aging

Pile capacity will increase after completed pile installation. First, due to dissipation of pore pressure and, then, due to aging — marginally or appreciably. Bullock et al. (2005) indicated a relation for increase of capacity with elapsed time after end of driving. The relation would allow a test performed before full set-up and the final capacity to be calculated — extrapolated — from the relation shown in Eq. 8.

$$(8) \quad Q_{REF} = \left(1 + A \lg \frac{t}{t_{REF}}\right)$$

where

Q_t	=	capacity at Time "t"
Q_{REF}	=	capacity at Time " t_{REF} "
A	=	a constant expressing ratio of capacity increase for a ten-fold increase of elapsed time (days) beyond t_{REF}

Bullock et al. (2005) adapted the capacity at the 1 day elapsed time after end of initial driving as the reference capacity. In contrast, Karlsrud et al. (2005) recommended that the reference capacity should be the capacity after full dissipation of pore pressures and suggested that the reference should be the capacity at 100 days after the driving, being the capacity after full dissipation of the excess pore pressures developed during the installation of the pile. The condition that the excess pore

pressures have indeed dissipated during the first 100 days is neither true for all single piles nor for all pile groups. For example, Fellenius (2006) reported that the dissipation time developed from the driving of a single pile in soft marine clay was six months, and Bozozuk et al. (1978) reported a dissipation time of eight months for a pile group in Champlain clay.

Karlsrud et al. (2005) indicated that the A-value in Eq. 8 would follow Eq. 9.

$$(9) \quad A = 0.1 + 0.4 \left(1 - \frac{I_p}{50}\right) (OCR)^{-0.8}$$

where A = the A-constant of Eq. 8
 I_p = plasticity index, $I_p \leq 50$
 OCR = overconsolidation ratio

As the lower limit values of plasticity index and OCR are zero and unity, respectively, the A-constant according to Eq. 9 cannot become larger than 0.5. Moreover, a plasticity index larger than 50 is input as 50. Therefore, however large the overconsolidation, the A-constant cannot be smaller than 0.1, i.e., $0.1 < A < 0.5$.

Waiting 100 days after driving before testing the pile is normally not practical, however. Therefore, a practical solution to find the 100-day capacity could be by performing a few, minimum two, tests during the set-up period and plotting the pile capacities versus time in a semi-log plot, as illustrated in Fig. 14. The 100 day capacity can then be determined from such a graph by extrapolation. Once the 100-day capacity is determined, Eqs. 8 and 9 could be used to indicate the increase of capacity due to aging beyond the 1 day and 100 day values, respectively.

However, extrapolation from capacities measured during the period of excess pore pressure to the capacity after the pore pressures have dissipated is rather inaccurate. Such hypothetical extrapolation are indicated in Fig. 14 for the Alberta case, taking the lead from the development for the Quebec case. It is only because of this lead, and because the actual four-year test results are known, that the Alberta extrapolations appear reasonable.

The Alberta case plasticity index and OCR values are about 25 and 3, respectively. Inserting these values into Eq. 9 returns $A = 0.4$. With this value for A, Eq. 8 indicates that, after twelve years, the capacity would increase by about two-thirds beyond that at 100 days. Similarly, the A-value for the Quebec case is 0.5, suggesting an 80 % capacity increase. Considering that, for both cases, capacity increase during the first four years and two years, respectively, is marginal, the calculated twelve-year capacity increases appear rather unrealistic.

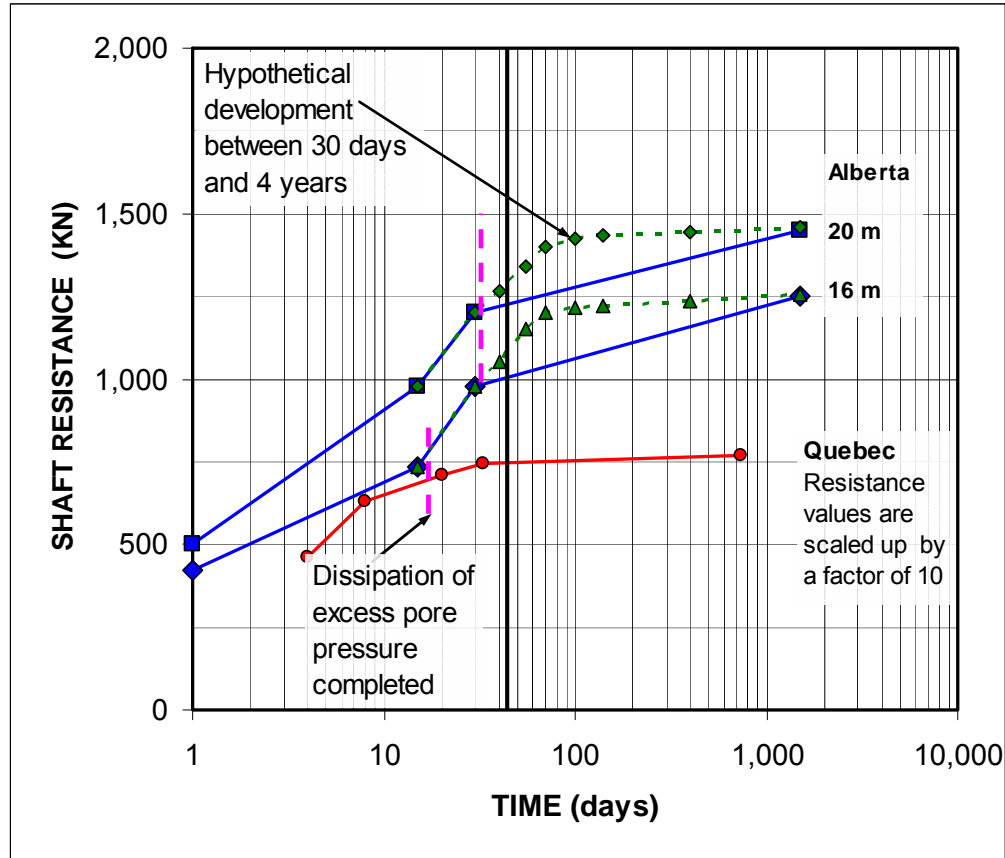


Fig. 14 Capacities versus days after initial driving.

