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DETERMINATION OF SOIL PROPERTIES FROM LABORATORY PIEZOCONE PENETRATION TESTS

DETERMINATION DES PROPRIETES DU SOL PAR DES MESURES DE LA PENETRATION PIEZOMETRIQUE AU LABORATOIRE

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SYNOPSIS: Soil properties from miniature piezocone penetration tests on cohesive specimens in a calibration chamber system are presented. Factors such as soil type, stress history, penetration boundary conditions and filter location are considered. An increase in the strength of the soil around the cone penetrometer was observed when penetration was resumed after a dissipation test. The soil type and the horizontal stress had a significant influence on the empirical cone factor and the empirical pore pressure factor used to estimate the undrained shear strength. Interpretation of the dissipation results to evaluate the radial coefficient of consolidation should be based on the initial dissipation values of the excess pore pressure and not the penetration pore pressure. The proposed time factors for the dissipation at the tip and that above the cone base should include the influence of the stress history of the soil.

INTRODUCTION

The use of the electronic piezocone penetrometer as a potential in-situ testing device to evaluate engineering soil properties is gaining wide importance. In order to calibrate and validate theoretical models used to interpret piezocone penetration test (PCPT) results, it is essential to perform accurately controlled field and laboratory tests. Field calibration tests have numerous limitations because of soil inhomogeneity and uncertainties regarding the magnitude of in-situ stresses and stress history of the deposit. Laboratory calibration tests have definite advantages in these respects. Homogeneous, reproducible and instrumented soil specimens, subjected to a known stress history can be prepared and tested under strictly controlled boundary conditions. Various parametric studies can also be conducted. Laboratory calibration chamber tests in cohesive soils are limited in number because of the extremely time consuming and laborious process involved in the preparation of such large specimens (Huang et al. 1988; McManus and Kulhawy 1991; Anderson et al. 1991; Voyiadjis et al. 1991).

This paper presents the interpretation of undrained shear strength and radial coefficient of consolidation from eight miniature piezocone penetration tests on large instrumented cohesive soil specimens in a calibration chamber system.

EXPERIMENTAL STUDY

The clay specimens were prepared in two stages: (1) slurry consolidation in a consolidometer from a high water content soil slurry (Krizek and Sheeran 1970), and (2) reconsolidation in a calibration chamber to higher stresses which is free from the rigid boundary effects of a slurry consolidometer.

Soil slurry was prepared by mixing kaolinite and fine sand (D_{60}/D_{10} = 1.4) with deionized water at a water content of twice the liquid limit. A mixture of 50% kaolin and 50% Edgar fine sand by weight was used to prepare the K-50 specimens (liquid limit, w_L = 30%; plastic limit w_P = 16%). The K-33 specimen (w_L = 20%, w_P = 14%) was prepared from a mixture of 33% kaolin and 67% fine sand. The specimens were normally consolidated to 138 kPa in a period of five weeks. At the end of the first stage of slurry consolidation, the specimen (525 mm in diameter and 812 mm in height) enclosed in the membrane was transferred into the calibration chamber (Figure 1) where it was subjected to a second stage of consolidation to higher stresses.

A summary of the stress history of the specimens is given in Table 1. Reference soil parameters (undrained shear strength, s_u; Skempton's pore pressure parameter, A_f; rigidity index, I_r, which is the ratio of the shear modulus to the undrained shear strength; radial coefficient of consolidation, c_r) from tests conducted on undisturbed samples are given in Table 2.

