CPT in glacial soils after deep excavation

T. Liao & K.R. Bell
Bechtel Power Corporation, Frederick, MD, USA

H. Senapathy
HSA, Inc, Washington, DC, USA

ABSTRACT: A 35-m high bluff along the shoreline of Lake Michigan was cut for construction of a large energy facility. Following the excavation, cone penetration tests (CPT) were performed in both the excavated area, as well as in the shoreline area between the bluff and the lake. For the underlying glacial soils in the excavated area, empirical correlations were applied to the CPT measurements to estimate the preconsolidation pressure, which is then compared with the calculated overburden pressure before excavation. In the glacial soil deposits, the CPT tests were found especially helpful in detecting seams of geomaterial that may be missed by other tests.

1 INTRODUCTION

A large energy facility was constructed along the shoreline of Lake Michigan in the State of Wisconsin. Before construction, this site was a steep bluff close to the shoreline of the lake, as shown in Figure 1a. The base of the bluff is at an elevation of about 180 m above mean sea level (MSL). The topography of the plateau is relatively flat to rolling with elevations generally ranging from about 210 m to 220 m above MSL. In order to utilize the lake water, the energy facility needed to be placed near the water level in the lake. Therefore, in an area of about 550 m x 350 m, approximately 35 m of the bluff was cut during construction to achieve the final grade close to the bluff base level at the site. Presented in Figure 1b is an aerial photo of the excavated area during construction.

Although the excavated area would accommodate most of the structures for the energy facility, some structures, including the chimney, would be erected near the shore in an area between the lake and the excavated area (Figure 1). The topography of the shoreline area is relatively flat with elevations generally ranging around 180 m above mean sea level (MSL).

Above bedrock, this site consists of vast quantities of glacial material due to multiple glacial incursions in the area. These glacial deposits are typically either well sorted water-laid deposits such as sand and gravel or poorly sorted ice-laid till deposits consisting of clay, silt, sand, gravel and boulders (Olcott, 1992). Since glacial soil deposits usually contain inter-bedded layers of fine and coarse grained materials, the cone penetration tests (CPT) can be especially helpful in providing a continuous soil profile and detecting seams of geomaterials that may be missed by other tests.

(a)

(b)

Figure 1. Construction site along the shoreline of Lake Michigan: (a) before excavation; (b) during construction.

2 EXCAVATED AREA

Subsurface investigation was performed in this area following a substantial amount of excavation to lower the ground surface to the proposed final grade elevation. The subsurface investigation consisted of cone penetration tests (CPT), standard penetration tests (SPT), suspension P-S velocity logging, and laboratory tests on selected soil samples. The groundwater table is at the ground surface after excavation.

2.1 Test Results

Presented in Figure 2 are the typical results from a CPT sounding performed in the excavated area. In the depth range from 1.5 m to 5 m, the tip resistance q_T and sleeve friction f_s are relatively high, and the measured porewater pressure u_2 is close to static water pressure u_0, indicating a quite clean sand layer. Although fluctuating CPT measurements were observed in the depth range from 5 m to 15 m, it can be seen that most of the soils are clay with seams of sandy material, based on the readings of q_T and u_2. Below 15 m in depth, two clay layers can be delineated with their boundary at about 20.4 m in depth. The unusually high q_T and u_2 measurement in the lower clay layer suggests it is very hard clay.

Figure 3 presents the blow counts N_{60} of four SPT borings performed within a distance of 50 m from the CPT sounding, along with selected lab test results (i.e., Atterberg Limits and fines content) for the soil samples from the excavated area. Based on the collected soil samples, most of the soils above 4 m in depth are sand, which is underlain by thick deposit of clay. The sand layer has SPT N-value around 15 and fines content around 10%. In the clay layer, the fines content is about 90%, and its SPT N-value generally increases with depth. The water content of the clay samples ranges from 10% to 20%, which is approximately in the same range of their plastic limit.

The liquid limit for the clay sample is in the range between 20% and 35%. Based on the unified soil classification system (USCS), the clay at this site can be classified as CL (Inorganic clays of low to medium plasticity).

