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# PREDICTION OF WICK DRAIN PERFORMANCE USING PIEZOMETER CONE DATA

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## ABSTRACT

The technique of installing prefabricated wick drains is well established in Europe and Japan, but is relatively new to North America. This paper describes the use of piezometer cone data to predict the performance of wick drains in a soft clayey silt in Burnaby, B.C. A description of the testing procedures and the theory to calculate the rate of consolidation is briefly presented. A comparison and discussion are presented between the predicted rate of consolidation using the piezometer cone data and the measured performance, for which good agreement was found.

Key words: Wick drains, Piezocone, prediction, preload, settlement.

# INTRODUCTION

Vertical drains have been used worldwide to increase the rate of settlement induced by external loading. In recent years many band-shaped wick drains have been introduced and used as vertical drains. The most difficult part in the design of a wick drain system is the estimation of the rate of settlement due to consolidation. The key design parameter is the coefficient of consolidation in the horizontal direction, $c_h$. The $c_h$ value is generally larger than the $c_v$ value. Therefore, the oedometer values of $c_v$ are generally too conservative. The ratio of $c_h$ to $c_v$ found by back-analysis of full-scale test data is often in the range of 2 to 7 depending on the degree of stratification (Hansbo et al, 1981). The need for better methods to determine the $c_h$ value is obvious. Hansbo and Torstensson (1977) proposed the use of the piezometer probe to determine the in-situ $c_h$ value by studying the dissipation of the excess pore pressure induced as the probe penetrates the clay.

The piezometer cone penetration test (CPTU) is becoming increasingly popular as a fast, efficient method for site investigation in soft, compressible soils.

Present day piezometer cones typically record cone resistance, $q_c$, sleeve friction, $f_s$, and pore pressure, u, continually (at intervals of 1 to 5 cm in depth) during penetration at a standard rate of 2 cm/ sec. Cone verticality and temperature are also commonly recorded. During a pause in the penetration, excess pore pressures generated during cone penetration immediately start to dissipate radially away from the cone. The rate of dissipation depends on the coefficient of consolidation, $c_h$, of the soil for a homogeneous deposit. The presence of silt and sand layers may greatly affect the average $c_h$ value. In cases of stratified soils with closely spaced, continuous silt and sand layers, vertical wick drains may be of little value.

This paper briefly describes the use of piezometer cone data to predict the performance of wick drains installed in a soft clayey silt in Burnaby, B.C.

## THEORETICAL CONSIDERATIONS

Vertical drains can be effective in promoting consolidation because they reduce the length of the drainage path and thus increase the consolidation rate. In practice, drains are usually installed in a triangular or rectangular grid. Therefore, in the theoretical analysis, each drain is assumed responsible for dewatering a vertical soil cylinder, concentric with the drain and with a diameter approximately equal to the drain spacing.

To predict the time of consolidation using vertical drains, various formulae have been presented (Barron, 1944, Hansbo, 1960). The newer approach by Hansbo (1960) is based on experience gained at the Ska-Edeby site in Sweden. Hansbo's approach is based on the assumption that under very low hydraulic heads the flow of porewater in a soil is not well represented by Darcy's Law. The exponential version of Darcy's Law in which $v = ki^n$, where $n = 1.5$ (Hansbo, 1960, Holtz & Broms, 1972) appears to give better agreement with observed results and is incorporated in the Hansbo (1960) theory. More recent studies have indicated that Darcy's law is valid in natural clays. However, a full discussion of this topic is beyond the scope of this note. The case history presented here uses the Hansbo (1960) theory in which

$$t = \frac{\alpha}{\lambda} D^2 \sqrt{\frac{D.g.\rho_\omega}{\Delta u_o}} \left(\frac{1}{\sqrt{1-\bar{U}}} - 1\right) \qquad (1)$$

where t = time to achieve a specified degree of consolidation

$\alpha$ = function of D/d (see Fig. 1)

D = effective diameter of dewatered soil cylinder

d = effective diameter of vertical drain

$\Delta u_o$ = average excess pore pressure at t = 0

$\bar{U}$ = average degree of consolidation

$\lambda = 0.5\ c_h$ (based on laboratory oedometer tests)

$\rho_\omega$ = density of water

g = acceleration due to gravity.

Figure 1 presents the variation of $\alpha$ suggested by Hansbo (1979). It is clear from Fig. 1 that the ratio of the effective diameters of the drain and dewatered soil has a significant influence on the time for consolidation.