The Louisiana State University Calibration Chamber System (LSU/CALCHAS) designed by de Lima and Tumay (1991) and Tumay and de Lima (1992) consists of a double walled flexible chamber, the operation of which is servo controlled. The LSU/CALCHAS is capable of simulating the four traditional boundary conditions commonly referred in literature as:

BC1: Constant vertical stress and constant lateral stress
BC2: Zero vertical strain and zero lateral strain
BC3: Constant vertical stress and zero lateral strain
BC4: Zero vertical strain and constant lateral stress

Eight miniature piezocone penetration tests (PCPT1 to PCPT8) were conducted in the soil specimens. The miniature piezocone penetrometer (Figure 2) fabricated by Fugro-McClelland Engineers B.V., The Netherlands, has a projected cone area of 1 cm^2 and a cone apex angle of 60°. Choice is available for the filter located either in the lowest 1/4 of the cone at the very tip (configuration used for tests PCPT2, PCPT4, PCPT6, PCPT8), or starting 0.5 mm above the base of the cone and 2 mm in vertical height (used for tests PCPT1, PCPT3, PCPT5, PCPT7).

TEST RESULTS AND INTERPRETATION

PCPT profiles for tests (PCPT1 to PCPT8) conducted in the specimens are shown in Figures 3 through 6. All tests were conducted under a back-pressure, u_0 of 138 kPa. Dissipation tests were performed at the end of all PCPT. For PCPT2, an additional dissipation test was performed at a depth of 390 mm. An initial increase in the cone resistance was observed when penetration was resumed following the dissipation test. This is probably due to an increase in the strength of the soil due to consolidation around the cone. In spite of the excellent saturation of the piezocone (as indicated by the instant response to the backpressure and the smooth dissipation profiles), it was observed that the penetration depth required to attain a steady pore pressure value (especially at the tip of the cone) was influenced by the stress

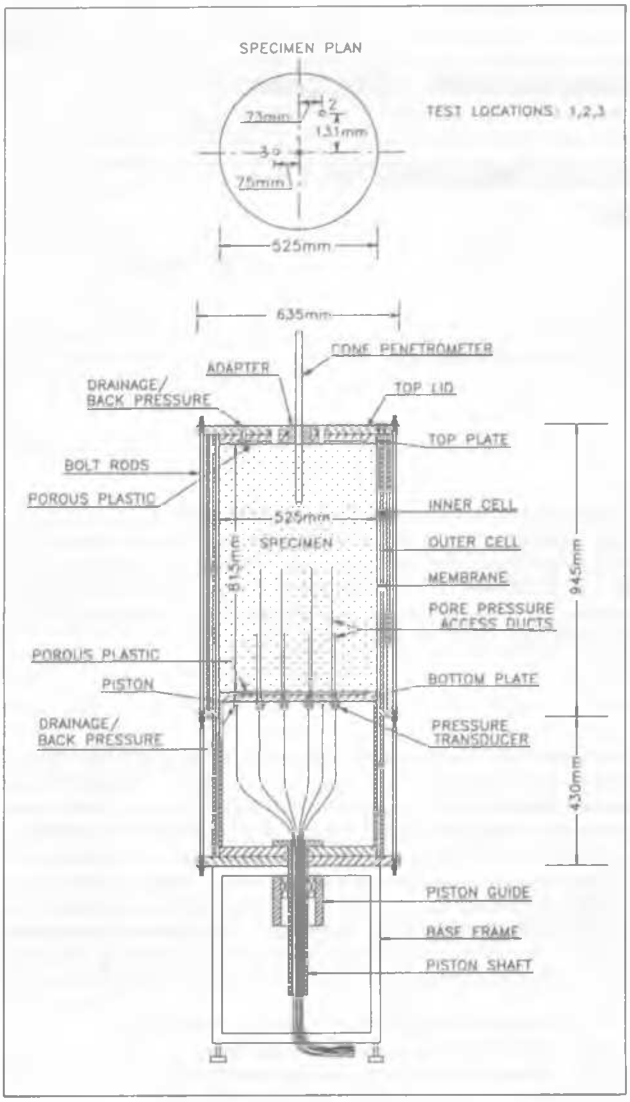

Fig. 1. Schematic of the LSU Calibration Chamber System (LSU/CALCHAS)

