**Proceedings of the XVIII ECSMGE 2024**
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY
© 2024 the Authors
ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-80
*Open Access: www.taylorfrancis.com, CC BY-NC-ND 4.0 license*

# Evaluation of some “similar” dissipation tests

## Évaluation de quelques tests de dissipation “similaires”

E. Imre*
*Óbuda University, Bánki Donát Fac. of Mechanical and Safety Eng., Budapest Hungary*

L. Bates
*University of Newcastle, Newcastle, Australia*

D. Marchetti
*Studio Prof. Marchetti, Italy*

*imre.emoke@uni-obuda.hu*

**ABSTRACT**: Various types of extremely short (<< 1 min) in situ, CPT related dissipation tests in 3 test sites were evaluated statistically using simple, closed form, parametric functions. The parameters identified from the measured data linked with soil plasticity information showed strong correlation for granular matter, more scatter for clays. A recently suggested horizontal consolidation model was used to explain the soil type dependence. The consolidation model - based on time dependent constitutive law and $K_0$ condition at the outer boundary – was verified by the evaluation of a few in situ total stress dissipation tests, where a similar, one-dimensional version of the model was used instead of the cylindrical version.

**RÉSUMÉ**: Des tests de dissipation totale courte de 3 sites ont été évalués par des fonctions paramétriques. Des diagrammes de profilage du sol ont été réalisés en reliant les parametres identifiés la plasticité du sol. Les diagrammes ont montré une forte corrélation pour la sols granulaire fins, une certaine dispersion pour les argiles. Les résultats ont été expliqués de models de consolidation couplés ont été élaborées. Dans l'évaluation précise d'essai de dissipation totale des contraintes longue in situ, la similarité du modele d'essai de relaxation et in situ a été utilisée.

**Keywords:** Oedometer relaxation test; sand; soil profiling; displacement pile; total stress dissipation test.

## 1 INTRODUCTION

The few-second-long, total stress related, $f_s$, $q_c$ and DMTA dissipation tests are made in the technical breaks of the steady penetration of the CPT equipment (S832 or Geomil CPT, see eg., Imre et al, 1995, 2023).

The test records are reflecting the following phenomena. (i) The CPT equipment is a model pile, and a residual stress state builds up at the stop of the penetration, which is negligible in soft clay but entails huge negative shaft resistance in sand (Poulos, 1987), which may redistribute (Prodinger, 1983). (ii) The dynamic-static transition in the soil constitutive equation, (iii) then horizontal consolidation (depending on soil type and initial condition) takes place.

The total stress dissipation tests are not evaluated (Baligh et al., 1985), or are evaluated approximately (Marchetti at al., 1986) in present practice.

This review type paper discusses how a newly suggested model family may explain the short, total stress related dissipation tests and the soil profiling charts. The content of the paper is as follows. The various dissipation tests, test sites and the statistical evaluation with the soil profiling charts are shown in Sections 2 to 3, the model is presented in Section 4, the model validation and the parameter analyses are presented in Sections 5 to 6.

## 2 MEASURED DATA

### 2.1 Szeged, Debrecen and Szeged-ELI sites

In the frame of the research started in 1998, some 10 to 20 m deep borings and S832 CPT’s short $f_s$ and $q_c$ dissipation tests were made in Szeged side of Tisza river and in Debrecen city.

The slightly OC (crust) and NC soil layers of Szeged and eolian sand layer of Debrecen city are listed in Figure 1 and Tables 1 to 3. Szeged city was divided into 3 parts, in parts A and B,the soil was non-saline, in part C (including the ELI site) the soil was spotlike saline (Imre et al., 2010).

In Szeged outskirts, 20 to 80 m deep borings and various dissipation tests were made in the expertise on ELI building (Figure 2).