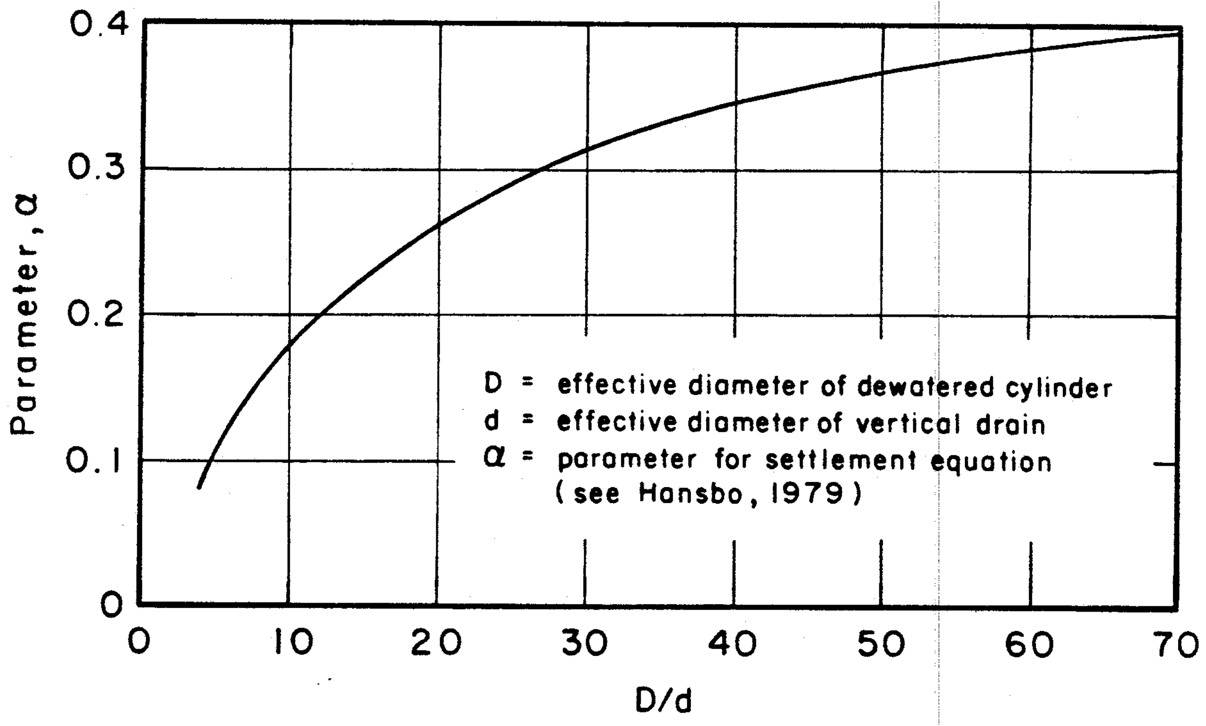

Fig. 1. Variation of 'α' parameter with diameter ratio for Hansbo Equation (Adapted from Hansbo, 1979).

In Eqn. 1 the magnitude of the consolidation load (i.e., the excess pore pressure at t = 0) also has an influence on the time of consolidation; i.e., the higher the load, the shorter the time for consolidation.

Traditionally, the coefficient of consolidation has been measured in a laboratory using an oedometer. Recent research (Hansbo et al, 1981) showed that the coefficient of consolidation measured using the CPTU dissipation, $c_h$, is from 2 to 7 times larger than $c_v$ as measured in an oedometer. Since $c_h$ in Eqn. 1 was initially based on laboratory testing (Hansbo, 1960), an adjustment will be needed to the $\lambda$ factor to account for the difference in measurement between oedometer and the piezometer cone dissipation method of calculation of $c_h$ in order to fit observed behaviour.

The $c_h$ of the soil can thus be determined by monitoring the rate of dissipation of excess pore pressures during a pause in piezometer cone penetration. Several theoretical solutions are available to obtain $c_h$ from dissipation data. A summary of the solutions was presented by Robertson and Campanella (1983) and

Levadoux and Baligh (1986). A summary of the solution proposed by Torstensson (1977) for cylindrical cavity expansion gives excellent results and is shown in Fig. 2. Figure 2 presents the variation of the degree of consolidation against a non- dimensional time factor, T, where

$$T = \frac{c_h t}{r^2} \tag{2}$$

and, $t$ = time recorded for a specified percentage of dissipation

$c_h$ = horizontal coefficient of consolidation

$r$ = radius of cavity or piezometer cone (usually 17.85 mm)

The solutions vary with the soil stiffness ratio or rigidity index ($E/S_u$) (see Fig. 2). The reason for this is that a stiff soil will extend a zone of influence much larger than a soft soil would due to the penetration of the piezometer cone. The result of a larger zone of influence is to decrease the rate of decay of excess pore pressures at the cone. Although some doubt surrounds the selection of an appropriate stiffness ratio, solutions are not overly sensitive to soil stiffness.