Table 1. Summary of the Stress History of the Specimens

Specimen No.	Soil Type	Chamber Consolidation	OCR	Final Effective Stresses (kPa)	
				Vertical	Horizontal
1	K-50	Isotropic	1	207	207
2	K-33	Isotropic	1	207	207
3	K-50	Isotropic	5	41.4	41.4
4	K-50	Anisotropic (K_o)	1	207	107.6

Table 2. Reference Soil Parameters

Specimen No.	Undrained Shear Strength s_u (kPa)	A_f	Rigidity Index I_r	Radial Coefficient of Consolidation (c_r x 10^{-3} cm^2/s)	
				Virgin	Reload
1	60	1.10	267	14.1	78.8
2	80	0.49	100	28.3	141.0
3	50	0.18	150	14.1	78.8
4	65	0.59	567	26.4	105.0

history and the lateral stress coefficient, K_o, which is the ratio of the lateral stress to the vertical stress in the specimen.

The undrained shear strength (s_u) may be estimated from the following empirical relations:

$$s_u = \frac{q_T - \sigma_{vo}}{N_{kT}} \tag{1}$$

$$s_u = \frac{\Delta u}{N_{\Delta u}} \tag{2}$$

where N_{kT} = empirical cone factor, σ_{vo} = total vertical stress, q_T = cone resistance corrected for pore pressure effects, Δu = excess pore pressure, $N_{\Delta u}$ = empirical pore pressure factor (from cavity expansion theories). The values of N_{kT} and $N_{\Delta u}$ computed from PCPT data and using the reference s_u are given in Table 3. The N_{kT} and $N_{\Delta u}$ values in the isotropically consolidated specimens were higher than those for the anisotropically consolidated specimen (specimen no. 4), signifying the importance of the horizontal stress. The N_{kT} and $N_{\Delta u}$ values in specimen no. 2 were lower than those in specimen no. 1, indicating the influence of the soil type (plasticity index).

A sudden drop in the excess pore pressure was observed in all the tests, as soon as penetration ceased. This drop was higher for pore pressures recorded at the tip than those recorded just above the base of the cone and is due to the normal stress reduction that occurs when penetration ceases. Hence, the interpretation of the dissipation results for the radial coefficient of consolidation, c_r, should be based on the initial dissipation values of the excess pore pressure, Δu_i (and not the penetration pore pressure). The radial coefficient of consolidation can be predicted from PCPT dissipation test results using the following expression:

$$c_r = \frac{T_{50}\, r_0^2}{t_{50}} \tag{3}$$

where T_{50} = time factor at 50% dissipation, t_{50} = time for 50% dissipation, and r_0 = penetrometer radius. Figures 7 through 10 show a comparison of the dissipation test results in the four specimens with those predicted by Levadoux and Baligh (1986), and Houlsby and Teh (1988). It can be seen that the method proposed by Levadoux and Baligh does not indicate significant difference in the time factors, T, for dissipation at the tip and above the cone base; whereas the method proposed by Houlsby and Teh overestimates the difference in the time factors. The method proposed by Houlsby and Teh takes into account the effect of the rigidity index. However, the influence of the stress history of the soil on the time factors at the tip and above the cone base is missing in both methods. Comparison between the reference c_r and the estimated c_r (using some of the existing methods) are shown in Table 4. A reasonable prediction of c_r was obtained for the specimens tested.