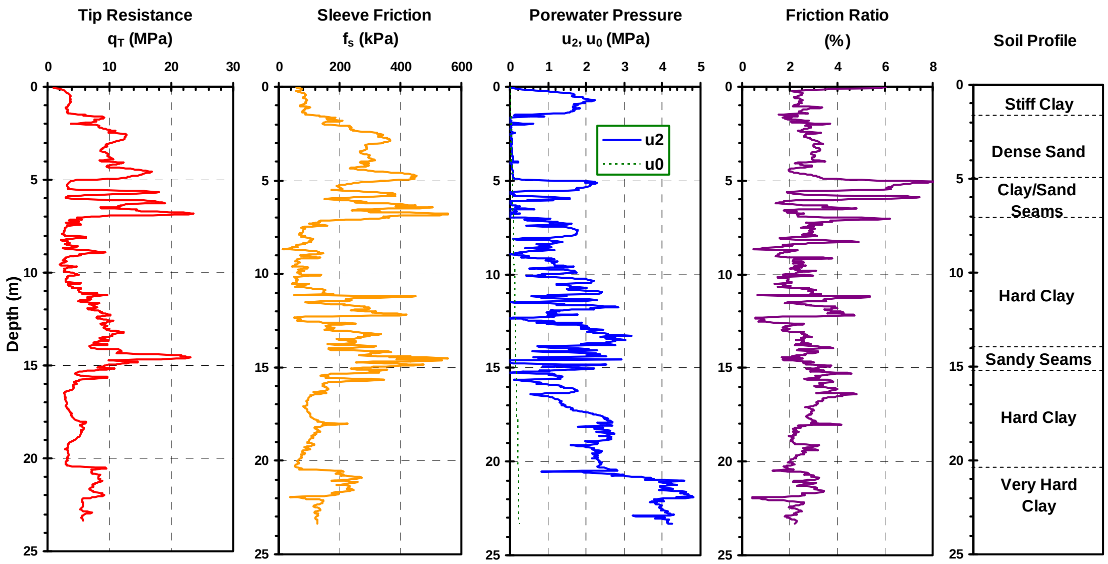

Figure 2. Typical CPT results in excavated area

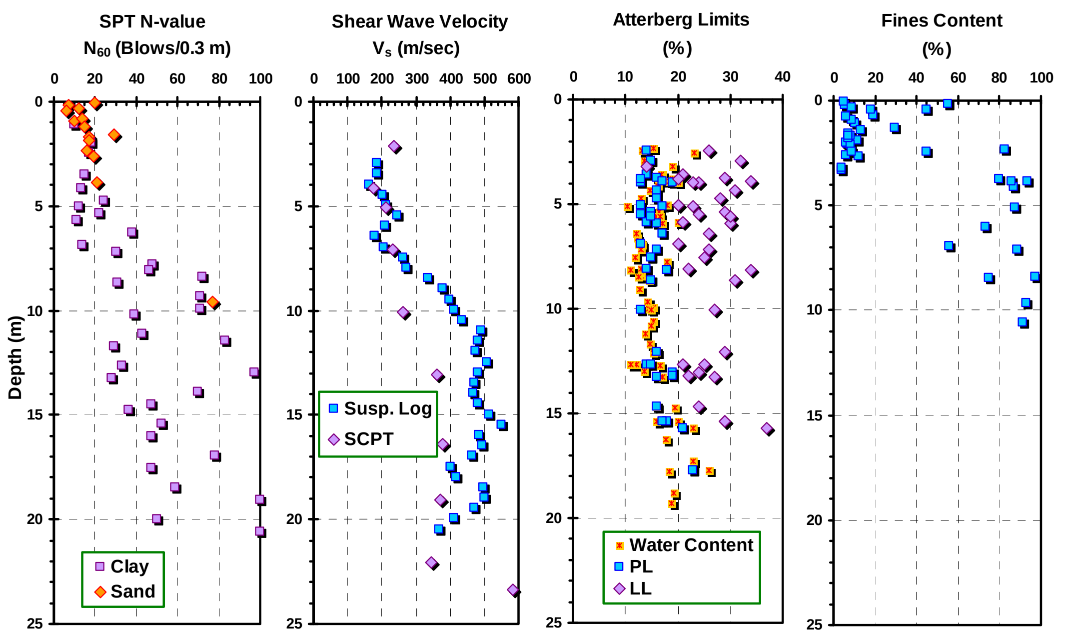

Figure 3. Other test results in excavated area

Figure 3 also shows the shear wave velocity V_s measured in the excavated area by both downhole P-S logging and a seismic cone penetration test (SCPT). Measurements from the two tests agree well except in the depth range from 10 m to 17 m. The significantly higher value of V_s at depth 23.5 m confirms the existence of a very hard clay layer interpreted from the CPT sounding shown in Figure 2.