Concerning the S832 short $f_s$ and $q_c$ dissipation tests made in Szeged side of Tisza river and in Debrecen, mean dissipation test was determined for each group (Figure 3, within layers 1 to 5 and part A, B and C in Szeged, various plasticity). The. Some DMTA dissipation test records and some Geomil $q_c$ dissipation test data were made in Szeged ELI site, the initial part of the tests are shown in Figure 4.

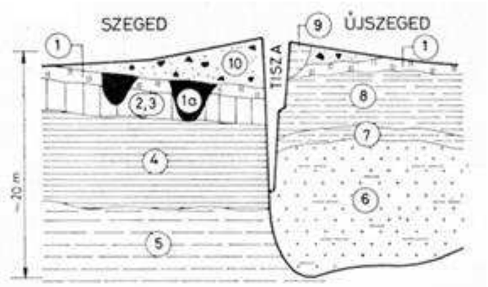



*Figure 1. Layers in Szeged side of river (Réthát, 1988).*

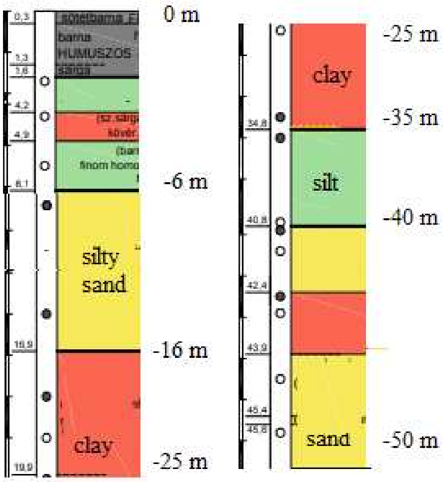



*Figure 2. Szeged, ELI, a profile indicating location of the soil samples and dissipation tests (u2, qc, DMTA). 0-50m.*

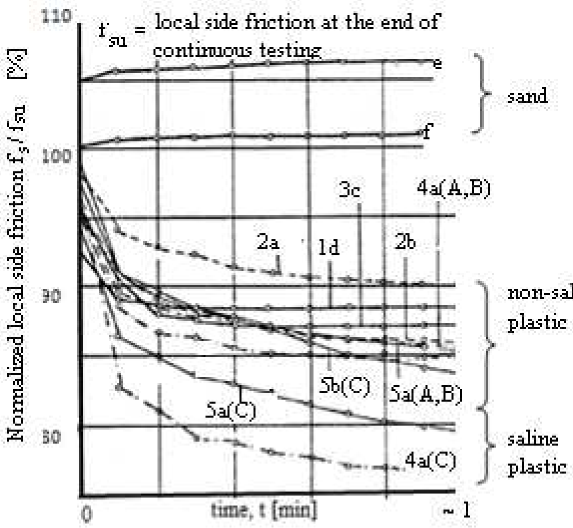



*Figure 3. Szeged city and Debrecen sand data, mean short CPT $f_s$ dissipation tests, see Tables 1-2, (Imre, 1995).*

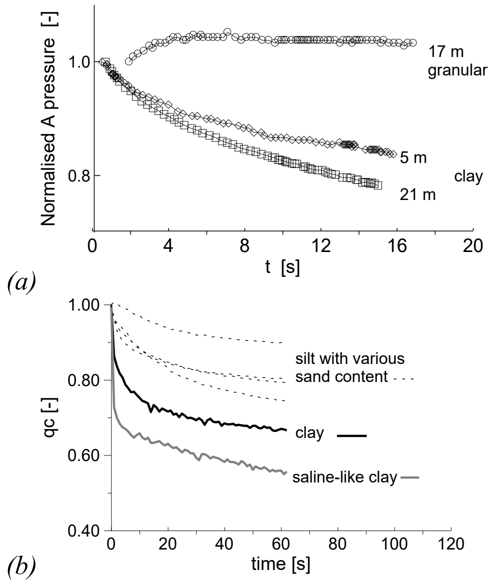



*Figure 4. Szeged, ELI site. (a) Short DMTA dissipation test results in sand and in clay (b) Geomil, short $q_c$ dissipation tests in sandy silt and in clay.*