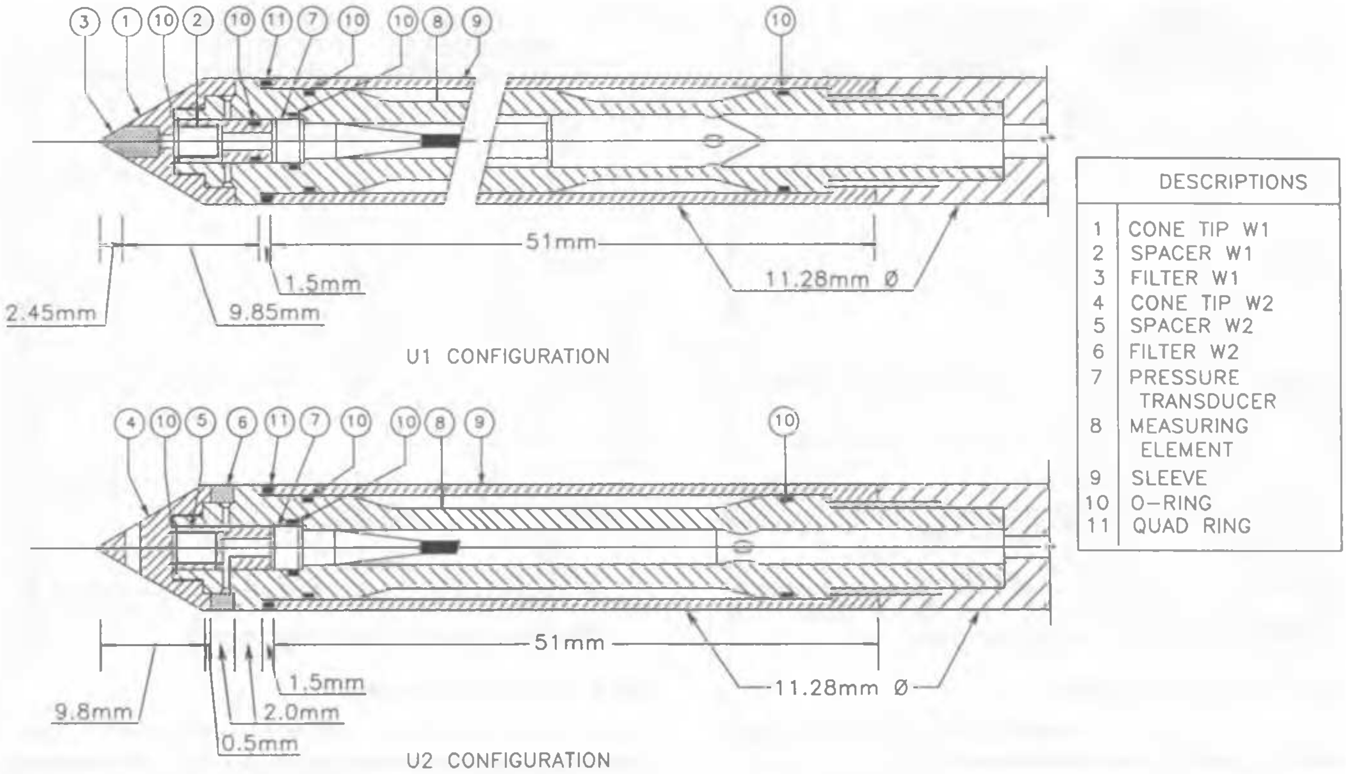

Fig. 2. Schematic of the miniature piezocone penetrometer

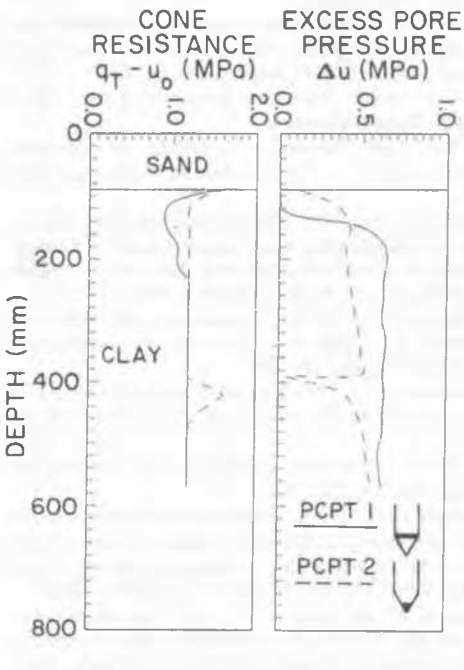

Fig. 3. PCPT profiles in specimen 1

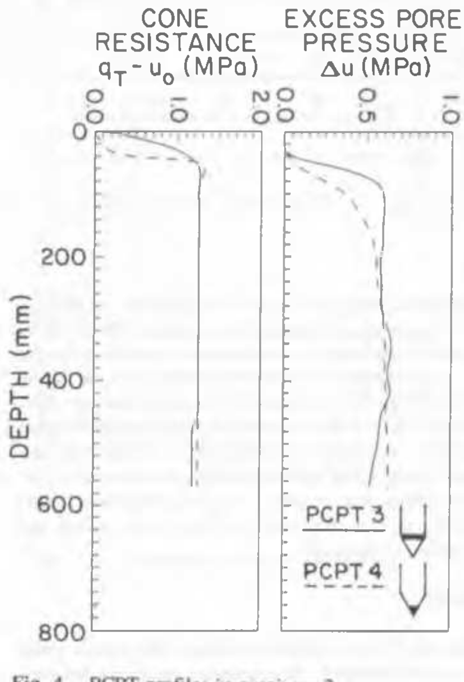

Fig. 4. PCPT profiles in specimen 2

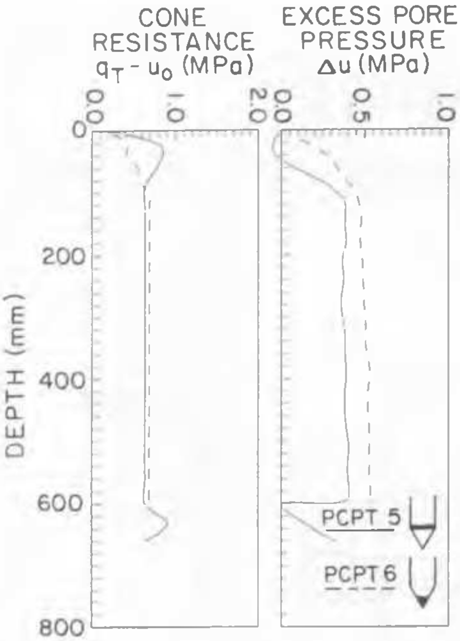

Fig. 5. PCPT profiles in specimen 3