2.2 Correlation Between CPT and SPT

The SPT N_{60}-value can be estimated from the CPT tip resistance using empirical correlations. The ratio of CPT tip resistance q_T and SPT N-value can be expressed as a function of the mean grain size D_{50} (Kulhawy & Mayne, 1990):

$$(q_T / p_a)/N_{60} = 5.44 D_{50}^{\ 0.26} \qquad (1)$$

where p_a is atmospheric pressure, q_T and p_a have the same units, and D_{50} is in mm.

Based on lab test results on the collected soil samples, the sand and clay soils from different depth are similar in terms of grain size distribution, with the average mean grain size D_{50} for the sand and clay soils being 0.2 mm and 0.006 mm, respectively. Thus, the CPT tip resistance q_T can be correlated from N_{60} using equation (1), as shown in Figure 4a. Comparison shows that the correlated q_T is quite consistent with the measured value in the clay layers, but they appear to be lower than the measured value in the depth range in the top 7 m, which primarily consists of sand.

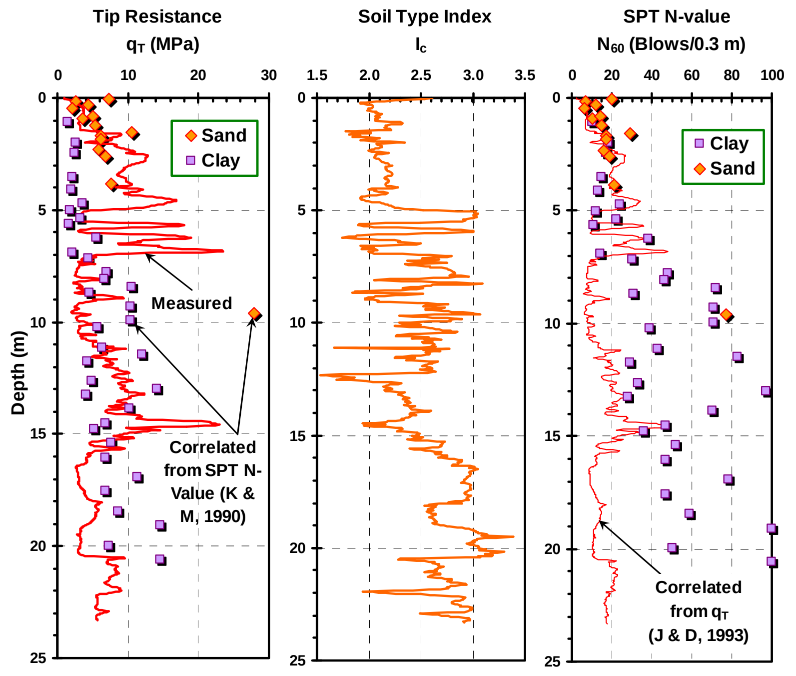

Figure 4. Presentation of empirical correlations based on CPT results in the excavated area: (a) Comparison of measured and correlated CPT tip resistance; (b) Soil type index; (c) Comparison of measured and correlated SPT N-value.

The SPT N_{60} value can also be estimated from CPT tip resistance using the following correlation suggested by Jefferies & Davies (1993):

$$q_T / N_{60} = 0.85(1 - I_c / 4.75) \qquad (2)$$

where q_T is in MPa, and I_c is the soil behavior type index which can be derived from the CPT measurements (Jefferies & Davies, 1993). The I_c values computed from the typical CPT sounding (Figure 2) are shown in Figure 4. The I_c value is generally in the range from 2 to 2.2 in the top sand layer. In the depth range from 5 m to 15 m, the I_c value fluctuates with an approximate average of 2.5. The clay layers below depth of 15 m have an I_c value between 2.6 and 3.0. The I_c value for the soil layers are consistent with those suggested by Jefferies & Davies (1993) for different soil types. With the I_c value, the SPT N-value can be correlated from the measured q_T as shown in Figure 4c. The correlated N_{60} value agrees well with the measured value in the top sand layer. However, in the clay layers below depth 5 m, the correlated N_{60} value seems to be considerably lower than the measured values.

