Design of axially and laterally loaded piles using *in situ* tests: A case history

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Received April 16, 1985
Accepted July 17, 1985

A 915 mm diameter steel pipe pile was driven and tested by the B.C. Ministry of Transportation and Highways as part of their foundation studies for the proposed Annacis channel crossing of the Fraser River. The pile was driven open ended to a maximum depth of 94 m. The pile was tested axially to failure when the pile tip was at depths of 67, 78, and 94 m below ground surface. Following the final axial load test, the pile was loaded laterally to a total deflection at the ground surface of 150 mm. A slope indicator casing was installed in the pile to monitor the deflected shape during lateral loading.

Adjacent to the pile, a piezometer-friction cone penetration test (CPT) and a full-displacement pressuremeter profile were made. Results of the axial and lateral load tests are presented along with the data from the CPT and the full-displacement pressuremeter tests. Results of several analyses using the data from the CPT and pressuremeter tests to predict the axial and lateral performance of the pile are presented. A comparison and discussion is presented between the predicted and measured axial and lateral behaviour of the pile, for which excellent agreement was found.

Key words: pile load test, cone penetration test, pressuremeter test.

Un pieu cylindrique en acier ayant un diamètre de 915 mm a été battu et testé par le "B.C. Ministry of Transportation and Highways" (ou ministère du Transport et des Routes de la Colombie-Britannique) dans le cadre de ses études de fondation pour l'ouvrage de traversée du canal Annacis du fleuve Fraser. Le pieu a été battu à bout-ouvert jusqu'à une profondeur maximum de 94 m. Le pieu a été chargé axialement jusqu'à la rupture lorsque la pointe atteignait les profondeurs de 67, 78 et 94 m sous la surface du sol. À la suite du dernier essai de chargement axial, le pieu a été chargé latéralement jusqu'à une déflexion de 150 mm mesurée au niveau de la surface du sol. Un tube d'inclinomètre a été installé dans le pieu pour mesurer la déflexion durant le chargement latéral.

Adjacent au pieu, un profil au piézocône avec manchon à friction et un profil au pressiomètre ont été réalisés. Les résultats des essais de chargements axial et latéral sont présentés de même que les données des essais au piézocône et au pressiomètre. L'on donne aussi les résultats de plusieurs analyses faisant usage des données du piézocône et du pressiomètre pour prédire le comportement du pieu sous charge axiale et latérale. La comparaison des résultats montre une excellente concordance entre les comportements prédits et mesurés du pieu tant pour le chargement axial que latéral; ces résultats sont présentés et discutés.

Mots clés: essai de chargement de pieu, piézocône, pressiomètre.

[Traduit par la revue]

Can. Geotech. J. 22, 518–527 (1985)

Introduction

The Annacis Throughway project near Vancouver, B.C. involves the construction of several bridges over the Fraser River. The high-level bridge over the main Fraser River channel will be the longest cable-stayed span in the world with a main span of 465 m. Another, low-level, bridge will be constructed over the Annacis channel of the Fraser River. The foundation soils for both structures are complex and include deep deposits of Fraser River sand, sensitive marine deltaic clays and silts, and surficial organic silts. Design of the support foundations for both bridges and approach structures has been complex and several full-scale pile load tests have been performed.

A 915 mm diameter steel pipe pile with a 19 mm wall thickness was driven and tested by the B.C. Ministry of Transportation and Highways as part of their foundation studies for the proposed low-level crossing of the Annacis channel. The pile was driven open ended to a total depth of 94 m and tested axially to failure when the pile tip was at depths of 67, 78, and 94 m below ground surface. Following the final axial load test, the pile was loaded laterally to a total deflection at ground surface of 150 mm. A slope indicator casing was installed in the pile to monitor the deflected shape of the pile during lateral loading. Adjacent to the test pile, a piezometer-friction cone penetration test (CPT) and a full-displacement pressuremeter profile were performed.

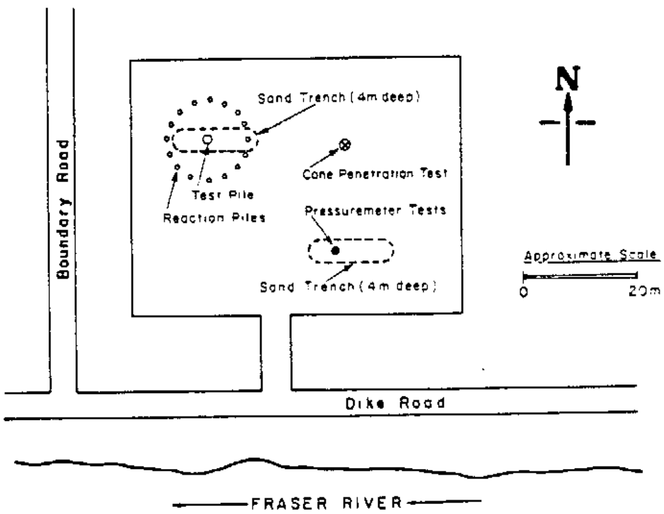

FIG. 1. Annacis Channel test pile site plan.

Test site

The site for the test pile is in a low-lying area on the north bank of the Annacis channel of the Fraser River at the boundary between New Westminster and Burnaby, B.C. Figure 1 shows the site plan and the location of the test pile, reaction piles, and *in situ* test locations.

