FLOW PROPERTIES from PIEZOCONE DISSIPATION TESTS

Soils exhibit flow properties that control ***hydraulic conductivity*** (k), rates of consolidation, construction behavior, and drainage characteristics in the ground. Field measurements for soil permeability include pumping tests with measured drawdown, slug tests, and packer methods. Laboratory methods include falling head and constant head types in permeameters, controlled gradient, and constant rate of strain consolidation (Leroueil, et al., *Geotechnique*, June 1992). An indirect assessment of permeability can be made from consolidation test data. Results of pressure dissipation readings from piezocone and flat dilatometer and holding tests during pressuremeter testing can be used to determine permeability and the coefficient of consolidation (Jamiolkowski, et al. 1985, *Proc. 11th ICSMFE*, San Francisco, Vol. 1). Herein, only the piezocone approach will be discussed.

The ***permeability*** (k) can be determined from the dissipation test data, either by use of the direct correlative relationship presented earlier, or alternatively by the evaluation of the ***coefficient of consolidation***, c_h. Assuming radial flow, the horizontal permeability (k_h) is obtained from:

$$k_h = \frac{c_h \gamma_w}{D'}$$

where D′ = constrained modulus obtained from oedometer tests. Note: results of high-quality lab testing of natural clays show $k_h \approx 1.1\ k_v$ unless the deposit is highly stratified or consists of varved materials (Tavenas, et al., Nov. 1983, *Canadian Geot. Journal*).

Piezocone Dissipation Tests

In a CPTu test performed in saturated clays and silts, large excess porewater pressures (Δu) are generated during penetration of the piezocone. Soft to firm intact clays will exhibit measured penetration porewater pressures which are 3 to 6 times greater than the hydrostatic water pressure, while values of 10 to 20 times greater than the hydrostatic water pressure will typically be measured in stiff to hard intact clays. In fissured materials, zero or negative porewater pressures will be recorded. Regardless, once penetration is stopped, these excess pressures will decay with time and eventually reach equilibrium conditions which correspond to hydrostatic values. In essence, this is analogous to a push-in type piezometer. In addition to piezometers and piezocones, excess pressures occur during the driving of pile foundations, installation of displacement devices such as vibroflots for stone columns and mandrels for vertical wick-drains, as well as insertion of other in-situ tests including dilatometer, full-displacement pressuremeter, and field vane.

How quickly the porewater pressures decay depends on the permeability of the surrounding medium (k), as well as the horizontal coefficient of consolidation (c_h). In clean sands and gravels that are pervious, essentially drained response is observed at the time of penetration and the measured porewater pressures are hydrostatic. In most other cases, an initial undrained response occurs that is followed by drainage. For example, in silty sands, generated excess pressures can dissipate in 1 to 2 minutes, while in contrast, fat plastic clays may require 2 to 3 days for complete equalization.

Representative dissipation curves from two types of piezocone elements (midface u_1 and shoulder u_2) are presented in Figure F-1. These data were recorded at a depth of 15.2 meters in a deposit of soft varved silty clay at the National Geotechnical Experimentation Site (NGES) in Amherst, MA. Full equalization tohydrostatic conditions is reached in about 1 hour (3600 s). In routine testing, data are recorded to just 50 percent consolidation in order to maintain productivity. In this case, the initial penetration pressures correspond to 0 percent decay and a calculated hydrostatic value (u_0) based on groundwater levels represents the 100 percent completion. Figure F-1 illustrates the procedure to obtain the time to 50% completion (t_{50}).

