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FACTORS AFFECTING THE PORE WATER PRESSURE AND ITS MEASUREMENT AROUND A PENETRATING CONE

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ABSTRACT

This paper discusses the practical aspects of measuring pore pressures during cone penetration and the factors that affect these measurements. Discussion covers the areas of pore pressure element location, soil type, soil stress history, sensitivity and stiffness, and cone design. Discussion is also provided regarding the use of pore pressure dissipation data. Field observations and experiences are used to support the concepts proposed.

INTRODUCTION

Use of the cone penetration test (CPT) has become increasingly more popular for site investigation. The addition of pore pressure measurements during the CPT, often noted as CPTU, is also becoming more popular. Much attention has been focussed in recent years towards a clearer understanding of the interpretation of the pore pressures measured during cone penetration. Many new interpretation techniques have been suggested to evaluate soil type, undrained shear strength and stress history (OCR) from the pore pressures measurements.

This paper discusses the practical aspects of measuring pore pressures during cone penetration and the factors that affect these measurements. Field observations and experiences gathered during more than seven years of piezometer cone testing are used to support the concepts proposed.

Discussion of the factors that affect pore pressure measurement around a penetrating cone are covered in the following subheadings;

- pore pressure element location
- soil type
- soil stress history, sensitivity and stiffness
- cone design.

Discussion is also given on the procedures and results from pore pressure dissipation tests performed during a pause in the cone penetration.

A detailed description and discussion on test procedures and saturation techniques are given by Robertson and Campanella (1984).

PORE PRESSURE ELEMENT LOCATION

Pore pressures can be generated in soils due to changes in both mean normal stresses and shear stresses. When saturated soils are subjected to increases in mean normal stresses, positive pore pressures are generated. When saturated soils are subjected to only shear stresses, such as in a simple shear test, pore pressures generated can be either positive or negative depending on the contractive or dilative response of the soil. During cone penetration the soil elements adjacent to the penetrating cone experience changes in both mean normal stresses and shear stresses. Cavity expansion theories and observations of CPT measurements have shown that total normal stresses are high on the cone tip and low immediately behind the tip. On the face of the cone during penetration normal stresses and shear stresses are highest, generally resulting in high positive pore pressures in fine grained soils. However, as an element of soil passes behind the tip, there is a small strain decrease resulting in a large total normal stress relief. This stress relief can be very large in sandy soils (Hughes & Robertson, 1985), due to arching effects, and less in soft clayey soils.

Therefore, pore pressures measured on the face of a penetrating cone are influenced by both large normal and shear stresses, whereas, pore pressures measured immediately behind the tip tend to be influenced more by high shear stresses. Therefore, pore pressures measured immediately behind the tip appear to be more representative of the shear induced behavior of the soil.

Figure 1 represents a summary of the observed pore pressures measured at different locations on a cone during penetration through a large variety of different soils. The observed pore pressures (u) have been normalized to the equilibrium pore pressures (u_o). In normally consolidated insensitive clays and silts the pore pressure on the face of the cone tip is usually in the order of 3 to 4 times larger than u_o, with the pore pressure immediately behind the tip approximately 15 to 20 percent smaller. However, for fine grained soils with progressively larger overconsolidation ratio (OCR), the pore pressures measured on