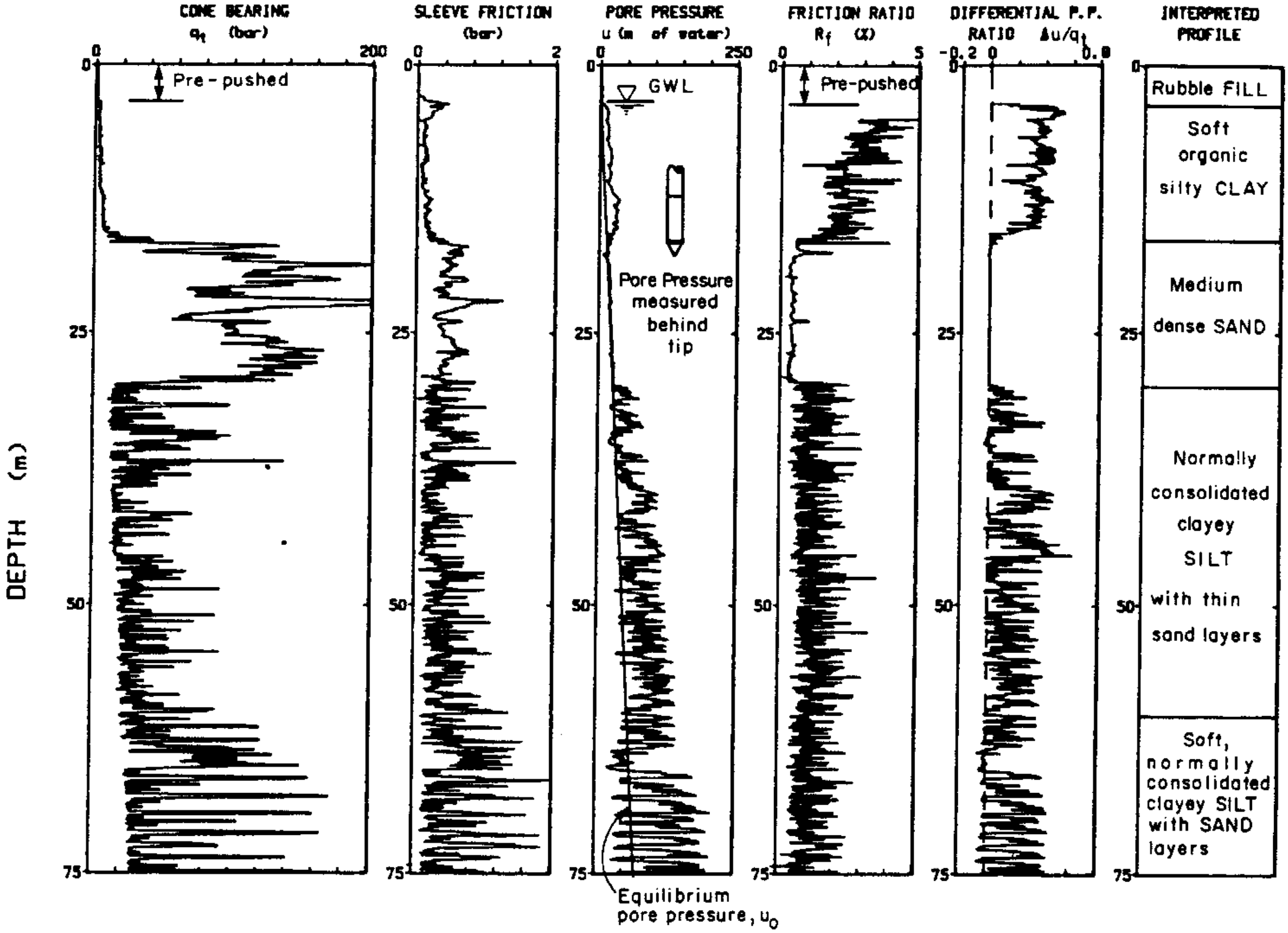

FIG. 2. Soil profile for test pile site (1 bar = 100 kPa).

A summary of the soil profile to a depth of 75 m based on sampling, laboratory tests, and a cone penetration test (CPT) is shown in Fig. 2. The upper 4 m of soil consists of random rubble fill. The CPT was performed in a prebored hole extending to a depth of 4 m. The prebored hole was formed by pushing a blank solid steel 15 cm^2 cone, to avoid damage to the 10 cm^2 electronic cone. From a depth of 4–16 m, the soil consists of soft, compressible organic silty clay. From 16 to 30 m is a layer of medium-dense sand. Extending below 30 m is a thick deposit of normally consolidated clayey silt. The clayey silt is estimated to extend to a depth of more than 130 m. The clayey silt deposit is interbedded with sand lenses to a depth of about 60 m. Below 60 m the sand layers become thicker and more well defined. The clayey silt deposit has a sensitivity of between 3 and 7 based on CPT data (Robertson and Campanella 1983) and about 5 based on laboratory vane testing.

Groundwater is approximately 4 m below existing ground surface and fluctuates with the river level. Equilibrium groundwater pressures are approximately hydrostatic for the depths shown in Fig. 2.

Pile load testing

Axial compressive tests

Reaction for the axial compression pile load tests on the 915 mm diameter pipe pile was developed from sixteen 762 mm diameter open-ended pipe piles driven to depths of 24–26 m. The reaction piles were driven in a circle of radius 4.6 m (center-to-center) around the test pile.

A schematic of the axial pile load test arrangement is shown in Fig. 3. Load was transferred to the reaction piles by 57 mm diameter Dywidag bars. The bars were threaded into the reaction frame and were connected to the tension reaction piles with a plate assembly welded inside the tension piles.

A 102 mm thick jacking plate was welded on top of the test pile and two 8900 kN capacity calibrated hydraulic jacks were placed on the jacking plate. Two, 150 mm thick, steel plates were placed on top of the jacks. A 13 000 kN capacity electronic load cell with a hemispherical bearing plate on top was placed on the steel plate, as shown in Fig. 3. Fluid pressure was supplied to both jacks by a common manifold on an electrically driven hydraulic pump.

The axial compressive load applied to the test pile was measured using the electronic load cell. The axial load was also measured from calibrated gauges on the hydraulic jacks.

Vertical and lateral pile deflections relative to two 300 mm H-section steel reference beams were measured by six electronic displacement potentiometers. Four potentiometers, placed at the corners of the 102 mm thick jacking plate, measured the vertical displacement of the pile head. Two potentiometers measured horizontal displacement of the pile head and were

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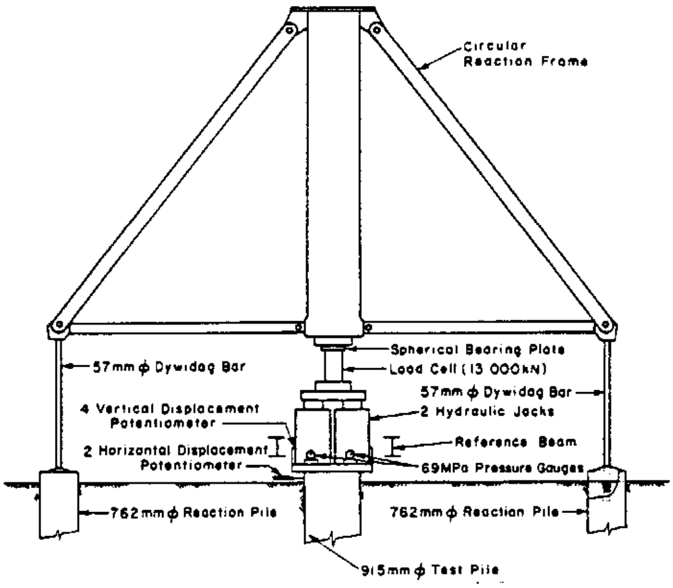

FIG. 3. Schematic of axial pile load test arrangement.

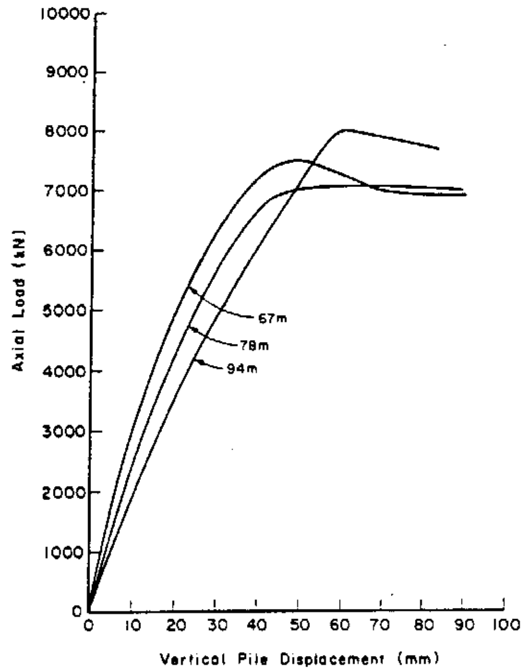

FIG. 4. Summary of axial pile load test results.

aligned in orthogonal directions. The reference beams were supported on timbers at a minimum clear distance of 5.5 m from the test pile.

