# INTERPRETATION OF PIEZOCONE DATA AFTER PARTIALLY DRAINED PENETRATION

by

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SYNOPSIS Piezocone has been generally accepted as a very effective tool in soil profiling and estimating engineering properties. However, most of the available correlations are only applicable for fully drained or totally undrained penetration. This paper presents a generalized method to interprete the pore pressure dissipation data after partially drained piezocone penetration in silty clay deposits in Taipei. The estimated permeability and coefficient of consolidation give a satisfactory agreement with relevant laboratory and field measurements.

## INTRODUCTION

Drainage condition during piezocone penetration depends on the magnitude of the soil permeability relative to the steady penetration rate of the cone. Presently the penetration rate has been standardized at 2 cm/sec. This rate ensures that minimal pore water migration occurs during penetration in very fine-grained soils of low permeability ($k=10^{-7}\sim10^{-10}$ cm/sec), i.e. clayey soils are sheared in an undrained mode. On the other hand, in coarse-grained soils with high permeability ($k=10^{-1}\sim10^{-4}$ cm/sec), almost no excess pore water pressure is generated and the penetration is a fully drained shearing process. For silty soils with intermediate values of permeability ($k=10^{-4}\sim10^{-7}$ cm/sec), partial dissipation of excess pore pressures can occur during penetration. Situations of partial drainage are generally very difficult to analyze and can involve significant uncertainties in the interpretation of field measurements (Campanella et al, 1981).

A procedure to determine the permeability and consolidation characteristics of the silty soil deposits is required for many excavation works in Taipei, such as the construction of Taipei Rapid Transit Systems. For example, coefficient of consolidation is required to determine the degree of swelling of the soil deposits below the excavation surface which is critical to the lateral wall deflection and the settlement of adjacent structures. In addition, the required penetration depth of the diaphragm wall is generally controlled by groundwater blow-in rather than toe stability, i.e. the wall length is determined by the cut-off length needed to penetrate into the lower less permeable soil layer. Since the depth to that lower layer is very variable, the length of each panel is not fixed during design stage and has to be determined at site before the construction of the diaphragm wall. Piezocone has been proposed by many design consultants to replace the conventional time-consuming and more expensive drilling and sampling practice to determine the termination depth of the wall. For quality control purpose, owners need a procedure to quantitatively determine the permeability of the deposits when piezocone is used.

This paper presents a procedure, on the basis of the Strain Path Method, to determine the permeability and consolidation characteristics of the silty clay deposits in Taipei. The comparisons between laboratory test results and interpreted field measurements indicate reasonably good agreements.

## BACKGROUND

An integrated and systematic framework for interpreting piezocone measurements has been established using the Strain Path Method (Baligh, 1985) which has been developed at the Massachusetts Institute of Technology by Baligh and his co-workers(e.g. Levadoux, 1980; and Elghaib, 1989). A rational interpretation method has been proposed to estimate the in situ consolidation and permeability characteristics of clays from pore pressure dissipation measurements after piezocone penetration (Baligh and Levadoux, 1980). The method is based on linear consolidation analyses and initial pore pressure distributions calculated by Strain Path Method for undrained penetration in Boston Blue Clay. Backfigured values of the coefficient of consolidation and permeability are in reasonable agreement with laboratory and field performance data. This method has now been generally accepted in interpreting piezocone dissipation tests after undrained penetration (Jamiolkowski et al., 1985).

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Based on the Strain Path Method approach, Elghaib (1989) presented his thesis showing that the piezocone measurements can be predicted over the full range of drainage conditions including the case of partial drainage. Using the results presented by Baligh and Levadoux (1980) and the analogy between partial drainage during penetration and dissipation after totally undrained penetration, a recommended method was proposed which requires an iteration procedure to determine the coefficient of consolidation (c) and the degree of partial drainage (Mp) where Mp is defined as the ratio of the excess pore pressure measurement to the excess pore pressure that would have been generated for fully undrained penetration. It should be noted that the parameter Mp is dependent on the location of the pore pressure measured. Interpretation method of the piezocone tip measurement was presented in Elghaib (1989). However there is no field data included in Elghaib's research to verify the proposed interpretation method. Required normalized curve to interprete dissipation test results for porous element located at shoulder of the cone has not been presented by Elghaib at that stage.