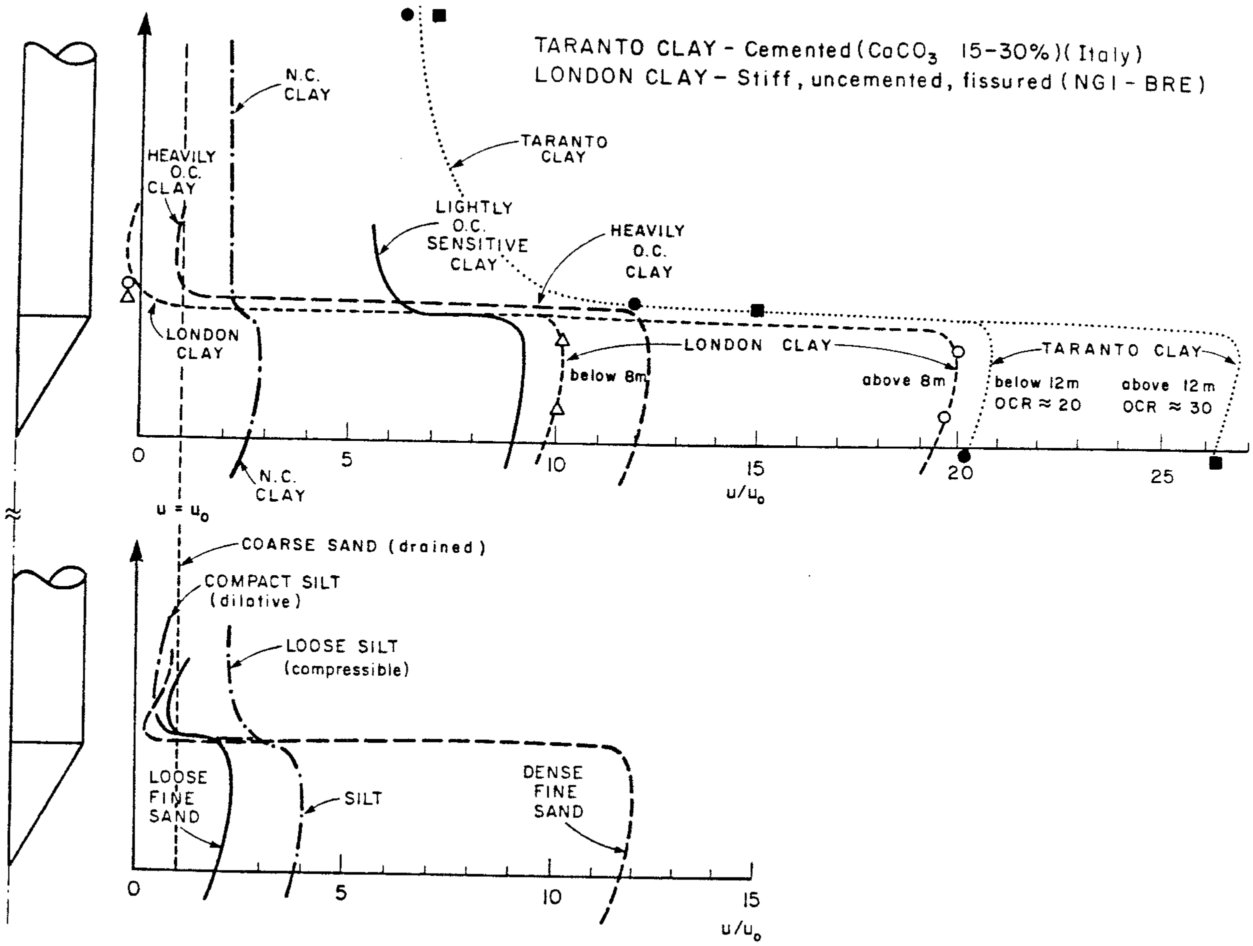


Fig. 1. Conceptual pore pressure distribution in saturated soil during CPTU based on field measurements. (After Robertson et al, 1986)

the face of the cone become progressively larger. Pore pressures on the face of the cone tip have been recorded up to 26 times larger than u_o for very stiff clays with OCR ≈30 (Jamiolkowski et al, 1985), whereas pore pressures measured behind the cone tip are generally significantly less. The high pore pressures on the face of the tip in stiff overconsolidated clays is due, mainly, to the large increases in normal stress due to cone penetration, whereas the area behind the tip is in a zone of normal stress relief. Both areas have large shear stresses but the large normal stresses dominate the pore pressure response on the face. Shear stresses appear to dominate the pore pressure response behind the tip.

Similar observations have been made during cone penetration in fine and silty sands. In dense silty sands pore pressures can be high on the face of the cone and low immediately behind the tip. Figure 1 indicates that pore pressures measured immediately behind the tip are often less than u_o, even in loose sands, provided penetration is undrained.

Figure 1 indicates that, regardless of soil type, the pore pressure along the face of the cone is relatively insensitive to the exact location of the sensing element. Lunne et al (1986) report that similar pore pressures have been measured on the cone tip and at the mid-height of the cone face. Similar findings were reported by Roy et al (1982). This is an important finding which allows comparison of pore pressure data collected from different cone designs, each having the pore pressure element located somewhere on the cone face.

Some of the data used to compile Figure 1 shows that in some overconsolidated and dense soils a large gradient in pore pressures can exist immediately behind the tip. This gradient can result in a variation in measured pore pressure for slightly different measurement locations in the area behind the tip. Figure 2 presents an example of data from the Imperial Valley, California, showing pore pressure measurements at different locations on a cone during penetration in an overconsolidated stiff clay from a depth of 6.5 m to 11.5 m. The pore pressure measured on the face of the tip with a 5 mm thick element is approximately 9 times larger than u_o. The pore pressure measured with a 5 mm thick element located 5 mm behind the shoulder (edge of shoulder to centre of element) of the tip measures a pore pressure slightly less than u_o. However, a 2-1/2 mm thick element located only 2-1/2 mm behind the shoulder of the tip measures a pore pressure almost twice u_o. The data in Figure 2 illustrates the high pore pressure gradient that can exist behind the tip in stiff soils.

An alternate location for measurement of pore pressures behind the tip is to locate the pore pressure element behind the friction sleeve (i.e., 135 mm behind the tip for a 10 cm² base area tip). Data collected in soft fine grained soils simultaneously behind the tip and behind the friction sleeve indicates that pore pressures are generally slightly lower behind the sleeve than behind the tip. Discussion will be given later regarding uses and advantages of measuring pore pressures behind the friction sleeve.

