EFFECTIVE STRESS ANALYSIS AND SET-UP FOR CAPACITY OF PILES IN CLAY

Bengt H. Fellenius, Bengt Fellenius Consultants Inc., 1905 Alexander St. SE, Calgary, AB, T2G 4J3; <Bengt@Fellenius.net>

ABSTRACT
Two case histories are used to show that soil response to axial loading of a pile in clay is similar to that of a pile in sand in that it is proportional to the effective stress. One case presents the results of static loading tests on three piles of different length and demonstrates how the shaft and toe resistance can be analyzed through the application of CPTU analysis. Another case is from Alberta and involves static and dynamic tests of two piles during ongoing set-up after driving. The calculated load distribution for the piles correlate well with effective stress distribution, but not with distribution of undrained shear strength. It is recommended to use correlations to density index and plasticity index only as identifiers for explicit cases of relevance to a specific design and, moreover, not to rely on a single index for determining pile capacity.

RESUME
Deux dossiers antécédents démontrent que le comportement d’un sol sous le chargement axial d’un pieux enfoncé dans l’argile est semblable à celui d’un dans le sable comme étant proportionnel aux contraintes effectives. Deux cas présentent trois résultats d’essais de chargement statique et démontrent comment les résistances agissant sur le fût et la pointe peuvent être séparées. Le deuxième cas présente des essais de chargement statique et dynamique effectués dans un site en Alberta sur deux pieux a différentes reprises après l’enfoncement initial. Pour tous les cas présentés, la distribution des résistances correspond bien avec la distribution des contraintes effectives mais pas avec la distribution de capacité de cisaillement non-drainé. On recommande d’utiliser seulement les indices de densité et de plasticité comme identificateurs de cas spécifiques pertinents à une certaine conception, et, en plus, jamais se fier seulement à un indice unique pour déterminer la capacité portante.

1. INTRODUCTION
It is well-accepted that load-transfer for piles in sand follows the principle of effective stress. That is, unit shaft resistance is proportional to the effective stress acting against the pile shaft. For piles in clay, however, many practitioners still consider the unit shaft resistance to be a function of the undrained shear strength of the clay, that is, they employ total stress analysis. To illustrate the usefulness of the effective stress analysis also for piles in clay, this paper presents a summary of three published case histories and details results from two studies involving driven pipe piles in clay.

2. LOAD-TRANSFER FOR PILES IN CLAY
The fact that load-transfer for piles in clay also follows the principles of effective stress is not fully recognized by the profession. Many still continue to use total stress analysis, that is, they calculate the unit shaft resistance along a pile from undrained shear strength values—determined from in-situ tests, such as vane-shear tests, or converted from results of CPT-soundings, or obtained from laboratory tests on recovered samples, using various types of undrained tests. Inasmuch the total stress 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 near the site or 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. The values do not directly correlate to pile unit shaft resistance. Instead, the correlation should be to a Bjerrum Burland beta coefficient—the “proportionality coefficient” applied to the actual effective overburden stress. Such correlation will benefit from an estimate of the ratio between vertical and horizontal effective stress—the K-coefficient—which is a function of preconsolidation stress and other aspects. Above all, the actual distribution of pore pressure must be known and included in the analysis in order to approximate the necessary actual stress environment.

Many full-scale studies have demonstrated the universality of effective stress analysis in clays. For example, tests in Oslo, Norway (Bjerrum et al. 1969), in Fukagawa, Japan (Endo et al. 1959), in Quebec (Konrad and Roy 1981), and Bangkok, Thailand, (Indranatna et al. 1992). This paper presents results from tests in Lulu Island, Vancouver, BC, and at Paddle River, Alberta.