*Table 1. Layers in the upper 20 m, Szeged Tisza bank.*

| Layer | Soil |
|---|---|
| 1 | Loess |
| 2 | Upper yellow lacustrine clay, OC |
| 3 | Silty inclusion |
| 4 | Lower yellow lacustrine clay, NC |
| 5 | Blueish fresh-water deposit, NC |

*Table 2. Soil classification, Szeged and Debrecen.*

| Notation | Category limits |
|---|---|
| a | > 25 $I_p$ [%] |
| b | 15-25 $I_p$ [%] |
| c | 10-15 $I_p$, [%] |
| d | 5-10 $I_p$, [%] |
| e | $d_{30}$>0.1mm |
| f | $d_{30}$<0.1mm |

*Table 3. Some soil properties in Szeged, non-saline soils.*

| Layer | *I*p [%] | *C*U [kPa] | *k*[m/s] | *e*[-] |
|---|---|---|---|---|
| 1 | 7.4 | 86.2 | $6.1.10^{-8}$ | 0.68 |
| 2 | 28.9+ | 92.7 | $3.9.10^{-8}$ | 0.76 |
| 3 | 19. | 75.3 | $3.8.10^{-8}$ | 0.76 |
| 4 | 36.3 | 95.0 | $2.5.10^{-g}$ | 0.85 |
| 5 | 30.01 | 60.0 | $6.3.10^{-8}$ | 0.80 |

## 2.2 The Araquari test site

The project was developed on a coastal plain in Araquari, Brazil (Figures 5, 6) where marine sandy terraces were formed in the Pleistocene. The testing program consisted of short DMTA dissipation tests in every 20 cm with data sampling in a frequency of 20 millisecond, in 3 days (Imre et al., 2023(a and b)).

## 3 SOIL PROFILING CHARTS

The following empirical parameters were defined for the various short dissipation test records (Imre, 1995; Imre et al, 2023(b)). A “stress delta” parameter:

$$\Delta_{\sigma} = \sigma(t_i) - \sigma(t_i + \Delta t); \quad (1)$$

In case of the short DMTA test, the initi time $t_i$ was the first recorded time and 15 s was the final time value.

In case of the short Sz832 CPT $f_s$ and $q_c$ dissipation tests, the $t_i$ was the time when the immediate stress drop (due to dynamic-static transition) was ended, and about 30 s was the final time value depending on the personnel.

Between these times, the stress variation is described by the following parametric functions for DMTA and CPT dissipation tests, resp.:

$$\sigma(t) = at^{b} \quad (2)$$

$$\sigma(t) = \sigma_{\infty} + (\sigma_0 - \sigma_{\infty})\ e^{-\frac{t}{\upsilon}} \quad (3)$$

Analysing the parameters, the stress delta parameter is negative/positive for stress increase/decrease, the reverse is true for the exponent $b$. Linking the parameters with plasticity index or soil type, soil profiling diagrams are shown in Figure 7 related to DMTA and fs dissipation tests.

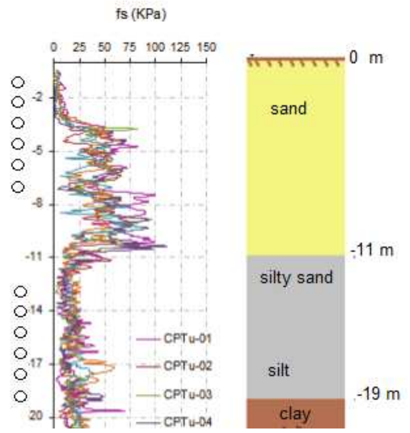



*Figure 5. Araquari test site. Soil profile and CPTu fs data (Brocheroa and Schnaid, 2020). No test in the dense sand.*

## 4 JOINED MODEL

A coupled 1 consolidation model family (oedometer, cylindrical, spherical model, space dimension 1, 2, 3), was derived by using displacement boundary condition at „outer“ boundary (Imre et al., 2010).