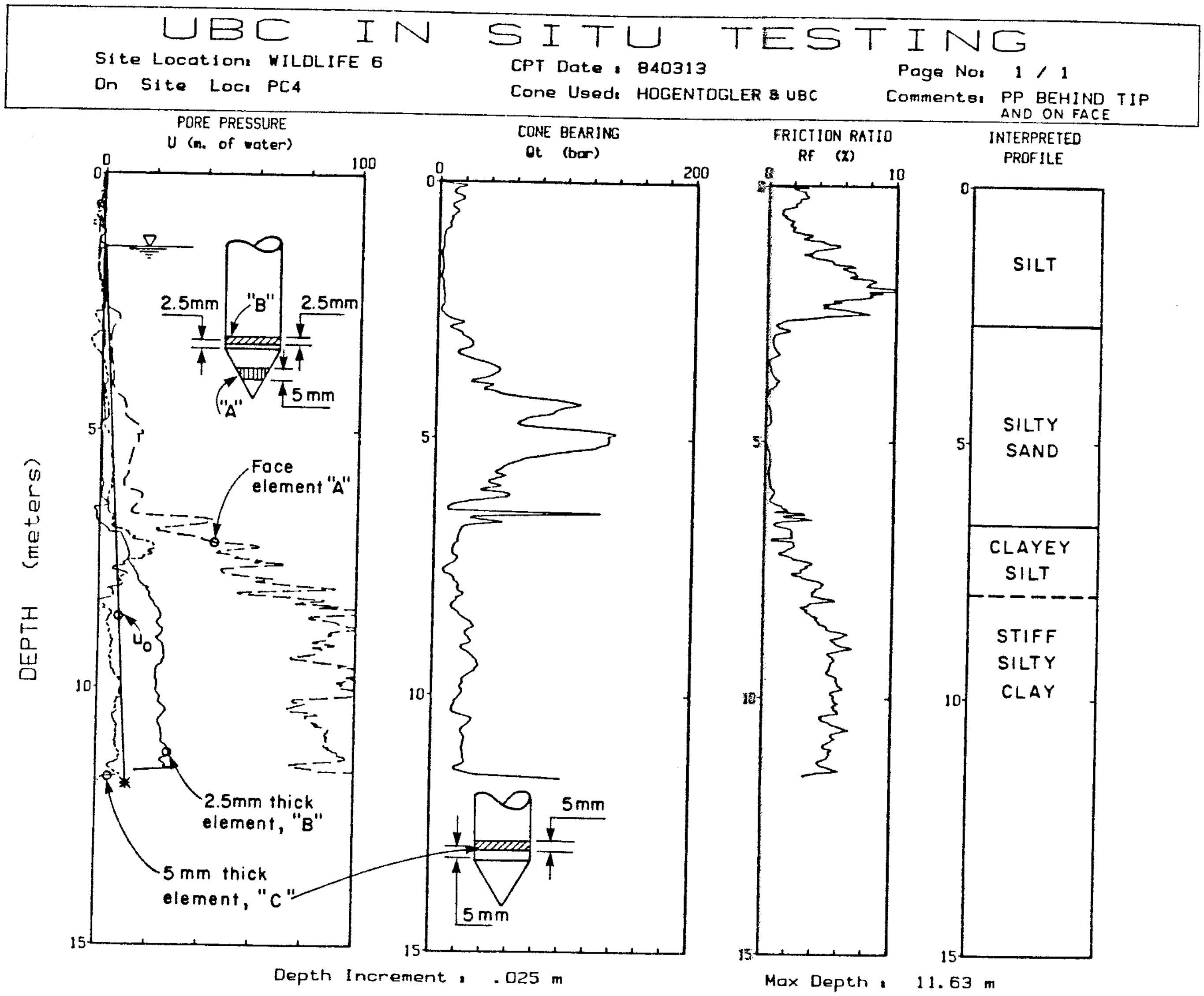


Fig. 2. CPTU pore pressures at different locations.

SOIL TYPE

Traditionally, soil classification from CPT data has been related to cone bearing, q_c, and friction ratio, FR, where,

$$\text{Friction ratio, FR} = \frac{f_s}{q_c} \times 100\%$$

Several charts have been developed that use these basic CPT data. All the charts are similar in that sandy soils generally have high cone bearings and low friction ratios; whereas clayey soils generally have low cone bearings and high friciton ratios. However, the measurement of sleeve friction is sometimes less accurate and reliable than cone resistance. Also cones of different designs will often produce variable friction sleeve measurements. This can be caused by variations in mechanical and electrical design features of the friction sleeve as well as unequal end areas.

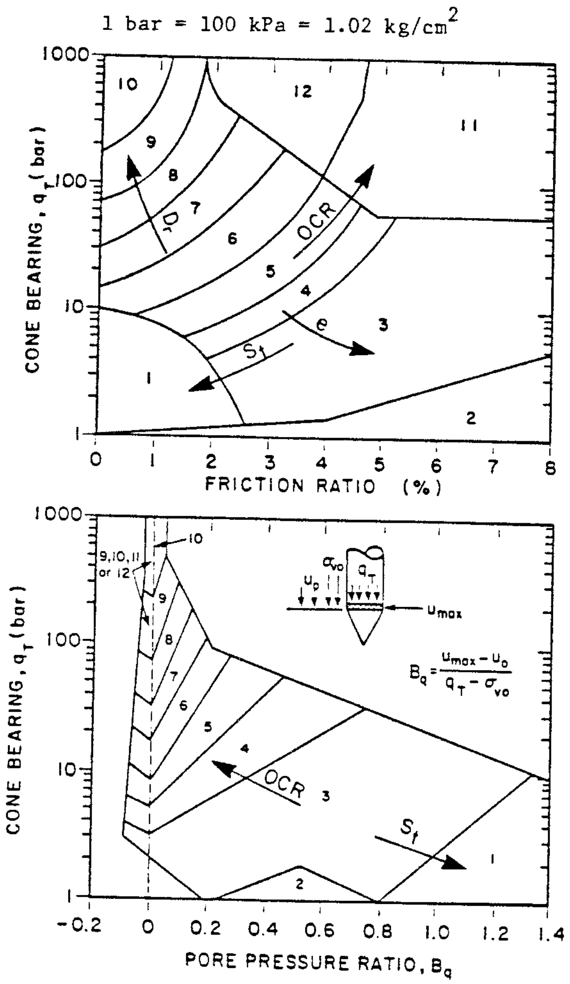
To overcome the problems associated with sleeve friction measurements, several soil classification charts have been proposed based on q_T and pore pressures (Jones and Rust, 1982; Baligh et al, 1980; Senneset and Janbu, 1984; Robertson et al, 1986). The more recent charts use the pore pressure parameter ratio, B_q, defined as:

$$B_q = \frac{\Delta u}{q_T - \sigma_{vo}}$$

where Δu = excess pore pressure, $\Delta u = u - u_o$
q_T = cone resistance corrected for pore pressure effects (Campanella et al, 1982)
σ_{vo} = total overburden stress.