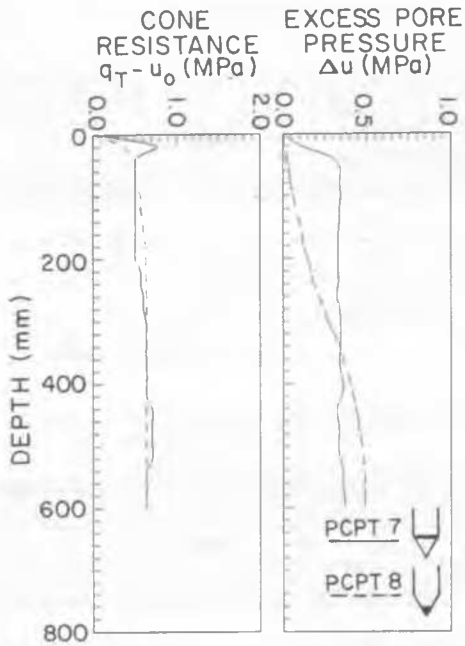

Fig. 6. PCPT profiles in specimen 4

Table 3. N_{kT} and $N_{\Delta u}$ Values Obtained from PCPT

Specimen No.	Test	$N_{kT} = \frac{q_T - \sigma_{vo}}{s_u}$	$N_{\Delta u} = \frac{\Delta u}{s_u}$
1	PCPT1	16	10.4
	PCPT2	16	9.4
2	PCPT3	13	7.4
	PCPT4	13	7.9
3	PCPT5	15	10.1
	PCPT6	16	13.2
4	PCPT7	7	5.6
	PCPT8	7	7.5