## RECOMMENDED PROCEDURES

The soil deposits in Taipei Basin can be generally described as six alternating cohesive and cohesionless soil layers. Table 1 shows a typical soil profile in downtown Taipei, or so-called T2 Area (MAA, 1987). One of the most important features of all these six layers are the high percentage of silt-size particles. Of particular interests are the silty clay deposits of Layer IV and Layer II since suffits of the deep excavations with a depth between 15 and 25m are almost all located in Layer IV, and if cut-off is required then the lower, less permeable layer would be Layer II. Recent study (Hwang, 1992) indicates that the field and laboratory permeability measurements of Layer IV and Layer II varies from 2 x $10^{-8}$ to 9 x $10^{-4}$ cm/sec. It does not only indicate a significant scatter of permeability measurements, but also implies that the drainage condition during cone penetration may neither be fully drained nor totally undrained. Thus a more generalized method is required to interprete piezocone dissipation test results in Layer IV and Layer II soils in Taipei. It is noted that most of the piezocones used in Taipei have their porous elements located at the shoulder of the cone. The authors derived normalized curves ( Figs. 1 and 2 ) to interprete pore pressure dissipation data obtained at the shoulder of the cone under any degree of partial drainage with various rates of cone penetration. They were derived by exactly the same rationale as that used by Elghaib (1989) on the basis of the standard curve proposed by Baligh and Levadoux (1980). The interpretation method, as suggested by Elghaib (1989), can be summarized as follows :

TABLE 1 TYPICAL SOIL PROPERTIES IN DOWNTOWN TAIPEI

| FORMATION | THICKNESS m | DRY UNIT WEIGHT t/m$^3$ | WATER CONTENT % | LIQUID LIMIT % | PLASTICITY INDEX % | PARTICLE SIZE DISTRIBUTION % | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | GRAVEL | SAND | SILT | CLAY(<5μ) |
| TOP SOIL (CL,CL/ML) | 1.6 | 1.68 | 26.6 | 33.3 | 11.2 | 0 | 34 | 31 | 25 |
| LAYER VI (CL/ML) | 4.5 | 1.45 | 31.2 | 35.8 | 12.9 | 0 | 10 | 58 | 32 |
| LAYER V (SM) | 10.1 | 1.54 | 26.3 | - | NP | 1 | 75 | 19 | 5 |
| LAYER IV (CL/ML) | 9.0 | 1.43 | 32.1 | 34.3 | 12.0 | 0 | 8 | 61 | 31 |
| LAYER III (SM) | 10.6 | 1.61 | 23.9 | - | NP | 0 | 60 | 34 | 6 |
| LAYER II (CL/ML) | 8.4 | 1.55 | 27.2 | 30.3 | 9.2 | 0 | 9 | 67 | 24 |
| LAYER I (SM) | 4.5 | 1.70 | 20.3 | - | NP | 1 | 62 | 30 | 7 |

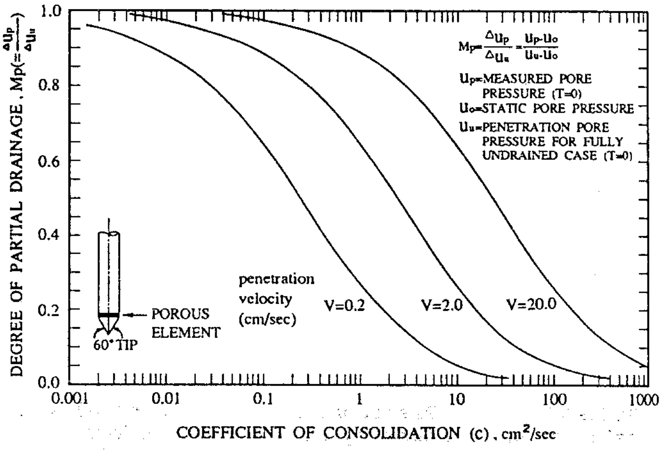

FIG. 1 PREDICTION OF PARTIAL DRAINAGE EFFECTS ON EXCESS PORE PRESSURE

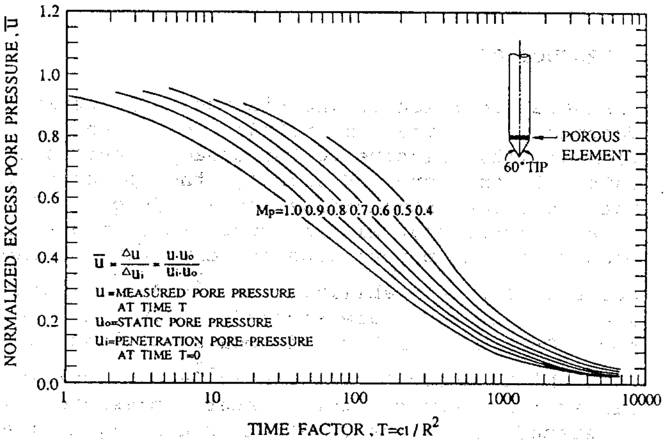

FIG. 2 EXCESS PORE PRESSURE DISSIPATION AFTER PARTIALLY DRAINED PENETRATION

(1) Select the penetration speed, V, used during penetration.
(2) Assume a value for the degree of partial drainage Mp (may start with the iteration process assuming undrained penetration; i.e. Mp=1).