The authors have found from their experience that it is not always possible to clearly identify a soil type based solely on the q_T and Δu data. Sometimes changes in the friction ratio have been able to more clearly define changes in soil type. Therefore, the authors recommend and use all three pieces of data (q_T, u, f_s) in the form of q_T, B_q and FR to define soil behavior type, as shown on Figure 3.

The charts in Figure 3 can be used as a guide to define soil behavior type based on CPTU data. Factors such as changes in stress history, sensitivity, stiffness and void ratio will influence the classification using either the FR or the B_q chart. Occasionally, soils will fall within different zones on each chart. In these cases judgment is required to correctly clas-



Zone	Soil Behaviour Type
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained*
12	sand to clayey sand*

* overconsolidated or cemented.

Fig. 3. Proposed soil behavior type classification system from CPTU data.

sify the soil behavior type. Often the rate and manner in which the excess pore pressures dissipate during a pause in the cone penetration will aid in the classification. For example, a soil may have the following CPTU parameters; q_T = 10 bars (10 tsf), FR = 4%, B_q = 0.1. It would classify as a clay on the FR chart and as a clayey silt to silty clay on the B_q chart. However, if the rate of pore pressure dissipation were very slow, this would add confidence to the classification of a clay. If the dissipation were rapid (t_{50} < 60 secs), the soil may be more like a clayey silt or possibly a clayey sand. The manner of the dissipation can also be important. In stiff, overconsolidated clay soils, the pore pressure behind the tip can be very low in comparison to the high pore pressures on the face. When penetration is stopped, pore pressures recorded immediately behind the tip may initially rise before dropping to the equilibrium pressure. The rise is caused by local equalization of the high pore pressures on the nearby cone face.

The standard rate of penetration for CPT is 2 cm/sec. During cone penetration, soils tend to generate excess pore pressures. However, for penetration in medium grained clean sands and coarser materials, these pore pressures dissipate almost as fast as they are generated and penetration takes place under drained conditions. For penetration in fine grained soils, such as clays and clayey silts, significant excess pore pressure can be generated because of their relatively low permeability, and penetration takes place under predominantly undrained conditions. Penetration into fine sands and silty sands can generate excess pore pressures, but penetration may be taking place under partially drained conditions.

Correct interpretation of the pore pressure data requires some knowledge that the penetration is predominantly undrained. Radial consolidation theory can be used to obtain a plausible estimate of the upper limit to soil permeability for which the piezometric element will observe undrained pore pressures. The estimate of the upper limit to permeability for undrained penetration depends on the soil compressibility and stiffness as well as the size of the cone and porous element. For standard 10 cm² base area cones and 5 mm thick porous elements, the plausible upper limit to soil permeability for undrained penetration (at 2 cm/sec) is in the order of 1×10^{-7} m/s. A partially drained CPT response may be observed for soils with a permeability in the range of 1×10^{-4} m/s to 1×10^{-7} m/s, that is, soils such as fine sands to silts. For permeabilities greater than about 1×10^{-4} m/s, penetration is most likely fully drained. These values have been generally confirmed by field observations.

As suggested earlier, an indication of the soils permeability can be made by observation of the rate of dissipation of any excess pore pressures. If the pore pressures dissipate fully in less than about 3 minutes, the penetration process was most likely partially drained and quantitative interpretation can be difficult. A full dissipation in 3 minutes represents a 50% dissipation (t_{50}) of about 30 seconds.

The above comments are given only as a guide to the problems of partial drainage during cone penetration. Other factors such as stratified deposits and poor saturation of the sensing element also influence the pore pressure response.

It has often been suggested that if penetration is partially drained at 2 cm/sec, the rate of penetration could be increased or decreased to produce an undrained or drained penetration, respectively. However, since the permeability of soils varies by orders of magnitude, the changes in penetration rate required to significantly change the drainage process would also vary by orders of magnitude. Penetration rates of 20 cm/sec or faster and 0.2 cm/sec or slower become impractical.