2. LULU ISLAND TESTS, VANCOUVER, BC
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. Fig. 1 presents the results of a CPTU sounding at near the test piles, showing the soil profile to consist 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. The vertical bars in the figure indicates the depths of the three test piles. 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 soils are very 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 day, 85 days, and 38 days after initial driving, respectively. As shown in Fig. 1, the shortest pile was terminated about 1.3 m above the sand, that is about 4 pile diameters. Pile 16.9 m was driven about 1.8 m into the 2 m thick weaker, clay and silt interspersed upper portion of the sand layer. Pile 31.1 m was driven about 3 m below the bottom boundary of the sand layer, that is, about 10 pile diameters below the sand/clay boundary. It is usually assumed that the influence of the soil above the pile toe extends to a distance above the pile toe of 8 pile diameters. That is, the presence of the sand layer would not have influenced the toe resistance of the 31.1 m pile. No pile was driven into the main sand layer between the depths of about 17 m and 26 m.

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 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. Simply, the pile capacities can be taken as lying in the range between the offset limit load and the maximum applied test load. While full dissipation has probably not occurred for the 38 day test on Pile 31.1 m, it is probably so for the 85 day test on Pile 16.8 m. At the 197 day test on Pile 13.7 m, the shaft resistance may have further increased due to aging.

In addition to knowledge of the distribution of effective stress, an effective stress analysis of pile capacity requires input of the response of shaft shear and toe resistances to the applied load, that is, the beta and toe coefficients. Upper and lower boundaries of the coefficients can be obtained from back-calculations of the pile capacities in the static loading tests using zero shaft resistance to zero toe resistance (i.e., all toe or all shaft). How much weight to be given to the beta-coefficient and how much to the toe coefficient is a matter of judgment. A quick, but approximate approach is to assume that the ratio between the beta-coefficient and the toe coefficient is equal to the average friction ratio of the cone sounding in the zone of influence around the pile toe: 8 pile diameters up and 4 pile diameters down from the pile toe. However, once cone sounding data are available, it is preferable to apply a method for determining pile capacity directly from the cone data.

Fig. 1 CPTU sounding diagrams from Lulu Island test.

Two methods for calculating the pile capacity were applied to the CPTU sounding: the CPT-based LCPC method (Bustamante and Gianeselli 1982; CFEM 1992) and the CPTU based Esami-Fellenius (Esami and Fellenius 1997) method. Fig. 3 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 main soil type, 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 separated on six soil types as applied to every single 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 in two steps. First, the analysis was fitted to the shaft resistance distribution determined by the CPTU method, resulting in β coefficients of 0.15, 0.20,
and 0.15 in the three main soil layers, respectively, and the distributions shown in Fig. 4. The 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.

In a second step, the so-established beta coefficients were used in an effective stress analysis of total capacity where the toe-coefficients were chosen to fit the calculations to the total capacity (value between offset limit load and maximum test value) of each of the static loading tests on the three piles. The calculated toe coefficients and resistance distributions are shown in Fig. 4 together with the capacity values of the test piles. As no pile test was made in the sand between the depths of 17 m and 28 m that could be used to correlate the effective stress input in this layer, the fit here was made to the mean of the LCPC and the Eslami-Fellenius CPTU-calculated capacity distributions.

3. TEST AT PADDLE RIVER, AB

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 at Paddle River, about 180 Km Northwest of Edmonton, Alberta, for which project the author served as a consultant. The site conditions are presented by Diyaljee and Pariti (2000).

The two 324 mm pipe piles were driven on October 30, 1990, about 2.4 m apart to depths of 16 m and 20 m and subjected to dynamic testing at End-of-Initial-Driving (EOID), and at restrike at 30 days, and 4 years (1,485 days; November 25, 1994, and only Pile 2) after EOID. The piles were also subjected to static testing at 15 days, 30 days, and 4 years (November 23, 1994) after EOID. The 15 day and 30 day restrikes were carried out the day before the static loading test and consisted of 5 to 8 blows to a total penetration of about 50 mm. The 4-year restrike was carried out two days after the static loading test and consisted of 11 blows to a total penetration of 80 mm. A Hera 1500 diesel hammer was used for the initial driving. The restrikes were made with an 18 KN drop hammer with a height-of-fall of 1.8 m.

The groundwater table lies at a depth of 1.5 m and the neutral pore pressures are hydrostatically distributed. The soil profile consists of 7.5 m of soft 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 %. The Janbu modulus number ranges from about 12 through 20. The clay is overconsolidated with an OCR of 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. The clay is a re-worked, transported, and re-deposited glacial till clay. The clay is interspersed at 5 m depth by an about 0.5 m thick layer of silty sand.
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 (the e0-values of the test specimens range from 0.18 through about 0.20). No recompression modulus is available, but the sandy clay till is clearly overconsolidated.