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(3) Use Fig. 2 to determine the time factor T corresponding to a given degree of consolidation (for example, $\bar{U}=0.5$ i.e. 50% consolidation).
(4) From dissipation data, determine the actual time, t, required to reach $\bar{U}=0.5$ after penetration.
(5) Compute the coefficient of consolidation $c=TR^2/t$ where R is the radius of the cone.
(6) Enter Fig. 1 with the computed value of c and the known penetration speed V, get an updated value of Mp and compare it to the assumed value in step 2.
(7) Use the updated value of Mp and repeat steps 2 through 6 until consistency between the updated and assumed value of Mp is obtained.

Once the coefficient of consolidation is obtained, the coefficient of permeability can be calculated following Baligh and Levadoux's approach (1980).

## VERIFICATION OF THE METHOD

The test site of this study is located in downtown Taipei. The soil deposits are generally characterized as T2 soils. In order to make some direct comparisons with laboratory test results, to reconfirm the repeatability of dissipation data and to verify the measured steady state water pressure, two piezocone probings with associated dissipation tests were carried out just 5 m away from the borehole. Figure 2A presents field measurements during piezocone penetration at the test site. A boring log at site is also presented in Table 2 for comparison. Continuous stationary piston tube samples were taken for laboratory tests and standpipe-type piezometers were installed at selected depths. A careful comparison between two penetration results indicate that the cone penetration test results are quite repeatable. In addition, the stratigraphy and classification (based on tip resitance and friction ratio) obtained from piezocone are in general agreement with the boring log. Figure 3 shows the normalized dissipation curves after piezocone penetration at depths of 21m and 21.25m. In addition to the abovementioned special testing program, many laboratory tests and piezocone test results in T2 area are collected for the evaluation of the proposed interpretation method.