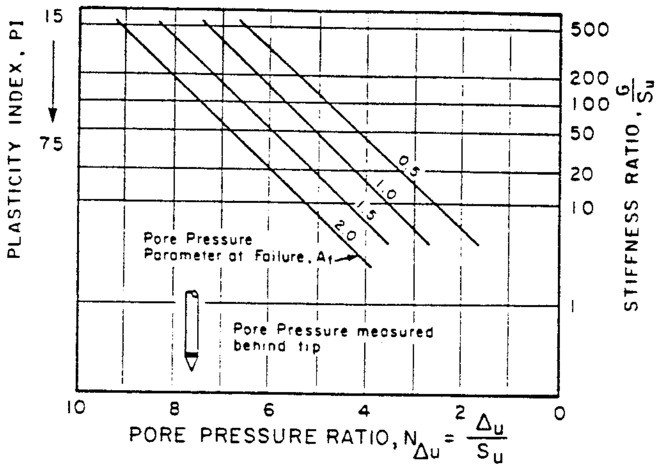
SOIL STRESS HISTORY, SENSITIVITY AND STIFFNESS

For undrained cone penetration into fine grained soils, the pore pressures generated are influenced by the soils strength, stress history, sensitivity and stiffness. Cavity expansion and other theoretical solutions have shown that for normally consolidated insensitive clays the excess pore pressures depend on the rigidity index, where

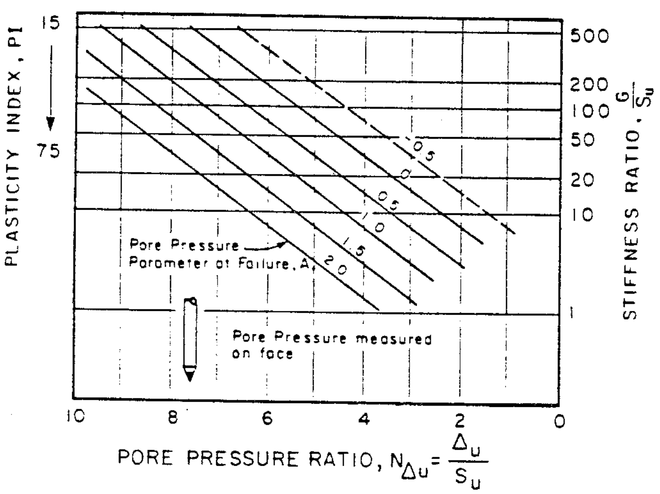
$$\text{rigidity index, } I_R = \frac{G}{S_u}$$

where, G = shear modulus
S_u = undrained shear strength.

Low values of I_R generally apply to highly plastic clays (plasticity index, PI > 80) which tend to generate low pore pressures. High values of I_R generally apply to low plastic clays and silts (PI < 15) which tend to generate high pore pressures. The excess pore pressures also tend to increase with increasing soil sensitivity and decrease with increasing overconsolidation ratio (stress history).



(a) Pore pressures measured behind tip.



(b) Pore pressures measured on face of tip.

Estimates of Skempton's Pore Pressure Parameter, A_f.

Saturated Clays	A_f
Very sensitive to quick	1.5-3.0
Normally consolidated	0.7-1.3
Lightly consolidated	0.3-0.7
Highly consolidated	-0.5-0.0

Fig. 4. Method to obtain undrained shear strength from excess pore pressures measured during CPTU test.

A semi-empirical solution was proposed by Massarch and Broms (1981) based on cavity expansion theories but including the effects of overconsolidation and sensitivity by using Skempton's pore pressure parameter at failure (A_f). Charts illustrating this approach are given in Fig. 4. Approximate values for A_f can be estimated from the table in Fig. 4. Clearly a knowledge of the shear modulus (G) or plasticity index (PI) would assist in the estimate of S_u. The addition of shear wave velocity measurements during seismic CPTU is a promising method to obtain an independent and economic measure of the shear modulus (Robertson et al, 1986).

If pore pressures are measured immediately behind the cone tip, the measured values may not have reached the true cylindrical cavity expansion value. Therefore S_u estimated from the chart with the pore pressures behind the tip may be slightly over estimated. Also because of the tendency for low or negative pore pressures measured behind the tip in insensitive, overconsolidated clays (see Fig. 1), the chart in Fig. 4 is not recommended for moderately highly overconsolidated clays ($-0.5 < A_f < 0$).

Although the charts in Fig. 4 are based on cavity expansion theories, they are basically semi-empirical in nature. The advantage in using the charts is that they provide some rational guide to the initial selection of the cone factor, $N_{\Delta u}$. The charts clearly show how the factor $N_{\Delta u}$ will vary with OCR, sensitivity and stiffness, but should always be adjusted to fit local experience.

The data in Fig. 4 show that no simple unique relationship exists between CPTU data and undrained shear strength, S_u, for all clay type soils. However, simple relationships are possible for site specific soils where sensitivity, stiffness and stress history may not be significant variables.

Several methods have been suggested to correlate pore pressure parameters, such as B_q, to OCR. However, any relationship between pore pressure and OCR would be influenced by variations in soil stiffness and sensitivity since the excess pore pressure is also a function of stiffness ratio and sensitivity.

Fig. 5 presents the pore pressure parameter B_q against the best estimate of the in situ overconsolidation ratio (OCR) for soils from four sites near Vancouver, B.C. Details of the soils are given by Robertson et al (1986). Generally, the OCR has been estimated from relationships between undrained shear strength, S_u, plasticity index, PI, and OCR. Admittedly, there are significant uncertainties in the values of OCR determined this way. However, the data shown in Fig. 5 show that B_q generally decreases with increasing OCR. Also for the same OCR, B_q increases with increasing sensitivity, S_t. The trends and variation in data shown in Fig. 5 could have also been predicted using the chart in Fig. 4a.

The charts and data presented in Figs. 4 and 5 illustrate how the pore pressures during cone penetration are influenced by the combined effects of soil strength, stress history, stiffness and sensitivity. Therefore, no unique global relationship exists between the pore pressure measured at one location on the cone and any one of the above soil variables, such as undrained shear strength or OCR. However, for a given geologic formation of, say, normally consolidated clay, the undrained shear strength will directly correlate with the measured pore pressure at a given location on the cone.