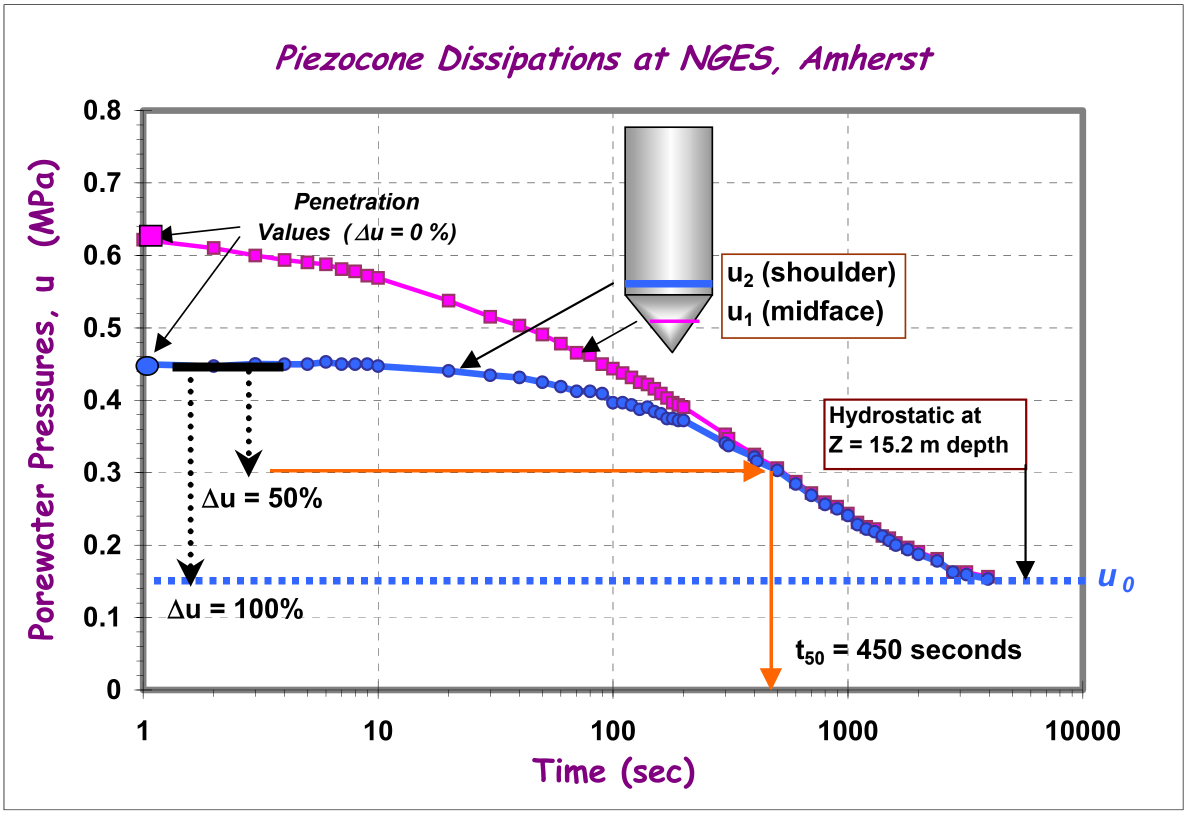


Figure F-1. Porewater Pressure Dissipation Response in Soft Varved Clay at Amherst NGES.
(Procedure for t_{50} determination using U_2 readings shown)

The aforementioned approach applies to soils that exhibit monotonic decay of porewater pressures with logarithm of time. For cases involving heavily overconsolidated and fissured geomaterials, a dilatory response can occur whereby the porewater pressures initially rise with time, reach a peak value, and then subsequently decrease with time.

For type 2 piezocones with shoulder filter elements, the t_{50} reading from monotonic responses can be used to evaluate the permeability according to the chart provided in Figure F-2. The average relationship may be approximately expressed by:

$$k \ \ (\text{cm/s}) \ \ \approx \ \ 1/(251 \cdot t_{50})^{1.25}$$

where t_{50} is given in seconds. The interpretation of the coefficient of consolidation from dissipation data is discussed subsequently and includes both monotonic and dilatory porewater pressure behavior.

Monotonic Dissipation

For *monotonic* porewater decays where the readings always decrease with time, these responses are generally are associated with soft to firm clays and silts. For these cases, the strain path method (Teh & Houlsby, 1991, *Geotechnique*) may be used to determine c_h from the expression:

$$c_h = \frac{T^* a^2 \sqrt{I_R}}{t_{50}}$$

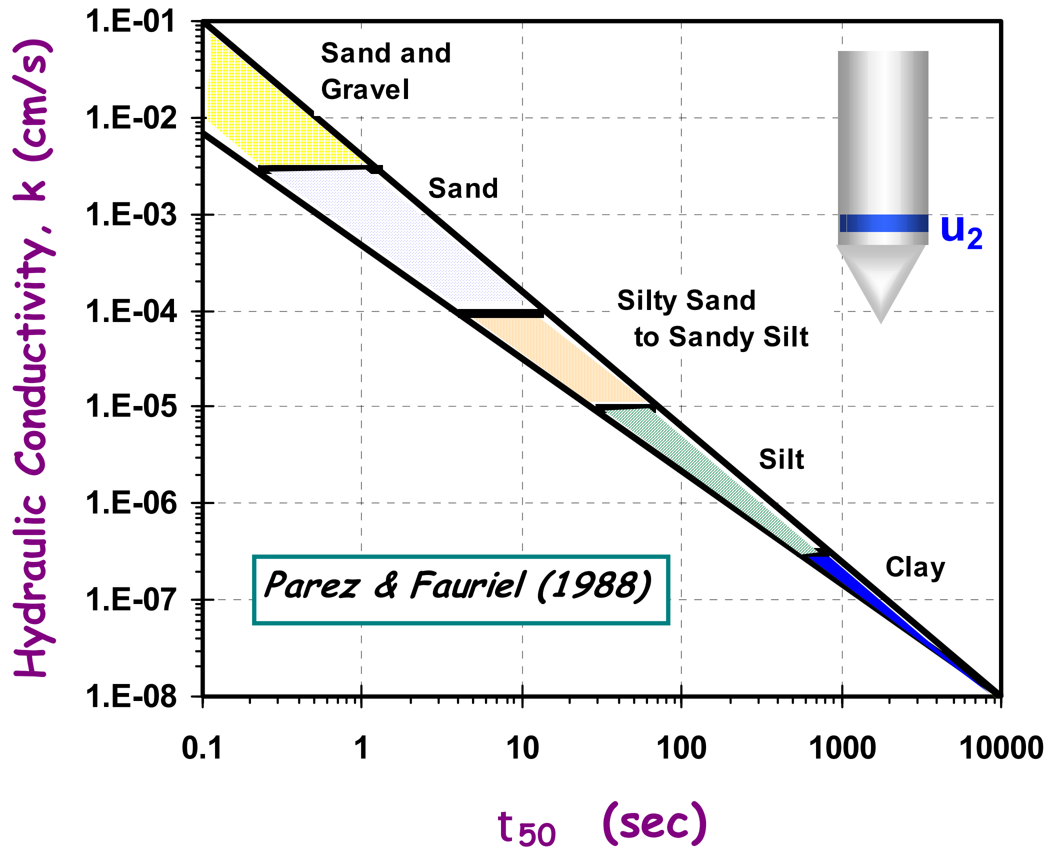


Figure F-2: Coefficient of Permeability (k = Hydraulic Conductivity) from Measured Time to 50% Consolidation (t_{50}) for Monotonic Type 2 Dissipations (from Parez & Fauriel, 1988).

where T* = modified time factor from consolidation theory, a = probe radius, $I_R = G/s_u$ = rigidity index of the soil, and t = measured time on the dissipation record (usually taken at 50% equalization). Several solutions have been presented for the modified time factor T* based on different theories, including cavity expansion, strain path, and dislocation points (Burns & Mayne, 1998, *Can. Geot. J.*). For monotonic dissipation response, the strain path solutions (Teh & Houlsby, 1991, *Geot.*) are presented in Figures F-3 and F-4 for both midface and shoulder type elements, respectively.

The determination of t_{50} from shoulder porewater decays is illustrated by example in Figure F-1. These strain path solutions can be approximately described by the following:

$$\frac{\Delta u}{\Delta u_{initial}} = \left(\frac{1}{1.12 + 30 \cdot T^*}\right)^{0.48}$$

$$\frac{\Delta u_2}{\Delta u_{2-INITIAL}} = \left(\frac{1}{1 + 10 \cdot T^*}\right)^{0.64}$$

For the particular case of 50% consolidation, the respective time factors are T* = 0.118 for the type 1 (midface element) and T* = 0.245 for the type 2 (shoulder element).