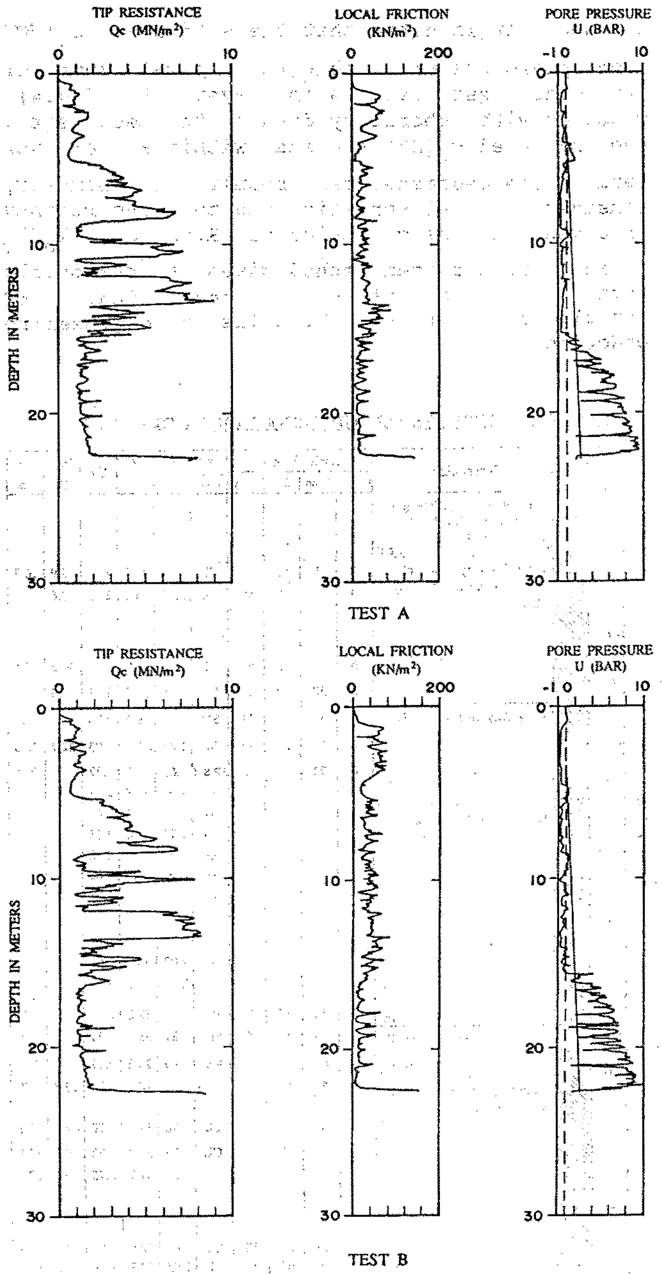

FIG. 2A FIELD MEASUREMENTS DURING PIEZOCONE PENETRATION

Figure 4 compares the coefficient of consolidation from piezocone measurements with relevant laboratory data of Layer IV soils. The measured c(probe), is regarded as the coefficient of horizontal consolidation in OC range. In order to calculate $c_v$(NC), the ratios of $k_h/k_v$ and CR/RR are required, in which $k_h$ is the horizontal permeability, $k_v$ is the vertical permeability, CR is the compression ratio and RR is the recompression ratio. The $k_h/k_v$ ratio used in this study is 1.5, on the basis of recent summary of laboratory permeability tests on vertically and horizontally trimmed samples. The ratio CR/RR varies from 6 to 10 depending upon the location of test site. The Mp value varies from 0.82 to 0.94. According to Elghaib (1989), dissipation with Mp varying between 0.95 and 0.05 should be regarded as partial drainage condition. Based on the proposed method, the estimated $c_v$(NC) from piezocone varies from $4.3 \times 10^{-3}$ cm$^2$/sec to $1.2 \times 10^{-2}$

cm$^2$/sec. It is noted that the estimated $c_v$(NC) from piezocone are within the typical range of oedometer test results in Layer IV (Fig.4). Compared with laboratory data of the test site, the estimated $c_v$(NC) are even within a narrower band of the oedometer test results. In summary, considering areal variation and the uncertainty involved in the selection of CR/RR and $k_h/k_v$ ratio, the proposed method gives a reasonably good estimate of the coefficient of consolidation of Layer IV soils in engineering practice.

TABLE 2 RESULTS OF SUBSURFACE EXPLORATION

| DEPTH IN METERS | LOG | Description | Grain Size% |  |  |  | USCS Classification | $\gamma_t$ t/m$^3$ | $W_n$ % | Gs | $W_L$ % | $I_p$ % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | Gravel | Sand | Silt | Clay |  |  |  |  |  |  |
|  | SF | Fill ; brownish yellow gray sandy clay, little brick and gravel. ( SF )<br>1.8m |  |  |  |  |  |  |  |  |  |  |
