PORE PRESSURES DURING CONE PENETRATION TESTING

Prepared for ESOPT II
2nd European Symposium on
Penetration Testing
Amsterdam, The Netherlands
May 24 - 27, 1982

R.G. CAMPANELLA, D. GILLESPIE AND P.K. ROBERTSON
Civil Engineering Dept., University of British Columbia, Vancouver, Canada

1 INTRODUCTION

The addition of pore pressure measurements during static cone penetration testing has added a new dimension to the interpretation of geotechnical parameters particularly in loose or soft, saturated, deltaic deposits. The continuous measurement of pore pressures along with bearing and friction has enhanced the electric cone penetrometer as the premier tool for statification logging of soil deposits. The purpose of this paper is to report on observations of pore pressure effects during penetration testing with a multichannel cone. These effects include both dynamic pore pressures during penetration as well as their dissipation to determine consolidation characteristics and equilibrium values.

A 5-channel cone has been developed at the Department of Civil Engineering at the University of British Columbia that enables continuous monitoring of bearing, friction, pore pressure, slope and temperature. The dimensions conform to the European Standard for electric cones. Full details of the cone and associated equipment are given by Campanella and Robertson (1981).

2 PORE PRESSURE EFFECTS ON MEASURED PARAMETERS

It has been observed by us and others (de Ruiter, 1981) that when the cone is subjected to an all around water pressure there is a shift in the zeros for both the friction and tip measurements.

The friction shift is due to unequal end area of the friction sleeve (see Fig. 1) and is usually negative or opposite to the soil friction but can be positive. Most friction cones in use today have unequal end areas, but corrections are not usually made, perhaps because dynamic pore pressure measurements are lacking. Friction corrections are especially significant in deep profiles beneath the water table and in low permeability saturated soils where very large dynamic pore pressures are generated. High pore pressure zero shifts in friction explain why some normally consolidated and sensitive clays have such low friction ratios. Baligh et al., 1981, reported negative friction values. All friction cones should be calibrated and readings corrected for dynamic pore pressure effects, if one is to develop confidence in its use. Of course, the best solution is the design and use of a friction sleeve which has equal end area requiring no pore pressure corrections such as the one described by Campanella and Robertson (1981).

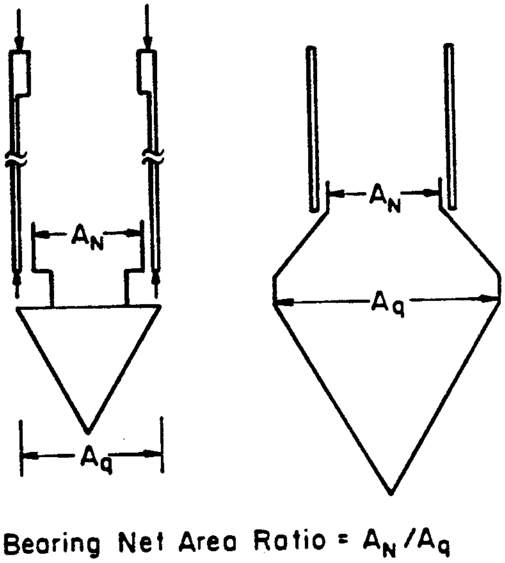


Fig. 1 Influence of unequal end areas.

Since the bearing tip is a total stress element, it should record a bearing stress equal to an all around applied pressure. To our knowledge, this is never the case and the tip always records a stress less than the applied all around pressure, again because of unequal areas at the tip (see Fig. 1). Thus, every cone has a given net area ratio associated with its design and dimensions. Most cones have net area ratios of from about 0.6 to 0.8, but a 20 cm^2 bulbous cone tip (de Ruiter, 1981) like the one shown in Fig. 1 would have a net area ratio less than 0.5 and probably close to 0.4. It is strongly recommended that all bearing cones be calibrated and when possible all bearing values reported as total stress where