In the cylindrical version of the model, the constant displacement boundary condition is the $K_0$ condition which is more realistic than the constant total stress boundary condition.

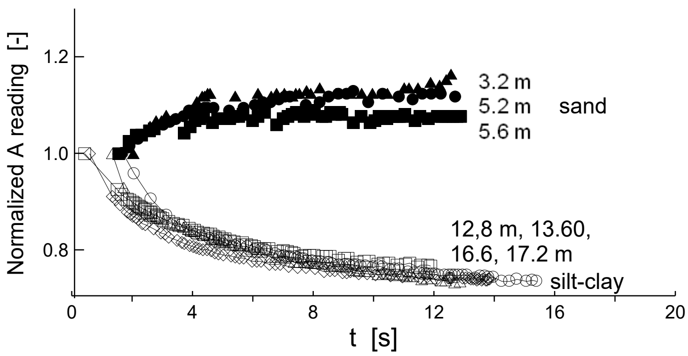
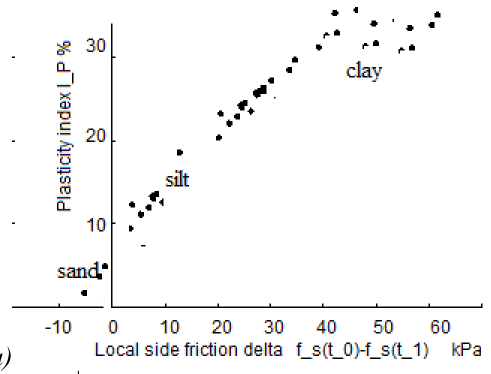
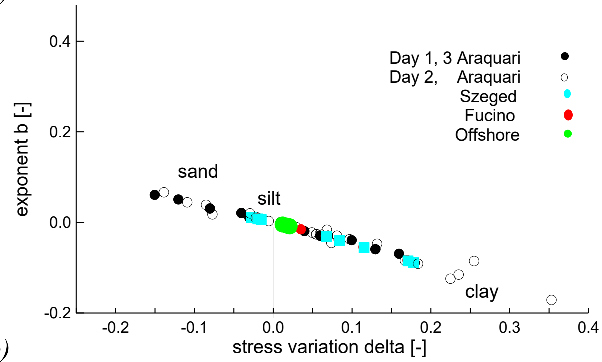



*Figure 6. Araquari test site. Short DMTA dissipation test records, in sand and in silt - clay layer.*




*(a)*




*(b)*

*Figure 7. Soil profiling diagrams by linking identified parameters with soil plasticity. (a) Szeged, S832 CPT fs diagram. (b) All test sites, DMTA diagram. Strong correlation for granular matter, scatter for clays.*

By adding a relaxation part-model, the joined 1 model family is resulted (Imre et al., 2010):

$$\sigma(t,r_0) = \sigma^{c}(t,r_0) + \Delta\sigma^{r}(t,r_0) = \sigma_{\infty} + \sigma^{t}(t,r_0) + \Delta\sigma^{r}(t,r_0) \quad (4)$$

where superscripts c and r indicate consolidation and, relaxation, resp. The relaxation model was follows:

$$\Delta\sigma^{r}(t,r_0) = -s\,\sigma(0,r_0)\frac{1}{1-s\,b}\log\left[\frac{t+t_1}{t_1+t_3}\right];\, t > t_1+t_3 \quad (5)$$