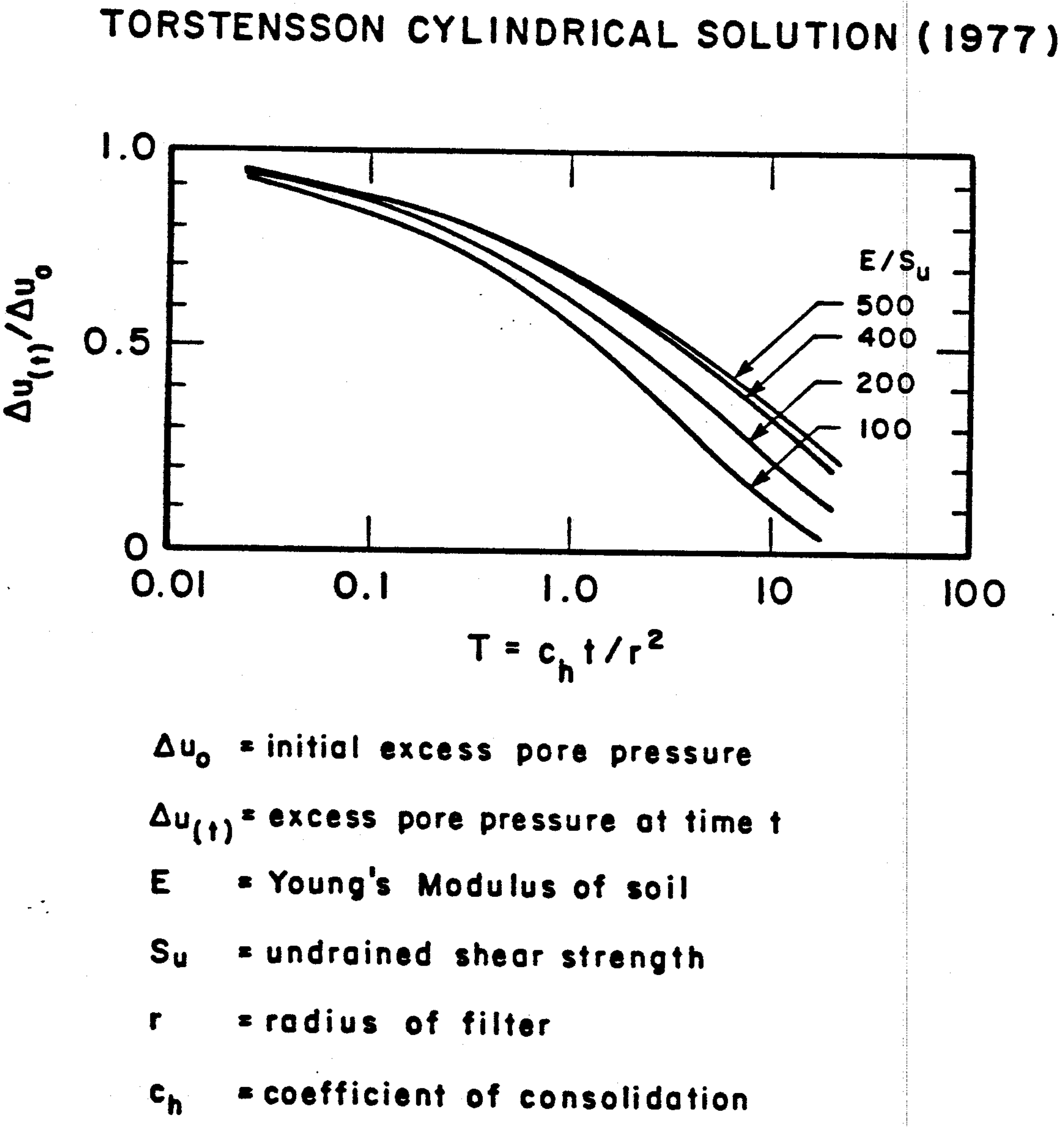

Fig. 2. Dissipation of initial excess pore pressure based on Torstensson cylindrical cavity solution (Adapted from Torstensson, 1977).

Figure 2, therefore, allows the calculation of a $c_h$ directly from the data provided from a dissipation test during a piezometer sounding. By allowing the initial excess pore pressure generated during penetration to dissipate by 50% ($\Delta U_t/\Delta U_o$ = 0.5), a time factor T of about 5 is determined for $E/S_u$ of 500 (see Fig. 2). Thus, from Eqn. 2, knowing the radius of the penetrometer and the time taken for 50% dissipation, the horizontal coefficient of consolidation ($c_h$) can be calculated. Figure 3 shows how $t_{50}$ is determined from a typical dissipation curve.

Experience has shown that the excess pore pressures generated during cone penetration are affected by several factors, the main one being the location of the porous element relative to the cone tip (Campanella et al, 1986). In general, more consistent dissipation data is obtained in soft, normally to lightly overconsolidated soils if the pore pressure is measured behind the cone tip. Further comments regarding the procedure to perform and the interpretation of dissipation tests are given by Robertson and Campanella (1983, 1986) and Campanella and Robertson (1988).