A review of Fig. 1 illustrates the potential of using the difference in pore pressures measured simultaneously at two different locations on the cones to

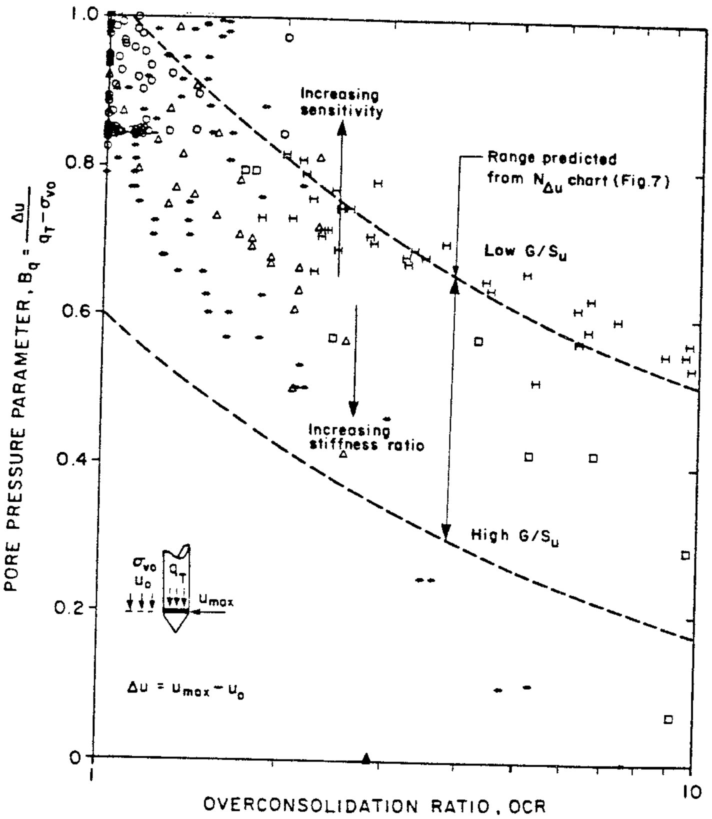


Fig. 5. Pore pressure parameter B_q versus overconsolidation ratio, OCR.

relate to paramaters such as OCR. In overconsolidated clays the pore pressures tend to be large on the face of the cone and small behind the tip. This is illustrated by the large difference in pore pressures measured between the face and behind the tip for the overconsolidated clay in Fig. 2.

The difference between pore pressures measured on the face and behind the tip also appears to be related to the dilatancy of sandy soils, as shown in Fig. 1. The data presented in Fig. 2 illustrates the pore pressure response in a silty sand deposit from a depth of 2.5 m to 6.5 m. At a depth of about 5 m the silty sand is dense and the difference between the pore pressures on the face and behind the tip are largest. At a depth of about 3 m the silty sand is loose and the difference between the pore pressure measurements is small. The pore pressure measured behind the tip at a depth of 3 m is less than u_o, although the silty sand is loose. This is most likely due to the very low overburden stress that exists at a depth of 3 m (i.e., $\sigma_{vo} \approx 30$ kPa). Even loose sands can show a dilatant behavior at very low confining stress.

Quantitative interpretation of the pore pressures in silty sand and sandy silt deposits is made difficult because of possible partial drainage. Research is underway at UBC to evaluate techniques of quantifying the pore pressure response in sandy soils related to liquefaction assessment.

CONE DESIGN

The extreme sensitivity of pore pressure measurements to the filter element location has already been discussed. For any given location, there are several other factors that influence the reliability and accuracy of the pore pressure results. These factors relate to design features of the cone.

The mechanical design of the cone must ensure that when the cone tip is stressed, no load is transferred to the pore pressure transducer, porous element or fluid volume. This problem can be checked by loading the tip of a fully assembled, saturated piezometer cone and observing the pore pressure response. If no mechanical transfer of load is taking place, there should be no pore pressure response.

Measuring dynamic pore pressures with the piezometer cone requires careful consideration of probe design, choice of the porous element and probe saturation. For a high frequency response (i.e., low response time), the design must aim at a small fluid filled cavity, low compressibility and viscosity of fluid, a high permeability of the porous filter and a large area to wall thickness ratio of the filter (Smits, 1982). To measure dynamic pore pressures rather than filter compression effects, the filter should be rigid. However, to maintain saturation, the filter should have a high air entry resistance, which requires a finely graded filter and/or high viscosity of the fluid. Clearly, not all of these requirements can be combined.

An essential requirement is to incorporate a small fluid cavity and a low compressibility of saturating fluid. A compromise is required between a high permeability of the porous filter to maintain a fast response time and a low permeability to have a high air entry resistance to maintain saturation. Good results have been obtained at UBC using glycerin as a saturating fluid and a porous plastic (hydrophilic polypropylene) filter with a nominal particle size of 120 microns before fusion. The permeability of the filter is approximately 0.01 cm/sec. The function of the filter is to allow rapid movements of the extremely small volumes of water needed to activate the presure sensor while preventing soil ingress or blockage.