$$q_T = q_c + u_T(1-a) \qquad (1)$$

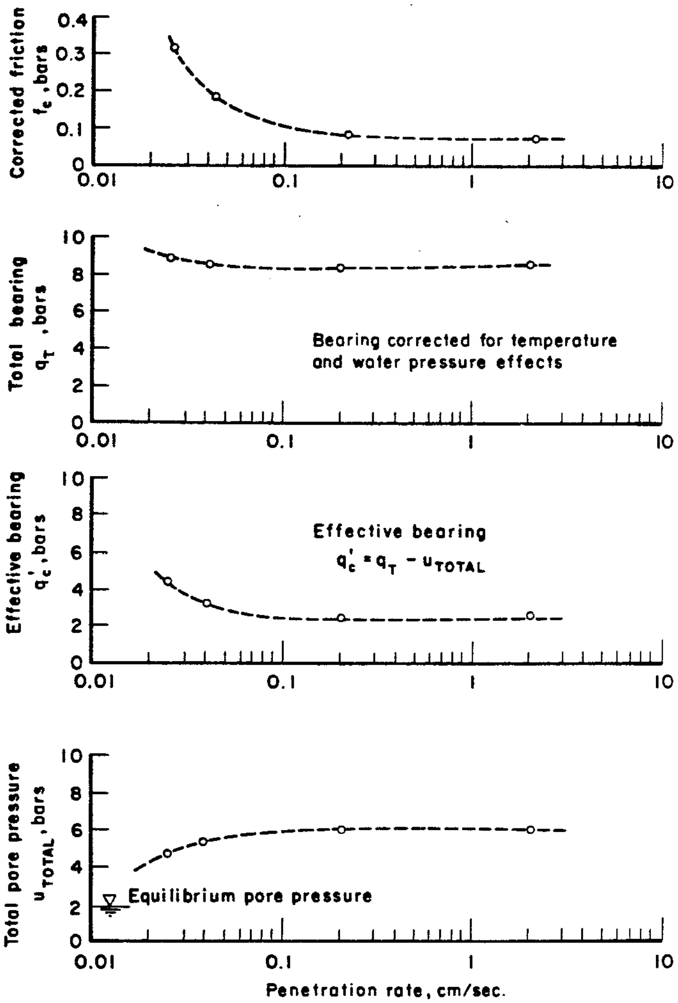
and q_T is total stress, q_c is measured bearing, u_T is total dynamic pore pressure and "a" is net area ratio. This correction can not be eliminated except with a unitized, jointless design where the sleeve is strain gauged to measure the tip load. Such a design is not yet available.

If the total tip resistance, q_T, is used, the differential pore pressure ratio, $\Delta u/q_T$, can be expected to relate more uniquely to the stress history of a clay deposit. Baligh et al. (1981) showed that the use of corrected total bearing in the ratio u_T/q_T reduced this ratio to values less than 1 which is the theoretical upper limit.

Also, not using total bearing, q_T, may account for some of the reported wide variations of the calculated bearing capacity factor, N_k, required to determine undrained shear strength from cone bearing.

3 RATE OF PENETRATION

The standard rate of penetration for a static cone test is 2 cm/sec. Traditionally cone penetration in sands have been considered to be drained and penetration in clays undrained. However, for mixed soils such as silts and clayey silts, the drainage condition during penetration is not well defined. Fig. 2 summarizes cone data as a function of penetration rate in a clayey silt at our research site. Our research site has a fairly uniform very deep (>100 m) clayey silt overlain by dense sands at a depth of about 15 m with the ground water table near the surface. Full details of the research site were given by Campanella and Robertson (1981). The clayey silt has a permeability in the order of 8 x 10^{-7}cm/sec. Fig. 2 shows that the penetration is essentially undrained down to a penetration speed of about 0.2 cm/sec. As the penetration speed is progressively decreased below this speed the total pore pressure during penetration decreases and a corresponding increase is observed for the total cone bearing and friction; both corrected for zero shift. The increase is particularly noticeable for the friction. The increase is less noticeable for the bearing, in part, because the bearing also records the water pressure. Thus, as the water pressure decreases the bearing tends to decrease, but this is offset by the increase in effective stress in the soil which increases the bearing. To illustrate this behaviour the **effective** bearing is also shown on Fig. 2. The effective bearing is defined as the total bearing (corrected for temperature and all around pressure effects) minus the total water pressure.