The electrical output from the six potentiometers and the load cell were digitized at 1 min intervals throughout the test by a 10-channel data acquisition system provided and operated by Weir-Jones Engineering Consultants Ltd. In addition to the electrical measurement devices, the vertical displacements of the pile head and reference beams were measured using a precise level.

The test pile was tested axially to failure when the pile tip was at depths of 67, 78, and 94 m below ground surface. Following the final axial load test, the pile was loaded laterally. The testing sequence was as follows: (1) drive pile to 67 m; (2) wait 3 weeks; (3) axial load test to failure; (4) drive pile to 78 m; (5) wait 3 weeks; (6) axial load test to failure; (7) drive pile to 94 m; (8) wait 3 weeks; (9) axial load test to failure; (10) wait 1 week; (11) lateral load test.

The axial load tests were performed in increments of 500 kN. Each load increment was held until the rate of vertical movement was less than 0.25 mm/h or for a maximum of 10 min. The load tests were carried out following a quick load test procedure, similar to that described in ASTM D1143-81, Section 5.6.

A summary of the load–deflection curves for the three axial load tests is shown in Fig. 4.

The results from all three axial load tests indicate that the pile behaved as a friction pile. The reduction in measured load occurred because, with the rapid axial deflections, the hydraulic jacks could not maintain the load.

Lateral load test

Figure 5 shows a schematic of the lateral pile load test arrangement. Struts were braced against five of the reaction piles, as shown in Fig. 5. A hydraulic jack was placed between the reaction beam and the test pile. A calibrated gauge on the hydraulic jack was used to record the applied lateral load. The pile head deflection was measured by two dial gauges attached to a 300 mm H-section reference beam. A slope indicator casing was mounted inside the pile to a depth of 20 m to record the deflected shape of the pile. A precise-level survey was also made to check the displacement readings.

The lateral load was applied in increments of approximately

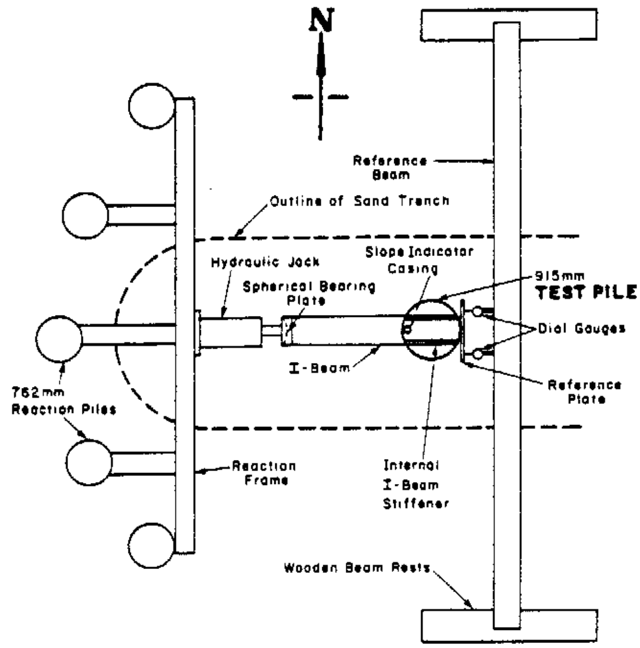

FIG. 5. Schematic of lateral pile load test arrangement.

100 kN. Each increment was maintained for periods of 10–15 min to allow time for readings to be taken on the gauges and slope indicator. The entire lateral load test was completed in less than 5 h.

At the test pile location, the surface 4 m of rubble fill was removed and replaced with loose sand fill before installation of the test pile. This was carried out to make the surface soils more representative of those in the adjacent riverbed area.

In situ testing

Adjacent to the test pile arrangement, a piezometer-friction cone sounding and a full-displacement pressuremeter profile were made. The cone and pressuremeter were installed using the

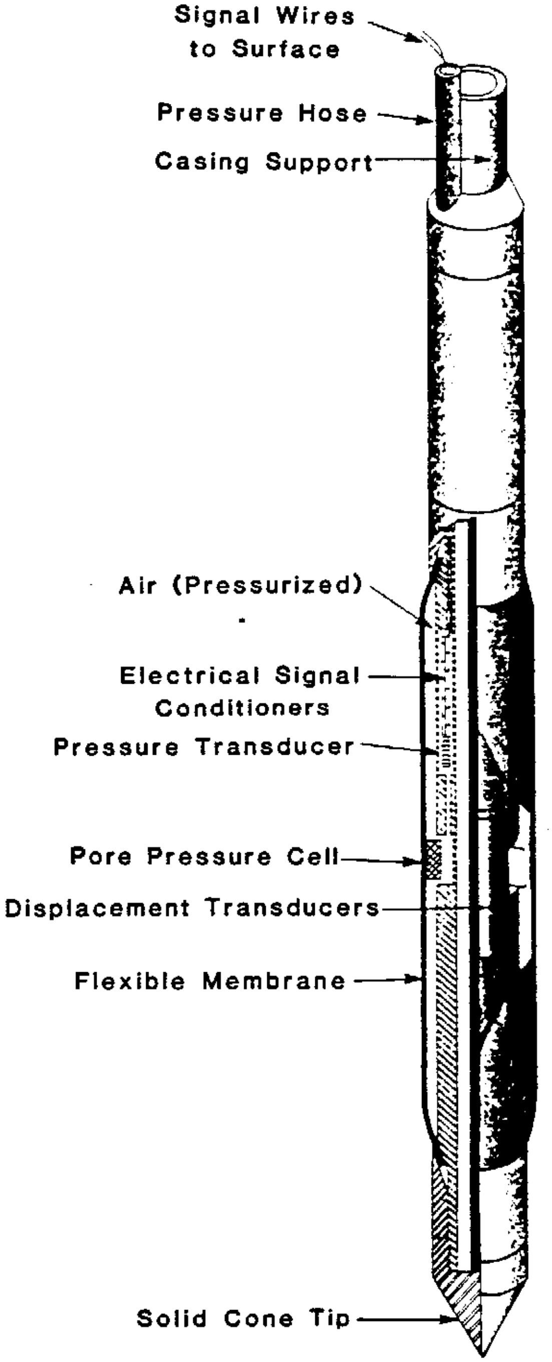

FIG. 6. Schematic of full-displacement pressuremeter probe in the inflated condition.

University of British Columbia (UBC) *in situ* testing vehicle. Full details of the vehicle are given by Campanella and Robertson (1981).

The cone penetration test (CPT) was made to a depth of 78 m below ground surface. A summary of the CPT data is shown in Fig. 2. A detailed description on the use and interpretation of CPT data is given by Robertson and Campanella (1983).

The pressuremeter profile was completed to a depth of 17 m. The instrument used was a self-boring-type pressuremeter fitted with a solid 60° steel cone shoe at the tip. A schematic of the pressuremeter in the inflated condition is shown in Fig. 6. The pressuremeter was pushed into the soil in a full-displacement mode in a manner similar to that used in the CPT. The instrument was 76 mm in diameter with a length-to-diameter ratio, for the membrane section, of 6. The center of the membrane was about 380 mm behind the solid cone tip. The solid 60° cone tip was of the same diameter as that of the membrane before inflation. The flexible membrane was protected by a stainless steel Chinese lantern sheath that was separate from the membrane. The lateral displacements of the central portion of the expanding membrane were measured electrically using three displacement transducers spaced at 120°. Pore pressure cells were incorporated on the membrane to monitor pore-water pressure; a pressure transducer inside the instrument monitored the applied internal gas pressure. The tests were carried out in a pressure-controlled manner by inflating the membrane in increments of 25 kPa or 50 kPa, depending on the soil stiffness. Pressure expansion tests were performed every 2 m from a depth of 3 m. The resulting pressure expansion curves showing the average strain arm measurements are shown in Fig. 7.