Filter element squeeze can also be important for cones that measure pore pressures at the tip or on the face of the tip. During penetration into a dense layer with high cone tip resistance, the filter element can become compressed and generate high positive pore pressures. This will occur unless the filter element has a very low compressibility or if filter and soil are of sufficient permeability to rapidly dissipate the pore pressure due to filter element compression. Experience at UBC with a relatively compressible porous plastic filter element has shown no evidence of induced pore pressure due to filter squeeze. This is likely due to the high permeability of the porous plastic element. In a recent field comparison study between porous polypropylene and ceramic filters there were no significant differences in dynamic pore pressures. Filter squeeze is mainly critical for pore pressure measurements on the tip during initial penetration into dense fine, silty sands and compact glacial silts.

It has previously been shown by several researchers (Campanella and Robertson, 1981) that complete saturation of the piezometer tip is essential. Pore pressure response can be inaccurate and sluggish for non-saturated piezometer systems. Both the maximum pore pressure and dissipation times can be seriously affected by air entrapment. Response to dynamic pore pressure can be significantly affected by entrapped air within the sensing element. This is particularly true for soft low permeability soils (Acar, 1981). Experience at UBC has shown that glycerin has worked effectively as a saturating fluid which is miscible with water, yet develops a high air entry tension to prevent loss of saturation during use and penetration through soils above the water table.

Glycerin has the advantage of being half as compressible as water, yet completely miscible with water. Pure glycerin has boiling and freezing points at 290°C and -17°C and a viscosity about 50% larger than water. The glycerin will probably diffuse into the water during the sounding but its effectiveness will be most important during the early part of the sounding above the water level or for shallow penetration below the water level where saturation is most important.

Unfortunately, it is not possible to check saturation before penetrating the soil. The need for a laboratory calibration of piezocone system compliance is questionable because even a small amount of entrapped air in the field can drastically increase the system compliance. Thus, it is difficult to assess if a laboratory calibration is realistic under field conditions.

Saturation Procedures

Saturation procedures generally consist of the following operations:

1) Deairing of filter stones.
2) Deairing of cone, especially with respect to the pressure chamber immediately adjacent to the pressure transducer.
3) Assembling of cone and filter.
4) Protection of system during handling, if required.

In the early days of piezocone sounding, it was normal practice to deair the filter stones and the cone by boiling the complete mantled cone with filter, but this proved to affect seriously the lifetime of the cone. Boiling time was about 2 hours.

General practice is now to either boil the filter stones or to leave them immersed in a suitable fluid in a vacuum chamber for approximately 3 hours until no air bubbles are seen. The practice at UBC has been to place the filter elements in warmed glycerin in a small ultra-sonic bath. A high vacuum is then applied via a sealed cover to the ultra-sonic bath. The combined effect of vibration and vacuum has proven very successful. After several hours vibration, the glycerin increases in temperature which reduces its viscosity and improves saturation. The filter stones are then placed in a small glycerin filled container ready for transportation into the field. Note that glycerin boils at over 200°C at atmospheric pressure which will damage porous plastic.

The voids in the cone itself should be deaired by flushing with a suitable fluid, such as glycerine, from a hypodermic needle. It is suggested that all piezometer cone designs should be made such that flushing the void within the cone tip can be performed with a hypodermic. The UBC cone system is fitted with a temporary funnel to facilitate this procedure (Campanella et al, 1983).

The next step after cone preparation and assembly is the lowering of the string of cone rods. A thin protective rubber sleeve is sometimes placed over the cone. To avoid premature rupture of the rubber sleeve, a small hole is pushed with a "dummy cone" of a larger diameter than the pore pressure cone. Sometimes a hand dug or a predrilled hole is made depending on circumstances and soil-stratigraphy. Experience at UBC has shown that glycerine develops a high air entry tension in the porous stone to prevent loss of saturation during use and penetration through most soils above the water level. Thus, predrilling is not necessary and no special precautions are needed like using a protective rubber or plastic bag over the cone.

It is recommended that the entire saturation procedure be repeated after each sounding, including a change of the filter element.

Techniques for saturating piezometer elements will depend on the individual cone design. The importance of initial complete saturation is reduced somewhat, once significant penetration below the water table has been achieved. The resulting equilibrium water pressure is then often sufficient to put any minor air bubbles into solution. Penetration through saturated sands at shallow depth, however, can produce negative pore water pressures behind the tip which may cause temporary cavitation if the pore pressure drops below about - 100 kPa.

Complete saturation of the internal cavity appears to be at least as important as complete saturation of the porous element. Thus, cone design becomes extremely important with regard to the saturation procedures required to obtain and maintain complete saturation of the internal cavity.

Piezometer Cone Dissipation Test

During a pause in the penetration any excess pore pressures measured on the cone will start to dissipate. The rate of dissipation depends upon the coefficient of consolidation which, in turn, depends on the compressibility and permeability of the soil.

A dissipation test can be performed at any depth. In the dissipation test the rate of dissipation of excess pore pressure to a certain percentage of the equilibrium pore pressure is measured. At the depth at which a dissipation test is needed, the penetration is stopped and sometimes it is required that the rod be clamped to the pushing rig.

Although theoretically this stops the movement of the cone rods instantaneously, in practice the cone will continue to move very slightly as the elastic strain energy in the rods releases and the tip load drops off. The longer the cone rods, and the greater the tendency for the soil to creep, the more significant this movement may be. This movement alters the total stresses in the soil around the conical tip and will influence the measured decay of pore pressure with time. It has been shown (Campanella et al, 1983) that this is only significant with the piezo element in the tip. With the piezo element behind the tip, it has not been necessary to clamp the rods.