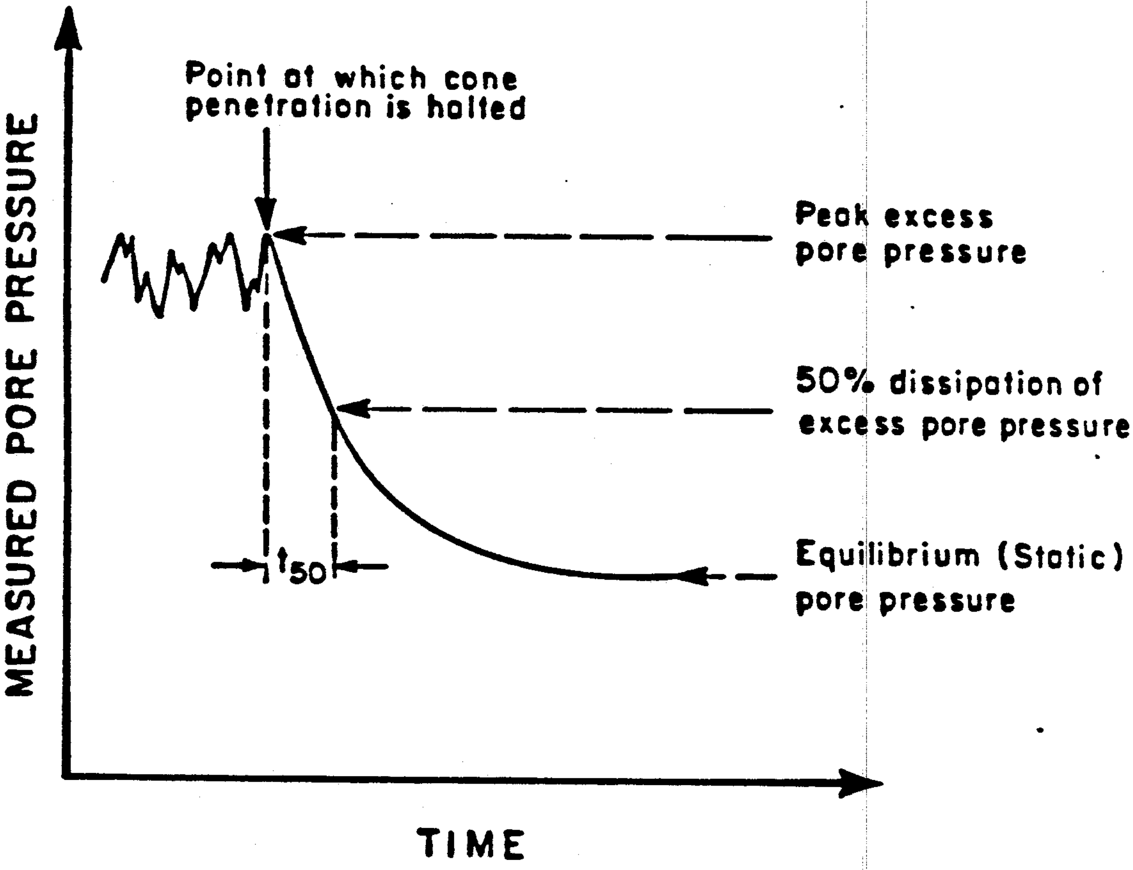

Fig. 3. Calculation of piezometer cone dissipation times. (Sensing element behind tip.)

## CASE HISTORY

Accelerated consolidation was required for a site near the Fraser River in Burnaby, B.C. Foundation design required placement of an approximately 5 m thick sand preload with wick drains. Without the use of wick drains, a preload up to 8 m thick left in place for 6 months or longer would have been required to meet tight construction schedules. The site was essentially flat-lying, with ground level at about elevation 2.5 m.

### Site Investigation

An example of the soil profile based on sampling and a piezometer cone penetration test (CPTU) is shown in Fig. 4. The soil stratigraphy is relatively uniform across the site, except for variations in thickness of the upper soft organic silt.

Surficial soils consist of about 1 to 1.5 m of sand fill overlying a soft organic clayey silt that extends to depths of about 4 to 9.5 m. Beneath the organic clayey silt is a dense sand layer that extends to a depth of about 24 m. The sand layer contains some random lenses and layers of softer silt, as shown