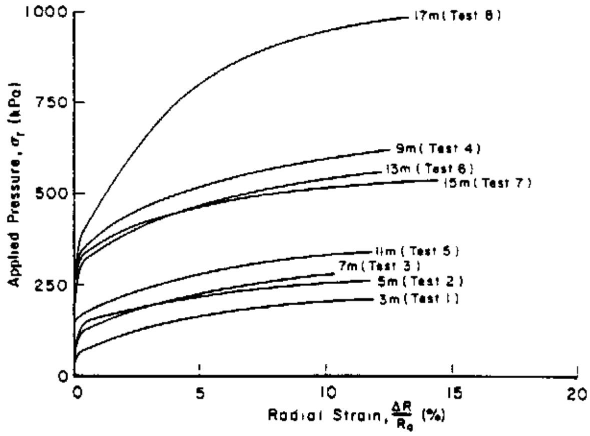

FIG. 7. Summary of full-displacement pressuremeter test results.

At the pressuremeter profile location, the surface 4 m of fill was removed and replaced with loose sand fill to simulate the conditions existing around the test pile.

The location of the pressuremeter tests with respect to the detailed stratigraphy determined by the CPT is shown in Fig. 8. The pressure expansion test at a depth of 3 m was performed in the loose sand backfill and showed a very soft response, as shown in Fig. 7. The pressure expansion test at a depth of 9 m shows a stiffer response than those immediately below. The CPT data show a stiffer layer of more sandy material between a depth of about 8 and 9.5 m, which would explain the stiffer pressuremeter response at this depth. Conventional drilling and sampling indicate that this sandy zone is not continuous across the site. The generally consistent agreement between the CPT and pressuremeter results, for the various soil layers, provides confidence in the pressuremeter data.

Prediction of pile behaviour

Ultimate axial capacity

Determination of pile capacity from the CPT is one of the earliest applications of the cone test. Pile foundations have been designed, more or less successfully, using empirical approaches, for a large number of years. The problem of estimating pile capacity, however, is complicated by the large variety of pile types and installation procedures, as well as soil type. The present state-of-the-art in pile design using CPT data is highly empirical.

A summary of the two main methods for using CPT data to predict vertical pile capacity for driven piles is given in the following sections.

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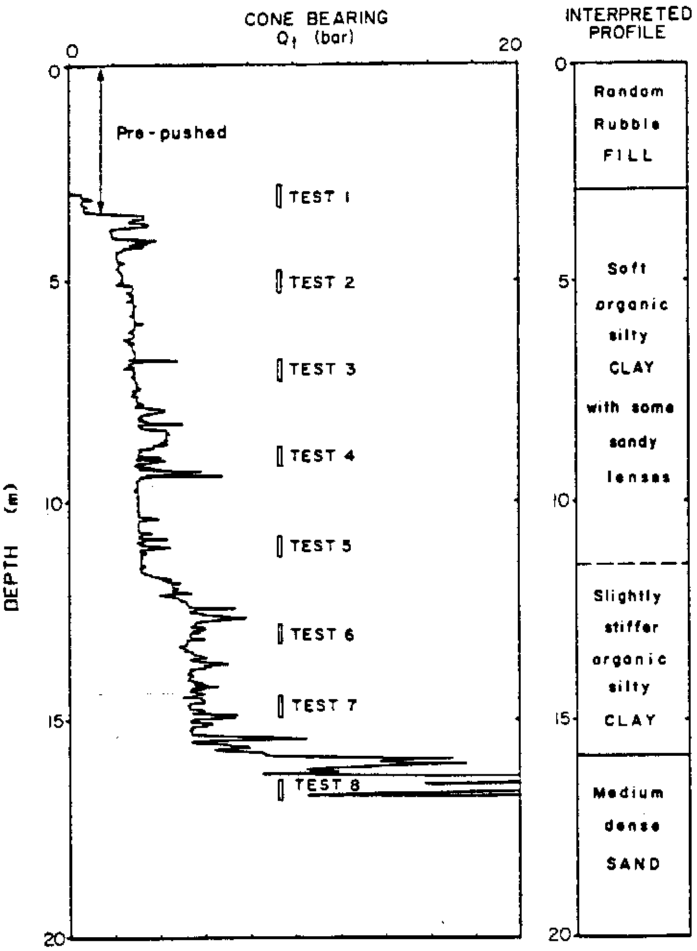

FIG. 8. Location of full-displacement pressuremeter tests relative to CPT soil profile (1 bar = 100 kPa).

European method (de Ruiter and Beringer 1979)—The CPT method used in many parts of Europe and especially for design of piles in the North Sea is summarized in Table 1. The unit end bearing for piles in sand is based on pile load test data and is governed by the cone bearing (q_c) in a zone of between 0.7D and 4D (where D is pile diameter) below the pile tip and 8D above the pile tip, as shown in the upper part of Fig. 9. Since overconsolidated cohesionless soils tend to experience some strength reduction during driving, a correction factor, shown in the lower part of Fig. 9, is applied to data obtained in these soils. Full details of this method are given by de Ruiter and Beringer (1979).

Schmertmann's method—The approach used extensively in North America was described in detail by Schmertmann (1978). The Schmertmann CPT method for design of piles is summarized in Table 2. The pile tip resistance in both sand and clay soils is the same as that recommended for sands by de Ruiter and Beringer (1979) using the upper part of Fig. 9. Figures 10 and 11 show the design curves used in the Schmertmann method for evaluating pile side friction in sand and clay soils, respectively.

The ultimate capacity in compression for an open-ended pipe pile as described by de Ruiter and Beringer (1979) is based on two criteria, representing two extreme conditions: either the pile can be considered as closed ended due to a soil plug or the pile behaves purely open ended.

For a closed-ended or plugged pile, the ultimate compressive capacity results from

$$[1] \quad Q_p = \sum f_o \cdot A_o + q_p \cdot A_p$$

where Q_p is ultimate compressive capacity plugged pile, f_o is outer unit skin friction, A_o is outer pile shaft area (πd_o), q_p is unit pile point end bearing, A_p is pile gross end-bearing area ($= \frac{1}{4}\pi d_o^2$), and d_o is outer pile diameter.

The ultimate compressive capacity for an open-ended or unplugged pile can be computed from

$$[2] \quad Q_u = \sum f_o \cdot A_o + \sum f_i \cdot A_i + q_w \cdot A_w$$

and

$$[3] \quad A_w = \pi(d_o - t)t$$

where Q_u is ultimate compressive capacity unplugged pile, f_o, f_i are outer and inner unit skin friction, A_o, A_i are outer and inner pile shaft area, q_w is unit pile wall end bearing, A_w is cross-sectional area of pile point, t is pile wall thickness, and d_o is outer pile diameter.