2.3 Preconsolidation Stress

The excavation at this site significantly changed the overburden stress of the underlying soils, which provides an opportunity to evaluate the various correlations to estimate the preconsolidation stress σ_p'. Methods of estimating σ_p' in clay from CPT measurements have been presented by a number of researchers (e.g., Schmertmann, 1978; Robertson, 1990). Some of the recently proposed methods are as follows (Demers & Leroueil, 2002; Chen & Mayne, 1996; Mayne, 2005):

$$\sigma_p' = 0.33(q_T - \sigma_{v0}) \tag{3}$$

$$\sigma_p' = 0.40(u_2 - u_0) \tag{4}$$

$$\sigma_p' = 0.60(q_T - u_2) \tag{5}$$

The σ_p' derived from the different expressions is separately plotted versus depth in Figure 5a, b, and c, and are compared with the removed overburden stress after excavation. It can be seen that the σ_p' values estimated using the three expressions all deviate from the calculated overburden stress to some extent. One possible reason is that the onsite soils consist of fissured clay, which the three expressions are known not to apply to. In addition, the soil might have experienced unknown stress history, besides the overburden removal during excavation.

The overconsolidation ratio for sand can be estimated from the following expression (Mayne, 2005):

$$OCR = \left[\frac{0.192(q_T / \sigma_{atm})^{0.22}}{(1-\sin\phi')(\sigma_{v0}'/\sigma_{atm})^{0.31}} \right]^{\left(\frac{1}{\sin\phi' - 0.27}\right)} \tag{6}$$

Where ϕ' is the effective stress friction angle of the sand, and σ_{atm} is atmosphere pressure. Taking $\phi' = 40^o$, Figure 5d shows overconsolidaton pressure derived using the above expression. As can be seen, the estimated σ_p' agrees well with the removed overburden pressure.

Mayne (2005) also suggested evaluating σ_p' from small-strain stiffness G_0:

$$\sigma_p' = 0.101 \sigma_{atm}^{0.102} G_0^{0.478} \sigma_{v0}'^{0.420} \tag{7}$$

Assuming the unit weight $\gamma = 17.5$ kN/m^3 for onsite soil, G_0 can be derived from the shear wave velocity V_s measured by P-S logging (i.e., $G_0 = (\gamma/g)\, V_s^2$). The σ_p' estimated from above expression is compared with the removed overburden pressure in

Figure 5e. Except in the top 3 m, the difference between estimated σ_p' and σ_{v0}' is less than 0.5σ_p'.

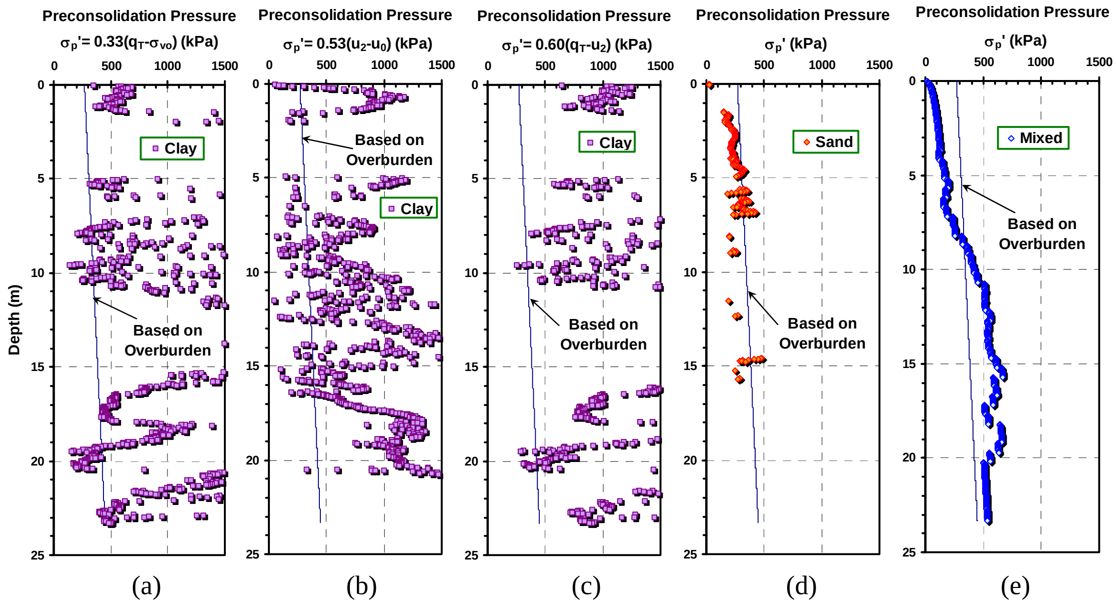

Figure 5. Evaluation of preconsolidation pressure of soils in the excavated area: (a) Clay - Equation 3; (b) Clay - Equation 4; (c) Clay - Equation 5; (d) Sand - Equation 6; (e) Mixed soils - Equation 7.