There are very few tests performed many years after the piles were installed, when also the 100-day capacity is determined. However, several such long-term tests have been performed in connection of measurements of drag load due to settling soil (Fellenius 2006). These tests do not indicate increase of resistance once the ultimate shaft resistance has been mobilized in either the negative or positive shear directions. It would seem that the capacity increase due to aging (as opposed to dissipation of excess pore pressure) is rather small and requires very long time to reach appreciable values. “Appreciable” refers to values that can be put to use in a design.

Bullock et al. (2005) suggested that actual design of a piled foundation should make use of the set-up with time determined according to Eqs. 8 and 9 in order to obtain a reference pile capacity larger than tested values. The approach presupposes that the so-extrapolated capacity value be given the same assurance (i.e., same factor of safety) as that found in the test. The author considers this to be a debatable suggestion. The capacity used as a tested capacity in a design should not be larger than the maximum capacity actually found in a test. It is a different matter that Eqs. 8 and 9 can be used to indicate the potential success of investing time and money to carry out a repeated test to establish a larger capacity for the design. Moreover, a designer is free to carry out a theoretical calculation of the capacity and apply a larger factor of safety (or smaller resistance factor) to the results of that calculation.

Verifying capacity through testing.

In addition to having the field investigation including CPTU soundings and vane shear tests, frequently, to optimize the use of the piles, testing for capacity is warranted. Note, a static loading test does not replace a thorough site investigation. Ideally, the test should not start until piezometer readings show that excess pore pressures developing from the construction have dissipated, i.e., after set-up is completed. Performing more than one test adds to the assurance of the design. However, static loading tests are expensive. The ideal approach is to combine static and dynamic testing, carrying out several dynamic tests for every static test. And, of course, correlating the test types to each other by carrying out a dynamic testing on the pile on which also a static test is performed. In this regard, the combination of static and dynamic tests on the Alberta piles show that to obtain a good correlation between the tests, the static test should be performed before the dynamic. This is not a trivial recommendation, because, in arranging for the static loading test, the pile head is usually cut close to the ground, which makes a subsequent dynamic test awkward. It is necessary, however, because the dynamic test involves several impacts, each of which move the pile more against the soil more than the movement imposed in the static loading test, causing a larger disturbance to the bond between the pile surface and the soil. Moreover, it is probable that many published correlations of capacity determined from static loading tests and dynamic tests, showing the dynamic test to "overestimate" the capacity (taking the subsequent static result as "true") involve this error. It is a different matter, and not any less important, that a reliable correlation must include the two tests carried within a very short time frame, if the tests are made while excess pore pressures still exist.

CONCLUSIONS

1. The increase of pile capacity demonstrated in the Quebec and Alberta cases is clearly a function of dissipation of excess pore pressures developed during the pile driving.
2. For each of the Quebec and Alberta cases, the same beta-coefficient fitted the back-calculated shaft resistance distribution for the tests performed at different times and at different level of dissipation of excess pore pressures.
3. For the Bangkok and Vancouver cases, the same beta-coefficient fitted the loading tests on the different length piles.
4. The correlation of beta coefficient to the plasticity index relation was close in the Vancouver and Bangkok cases, but very much off in the Quebec and Alberta cases.
5. To be reliable for a design case, analysis from CPTU tests requires calibration to each particular design project site.
6. For the Bangkok and Quebec cases, the undrained shear strength was close to the shaft resistance of the static loading tests after full dissipation of the pore

- pressures. This is also the finding for the Alberta case, provided unreduced vane shear values were used (alpha coefficient equal to unity).
7. The undrained shear strength cannot be used to estimate the shaft resistance before the excess pore pressures have dissipated.
 8. Eqs. 5 - 7 for combining the undrained shear strength and the effective stress, applied to Bangkok, Quebec, and Alberta cases, did not provide a good agreement to the shaft resistance values of the static tests.
 9. For the Alberta case, the increase of capacity with time after full dissipation of the excess pore pressures was small and did not agree with the aging relations, Eqs. 8 and 9.
 10. The shaft resistance calculated from the CPTU soundings agreed very well with the static tests for the Vancouver case, but disagreed considerably for the Alberta case.
 11. The pile capacity determined in the 30-day dynamic tests on the Alberta piles agreed very well with static tests, when the disturbance from one test to the following was considered. The results showed that when carrying out static and dynamic testing, it is important to perform the dynamic test after the static test.
 12. The results show that in calculating pile capacity for a specific design case, the effective stress method is the most reliable approach. However, the beta-coefficients cannot be taken uncritically from published compilations of case histories, but must be taken from a well-designed test on a instrumented pile at the considered site or from a case record of such a case at a representative site. Those same conditions apply also to calculations from CPTU soundings as well as to geotechnical parameters and indices used when calculating pile response to load.

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