For any given penetration depth, Q_p and Q_u can be computed. The final assigned capacity is governed by the lower value of the two. Hence, comparing [1] and [2], the pile will be plugged (i.e. closed-ended behaviour) when

$$[4] \quad \sum f_i \cdot A_i > q_p \cdot A_p'$$

where A_p' is end-bearing area of pile plug ($= \frac{1}{4}\pi d_i^2$) and d_i is inner pile diameter. This plugged condition occurs in relatively low end-bearing soils, such as clays and silts, where the friction developed up the inside of the pile exceeds the bearing capacity at the tip. In dense sands, the reversed condition often governs. Hence

$$[5] \quad q_p \cdot A_p' > \sum f_i \cdot A_i$$

In that case, open-ended piles are unplugged. Artificial plugs (concrete) can be put in the pile, in order to develop full end-bearing capacity, if required.

Equations [1] and [2] present the fundamental basis of any axial capacity determination, irrespective of the method used to establish the unit skin friction (f) and unit end bearing (q).

Using the above approach, and either the European method or Schmertmann's method, the 915 mm open-ended pipe pile can be expected to plug within the first 5 m and remain plugged until the pile tip reaches the sand at a depth of 17 m. During penetration through the sand the pile can then be expected to unplug because of the high end-bearing resistance. The pile can then be expected to plug again during penetration through the underlying soft clayey silt. A schematic presentation of the calculated development of the soil plug during driving is shown in Fig. 12.

The sand layers at depth are relatively thin (less than 500 mm thick) in comparison with the diameter of the pile (915 mm), and the pile would not be expected to unplug beneath a depth of 30 m. The only exception to this may occur at a depth of 64 m where a sand layer, approximately 2 m thick, exists.

Research (Treadwell 1975) has shown that a cone or pile senses an interface 5–10 diameters ahead and behind. Therefore, the pile would start to unplug a short distance above the sand deposit at 17 m and plug again shortly above the clayey silt deposit at 30 m.

The ultimate length of soil expected to remain inside the pile after penetration of the pile tip to any depth greater than 30 m is about 17–18 m, as shown in Fig. 12. The actual size of the plug within the pile was not monitored during driving. However, after driving the pile, the soil plug was measured to be

TABLE 1. Summary of European CPT design method (de Ruiter and Beringer 1979)

Sand		Clay	
Unit skin friction, f_p	Unit end bearing, q_p	Unit skin friction, f_p	Unit end bearing, q_p
Minimum of: $f_1 = 0.12$ MPa f_2 = CPT sleeve friction, f_s $f_3 = q_c/300$ (compression) $f_4 = q_c/400$ (tension)	Minimum: q_p from Fig. 9	$f = \alpha c_u$ where: $\alpha = 1$ for N.C. clay $= 0.5$ for O.C. clay	$q_p = N_c \cdot c_u$ where: $N_c = 9$ $c_u = q_c/N_k$, $N_k = 15$–20

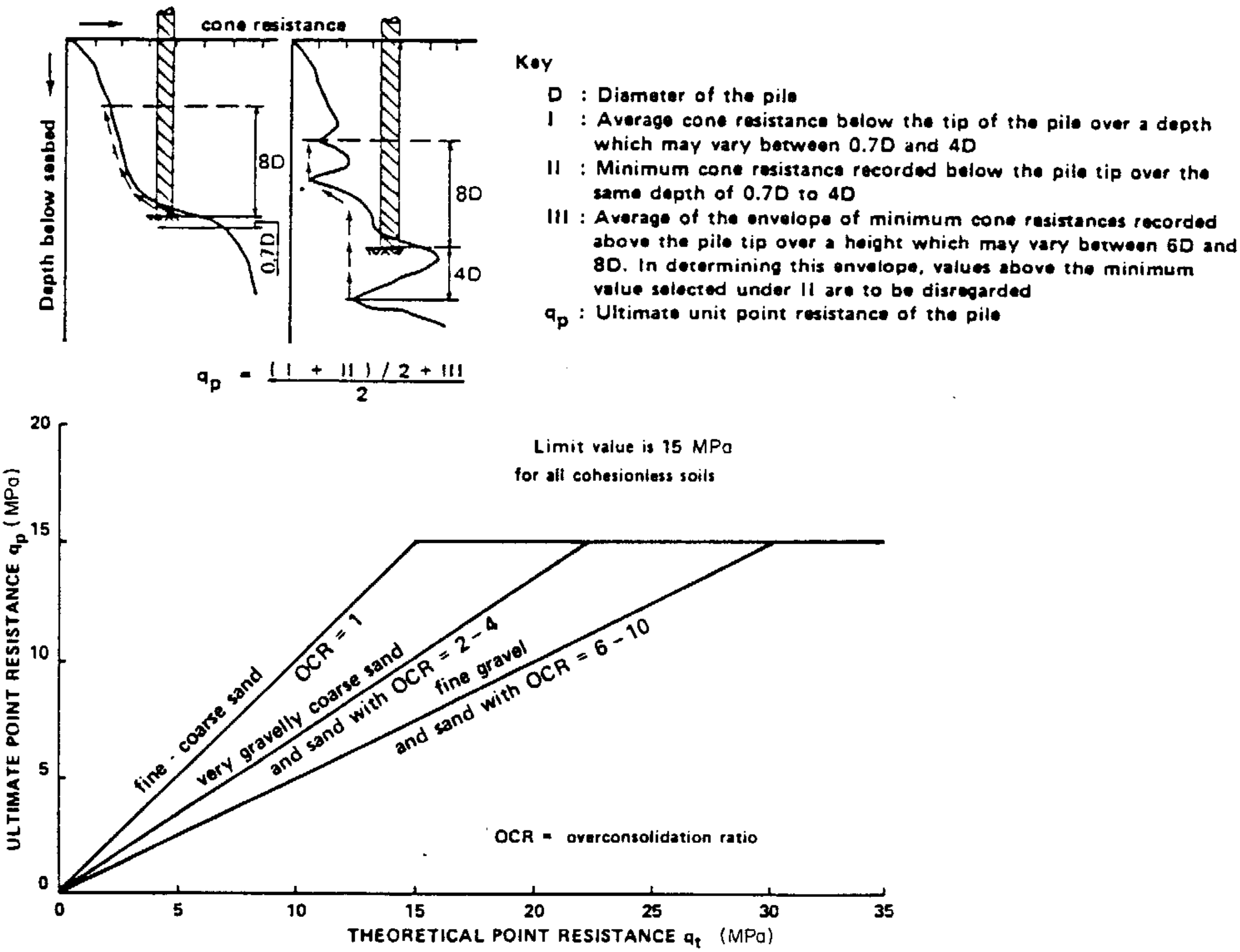

FIG. 9. Application of CPT data to pile design (after de Ruiter and Beringer 1979).

approximately 16 m below ground surface. The method of analysis used to calculate the size of the plug is based on the static final conditions of the pile. However, during driving, very high pore pressures are generated on the outside and inside of the pile. Hence, very low side friction is generated during driving. The static analysis, therefore, tends to underestimate the size of the plug during driving.