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The results of a CPTU sounding at the pile testing site are presented in Fig. 5 and indicate that the clay and the clay till are soft with a relatively homogeneous cone stress.

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. 6, the piezometer measurements show that the pile driving produced considerable excess pore pressures that dissipated with time. In Fig. 7, the vertical distribution of pore pressures are shown as measured before and 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. The measurements taken 4 years after the pile driving indicate no remaining excess pore pressures.

Figs. 8 and 9 present the load-movement curves of the static loading tests on Piles 16 m and 20 m, respectively, together with the capacities determined in the CAPWAP analyses (the dotted horizontal lines). The 15 day and 30 day tests on Pile 16 m were carried to movements in excess of 100 mm. The first observation is that the applied load reached a peak and thereafter the load required to move the pile diminished. This is an indication of 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 will could falsely indicate a capacity increase with time, or exaggerate an actual small continued increase of capacity.

The second observation is that the ultimate resistance (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. An additional insight is gained from comparing the capacities determined by the CAPWAP analysis and the static loading test. While no PDA testing and analysis was performed on the 15-day restrike, for both piles, PDA testing and CAPWAP analysis were performed on the 30 day restrikes. The CAPWAP analyses showed the capacity to reduce from the first to the fifth blows. The subsequent (next day) static loading tests showed capacities that were smaller than the CAPWAP-determined capacities for the first blow, but
close to the CAPWAP-determined values for the fifth blows; slightly smaller for Pile 16 m and slightly larger for Pile 20 m. For the 4-year test, however, where the static loading test was performed before the restrike, the capacity determined from the static loading test is larger than the CAPWAP capacity determined for the first blow. Similar to the 30 day restrike, the capacities reduced as the restriking continued and the capacity for the 11th blow is significantly smaller than the capacity for the 1st blow. It is evident that the restriking of the piles affected—the static resistance of the pile for the subsequent static loading test. The 30 day static loading test would have shown a higher capacity had the restrike been performed after the static loading test instead of the day before, as was done for the 4 year test.

Both the CPTU and CAPWAP 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, while the CAPWAP determined capacity agrees well with the capacity determined in the static loading tests (with due consideration of the effect of which type of test is performed first), the methods based on the cone sounding underestimate the capacity of the test piles. Calculation using the CPTU method (Eslami-Fellenius) indicate capacities of 820 KN and 1,050 KN for Piles 16 m and 20 m, respectively, which values are coincidentally close to the results of the 30-day static loading tests, as opposed to the capacities after full set-up. Calculation using the CPT LCPC method indicates 300 KN and 370 KN, respectively. That is, unlike the Vancouver case, the two methods give very different results. The reason lies in the somewhat arbitrary cut-off limits of shaft resistance imposed in the LCPC method.

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 Figs. 6 and 7 above. 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 15 day and 30 day resistance distributions are determined. The so-determined distributions are shown in Figs. 10 and 11 for the three static loading tests on each of 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 restrike and the calculated capacity should indeed come out larger than found in the static test.

It should be recognized that the beta-method (as the effective stress method often is called) is a simplistic method. It neither considers rotation of principle stresses along the pile surface, nor that the imposed movement against the soil of the pile surface in a static loading test generates small additional pore pressures that will reduce the effective stress against the pile surface. Still, the main finding is that the only way to achieve a reasonable agreement between the three pairs of static loading tests and calculations is to assign the soil-pile response to be proportional to the effective overburden stress.

4. DISCUSSION

4.1 Effective stress analysis

The presented case histories demonstrate that pile shaft resistance is governed by effective stress and can be expressed as a proportionality coefficient, the $\beta$-coefficient.
That effective stress governs the load transfer means that the pore pressure distribution actually acting in the soil body during the testing should always be established in order to have the test results guide the evaluation of expected long-term performance of the tested pile. Moreover, on completion of a test, all reporting should include a section relating the pile load-transfer to the actual effective stress distribution (and soil profile).