All measurements at 20m depth

Fig. 2 Penetration rate effects in clayey silt (McDonald's farm, Sea Island).

It is interesting to note that this effective bearing data shows how small an effective pressure (bearing) was required to penetrate the soft clayey silt under undrained conditions; the effective bearing was about 1/4 of the total bearing. These values are still extremely small when compared to the almost two orders of magnitude larger bearing values for the overlying dense sand which is being sheared under drained conditions. The data also illustrates the marked increase in the effective stresses around the tip as the rate of penetration decreases and the pore pressures drop by over 15 m of water pressure or about 1.5 bar (150 Kpa). The resulting change in effective stresses due to partially drained conditions around the tip produce an almost twofold increase in the effective bearing required to penetrate the silt. This behaviour is analogous to observed triaxial test behaviour of normally consolidated clays when comparing undrained with drained strength results.

The proposed concept of effective bearing defined as the total bearing stress minus the total water pressure represents a first order attempt at interpreting cone results as an effective strength characteristic. This would allow comparison of measured bearing stresses in undrained and partially drained soils with those in drained soils. Of course, the in-situ effective normal stress at the cone tip is still a missing essential parameter and must be estimated if one is to attempt a complete effective stress analysis. It is believed that the friction sleeve measurement may correlate well with the lateral effective normal stress and may provide the missing parameter. The concepts of effective bearing and effective stress interpretation of cone soundings are currently topics of intense research. It has become apparent, however, that it is essential to continuously monitor both pore pressure and bearing during penetration and to consistently work with total bearing (corrected for net area) in undrained soils.

4 DISSIPATION OF EXCESS PORE PRESSURE

Upon the arrest of penetration high positive excess pore pressures generated during cone penetration in cohesive soils such as the clayey silt at our research site, immediately start to dissipate. The rate of dissipation of the positive excess pore pressure is well known to depend upon the coefficient of consolidation of the soil. By monitoring the rate of dissipation of the excess pore pressure, an estimate of the coefficient of consolidation of the soil may be obtained. Several theoretical solutions are available to obtain the coefficient of consolidation from the dissipation of excess pore pressures generated by cavity expansion. The applicability and meaning of the solutions is complicated by several phenomena. These phenomena include:

- the importance of vertical as well as cylindrical diffusion,
- the effect of soil disturbance, and
- uncertainty over the distribution, level and change of total radial stresses.

In spite of these limitations, the usefulness of the theoretical solutions is encouraged by the repeatability of the test and the vast range in the dissipation rates measured for various soils encountered.

The influence of vertical dissipation was shown by Gillespie and Campanella (1981) by comparing dissipations recorded at various distances up the shaft from the tip. Dissipations immediately behind the tip, as in the UBC cone, were similar to dissipations further up the shaft. Also, horizontal dissipation appears to dominate the consolidation process. Hence, a cylindrical dissipation solution such as that by Torstensson (1977) was chosen in order to compare field dissipation results to laboratory test results.

Constant rate of strain consolidation tests were performed on 76 mm diameter samples trimmed in horizontal and vertical directions from a 89 mm diameter undisturbed sample obtained adjacent to the location of the pore pressure dissipations. In this manner, the predicted value could be compared directly to the laboratory measured values for the coefficient of consolidation.

A typical pore pressure dissipation, results of the laboratory testing, as well as the predicted coefficient of consolidation from the pore pressure dissipation, are shown in Fig. 3. The coefficient of consolidation was predicted using the cylindrical solution of Torstensson (1977) at the 50% level of dissipation. Comparison between the predicted coefficient of consolidation and the laboratory measured value reveals that the predicted value compares favourably with the horizontal coefficient of consolidation in the over consolidated state.