Sometimes a fixed period of dissipation for all soil layers is used and sometimes dissipation is continued to a predetermined percentage of the hydrostatic pressure. A common period is t_{50} or the time to record a dissipation of half the excess pore pressure.

The pore pressure is recorded in a time base mode and the measurement of equilibrium pressures provides important hydro-geologic information. For example, equilibrium pore pressures may not be hydrostatic.

Several theoretical solutions are available to obtain the coefficient of consolidation, c_h, from dissipation data. A summary of the solutions was presented by Robertson and Campanella (1983). The theoretical solutions are generally applicable only to soft, normally consolidated clayey soils, where the initial pore pressure distribution around the cone is reasonably well defined. In stiff overconsolidated clayey soils the pore pressure gradient around the cone can be extremely large (see Fig. 1). This gradient of pore pressure often results in dissipations recorded behind the tip that initially increase before decreasing to the final equilibrium value. An example of pore pressure dissipations recorded simultaneously at

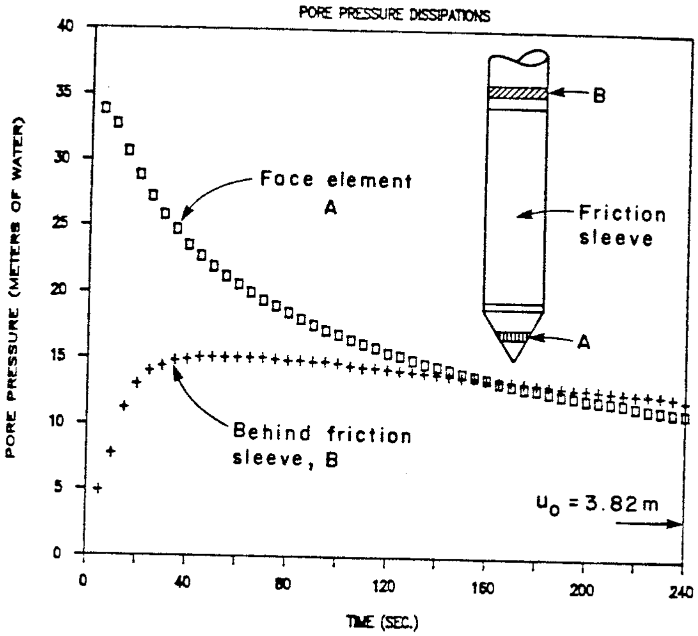


Fig. 6. Pore pressure dissipation at different locations on cone in overconsolidated clay.

two different locations on the cone in an offshore overconsolidated clay at a depth of 3.82 m is shown in Fig. 6. Pore pressures measured on the face of the cone start to dissipate as soon as penetration is stopped. The pore pressure response measured behind the friction sleeve shows an increase followed by a slow decrease. This type of response is believed to be due to the redistribution of excess pore pressures around the cone before the primarily radial drainage.

SUMMARY

Pore pressures measured during cone penetration are influenced by many factors. The primary factor is pore pressure element location. Significantly different pore pressures can be recorded if the sensing element is located in different positions on and behind the cone tip. No single location can provide information for all applications of pore pressure interpretation. Some of the main advantages of locating the pore pressure element behind the tip are; good protection from damage, generally easier to saturate, measured pore pressure dissipations unaffected by procedures and, most importantly, it is the best location to apply pore pressure corrections to cone bearing (Campanella and Robertson, 1981). However, the main disadvantage is the critical influence of element thickness. However, provided the element thickness is at least 5 mm and is located a minimum of 2.5 m behind the tip, experience has shown this problem can be avoided.

Some of the main advantages of locating the pore pressure element on the face of the cone are; best location to record maximum pore pressure and generally better repeatability. However, some of the disadvantages are; easily damaged, can be influenced by element squeeze, dissipations affected by procedure and incorrect pore pressure to apply corrections to cone bearing.

In general, no single location can provide information for all applications of pore pressure interpretation. Therefore, a very convincing argument can be made to standardize the location to provide a wide range of applications but yet maintain a practical location for saturation and protection. However, it would also appear logical that the overall cone design standards should be such that the porous element location can be changed in the field to allow soundings to be carried out to obtain specific pore pressure data. All pore pressure measurements from cone testing must clearly identify the **location** and **size** of the sensing element. Alternatively, a cone could be used with piezometer elements both in the tip and behind the tip. However, saturation of the piezometer element behind the tip can become very difficult unless the cone is designed carefully.

A cone with dual piezometer elements is currently used at UBC. The dual piezometer elements allow pore pressures to be measured simultaneously behind the friction sleeve and either on the cone face or immediately behind the cone tip. (See Fig. 6.)

Pore pressures measured during cone penetration are also influenced by soil type, strength, stress history, sensitivity and stiffness. Guidelines and charts have been presented to illustrate how these variables influence the pore pressures measured on or behind the cone tip.

Pore pressure dissipation data can be extremely useful in providing additional information on the soil type and the drainage conditions during cone penetration.

Cone design, saturation and test procedures also influence the pore pressure response. Good experience has been gained at UBC using a combination of glycerine as the saturation fluid and a porous polypropylene for the filter element. The porous plastic has a high permeability and when combined with the glycerine develops a high air entry tension to prevent loss of saturation during use and penetration through most soils above the water level. It is essential to repeat the entire saturation procedure and change the filter before each sounding.

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