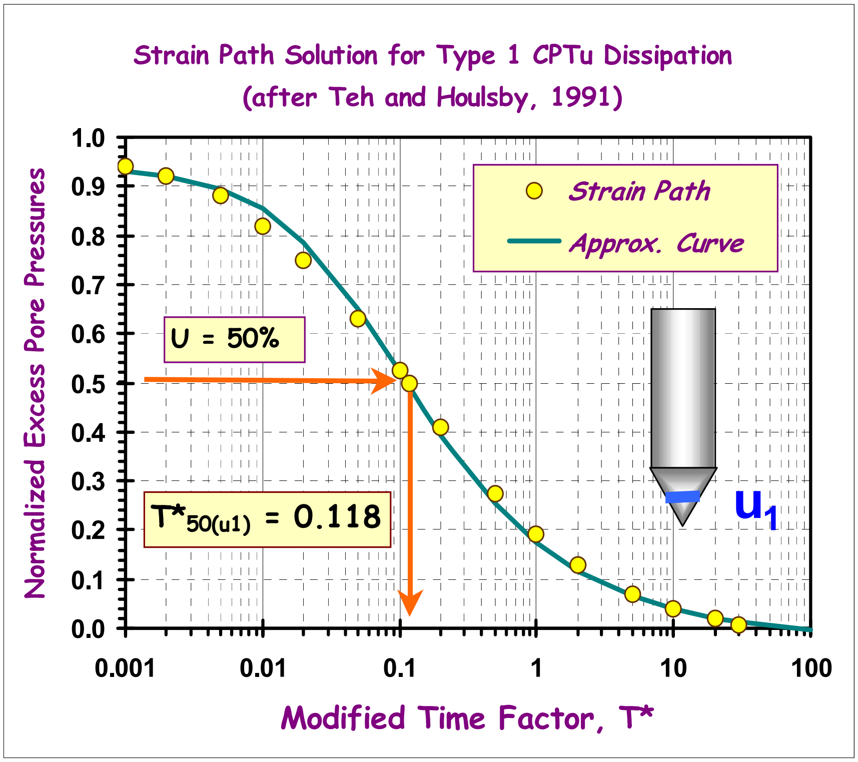


Figure F-3. Modified Time Factors for u_1 Monotonic Porewater Dissipations

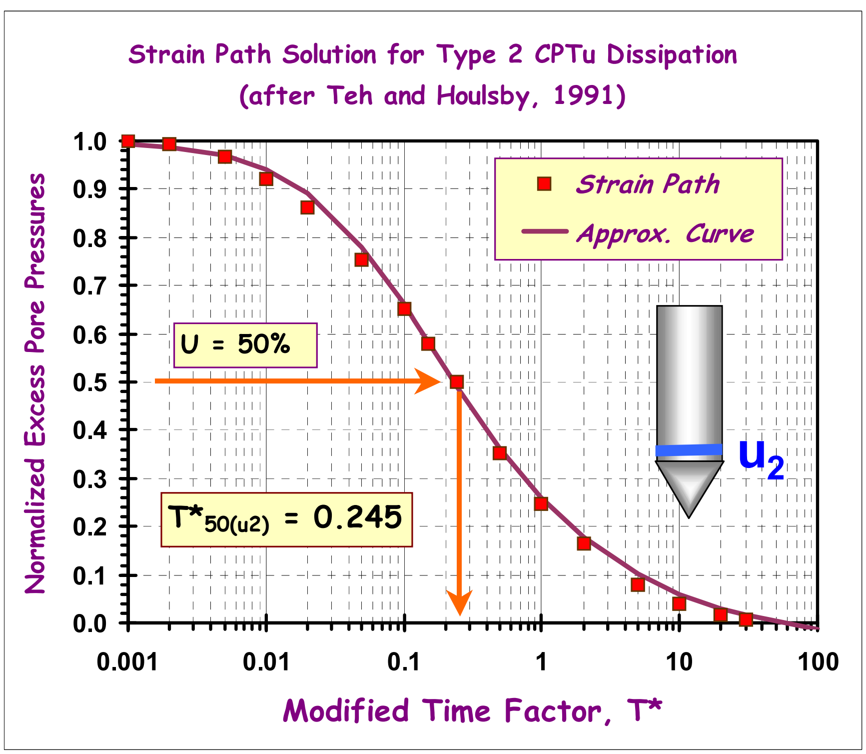


Figure F-4. Modified Time Factors for u_2 Monotonic Porewater Dissipations

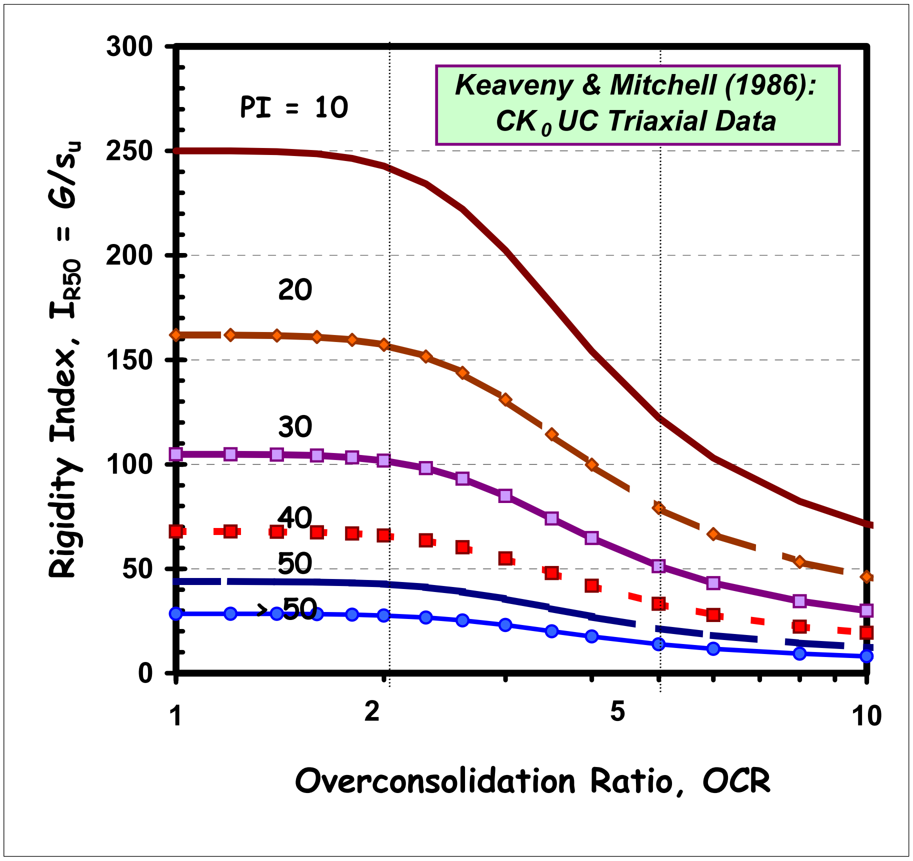


Figure F-5. Estimation of Undrained Rigidity Index of Clays and Silts from OCR and Plasticity Index (Keaveny & Mitchell, 1986).

For clays, the undrained rigidity index (I_R) is the ratio of shear modulus (G) to shear strength (s_u) and may be obtained from a number of different means including: (a) measured triaxial stress-strain curve, (b) measured pressuremeter tests, and (c) empirical correlation. One correlation based on anisotropically-consolidated triaxial compression test data expresses I_R in terms of OCR and plasticity index (PI), as shown in Figure F-5. For spreadsheet use, the empirical trend may be approximated by:

$$I_R \approx \frac{\exp\left[\frac{137 - PI}{23}\right]}{\left[1 + \ln\left\{1 + \frac{(OCR - 1)^{3.2}}{26}\right\}\right]^{0.8}}$$

Additional approaches to estimating the value of I_R are reviewed elsewhere (Mayne, *Proc. In-Situ 2001*, Bali). To facilitate the interpretation of c_h corresponding to t_{50} readings using the standard penetrometer, Figure F-6 presents a graphical plot for various I_R values.