As well as predicting an accurate coefficient of consolidation, to be useful, theories of pore pressure dissipation should predict a coefficient of consolidation that is independent of the degree of dissipation. Figure 3 shows a reasonable comparison between the measured pore pressure decay and the theoretical dissipation rate plotted using the value for the coefficient of consolidation predicted at the 50% level of dissipation. The results obtained to date support the view that consolidation takes place in the recompression mode after cone penetration testing in cohesive soils. Use of existing theoretical solutions such as those by Torstensson (1977) would seem, at least from the

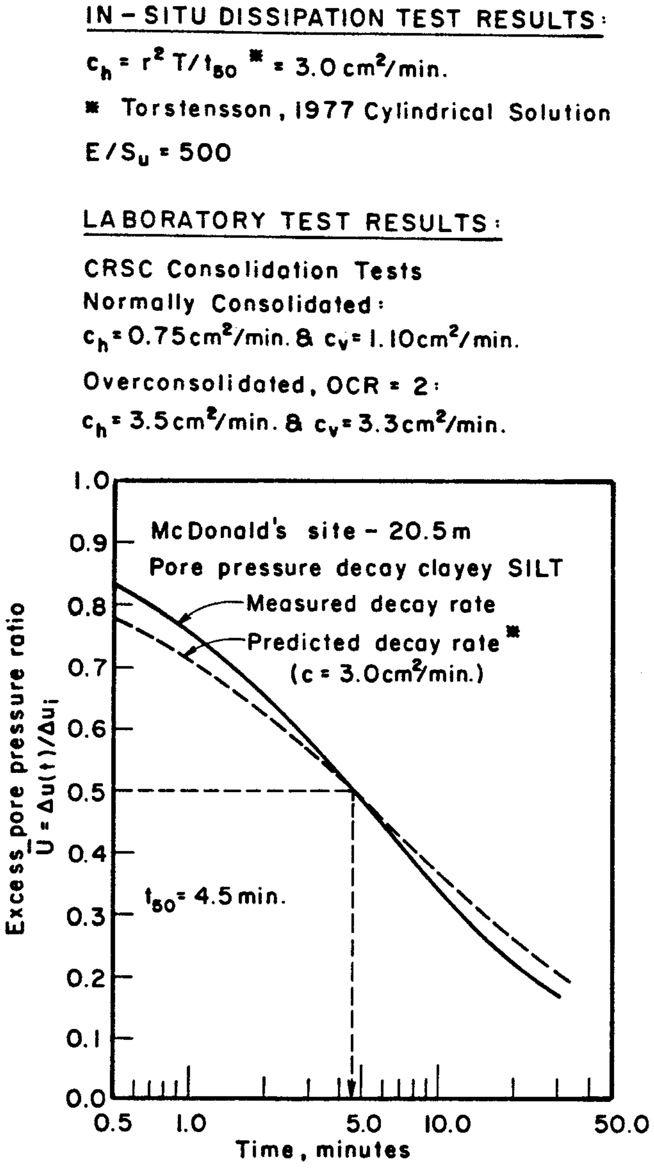


Fig. 3 Measured and predicted coefficient of consolidation.

experimental results obtained during this study, to be applicable for calculating the coefficient of consolidation in the horizontal direction for soil in the slightly overconsolidated state (OCR ≈ 2). Furthermore, a knowledge of the compressibility of the soil during recompression would allow direct calculation of the coefficient of permeability.

5 PROCEDURE USED WHILE RECORDING DISSIPATIONS

There is very little literature describing the procedure used while recording pore pressure dissipations. Some report that they found it necessary to clamp the penetration rods at the ground surface while recording pore pressure dissipation. It appears that if the rods were not clamped that a drop in the measured pore pressure would result when load was released from the tip. In our studies variations in the amount of load applied to the tip was made during several dissipations. Only insignificant differences in the decay response were observed when using cones with pore pressure sensing element immediately behind the tip.

The location of the sensing element may explain the sensitivity of the decay response to the procedure used. When load is released, pore pressures at the tip immediately drop in response to the decrease in total stress. Whereas behind the tip, in the zone of failed soil the stress level does not change significantly when load is released. It therefore appears that with the location of the piezometer element behind the tip the test is much less sensitive to the procedure used. This may be an important point because the amount of load applied to the tip, even with the rods clamped will change with time.