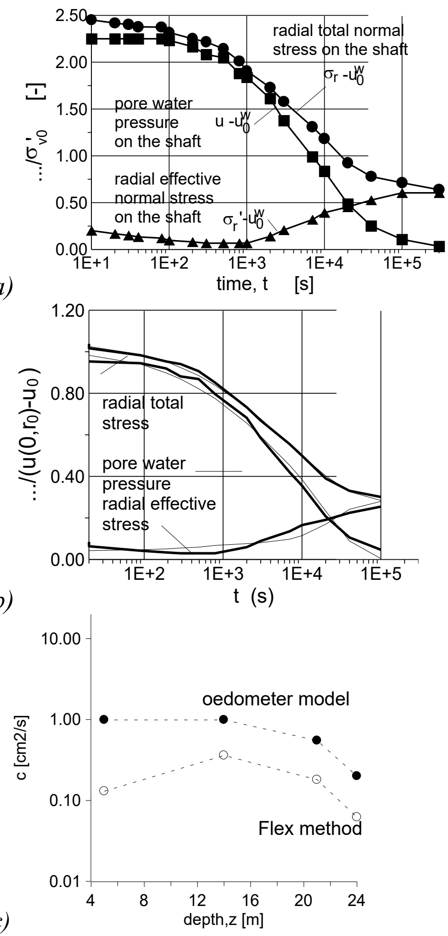
where s is coefficient of relaxation, $t_1$ and $t_3$ time constants, $b$ model parameter.

It follows from Eq (5) that the total stress decreases at $r_0$ with time due to relaxation if the elapsed time is greater than $t_1 + t_3$. The $t_3$ and $b$ are zero if no partial unloading takes place in the displacement load at the boundary $r_0$.

The qualitative behaviour of the consolidation model is determined by the initial condition as follows. Concerning the total stress variation, the transient part of the horizontal total stress at the boundary $r_0$ is equal to the mean excess pore water pressure (ment on the space domain):

$$\sigma_{r'}^{t}(t,r_0) = u_{mean}(t) \quad (6)$$

Its final value is equal to zero. It follows that the total stress changes with time due to consolidation by the initial value. This is a decrease or stress drop if the initial mean pore water pressure is positive and an increase if it is negative (see Eq (6)).




(a) (b) (c)

*Figure 8. (a) and (b): The piezo-lateral stress cell test data and evaluation results. (d) Evaluation results of DMTA long dissipation tests, made in Szeged-ELI site.*

Concerning the horizontal effective normal stress at boundary $r_0$, if positive initial excess pore water pressure dissipates, then it increases due to consolidation and decreases due to relaxation, according to Eq (5).

## 5 MODEL VALIDATION

In the case of the piezo-lateral stress cell test made in Boston Blue Clay (Figure 8(a), Baligh et al., 1985), the measured radial total stress decreases by 73%. The measured radial effective stress at the shaft decreases then increases with time. (No similar measured data set is available in case of granular matter.).

The evaluation was made by using the oedometer relaxation test model. The measured and fitted data are shown in Figure 8(b). The identified $c$ and the identified initial pore water pressure distribution were slightly over-predicted in both cases due to the elastic deformation of the equipment (since during the time dependent stress release, the diameter of the equipment slightly decreases).

The same method was applied to evaluate the DMTA dissipation tests in Szeged-ELI site. Some pore water pressure data were derived beforehand using Eq (6), then the oedometer test model was fitted on the A readings and the generated pore water pressure data. Some results are shown in Figure 8(c).

## 6 SIMULATIONS

A parameter analyses was made assuming positive initial excess pore water pressure distribution, the radius $r_1 = 37r_0$ was used, the steady-state part of solution and initial condition was estimated from the PSL cell test record in Boston Blue Clay (Figure 8(a)).

In case of varying coefficient of consolidation $c$ and relaxation $s$, the parameter analysis gave the following result. The radial total stress at $r_0$ decreased with time both due to consolidation and relaxation, with rate depending on the foregoing parameters (Figure 9), the total stress drop in short testing in soft clay can be explained by this (Figures 4, 6).

The time variation of the effective normal stress at boundary $r_0$ was an increase due to consolidation and a decrease due to relaxation, with rate depending on the coefficient of consolidation ($c$) and the coefficient of relaxation ($s$), see Figure 9. For large/small permeability soil, the increase/decrease with time was dominant in the first seconds, respectively.