Using [1] and [2] to calculate whether the pile plugs or unplugs, the ultimate capacity of the test pile was determined using the method proposed by Schmertmann (1978) and the European method (de Ruiter and Beringer 1979). The calculated ultimate capacities using both methods are plotted versus depth and compared in Fig. 13 with the measured capacities. For the soils at the test pile site, the method described by Schmertmann (1978) clearly provides an excellent assessment of the ultimate axial compresive capacity.

The two methods provide essentially the same ultimate capacities to a depth of 30 m. Below 30 m, the European method

TABLE 2. Summary of Schmertmann CPT design method

Sand		Clay	
Unit skin friction, f_p	Unit end bearing, q_p	Unit skin friction, f_p	Unit end bearing, q_p
Minimum of: $f_1 = K\left[\sum_0^{8D}\left(\frac{l}{8D}\right)\cdot f_s + \sum_{8D}^{L}\cdot f_s\right]$ where: K = ratio of f_p/f_s (Fig. 10) l = depth to f_s considered D = pile width L = pile length f_2 = 0.12 MPa $f_3 = c \cdot q_c$ where c varies from 0.009 to 0.018 depending on pile type	Minimum q_p from Fig. 9	Minimum of: $f_1 = \alpha' c_u$ (see Fig. 11) $f_2 = \lambda(\bar{p}' + 2\bar{c}_u)$ where: $\bar{p}'$ = ave. σ_{vo}' along pile length $\bar{c}_u$ = ave. c_u along pile length λ = 0.3 for L/B = 10 = 0.2 for L/B = 20 = 0.14 for $L/B > 60$ $f_3 = \alpha'\left[\sum_0^{8D}\left(\frac{l}{8D}\right)\cdot f_s + \sum_{8D}^{L}\cdot f_s\right]$	Minimum q_p from Fig. 9

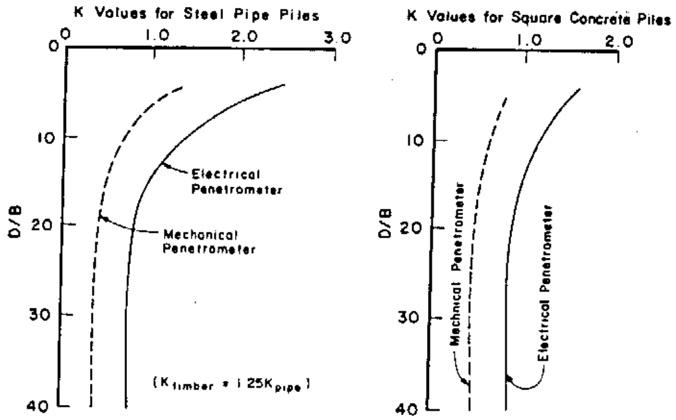

FIG. 10. Design curves for pile side friction in sand (after Schmertmann 1978).

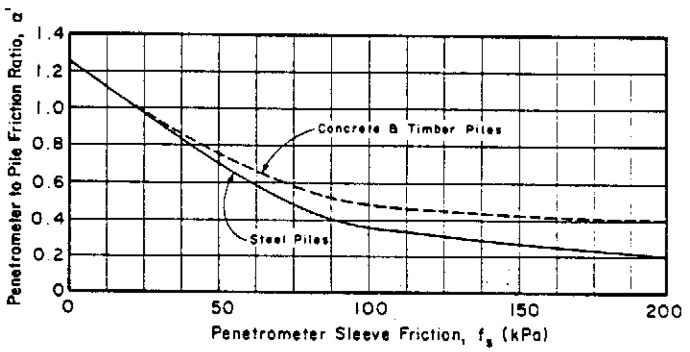

FIG. 11. Design curves for pile side friction in clay (after Schmertmann 1978).

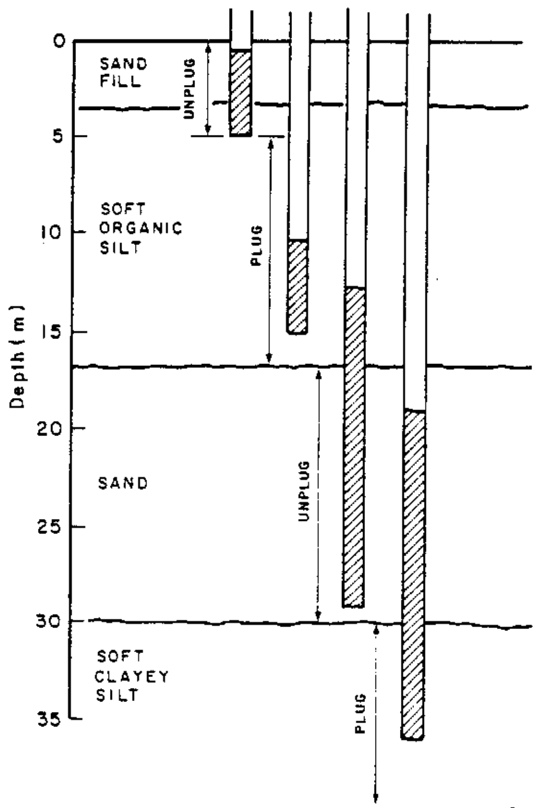

FIG. 12. Schematic presentation of the calculated development of soil plug during driving.

overpredicts the pile capacity substantially. Both methods predict that the ultimate capacity is controlled by the unit skin friction. When the pile tip is at a depth of 80 m, the European method predicts that 95% of the ultimate capacity is derived from skin friction. Schmertmann's method predicts that 84% of the capacity is derived from skin friction at the same depth.

The European method calculates the unit skin friction (f) for clayey soils from the calculated peak undrained shear strength. Schmertmann's method, on the other hand, employs the minimum of three calculated unit skin frictions. In this case, the minimum skin friction in the underlying clayey silt is always f_3 (see Table 2). This minimum unit skin friction uses the measured sleeve friction (f_s) from the CPT. In sensitive clayey soils, the sleeve friction is often assumed to be close to the remolded shear strength. Thus, since the clayey silt deposit has a sensitivity of around 5, Schmertmann's method was able to take this into account.