6 STRATIGRAPHIC LOGGING

Specific examples are presented in this section to illustrate the importance of simultaneous measurement of dynamic pore pressure as well as friction and bearing values. A portion of a cone log obtained from a site in the Fraser River delta near Vancouver, B.C. is shown in Fig. 4. The material types interpreted from the cone logs are also shown in Fig. 4. Within each unit shown, the greatest detail can be seen from the pore pressure log. For example, within the silt layers, the bearing and friction values, which reflect the properties of a larger zone of soil, are nearly constant, whereas the pore pressure log reveals small sand partings within each silt layer.

The material classification shown in Fig. 4 includes the distinction between sand, silt, and silty sand deposits. Although the sand and silt deposits are easily distinguished using only the bearing and friction measurements, the silty sand layers can only be distinguished from the silt layers by the pore pressure measurements. The bearing and friction ratio for the two materials are similar, each has a value of approximately 20 bar and approximately 3, respectively. If only the friction and bearing measurements are available, each would be classified as a silt. Inspection of the dynamic pore pressure logs, however, reveals a distinctly different behavior of the two materials. Compared to the silt, the silty sand has a much lower dynamic pore pressure. The variation in dynamic pore pressure can occur because either the silty sand is of

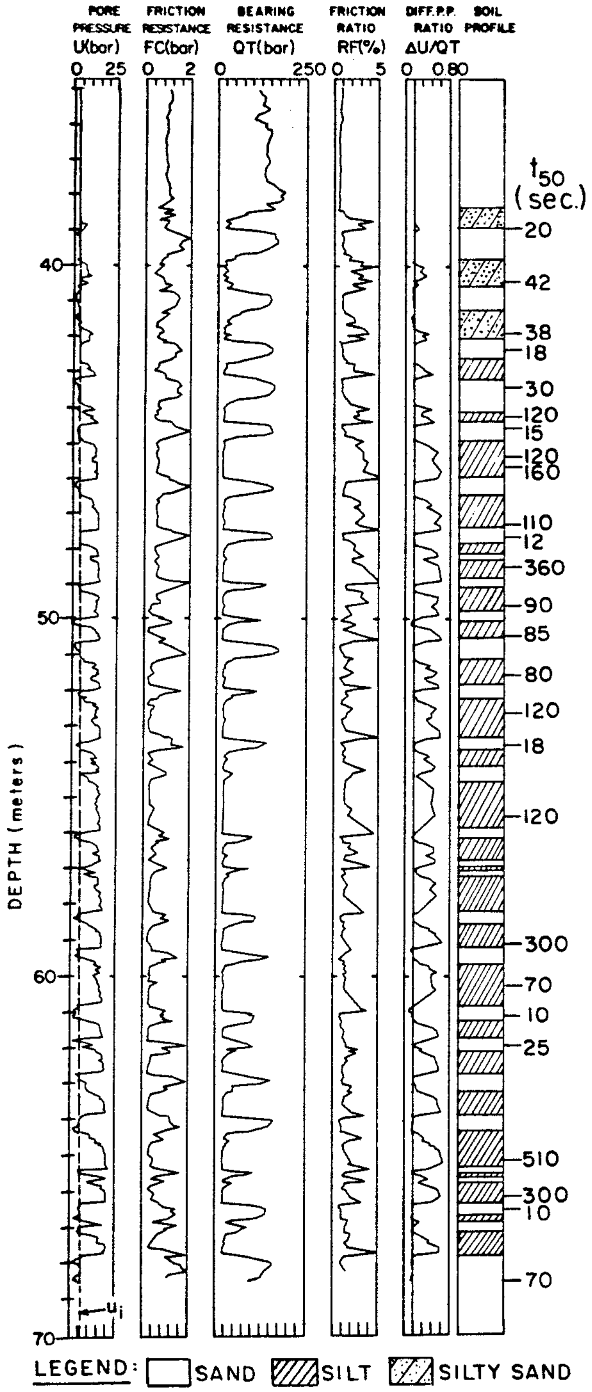


Fig. 4 Piezometer friction cone logging in stratified soils (Annacis Site).

smaller compressibility or is a freer draining material. Since the materials have the same stress history and equivalent bearing values, it appears that penetration of the silty sand results in a lower dynamic pore pressure because of partial drainage.