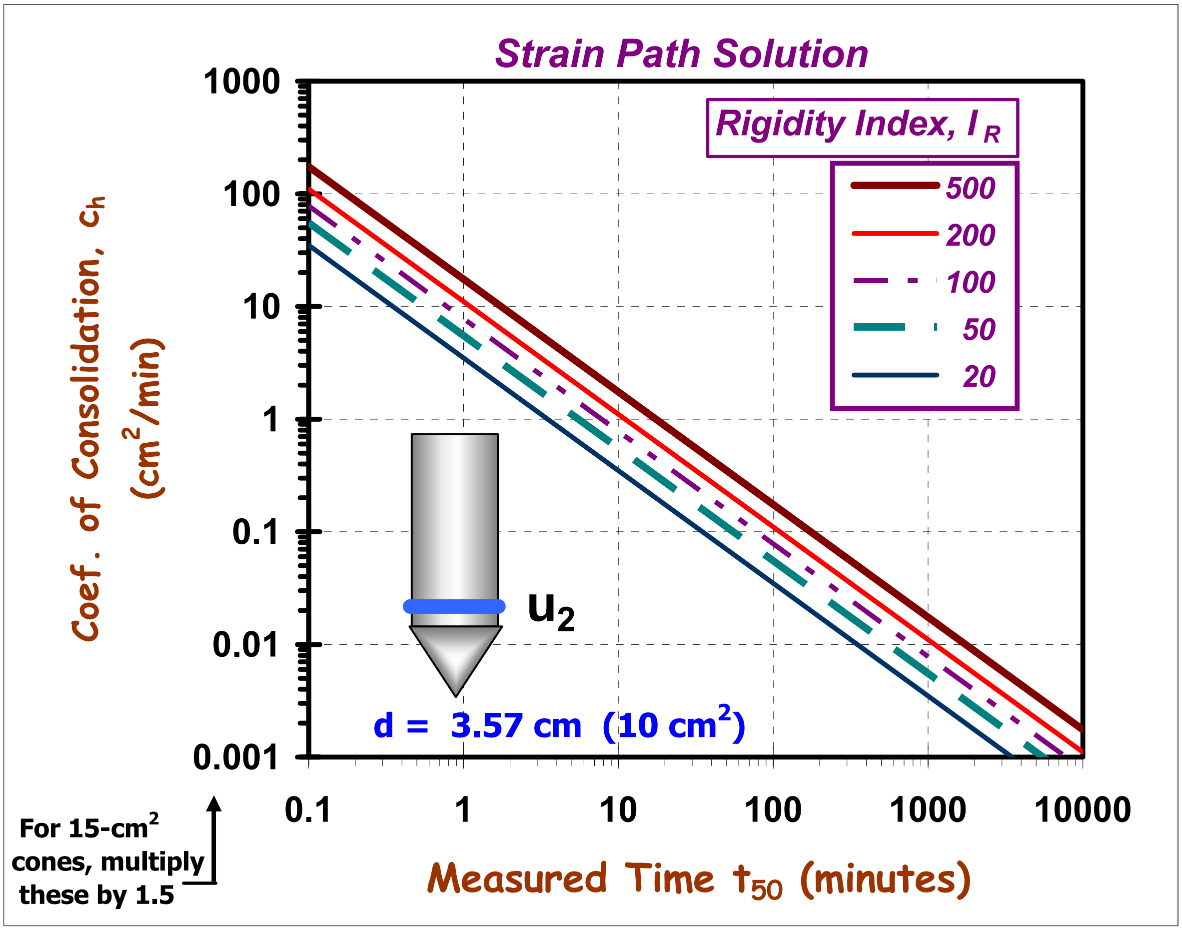


Figure F-6. Coefficient of consolidation at 50% dissipation for shoulder elements.

Dilatory Dissipations

In many overconsolidated and fissured materials, a dissipation test may first show an increase in Δu with time, reaching a peak value, and subsequent decrease in Δu with time (e.g., Lunne, et al. 1997). This type of response is termed *dilatory* dissipation, referring to both the delay in time and cause of the phenomenon (dilation). The dilatory response has been observed during type 2 piezocone tests as well as during installation of driven piles in fine-grained soils. The definition of 50% completion is not clear and thus the previous approach is not applicable.

A rigorous mathematics derivation has been presented elsewhere that provides a cavity expansion-critical state solution to both monotonic and dilatory porewater decay with time (Burns & Mayne, 1998). For practical use, an approximate closed-form expression is presented here. In lieu of merely matching one point on the dissipation curve (i.e, t_{50}), the entire curve is matched to provide the best overall value of c_h. The excess porewater pressures Δu_t at any time t can be compared with the initial values during penetration (Δu_i).