| 2 |  | Brownish gray sandy clay, trace organic occasionally. (CL) | 0 | 3 | 49 | 48 | CL | 1.97 | 33.1 | 2.72 | 38.9 | 16.7 |
|  |  |  | 0 | 2 | 49 | 49 | CL | 2.06 | 36.2 | 2.73 | 38.6 | 16.2 |
| 4 |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  | 0 | 1 | 42 | 57 | CL | 1.80 | 42.0 | 2.73 | 44.2 | 20.4 |
|  |  | 5.8m | 0 | 30 | 59 | 11 | CL | 1.96 | 30.9 | 2.71 | 25.3 | 9.30 |
| 6 |  | Gray silty fine sand. (SM) | 0 | 56 | 33 | 11 | SM | 2.29 | 19.7 | 2.70 |  | NP |
|  |  |  | 0 | 54 | 21 | 25 | SC | 1.88 | 33.7 | 2.73 | 22.7 | 7.60 |
|  |  |  | 0 | 94 | 5 | 1 | SP-SM | 1.68 | 17.6 | 2.72 |  | NP |
| 8 |  | 8.6m |  |  |  |  | — | 2.06 | 29.9 | 2.72 |  |  |
|  |  | Gray sandy silt. (ML) |  |  |  |  | — | 1.93 | 41.0 | 2.68 |  |  |
|  |  |  |  |  |  |  | — | 1.93 | 41.0 | 2.68 |  |  |
| 10 |  |  |  |  |  |  | — | 1.95 | 33.4 | 2.72 |  |  |
|  |  |  |  |  |  |  | — | 2.06 | 29.3 | 2.68 |  |  |
|  |  |  |  |  |  |  | — | 2.06 | 29.3 | 2.68 |  |  |
| 12 |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  | — | 2.02 | 30.4 | 2.67 |  |  |
|  |  |  |  |  |  |  | — | 2.09 | 28.6 | 2.72 |  |  |
| 14 |  | 14.7m | 0 | 42 | 47 | 11 | ML | 1.89 | 32.0 | 2.72 |  | NP |
|  |  | Gray silty fine sand. (SM) | 0 | 76 | 19 | 5 | SM | 1.87 | 35.7 | 2.71 |  | NP |
| 16 |  | 16.1m | 0 | 88 | 12 | 0 | SP-SM | 1.89 | 23.2 | 2.72 |  | NP |
|  |  | Gray silty clay, trace organic and shell. (CL) | 0 | 42 | 51 | 7 | CL-ML | 1.84 | 28.2 | 2.71 | 23.8 | 5.30 |
|  |  |  | 0 | 2 | 58 | 40 | CL | 1.90 | 34.0 | 2.70 | 42.5 | 20.8 |
| 18 |  |  | 0 | 2 | 68 | 30 | CL | 1.83 | 37.9 | 2.73 | 39.8 | 19.9 |
|  |  |  | 0 | 1 | 61 | 38 | CL | 1.82 | 37.9 | 2.75 | 40.9 | 19.2 |
| 20 |  |  |  |  |  |  |  |  |  |  |  |  |
|  |  |  | 0 | 15 | 67 | 18 | CL | 1.86 | 32.8 | 2.67 | 38.1 | 12.8 |
|  |  | 21.7m | 0 | 7 | 62 | 31 | CL | 1.95 | 30.2 | 2.69 | 35.8 | 11.9 |
| 22 |  | Gray silty fine sand. (SM) |  |  |  |  |  |  |  |  |  |  |
| 24 |  | 25.0m |  |  |  |  |  |  |  |  |  |  |