The time to reach 50% dissipation or t_{50} is a useful parameter for distinguishing stratigraphic types and indicating drainage conditions. For many soil types such as those shown in Fig. 4 the time to reach 50% dissipation of excess pore pressure is often achieved during rod breaks, with very little standby time. Similar results can be obtained using a smaller degree of dissipation of say 25% in less permeable soils.

7 DIFFERENTIAL PORE PRESSURE RATIO

Fig. 5 shows a piezometer cone log in hydraulically placed sandy and silty soil at a test section before and after treatment by dynamic consolidation. The sandy soil exists to a depth of about 5 m and a silty layer lies between about 5 and 8 m depth. It is interesting to note that before treatment the silty layer generated high excess pore pressures as indicated by the large differential pore pressure values. However, after treatment, there was a remarkable change in the pore pressure behaviour with significant pore pressures **below** hydrostatic and some actually negative or below atmospheric. A similar pore pressure behaviour was observed for a second test section where treatment was by vibro-compaction.

The very different pore pressure behaviour suggests that the volume change characteristics of the silty layer have been altered dramatically even though the bearing appears to have increased only a small amount. Before treatment the silt was contractive and after treatment dilative, at least, for the pore pressure element behind the tip. From the standpoint of liquefaction resistance, volume change characteristics are very important.

Sandy soils tend to generate pore pressures, but they dissipate almost as fast as they are generated and the high bearing in sands gives differential pore pressures of essentially zero. Also, it has been observed that because of their relatively low permeability, silty soils can give significant pore pressures. Thus, the differential pore pressure ratio gives an excellent indication of both the volume change characteristics and relative permeability. Also, it should be appreciated that the pore pressure ratio is sensitive to the bearing net area ratio correction in poorly drained soils and q_T must be used if correlations are to be meaningful.

8 POROUS FILTER LOCATION ON CONE

We take this opportunity to list here the following advantages of having the pore pressure element immediately behind the cone:

1. good protection and less prone to damage,
2. easier to saturate,
3. gives reasonably stable pore pressure response,