The local side friction increase for large $c$ and decrease for small $c$ (shown in Figure 3) can be explained by the model simulations including the scatter due to effect of parameter $s$ for soft clays.

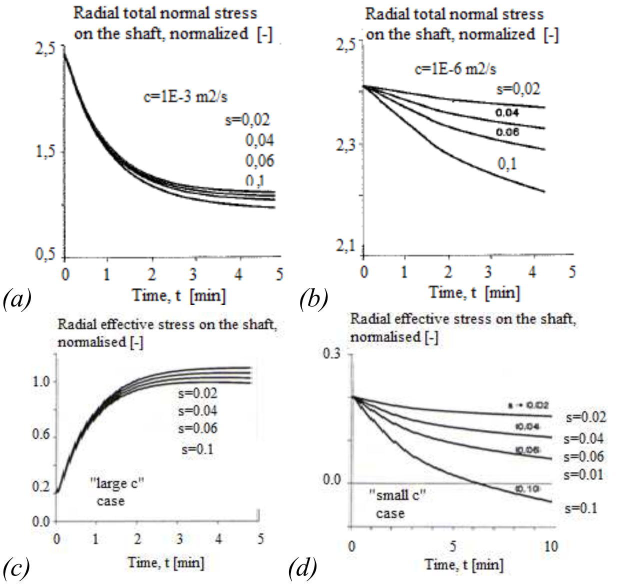



*Figure 9. Parameter analysis, various c and s values (positive initial excess pore water pressure distribution). (a) and (b) The radial total normal stress on the shaft, large / small c (or permeability), resp. (c) and (d) The radial effective stress on the shaft, large and small c, resp.*

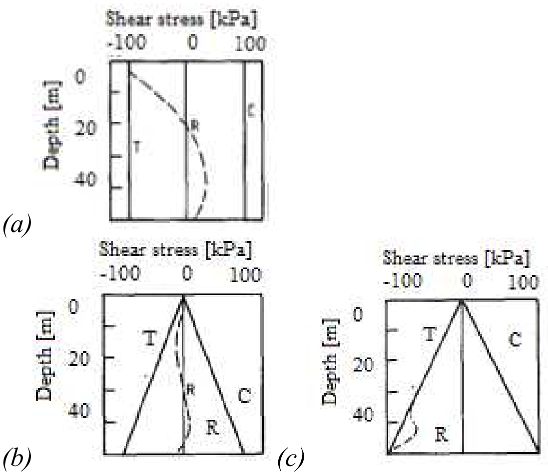



*Figure 10. (a) to (c) to (d): Residual shear stresses, stiff clay, soft clay, sand, resp. Legend: R: Residual stress. T: Stress for tensile failure. C: Stress for compression failure. (after Poulos, 1987).*

## 7 DISCUSSION

The excess pore water pressure in coarse sands is assumed as zero being the state fully drained by Campanella & and Robertson, 1991.

The theoretical works (Kouretzis et al., 2014) indicate high negative excess pore water pressures and phase change in coarse, dense sand after penetration. Negative excess pore water pressure is generally measured in most sands after penetration (see eg., Imre et al, 2021), which may increase the shaft resistance.

The theoretical work of (Poulos, 1987) indicate that the residual stress state is different in sand and in soft clay (see Figure 10). In sand, there are high negative, residual shaft stresses at the stop of the penetration. In accordance to this, Schnaid et al (2015) mention that in Araquari sand site „the equipment was unable to push the standard DMT through the upper dense sand due to a buildup of resistance during the pause to perform the standard DMT measurements“ (Figure 5). Similarly, huge re-driving resistance of displacement piles is mentioned by (Yang 1956) in sand.

Measurements during a time interval show that time dependent stress re-distribution takes place (Prodinger, 1983). The vertical equilibrium condition of the CPT equipment - soil system (Imre, 1988) and the frictional nature of the shaft force may result in a saw-tooth-like stress variation at the tip level.

## 8 CONCLUSION

The evaluation of the total stress dissipation tests is hindered by the fact that the CPT equipment is a displacement model pile with residual stress state, which is less important in soft clay and more important in sand.