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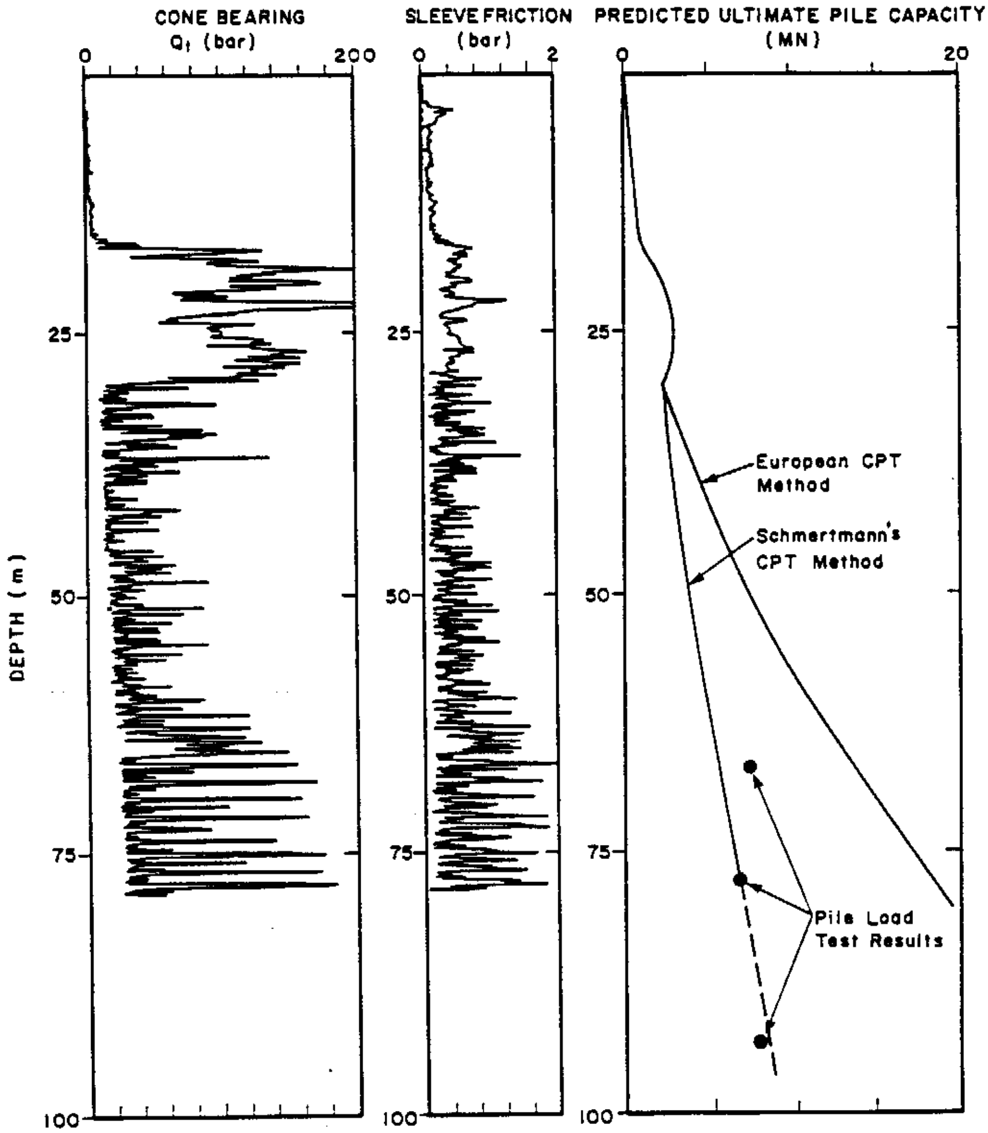

FIG. 13. Predicted and measured ultimate pile capacities versus depth for Annacis Channel test pile (1 bar = 100 kPa).

The calculation for pile capacity in the underlying clayey silt used only the lower cone bearing and sleeve friction values applicable to the clayey silt. The higher values caused by the thin sand layers were ignored because the pile is large relative to the thickness of the layers and some smearing could be expected across the layers. However, if the higher values from the thin sand layers are included, the calculated capacity of the pile at a depth of 80 m would increase by about 20%.

Schmertmann's method also shows that very little increase in ultimate capacity can be expected with increasing depth below 30 m. A possible reason for the decrease in measured ultimate capacity from a depth of 67–78 m may be caused by a small loss in available skin friction through the sand layer (17–30 m) due to the redriving of the pile between tests. Another possible cause for the slightly higher resistance at 67 m may be that the pile tip was located in the thicker sand layer at about that depth (see Fig. 2).

During the CPT several pore pressure dissipation tests were performed for the piezometer element located behind the tip. These were carried out by recording the rate of pore pressure dissipation during a stop in the cone penetration. In the underlying clayey silt deposit, the average time for 50% dissipation was 2–3 min. The projected time for 90% dissipation of pore pressures around the 36 mm diameter cone was therefore about 40 min (Torstensson 1977). The time for 90% dissipation of pore pressures around the driven 915 mm pile is then approximately equal to the ratio of the radius squared, i.e. 20 days. Therefore, the CPT pore pressure dissipation data indicate that the time periods between driving the pile and testing were sufficient to allow about 90% of the excess pore pressures to dissipate. It is also reasonable to expect that the extensive stratification would allow a more rapid dissipation of excess pore pressures.

Lateral loading behaviour

The nonlinear subgrade reaction method is widely used for the design of laterally loaded piles. This method replaces the soil reaction with a series of independent springs. The nonlinear behaviour of the soil springs is represented by P–y curves, which relate soil reaction and pile deflection at points along the pile length. Most of the existing methods for obtaining P–y curves are highly empirical. Often, little account is taken of the

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method of pile installation and the influence that this may have on the soil behaviour. The pressuremeter, however, offers the potential to measure the soil reaction *in situ*, under similar loading conditions.

Several methods have been proposed for the design of laterally loaded piles using pressuremeter data (Briaud *et al.* 1983; Baguelin *et al.* 1978; Robertson *et al.* 1983; Baguelin 1982). Most of these methods make use of preboring pressuremeter results, using a Ménard-type pressuremeter, and cannot model the disturbance caused from a driven pile since the pressuremeters are placed in a prebored hole. It is possible to install the pressuremeter in a manner that models the disturbance caused during pile installation. For driven displacement piles, the pressuremeter can be pushed into the soil in a full-displacement manner. For cast-in-place or bored piles, a prebored or self-bored pressuremeter test can model the disturbance during pile installation. The method by Robertson *et al.* (1983) uses the results from a pressuremeter pushed into the soil to model the installation of a driven displacement pile.

During the subsequent pressuremeter test, the soil deforms in a simple radial direction, whereas the displacements in the soil surrounding a laterally loaded pile are more complex as the soil moves radially away from the front face of the pile and inwards towards the back face. However, it is reasonable to expect that the soil in the center region of the front face of the pile and that around a pressuremeter would deform in similar manners. Therefore, it is reasonable to suppose that the geometric form of the pressure expansion curve obtained from the pressuremeter would be similar to the load displacement (P–y) curve for the soil acting on the front face of the pile.

Hughes *et al.* (1979) suggested that the pressuremeter curves should be increased by some factor (α) to give the correct P–y curves for the pile. Hughes *et al.* (1979) and Robertson *et al.* (1983) suggested that the multiplying factor is about 2 for clays and 1.5 for sands. These multiplying factors are applied to the pressure component of the pressure expansion curves, as shown in Fig. 14. These multiplying factors were also confirmed by Byrne and Atukorala (1983) using finite element analyses.

The approach given above can be applied to pressuremeter tests performed at some depth remote from the surface where the lateral displacement of the pile is not influenced by the surface. Generally, this critical depth is assumed to be about four pile diameters below the ground surface (Briaud *et al.* 1983). To account for the reduced soil reaction mobilized within the piles to critical depth, the multiplying factors are progressively reduced. The writers recommend that the factors be reduced from a value of 2 at four pile diameters to 0.67 at the ground surface for cohesive soils and from 1.5 down to 0 for cohesionless soils, as shown in Fig. 15.

The lateral stresses acting against the sides of the pile will be influenced by the pile driving and subsequent waiting or setup period. By installing the pressuremeter in a similar manner, the pressure expansion curve will have a lift-off pressure approximately equivalent to the initial lateral stress around the pile. Therefore, to obtain the subsequent P–y curve for lateral loading, the pressure axis for the P–y curve is moved to the lift-off pressure from the pressuremeter curve, as shown in Fig. 14.

