# **Predissolution and Postdissolution Penetration Resistance**

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**Abstract:** Mineral dissolution is a common chemomechanical digenetic process in geological systems. The penetration resistance in sediments that have experienced dissolution is studied using a laboratory-scale cone penetration test device and a calibration chamber. Variables include the initial sediment density and mass fraction of soluble grains. Results show that the void ratio increases with the extent of mineral dissolution; the magnitude of the void ratio change is higher in initially dense sediments. A terminal void ratio is found for dissolution; the void ratio after dissolution will not exceed this terminal void ratio regardless of the extent of dissolution. For boundary conditions applied in this study, the terminal void ratio for dissolution corresponds to a relative density of  $Dr \approx 15\%$ , which is attained when dissolution exceeds a mass fraction loss of 10%. While the tip resistance decreases after dissolution, the drop in tip resistance is most pronounced in initially dense sands. A single penetration resistance versus density trend is observed for all tests, regardless of the changes in lateral stress and fabric that soils may have experienced as a result of mineral dissolution. **DOI: 10.1061/(ASCE)GT.1943-5606.0000949.** © 2013 American Society of Civil Engineers.

CE Database subject headings: Minerals; Cone penetration resistance; Calibration.

Author keywords: Mineral dissolution; Cone penetration resistance; Calibration chamber; Terminal density.

# Introduction

Sediments undergo either dissolution or precipitation in response to changes in hydrochemical conditions. Particle-scale dissolution is a ubiquitous diagenetic process that contributes to changes in soil structure and properties. Mineral dissolution is triggered by changes in pH and/or pore water chemistry, and continues while the pore water remains chemically undersaturated. Rain and groundwater flow are the natural drivers for these changes. Water pumping, imposed thermal changes, and pollutants can increase dissolution kinetics. The formation of karst terrain and associated ground collapse are remarkable natural examples of mineral dissolution (Waltham et al. 2005; Hunt 2007). Short-term geothermal projects, methane hydrate dissociation, and long-term dissolution in  $CO_2$  storage reservoirs are relevant examples in the energy sector (Le Guen et al. 2007; Espinoza et al. 2011).

The consequences of dissolution on sediment behavior and the characterization of sediments in view of diagenetic dissolution remain largely uninvestigated. This study explores the effects of initial density and lost mass fraction on dissolution-driven changes in density and penetration resistance for normally loaded uncemented sands. A review of available information about the consequences of mineral dissolution is presented first.

#### **Previous Studies on Mineral Dissolution**

Selective or differential mineral dissolution reflects differences in mineral solubilities and reaction kinetics (Stumm and Morgan 1996).

Minerals that form at higher temperatures and pressures are less stable under shallow, near-surface conditions (Goldich 1938). Dissolution is often accompanied by precipitation; in particular, the weathering of many rocks yield clay minerals (Eggleton et al. 1987; Schlesinger 1991; Polyak and Güven 2000). For example, kaolinite precipitates when feldspar is weathered, and metal ions are leached away (Zhu and Lu 2009). The dissolution of grains in sediments affects the sediment's physical properties. Results from previous studies are summarized next [a comprehensive study is documented in Cha (2012)].

#### Void Ratio, Coordination Number, and Compressibility

The sediment contracts yet the void ratio increases during grain dissolution (Fam et al. 2002; Truong et al. 2010). Volume contraction and changes in void ratio are proportional to the initial mass fraction of dissolvable grains (Shin and Santamarina 2009; Truong et al. 2010). The coordination number decreases after dissolution, and compressibility increases during postdissolution zero lateral strain loading (Cha 2012).

#### Skeletal Stiffness G or Vs

The shear wave velocity decreases and the material attenuation increases during dissolution (Fam et al. 2002; Truong et al. 2010). Similarly, the shear modulus G decreases in hydrate-bearing sediments after hydrate dissociation (numerically: Holtzman et al. 2008 and Jung et al. 2012; experimentally: Espinoza and Santamarina 2011). The dissolution or softening of cementing bonds in soils can not only increase compressibility but also leads to sediment collapse (Abduljauwad and Al-Amoudi 1995; Ismael and Mollah 1998; Rinaldi et al. 1998; Mansour et al. 2008, 2009).

#### Fabric-Microstructure Changes

Distinct force chains and a honeycomb fabric characterize sediments after selective mineral dissolution (Shin et al. 2008; Shin and Santamarina 2009). Load-carrying grain arches develop around the dissolving particles and local porosity increases (Cha 2012).

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Note. This manuscript was submitted on September 22, 2012; approved on April 11, 2013; published online on November 15, 2013. Discussion period open until May 1, 2014; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 139, No. 12, December 1, 2013. @ASCE, ISSN 1090-0241/2013/12-2193–2200/\$25.00.