Figure 5 shows the comparison between estimated permeability from piezocone with relevant measurements in Layer IV. The value of RR used in estimating k from piezocone measurement varies from 0.014 to 0.024 depending on the test site location. It is reported that the field permeability measurement of Layer IV soils varies significantly from 9.6 x 10$^{-7}$ to 9.1 x 10$^{-4}$ cm/sec with a mean value of

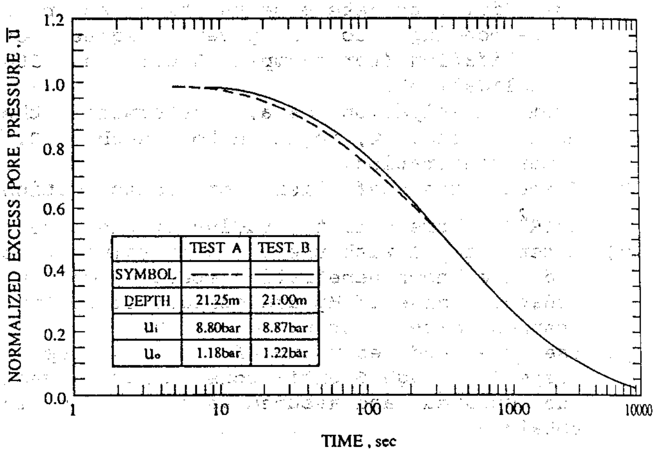

FIG. 3 NORMALIZED DISSIPATION CURVES AFTER PIEZOCONE PENETRATION

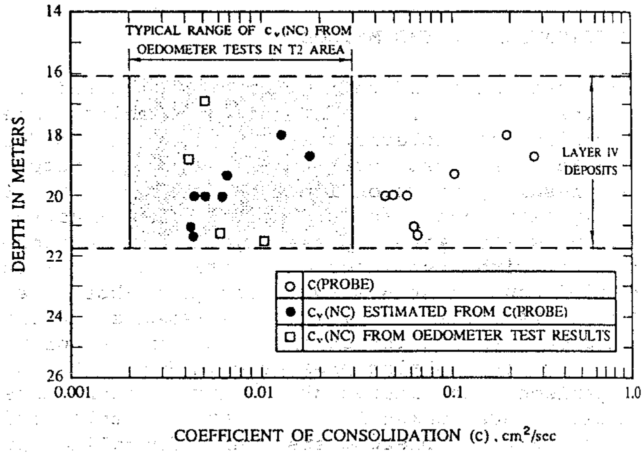

FIG. 4 COMPARISON OF PREDICTED AND MEASURED COEFFICIENTS OF CONSOLIDATION IN TAIPEI

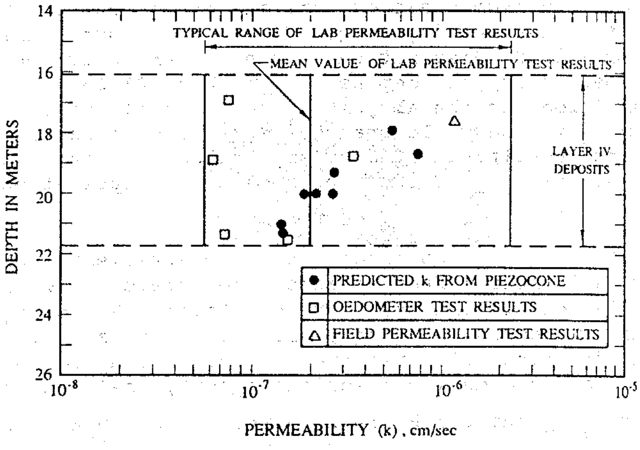

FIG. 5 COMPARISON OF PREDICTED AND MEASURED COEFFICIENTS OF PERMEABILITY IN TAIPEI

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$3.8 \times 10^{-5}$ cm/sec (Hwang, 1992) which is about 2 orders of magnitude higher than the mean value of laboratory test results. Figure 5 indicates that the interpreted k values from piezocone are lower than the field permeability test result. On the other hand, the predicted values are within the typical range of laboratory test results. The variation of predicted values are within a factor of about 5. In spite of the uncertainties involved in selecting RR and simplifications needed to estimate $k_h$ from piezocone data, the recommended procedure yields a reasonable prediction. This recommended procedure become even more valuable if compared with results of other existing in situ permeability testing methods which are always more expensive, time-consuming, and difficult to interprete.

Preliminary evaluation of Layer II data gives similar conclusions on the estimated c and k. More laboratory data are required for further verification.

## CONCLUSIONS

For silty clay deposits in Taipei, partial drainage of excess pore pressure can take place during piezocone penetration. This paper presents a generalized method to use dissipation data to estimate permeability and consolidation characteristics of soil deposits with varying degree of drainage at different speed of penetration.

It has to be noted that the existing laboratory and field testing methods generally give very wide range of estimates of permeability and consolidation characteristics of fine-grained soils. Based on the proposed approach, the backfigured values of permeability and coefficient of consolidation in Layer IV silty clay deposits in Taipei are consistently within relatively narrow ranges. Further evaluation of the recommended interpretation method was achieved by means of comparisons with laboratory measurements and field performance data. In spite of the simplications and assumptions required to estimate the coefficient of consolidation from dissipation data, the estimated coefficients of consolidation in NC range are in a reasonably good agreement with oedometer test results. The backfigured values of permeability are also considered quite satisfactory especially when compared with other existing methods. It can thus be concluded that the proposed method be accepted for engineering practical use.

## REFERENCES

Baligh, M.M. (1985). Strain Path Method. ASCE J. of Geot. Engrg. Div., III(9), pp.1108-1136.

Baligh, M.M. and Levadoux, J.N. (1980). Pore Pressure Dissipation After Cone Penetration. Research Report R80-11, Dept. of Civil Engrg., MIT, Cambridge, MA.

Campanella, R.G., Robertson, P.K., and Gillespie, D. (1981). In Situ Testing in Saturated Silt (Drained or Undrained ?). Vol. 1, Proc., 34th Canadian Geot. Conf.

Elghaib, M.K. (1989). Prediction and Interpretation of Piezocone Data during Undrained, Drained and Partially Drained Penetration. Ph.D. Thesis, Dept. of Civil Engrg., MIT, Cambridge, MA.

Hwang, R. (1992). Permeability of Taipei Soils. Unpublished Internal Research Report, Moh and Associate, Inc.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T. and Lancellota, R. (1985). New Developments in Field and Laboratory Testing of Soils. State-of-the-Art Report, Proc., Int'l Conf. on Soil Mech. and Found. Engrg., San Franciso, CA.

Levadoux, J.N. (1980). Pore Pressures in Clays Due to Cone Penetration. Ph.D. Thesis, Dept. of Civil Engrg., MIT, Cambridge, MA.

Moh and Associates (1987). Geotechnical Characterization of Taipei Basin Deposits. Report No. 85043, Submitted to the Ret-Ser Engineering Agency, Taipei Public Works Department, 201pp. (in Chinese).