CONCLUSION

The soil type and the horizontal stress had a significant influence on the N_{kT} and $N_{\Delta u}$ values used to estimate the undrained shear strength. The penetration depth required to attain a steady pore pressure value, especially for the filter located at the tip, was influenced by the stress history and the lateral stress coefficient, K_o There is an initial increase in the cone resistance when penetration is resumed following a dissipation test because of the increase in strength due to consolidation. Interpretation of the radial coefficient of consolidation should be based on the initial dissipation values of the excess pore pressure and not the penetration pore pressure. The proposed time factors for the dissipation at the tip and that above the cone base should include the influence of the stress history of the soil.

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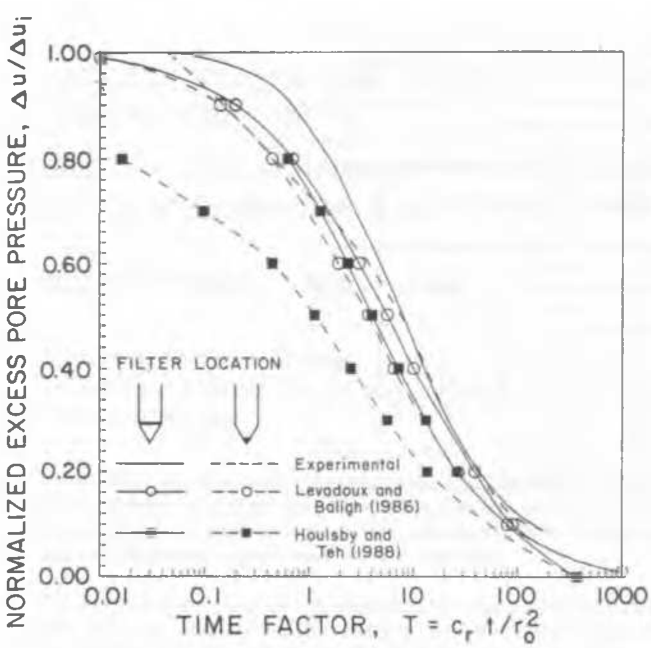