3 SHORELINE AREA

Figure 6 shows a typical CPT sounding performed on the shoreline area, which lies between the lake and the excavated area. From the CPT measurements, the soil profile can be derived, with a top sand layer of 4.8 m in thickness overlying a clay layer of about 8 m in thickness. It was found from the CPT tip resistance q_T that this clay layer varied in consistency from relatively soft in the top to hard below depth of 6 m. This clay layer was followed by a sand layer of about 2 m in thickness, and then underlain by a stiff clay layer.

Similar to the CPT sounding performed in the excavated area, the blow counts N_{60} of 3 SPT borings performed nearby are compared with the CPT tip resistance through correlations, as shown in Figure 7. The correlation proposed by Kulhawy & Mayne (1990) seems to give a better predication than that by Jefferies & Davies (1993). The soil type index I_c value in the clay layers is consistently around 2.5 in most of the depth range. The I_c value in the sand layers fluctuates, but is generally less than 2.0. The I_c appears to be an excellent index to distinguish sand from clay at this site.

A 185-m tall chimney was planned to be built on the shoreline area (Figure 1b). From the CPT measurements presented in Figure 6, there is a weak zone in the depth range from 5 m to 7 m, which requires ground improvement to achieve relatively high bearing capacity and reduce predicted settlements to less than 2.5 cm. However, the SPT N-values shown in Figure 7c missed such a soft zone. Jet-grouting was later used for ground improvement for the chimney foundation. Post-grouting settlement measurements taken during construction of the chimney showed that the improved bearing capacity had reached expected range and that the settlement was within the acceptable range.