The measured initial excess porewater pressure ($\Delta u_i = u_2 - u_o$) is given by:

$$\Delta u_i \quad = \quad (\Delta u_{oct})_i \quad + \quad (\Delta u_{shear})_i$$

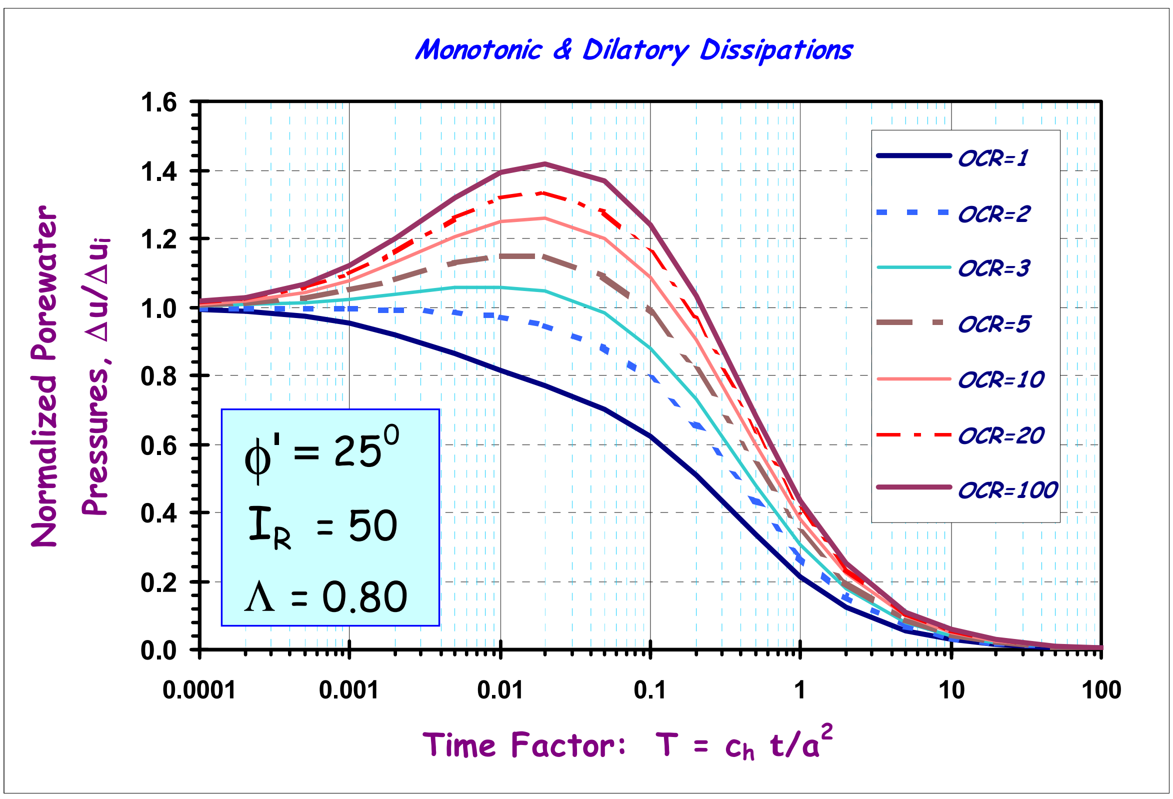
where $(\Delta u_{oct})_i \;=\; \sigma_{vo}'(2M/3)(OCR/2)^{\Lambda}\ln(I_R)$ = the octahedral component during penetration;

and $(\Delta u_{shear})_i \;=\; \sigma_{vo}'[1 - (OCR/2)^{\Lambda}]$ is the shear-induced component during penetration.

The porewater pressures at **<u>any</u>** time (t) are obtained in terms of the modified time factor T* from:

$$\Delta u_t \quad = \quad (\Delta u_{oct})_i\,[1 + 50\,T']^{-1} \quad + \quad (\Delta u_{shear})_i\,[1 + 5000\,T']^{-1}$$

where a different modified time factor is defined by: $T' = (c_h\,t)/(a^2\,I_R^{0.75})$. On a spreadsheet, a column of assumed (logarithmic) values of T' are used to generate the corresponding time (t) for a given rigidity index (I_R) and probe radius (a). Then, trial & error can be used to obtain the best fit c_h for the measured dissipation data. Series of dissipation curves can be developed for a given set of soil properties. One example set of curves is presented in Figure F-7 for various OCRs and the following parameters: $\Lambda = 0.8$, $I_R = 50$, and $\phi' = 25^{\circ}$, in order to obtain the more conventional time



factor, $T = (c_h\,t)/a^2$.

Figure F-7. Representative Solutions for Type 2 Dilatory Dissipation Curves at Various OCRs (after Burns & Mayne, 1998, *Canadian Geotechnical Journal*).