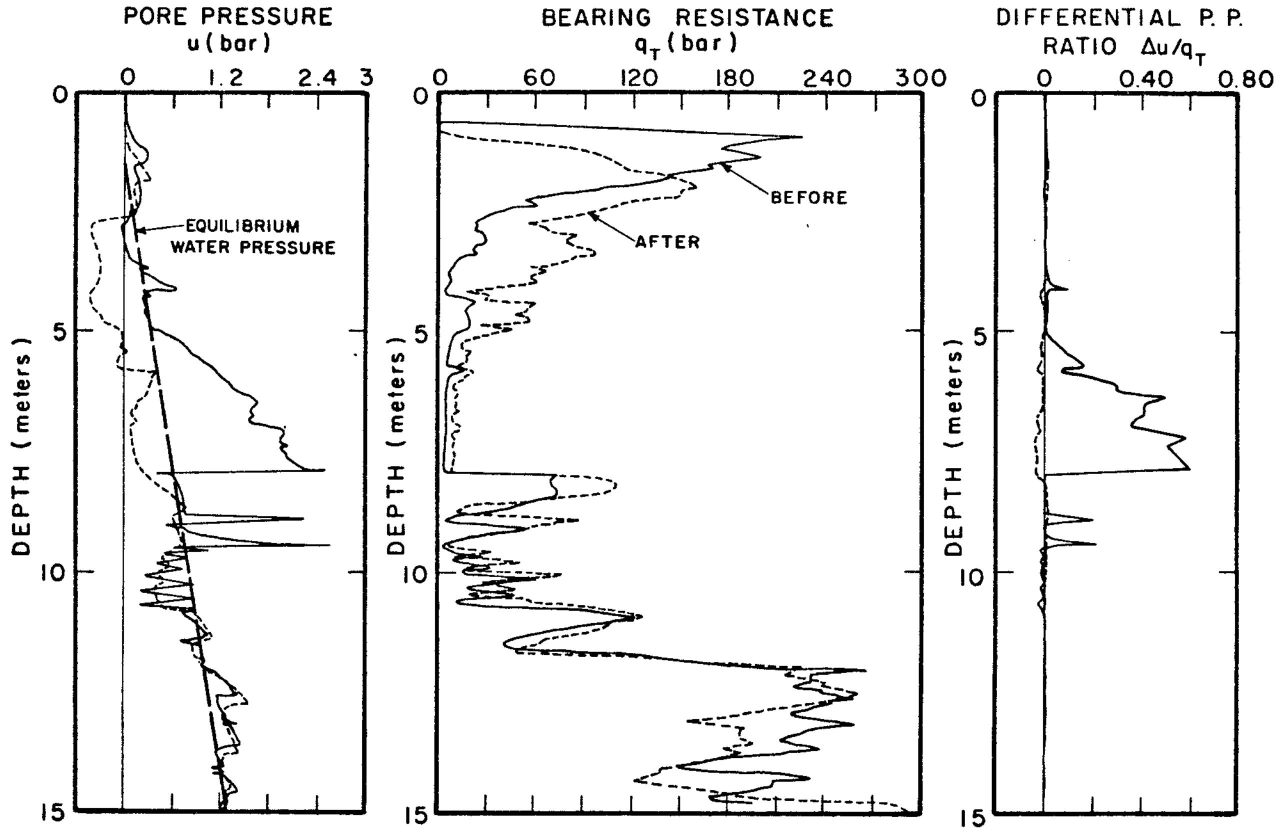


Fig. 5 Piezometer cone logging before and after dynamic compaction.

4. gives good range of dynamic pore pressures from negative to very positive,
5. good location for use of cylindrical solution to obtain consolidation characteristics from pore pressure decay,
6. best location to apply net area correction to calculate actual total cone bearing,
7. good location to apply pore presure correction to friction if end areas of sleeve are not equal.

The main disadvantage appears to be that the area behind the cone does not record the dynamic pore pressure which exists in the zone of maximum shear and total stress which is in front or on the face of the cone. It is believed that the location of the porous element immediately behind the cone encourages the measurement of low or negative dynamic pore pressures in fine sands and silts, yet records high positive pore presures in clays which are close to the maximum value. Furthermore, it is believed that the measurement of maximum pore pressure should not be the highest or overriding priority for a general investigative tool, and it is recommended the element be located immediately behind the tip.

9 ACKNOWLEDGEMENT

The assistance of NSERC and EMR, Canada; Ertec Western; Fugro B.V. and the Civil Engineering technical staff at the University of B.C. is most appreciated. Steve Brown and Ian McPherson helped with the field testing.

10 REFERENCES

de Ruiter, J., 1981, Current penetrometer practice, Sym. on Cone Penetration Testing and Experience, Geotechnical Div., ASCE, pg. 1-48.

Campanella, R.G. and Robertson, P.K., 1981, Applied cone research, Sym. on Cone Penetration Testing and Experience, Geotechnical Div., ASCE, pg. 343-362.

Baligh, M.M., Azzouz, A.S., Wissa, A.Z.E., Martin, R.T. and Morrison, M.J., 1981, Piezometer probe test in cohesive deposits, Sym. on Cone Penetration Testing and Experience, Geot. Div., ASCE, 247-264.

Gillespie, D. and Campanella, R.G., 1981, Consolidation characteristics from pore pressure dissipation after piezometer cone penetration, Soil Mechanics Series No. 47, Dept. of Civil Eng., Univ. of British Columbia, 17 pgs.

Torstensson, B.A., 1977, The pore pressure probe, Nordiske Geotekniske Mote, Oslo, Paper No. 34, pp. 34.1-34.15.