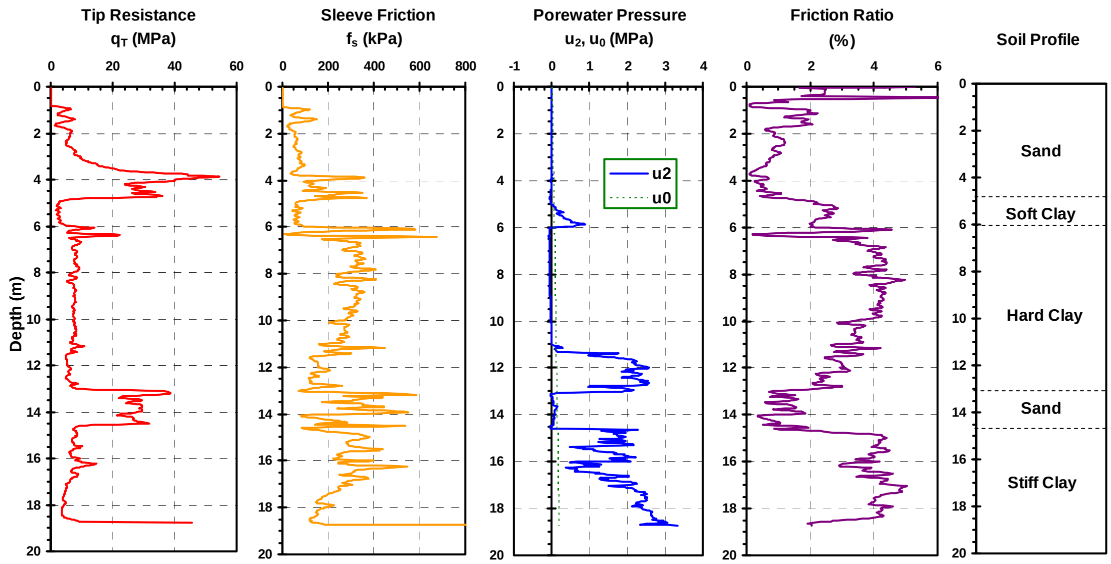

Figure 6. Typical CPT results in shoreline area

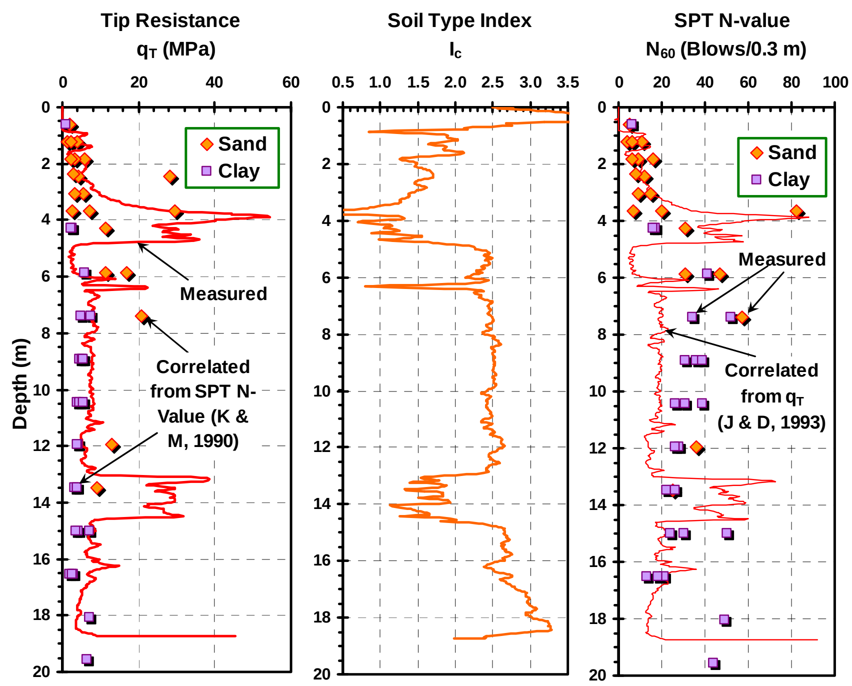

Figure 7. Presentation of empirical correlations based on CPT results in the shoreline area: (a) Comparison of measured and correlated CPT tip resistance; (b) Soil type index; (c) Comparison of measured and correlated SPT N-value.

4 CONCLUSIONS

For construction of a large energy facility, a 35-m high steep bluff was cut close to the shoreline of Lake Michigan. Following the excavation, CPT tests were performed in the excavated area, as well as in the shoreline area on the lake beach.

The change of overburden stress due to excavation provides an opportunity to evaluate various empirical correlations to estimate preconsolidation stress from CPT measurements. It was found that for sand the estimated preconsolidation pressure agrees well with the overburden stress before excavation. For clay soils, data scattering exists for all the correlation expressions, possibly due to fissures in clay or unknown stress history.

Because CPT tests can provide a continuous soil profile, it is very helpful in detecting inter-bedded layers of geomaterials. This is evident by the CPT tests performed in the shoreline area. The CPT measurements detected a weak zone of 1 m in thickness, which was missed by the other tests.

5 REFERENCES

Chen, B.S-Y. & Mayne, P.W. 1996. Statistical relationships between piezocone measurements and stress history of clays. *Canadian Geotechnical Journal*, 33(3): 488-498.

Demers, D., & Leroueil, S. 2002. Evaluation of preconsolidation pressure and the overconsolidation ratio from piezocone tests of clay deposits in Quebec. *Canadian Geotechnical Journal*, 39(1): 174-192.

Jefferies, M.G. & Davies, M.P. 1993. Use of CPTu to estimate equivalent SPT N_{60}. *ASTM Geotechnical Testing Journal*, 16(4): 458-468.

Kulhawy, F.H. & Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. Report EPRI EL-6800, Electric Power Research Institute, Palo Alto, 306 pp.

Mayne, P.W. 2005. Integrated ground behavior: In-situ and lab tests. In *Deformation Characteristics of Geomaterials*, Vol. 2 (Proc. Lyon), Taylor & Francis, London, 155-177.

Olcott, P.G. 1992. *Ground Water Atlas of the United States, Segment 9, Iowa, Michigan, Minnesota, Wisconsin*. U.S. Geological Survey, Hydrologic Investigations Atlas 730-J.

Robertson, P.K. 1990. Soil classification using the cone penetration test. *Canadian Geotechnical Journal*, 27(1): 151-158.

Schmertmann, J.H. 1978. *Guidelines for Cone Penetration Tests: Performance and Design*. Report No. FHWA-TS-78-209, Federal Highway Administration, Washington, D.C., 145 pp.