The evaluation of simultaneous total stress and pore water pressure dissipation records were successful in soft clay verifying the suggested model. However, the result (identified positive excess pore water pressure initial condition, parameter $c$) reflected that the stress release in the equipment modified the rigid body boundary condition. Similar evaluation can theoretically be made in the case of medium amd coarse sand, however, very few pieces of information on the long-term behavior and on the initial condition are available.

The result of the statistical evaluation - showing soil type dependences – were explained here by a point-symmetric, coupled 1 consolidation model completed with a relaxation part-model (Imre et al., 2010) as follows. Concerning fine-graded soils, the parameter analysis made using a fixed, positive initial excess pore water pressure distribution and varying (the soil type dependent) coefficient of consolidation $c$ and coefficient of relaxation $s$ explained the short dissipation test results and the soil profiling diagrams constructed from short, DMTA, $f_s$ and $q_c$ dissipation test records, including the scatter for soft clays.

Concerning medium and coarse sands, the time dependent A pressure increase during the DMTA dissipation test was tentatively explained by the suggested consolidation model assuming negative initial excess pore water pressure distribution and partial unloading at the soil-blade interface.

Experiences and theoretical works support these assumptions since negative excess pore water pressure is measured and theories predict dilating sand behaviour along the shaft after penetration, moreover, significant change in equipment forces with negative residual side friction forces occurs at the stop of penetration. However, very few pieces of information are available on long-term phenomena in medium and coarse sand.

## REFERENCES

Baligh M. M.; Martin R. T.; Azzouz A. S. & Morrisson M. J. (1985). The piezo-lateral stress cell. *Proc. of the 11th ICSMFE* San Francisco. (2) pp. 841-844.

Imre, E. (1995). "Statistical evaluation of simple rheological CPT data". *XI. ECSMFE*, 1995. 1. 155-161.

Imre, E. (1988). Skin bearing capacity of piles. *Proc. 1st Int. Geotech. Seminar on Deep Foundations on Bored and Auger Piles* pp. 421-429. , 9 p.

Imre E., Rózsa P., Bates L., Fityus S. (2010). Evaluation of monotonic and non-monotonic dissipation tests. *Computers and Geotechnics*. (37): pp. 885-904. https://doi.org/10.1016/j.compgeo.2010.07.008.

Imre E.; Schanz T.; Hortobágyi Zs.; Singh V.P;. Fityus S. (2015). Oedometer relaxation test. *XVI ECSMGE* London, UK, ICE Pub. pp. 3347-3352.

Imre, E., Bates, L., Fityus, S.; Hegedűs, M., Singh P. (2021). Coupled models for dissipation tests *ISC6*. https://doi.org/10.53243/ISC2020-269.

Imre E., Marchetti D., Juhász M., Bates L., Fityus S. (2023a). Least Squares evaluation of DMT dissipation test data –preliminary results. *Proc. 8th IS-Porto*, isdcg2023-216-1-c.pdf. 8p.

Imre E., Marchetti D., Juhasz M., Bates L., Fityus S. (2023b). The short DMTA dissipation test. *Proc. 8th IS-Porto*, Sept. 3-6, 2023.isdcg2023-217-1-c.docx (issmge.org). 8p.

Kouretzis G. P., Sheng D. & Wang D. (2014). Numerical simulation of cone penetration testing using a new critical state constitutive model for sand. *Computers and Geotechnics* (56) pp. 50–60. 10.1016/j. compgeo. 2013.11.002.

Marchetti S., Totani G., Campanella R. G., Robertson P. K., Taddei B. (1986). The $DMT_{hc}$ Method for Piles Driven In Clay, *Proceedings of In Situ '86.* GT. Div. ASCE, June 23-23, Blacksburg. 765-779.

Poulos H. G. (1987). Analysis of residual stress effects in piles. *J. Geotech. E. ASCE*. 113:216-22. doi.org/10.1061/(ASCE)0733-9410(1987)113:3(216).