Fig. 7. Dissipation results in specimen 1

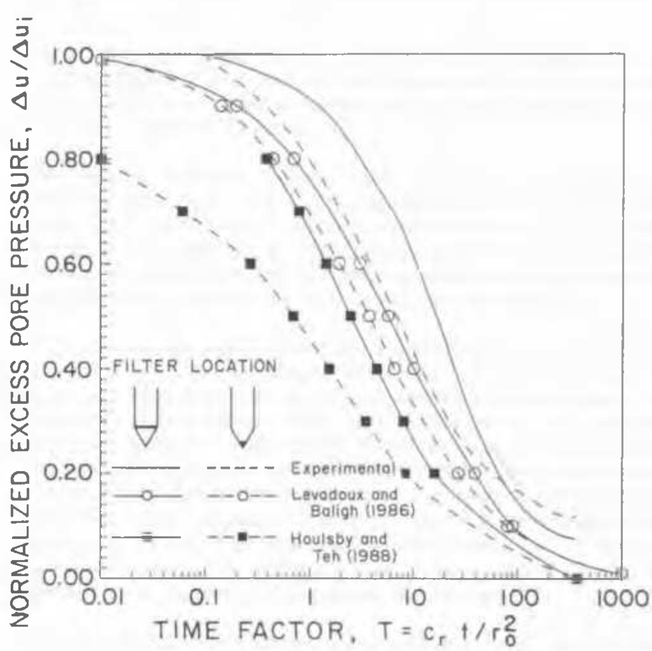

Fig. 8. Dissipation results in specimen 2

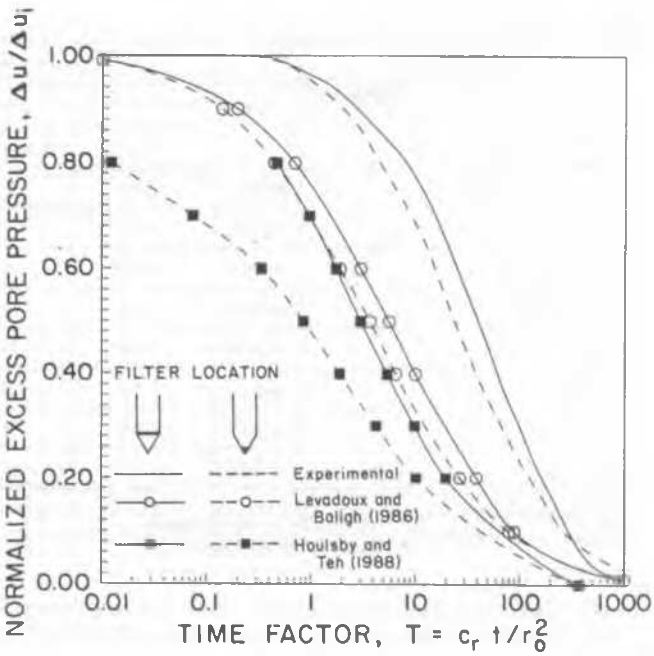

Fig. 9. Dissipation results in specimen 3

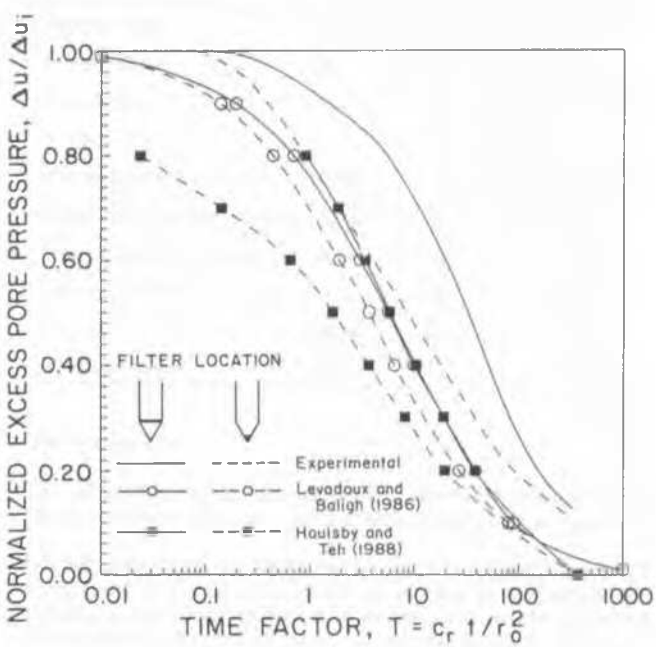

Fig. 10. Dissipation results in specimen 4

Table 4. Comparison between the Estimated and Reference c_r Values at 50% Dissipation Level

Specimen No.	Test	Estimated ($c_r \times 10^{-3}$ cm^2/sec)			Reference ($c_r \times 10^{-3}$ cm^2/sec)	
		Torstensson (1977)	Levadoux and Baligh (1986)	Houlsby and Teh (1988)	Virgin	Reload
1	PCPT1	10.89^c	10.01	7.16	14.1	78.8
	PCPT2	5.82^s	7.18	2.18		
2	PCPT3	4.86^c	9.9	4.33	28.3	141.0
	PCPT4	7.51^s	15.7	2.93		
3	PCPT5	7.68^c	11.14	5.97	14.1	78.8
	PCPT6	7.70^s	12.66	2.89		
4	PCPT7	9.09^c	5.09	5.31	26.4	105.0
	PCPT8	28.65^s	11.78	5.23		

c - cylindrical cavity expansion
s - spherical cavity expansion