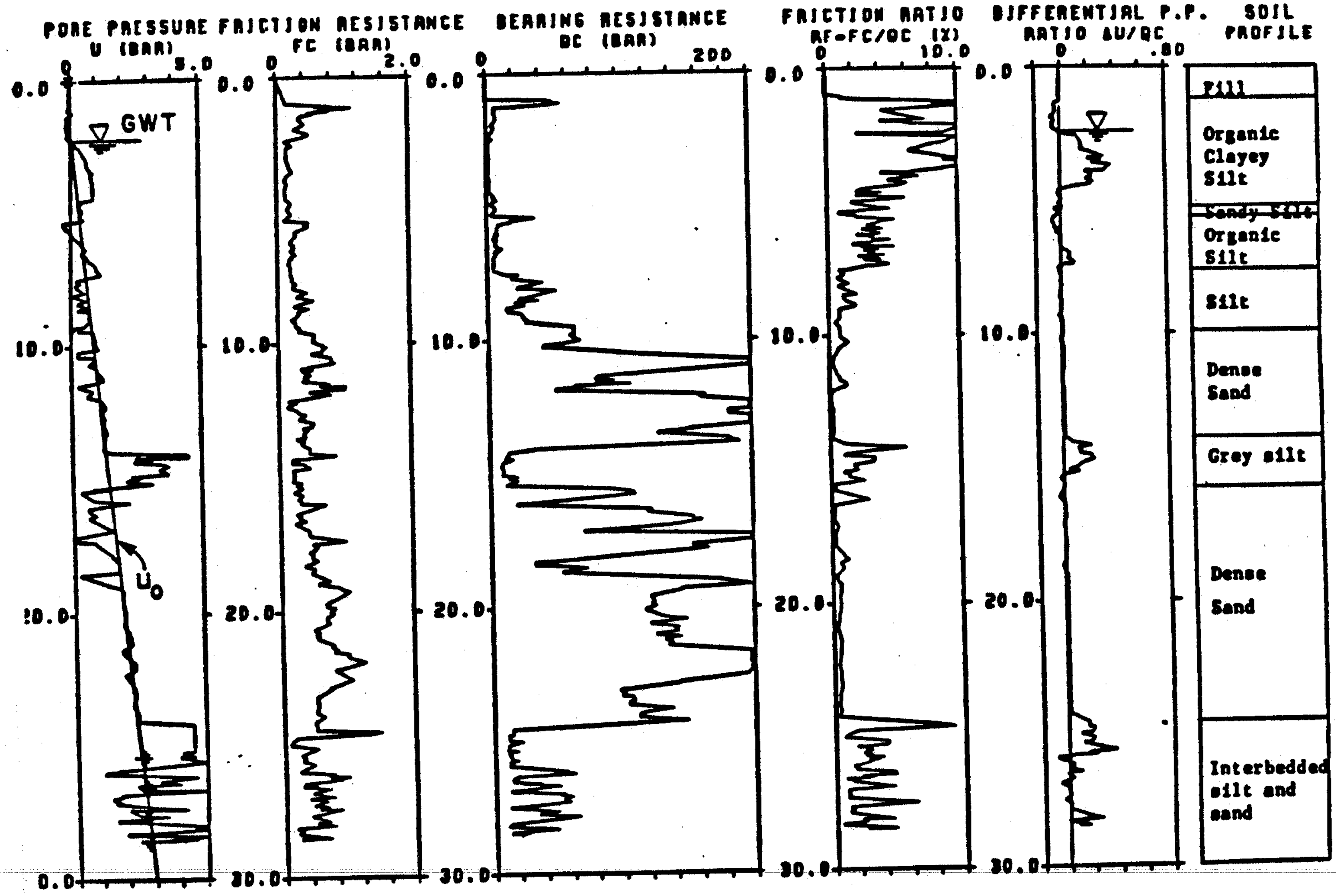

Fig. 4. Summary of soil profile at wick drain site in Vancouver, B.C.

in Fig. 4. Below the sand is an interbedded, soft to stiff, clayey silt and sand that extends to a depth of at least 60 m.

Laboratory and CPTU data indicate that the upper soft soils and the interlayered materials below 24 m are effectively normally consolidated and compressible. A total of 3 CPTU soundings were performed. The CPTU profile shown in Fig. 4 was the deepest to a depth of 29 m. The remaining CPTU profiles were both carried out to a depth of 7 m primarily to perform dissipation tests in the overlying soft organic silt.

During the CPTU several pauses were made in the penetration to monitor the dissipation rate of the excess pore pressures. Dissipation data was collected every 0.5 to 1.0 m through the upper clayey silt and silt deposit. The time for 50% dissipation ($t_{50}$), as indicated in Fig. 3, was determined and used to calculate $c_h$ values using the method proposed by Torstensson (1977). The average time for 50% dissipation was about 3 minutes for the pore pressure sensor located immediately behind the cone tip.

During the dissipation tests excess pore pressures were allowed to come to their equilibrium condition to obtain the overall equilibrium piezometric profile. A summary of the results from a series of dissipation tests from the shallowest CPTU profile is shown in Fig. 5.

The calculated $c_h$ values in the organic clayey silt layer are consistent with an average value of about $1x10^{-5}$ $m^2/s$. The equilibrium pore pressure data shown in Fig. 5 obtained after complete dissipation shows some interesting features. The sand layer from a depth of 3.8 m to 5.5 m shows a pore pressure profile that appears to reflect the water level in the adjacent Fraser River. The excess equilibrium pore pressures in the overlying clayey silt may reflect the downward seepage of water due to heavy rains experienced at the site the two months prior to the cone sounding. The CPTU tests were performed prior to preload placement.

## Wick Drain Installation

The installed pattern of wick drains was triangular and measured in the field to be 2.1 m between centres. The pattern was extended 3 to 8 m beyond the sides and up to 15 m beyond the ends of the proposed building perimeters. This was to ensure consolidation beyond the building walls and to assist in pore