The P–y curves required for the analysis are in units of force per unit length (P) and displacement (y), whereas the pressuremeter curves are in units of stress (σ_r) and circumferential strain ($\Delta R/R$), where R is the initial radius of the probe and ΔR is the change in radius. Thus, to convert the pressuremeter stress to force per unit length, the stress data is multiplied by the pile

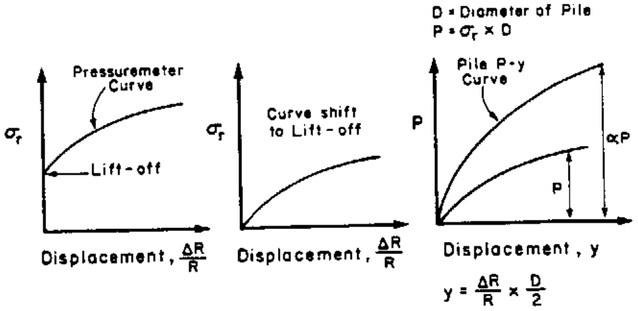

FIG. 14. Schematic representation of development of pile P–y curves from pressuremeter curves.

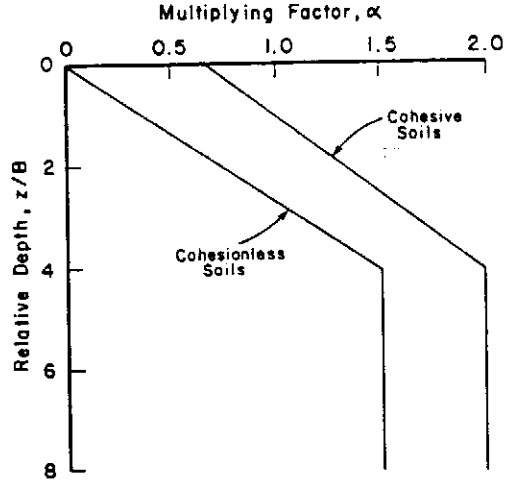

FIG. 15. Variation of multiplying factor with relative depth.

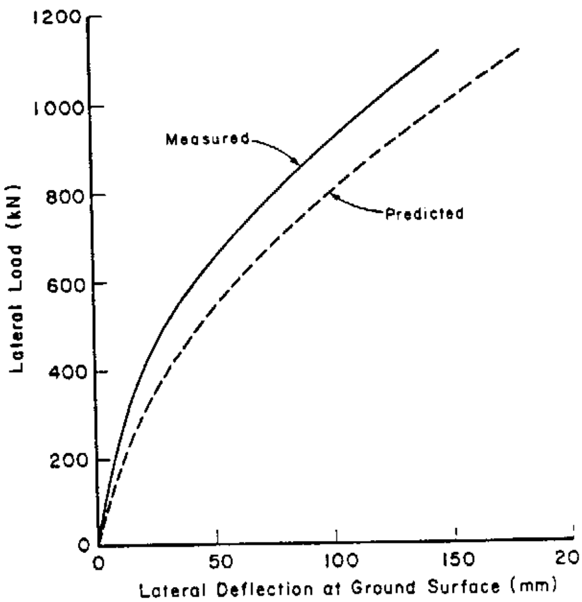

FIG. 16. Summary of measured and predicted lateral load versus deflection at ground surface for Annacis Channel test pile.

width (i.e. 915 mm). The pressuremeter strain data is multiplied by the pile half-width (i.e. 457.5 mm) to obtain the displacement (y).

P–y curves determined using the above approach were applied to a computer program (LATPILE) developed at the University of Texas (Reese 1977) and modified at UBC. The

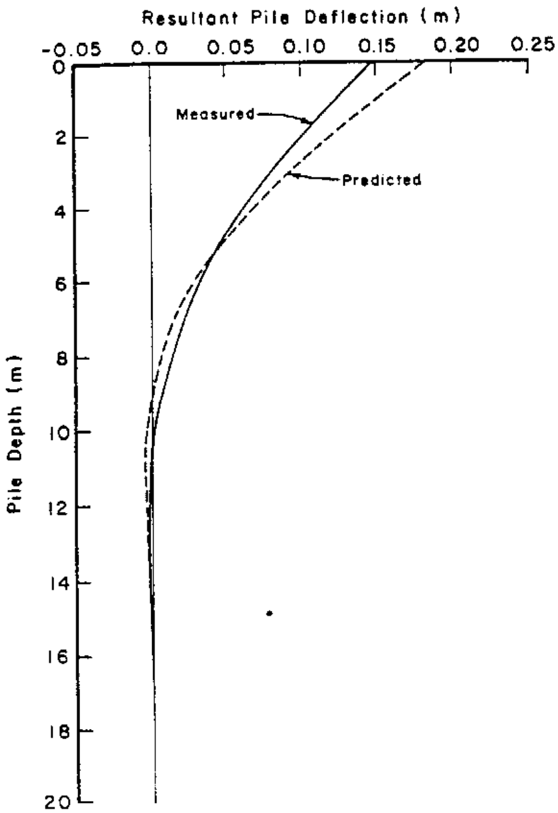

FIG. 17. Summary of measured and predicted lateral pile deflection versus depth at a lateral load of 1100 kN for Annacis Channel test pile.

calculated and measured load–deflection curves at the pile head are compared in Fig. 16. The predicted deflection agrees remarkably well with the measured deflection and is approximately 20% larger at the pile head. The predicted and measured deflected shapes of the pile at a lateral load of 1100 kN are shown in Fig. 17. The predicted location of zero deflection is at a depth of 9 m compared with the measured location of 11 m. Generally, the predicted behaviour of the pile under lateral loading agrees well with the measured behaviour. It is worth noting that the prediction of lateral behaviour was made based solely on the pressuremeter and CPT results *before* the measured results were made available from the lateral load test.

Conclusions

The installation and testing of a large-diameter steel pipe pile has provided an excellent case history to evaluate the use of *in situ* testing techniques, such as the CPT and full-displacement pressuremeter, for design of axially and laterally loaded piles in deep deposits of soft soils. The results of the test program performed at the site of the proposed bridge crossing of the Annacis channel have shown that for Fraser River soils the method proposed by Schmertmann (1978) for the design of axially loaded piles using CPT data can provide an excellent assessment of ultimate pile capacity. The reason for the poor prediction when the European method was used was thought to stem from the inability of the European method to account for soil sensitivity as indicated by the CPT friction sleeve stress. Both CPT methods for the design of the pile clearly predicted the dominant influence of the pile side friction. Schmertmann's method showed that very little additional axial capacity would be obtained by increasing the length of the pile in the underlying clayey silt deposit.

The results of the lateral pile load test show that the use of the full-displacement pressuremeter to define the nonlinear $P-y$ curves of the soil can provide an excellent assessment of the lateral loading behaviour of a driven displacement pile. The use of pressuremeter-derived $P-y$ curves provided an excellent prediction of the deflected shape of the pile.

Acknowledgements

The assistance of the Natural Sciences and Engineering Research Council of Canada, the British Columbia Ministry of Highways and Transportation, and the technical staff of the Civil Engineering Department, University of British Columbia is much appreciated. The assistance of Western Geosystems for the use of their pressuremeter equipment is also appreciated.

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