Prodinger W. (1983). Measurement of the Redistribution of Stress Around Deep Foundations. *Proc. Int. Symposium*. Zurich, 09. 5-8, 1983 543-553.

Réth// áti, L. (1988). *Probabilistic solutions in Geotechnics* Amsterdam. Elsevier.

Robertson P.K., Campanella R. G., Gillespie D.G., Greig J. (1986). Use of piezometer cone data. *ASCE Specialty conference IN-SITU'86.* SM 92.1263-1280.

Schnaid, F., Odebrecht, E., Sosnoski, J. and Robertson, P.K. (2016). Effects of test procedure on flat dilatometer test (DMT) results in intermediate soils. *Canadian Geot. Journal*, 53(8), pp.1270-1280. doi.org/10.1139/cgj-2015-0463.

Yang N.C. (1956). Re-driving characteristics of piles. *ASCE Jl. of Soil Mech. and Found. Div.* 1026-1.

## APPENDIX

Two equations of coupled consolidation:

$$E_{oed}\frac{\partial \varepsilon}{\partial r} - \frac{\partial u}{\partial r} = 0 \quad (7)$$

$$-\frac{k}{\gamma_v}\Delta\ u + \frac{\partial \varepsilon}{\partial t} = 0 \quad (8)$$

where $\varepsilon$ volumetric strain, $v$ is the radial displacement, $u$ is the excess pore water pressure, $r$ and $t$ are the space and the time co-ordinates respectively, $E_{oed}$ is the oedometric modulus. The boundary conditions:

$$u(t,r)|_{r=r_1} = 0,\ \frac{\partial u(t,r)}{\partial r}|_{r=r_0} \equiv 0,\ v(t,r)|_{r=r_0} \equiv v_0 > 0,$$
$$v(t,r)|_{r=r_1} \equiv 0 \quad (9\text{-}12)$$

The solution of the excess pore water pressure and the transient part of the radial total stress in the cylindrical case is:

$$u = \sum_{k=0}^{\infty} \lambda_k C_k\, e^{-\gamma_k^2 \cdot c_h \cdot t} \begin{Bmatrix} [I_0(\lambda_k r) + \mu_k Y_0(\lambda_k r)] \\ -[I_0(\lambda_k r_1) + \mu_k Y_0(\lambda_k r_1)] \end{Bmatrix} \quad (13)$$

$$\sigma = \sum_{k=0}^{\infty} \lambda_k C_k\, e^{-\gamma_k^2 \cdot c_h \cdot t} \left\{-[I_0(\lambda_k r_1) + \mu_k Y_0(\lambda_k r_1)]\right\} + \sigma_\infty \quad (14)$$

where $I_0$ and $Y_0$ are Bessel functions, $\lambda_k$, $\mu_k$ are the roots of the boundary condition equations; $C_k$ (k=1, 2, 3) are the l coefficients determinable from the initial condition, $c_h$ is coefficient of consolidation.

The solution of the excess pore water pressure and the transient part of the radial total stress for the oedometric relaxation test model is:

$$u(t,y) = -\sum_{i=1}^{\infty} \varphi_i [\cos(y(i\pi/H) - 1]\ e^{-i^2 \pi^2 T} \quad (13)$$

$$\sigma(t) = -\sum_{i=1}^{\infty} \varphi_i\, e^{-i^2 \pi^2 T} + \sigma_\infty \quad (14)$$

where $T = ct/H^2$ the time factor, $c$ coefficient of consolidation, $\phi_i$ are Fourier coefficients related to the initial condition, $H$ length between model boundaries.

INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

*This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:*

*https://www.issmge.org/publications/online-library*

*This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.*

*The paper was published in the proceedings of the 18th European Conference on Soil Mechanics and Geotechnical Engineering and was edited by Nuno Guerra. The conference was held from August 26th to August 30th 2024 in Lisbon, Portugal.*