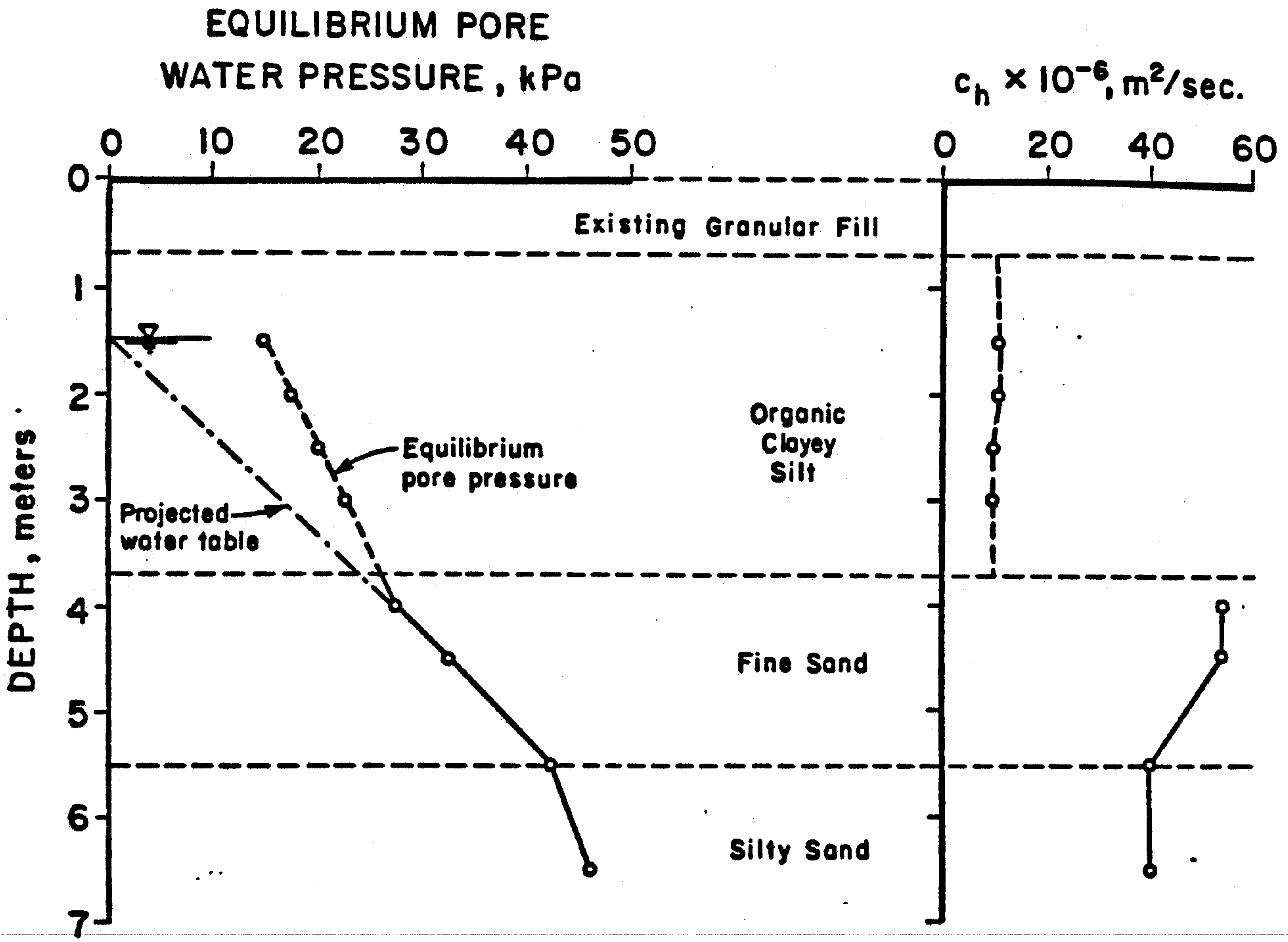

Fig. 5. Summary of calculated coefficients of consolidation and measured equilibrium porewater pressures from one CPTU sounding.

pressure dissipation at the edges of the relatively high preload fills, where stability was a concern.

The wick drains consist of a sleeve of porous, synthetic fibre surrounding a corrugated plastic band. The wick was about 100 mm wide by 5 mm thick and was manufactured by Alidrain in 150 m rolls. The wicks were installed with a hollow-core mandrel suspended from a boom attached to a backhoe. The wick was threaded inside the mandrel, and pushed hydraulically into the ground. The effective diameter of the drain, d, was assumed to be 50 mm (Hansbo and Torstensson, 1977). The effective diameter of the dewatered soil cylinder, D, was calculated to be 2.2 meters. A total of 26,175 m of wick was installed to depths of 4 to 9.5 m at 4328 locations for 3 building/warehouse facilities spread over the site.

## Monitoring Programme

Because of the short period available for preloading (less than 4 months), the potential for edge instability and the concern about effectiveness of the wick drains, a comprehensive monitoring programme was instigated by the geotechnical consultants. A total of 44 settlement plates, 36 movement hubs, 16 pneumatic piezometers and 2 screw-plate deep settlement gauges were installed. The pneumatic piezometers were placed in the middle of the upper clayey silt compressible layer. The screw-plate deep settlement gauges were placed in the sand below the upper clayey silt deposit. Full details of the monitoring programme, and a discussion of the results, are given by Robinson and Eivemark (1985).

Monitoring of the settlement and pore pressures during preloading was undertaken by the consultants to follow the progress of foundation compression, assess the adequacy of the design assumptions, and determine when subsequent phases of site work could be initiated.