The important question is now how to determine the actual coefficient value from knowledge of the soil profile. With regard to the results presented from of the cases of piles in clay, the beta-coefficients determined from the cases in Vancouver and Alberta are compared in Fig. 12 to values suggested by Karlstrud et al. (2005), who show a relation between the beta coefficient and the plasticity index, \( I_P \), of the clay. The coefficients from Thailand and Vancouver cases lie well within the original plot. However, the coefficients from Japan and Alberta lie outside the boundaries of the data of Karlstrud et al. (2005). Again, while the scatter of data makes the plasticity index not directly useful for a specific design case, the index is considered useful as a label identifying a clay soil in a similar vein as the density index for sand. It is prudent to expand the identification label to encompass undrained shear strength and other indices in common geotechnical use. No indices should be used without simultaneous anchoring to the geological background of the records relied on.

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. The scatter
is far too large for this. The beta-coefficients to apply must be taken from a well-designed test on an instrumented pile at the considered site or from a case record of such a case at a representative site. For the latter case, geotechnical parameters such as the various indices and the geologic background must be reasonably the same or similar to that of the specific design site so as to verify the applicability of the reference case.

4.2 Set-up due to aging

Pile capacity will continue to increase also after full dissipation of the excess pore pressures. In the Alberta case, the time for full dissipation was not measured. However, as indicated by the extrapolation of the measurements shown in Fig. 16, full dissipation is expected to have developed at about 70 days after the end of the driving.

![Graph showing dissipation of excess pore pressures versus days after initial driving.](image)

**Fig. 16** Dissipation of excess pore pressures versus days after initial driving.

Pile capacity will increase after the completed dissipation due to aging — marginally or appreciably. Bullock et al. (2005) indicated a relation for increase of capacity with time after driving, i.e., set-up, according to the relation shown in Eq. 1.

\[
R_t = R_{REF} \left( 1 + A \log \frac{t}{t_{REF}} \right)
\]

where:
- \( R_t \) = capacity at Time "t"
- \( R_{REF} \) = capacity at Time "t_{REF}"
- \( A \) = a constant expressing ratio of capacity increase for a ten-fold increase of time
- \( t_{REF} \) = time after end of driving used as reference

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, which capacity should then be the one after full dissipation of the excess pore pressures developed during the installation of the pile. It is therefore a condition that the excess pore pressures have indeed dissipated during the first 100 days. They indicate relation for \( A \) according to Eq. 2.

\[
A = 0.1 + 0.4 \left( 1 - \frac{t}{100} \right) (OCR)^{-0.8}
\]

where:
- \( A \) = a A-constant
- \( IP \) = plasticity index
- \( OCR \) = overconsolidation ratio

As the lower limit values of plasticity index and OCR are zero and unity, respectively, the A constant according to Eq. 2 cannot become larger than 0.5. Any larger value of plasticity index 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. 17. The 100 day capacity from this graph by extrapolation of the capacities determined. Once the 100-day capacity is determined, Eqs. 1 and 2 could be used to indicate the increase of capacity due to aging beyond the 100 day value.

![Graph showing capacities versus days after initial driving.](image)

**Fig. 17** Capacities versus days after initial driving.

Karlsrud et al. (2005) indicates a range of the constant of \( 0.1 < A < 0.5 \), and a relation for \( A \) according to Eq. 2.

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 is indicated in Fig. 17 for the Alberta case.
The Alberta case plasticity index and OCR values are about 25 and 3, respectively. Inserting these values into Eq. 2 returns \( A = 0.4 \). With this value for \( A \), Eq. 1 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.

There are very few tests performed many years after the piles were installed, when also the 100 days capacity is determined. The author only knows about the various piles were installed, when also the 100 days capacity is. There are very few tests performed many years after the piles were installed, when also the 100 days capacity is determined. The author only knows about the various long-term tests involving 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 requiring very long time to reach appreciable values. “Appreciable” refer 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 Eq. 1 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 dangerous 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 Eq. 1 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 and apply the larger factor of safety (or smaller resistance factor) to the results of that calculation.

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