Figure 6, after Robinson and Eivemark (1985), presents a summary of the settlement, load and pore pressure data for an area of the site where the upper compressible soil was near the maximum thickness. The results show that pore pressures due to the final stage of preload were essentialy fully dissipated after a period of about 50 days from the start of the preloading, only 30 days after the final lift of preload fill had been placed. The 'theoretical consolidation curve' indicated in Fig. 6 bears no relation to this paper. Monitoring settlements up to the end

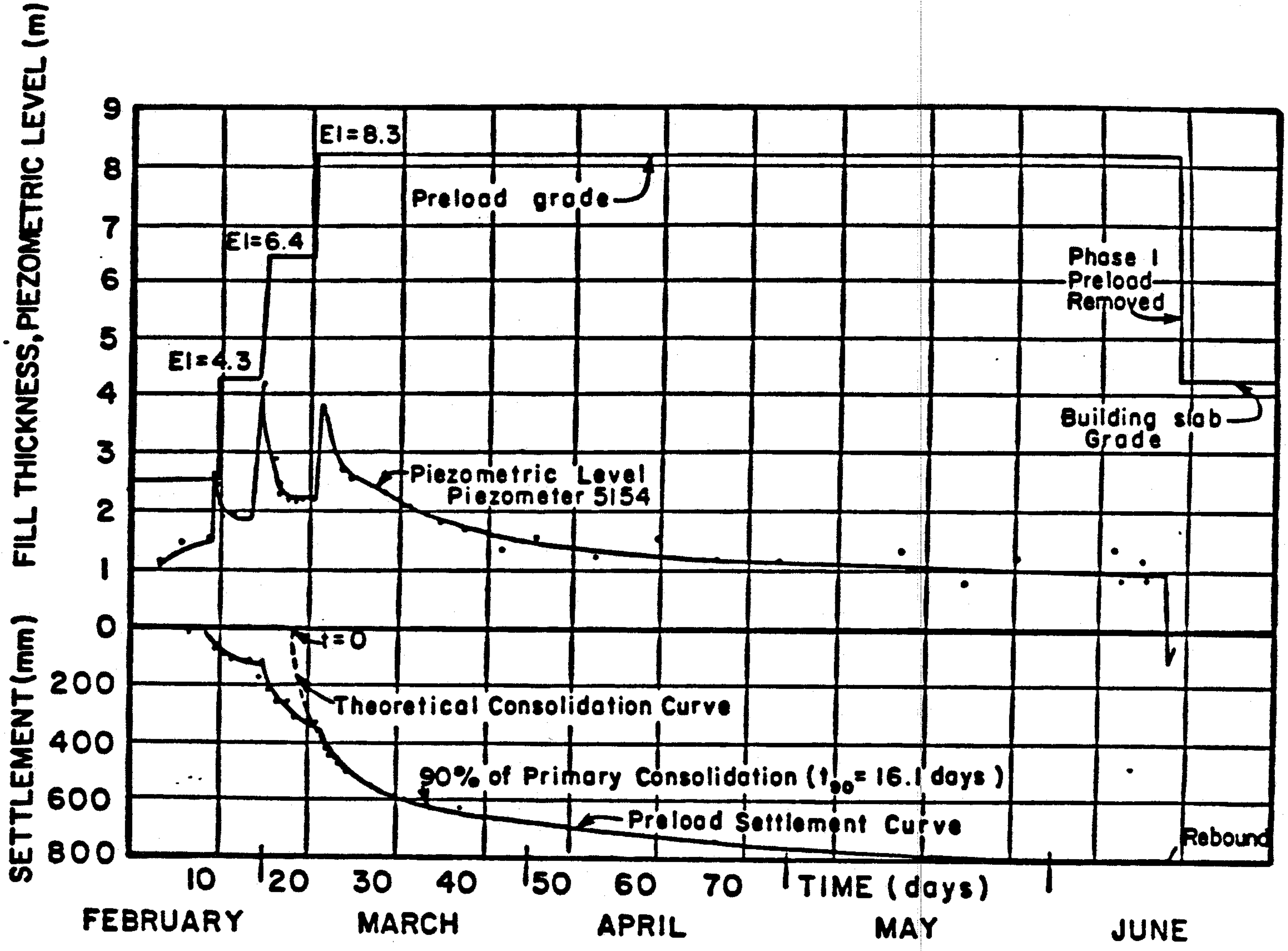

Fig. 6. Summary of settlement, load and pore pressure data from one area of the wick drain site (After Robinson and Eivemark, 1985).

of preloading (about 3 months after the fill had been placed) showed a constant rate of settlement as a function of the logarithm of time. At the time of preload removal, the long-term settlement rate varied from 75 to 130 mm per log cycle of time in days.

The variation of settlement across the site corresponds reasonably well with the variation in thickness of the soft surficial clayey silt deposit. Based on the screw-plate deep settlement markers, deep-seated settlements were in the order of 50 mm after 3 months. At the time of preload removal the measured total settlements ranged from 400 to 900 mm.

## Prediction of Wick Drain Performance

The rate of consolidation of the upper clayey silt deposit with wick drains installed was predicted using the piezometer cone penetration data. The rate of consolidation was calculated using the approach suggested by Hansbo (1979) as given in Eqn 1. An average value of $c_h = 1 \times 10^{-5}$ was used in the analysis (see Fig. 5). Full details of the calculation are given by Brown (1983).

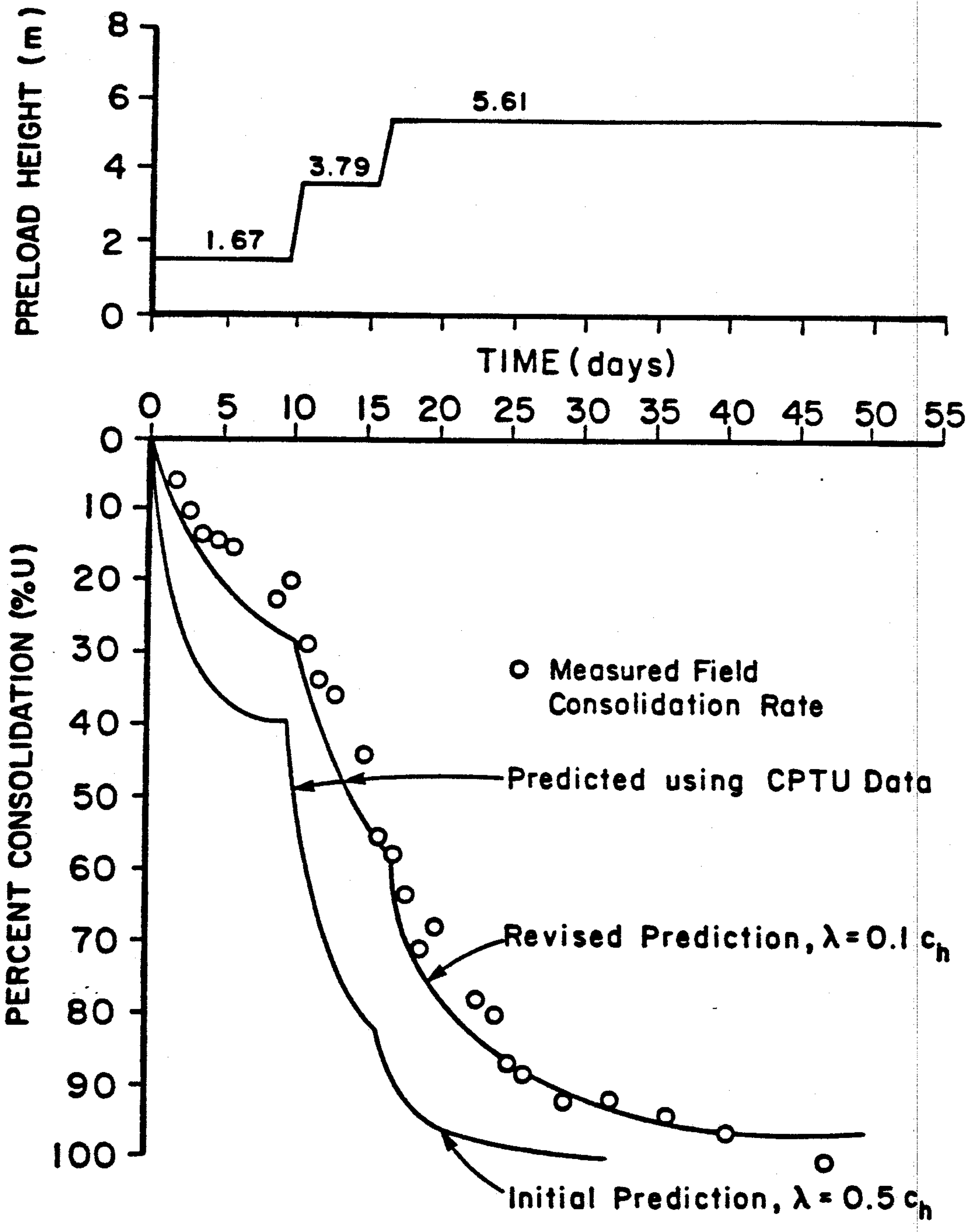

Fig. 7. Comparison of predicted and measured settlement rates at wick drain site.

A comparison between the calculated and measured settlement rates for one area of the site is shown in Fig. 7. The initial prediction using Hansbo's (1960) formula with $\lambda = 0.5\ c_h$ was refined to take into account the difference between laboratory and CPTU measured coefficients of consolidation and the highly organic nature of the soil under study. A factor of 5 was selected as an average difference between laboratory $c_v$ and piezocone $c_h$ (range of 2 to 7, Hansbo et al, 1981) to fit the response of the soil encountered. The revised prediction, which thus uses $\lambda = 0.1\ c_h$, is also presented in Fig. 7 and agrees well with the measured rate of settlement. Similar agreement was obtained in other areas of the site (Brown, 1983). Note that secondary consolidation is not considered in these settlement calculations and the measured field settlement records were corrected to remove the deep seated settlements recorded by the screw-plate settlement gauges.

The unusual 'scalloped' shape of the consolidation-time plots in Fig. 7 is due to the relatively long periods between preload increments. In most cases, the time to raise the preload fill to full height is often small in comparison with the total duration of the loading. However, due to the wick drains at this site, a significant amount of consolidation takes place between successive preload lifts.

## SUMMARY

A case history has been presented that shows the effectiveness of using pore pressure dissipation data obtained from piezometer cone penetration tests (CPTU) to calculate $c_h$ values for the design of wick drains. The calculated rates of consolidation using the CPTU derived $c_h$ values compare quite well with the measured performance when corrected for the difference between laboratory and field measured coefficients of consolidation. Similar experience has been reported by Hansbo et al (1981) and Battaglio et al (1981).

It is recommended that for similar soil conditions described herein a value of $\lambda = 0.1\ c_h$ should be used when $c_h$ is determined by in-situ pore pressure dissipation using a piezocone with sensing element behind the tip.

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