# Stress Ratio ko

The evolution of the stress ratio  $k_0$  between the horizontal and vertical effective stresses during dissolution under zero lateral strain conditions has been investigated experimentally, analytically, and numerically. Results show that mineral dissolution causes a decrease in the coefficient of lateral stress from  $k_0$  to the active condition  $k_a$ , and internal shear planes may develop (Shin and Santamarina 2009; Cha 2012). Contraction-driven shear failures may even cause polygonal faults (Shin et al. 2008). The load-cementation-dissolution history is important. For example, consider sand cemented at a stress  $\sigma'_0$  and loaded to a higher stress  $\sigma'_1 > \sigma'_0$ ; if cementation is dissolved at a constant vertical stress  $\sigma'_1$ , the lateral stress increases to  $k_0 \cdot \sigma'_1$ , similar to that of uncemented soils (Castellanza and Nova 2004).

# Shear Resistance

Sediments that experienced dissolution exhibit a lower peak shear resistance and higher contraction than the original sediment; in fact, initially dilative soils may become contractive as the extent of dissolution increases (Fam et al. 2002; Cha 2012). A higher contractive tendency implies higher vulnerability to seismic-induced settlement and liquefaction. At large strains, the original and postdissolution specimens converge to the same critical state line. The effective shear strength parameters c' and  $\phi'$  decrease because of bond erosion in cemented sands (Ismael and Mollah 1998).

# Penetration Resistance: Previous Studies

The most common site characterization tools in geotechnical engineering are based on penetration resistance. The penetration resistance in sands reflects relative density, effective stress, and the friction angle, while soil compressibility and age have a secondary effect.

# **Relative Density**

Cone resistance is strongly affected by relative density in sandy soils (Mayne and Kulhawy 1991; Salgado et al. 1997). Consequently, cone penetration is frequently used to assess the liquefaction potential at a site (Robertson and Wride 1998; Chang et al. 2006; Mayne et al. 2010).

# Friction Angle

Cone tip resistance depends on the friction angle as predicted by bearing-capacity equations (Terzaghi 1943; Meyerhof 1963; Hansen 1970), and semiempirical relationships have been proposed to correlate friction angle and cone tip resistance (Kulhawy and Mayne 1990; Chen and Juang 1996).

#### State of Stress

Tip resistance,  $q_c$ , is more sensitive to lateral stress,  $\sigma'_h$ , than vertical effective stress (Houlsby and Hitchman 1988; Salgado et al. 1997; Ahmadi et al. 2005). The proportionality between  $q_c$  and  $\sigma'_h$  depends on the soil friction angle (Houlsby and Hitchman 1988).

#### Soil Compressibility

Sands with high compressibility produce lower cone resistance for the same relative density compared with sands with low compressibility. The compressibility of sands is controlled by grain characteristics, such as grain mineralogy and angularity, e.g., carbonate sands are more compressible than silica sands, and angular silica sands are more compressible than rounded silica sands (Ghafghazi and Shuttle 2008; Robertson 2009).

# Cementation

Extensive cementation can overshadow the effects of stress and density on cone tip resistance (Rad and Tumay 1986; Puppala et al. 1995; Lee et al. 2010).

#### Aging

The cone resistance in sand increases with time after deposition or densification (Mitchell 1986; Mesri et al. 1990; Schmertmann 1991; Baxter and Mitchell 2004; Leon et al. 2006).

#### **Other Parameters**

Other parameters affecting the cone resistance include stress history, mineralogy, grain crushing, grain size distribution, and angularity. The effect of these parameters is either small or taken into consideration through the variables listed previously.

# **Devices and Procedure**

Calibration chambers have been extensively used to establish relationships between cone resistance and soil properties. Because of boundary effects, the difference between chamber and field cone resistance values decreases as the ratio of chamber to cone diameter increases (Mayne and Kulhawy 1991; Salgado et al. 1998). Boundary effects increase with soil dilatancy, i.e., dense packing and low effective stress (Been et al. 1988; Mayne and Kulhawy 1991; Schnaid and Houlsby 1991; Salgado et al. 1998; Ahmadi and Robertson 2008). A constant lateral stress condition underestimates field  $q_c$  values, while a zero lateral strain condition overestimates field  $q_c$  values (Parkin and Lunne 1982; Iwasaki et al. 1988).



**Fig. 1.** Mineral dissolution under constant vertical effective stress at zero lateral strain conditions; spring-loaded calibration chamber facilitates penetration testing at constant vertical load

Table 1.1	Materials
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	Insoluble grains	Dissolvable grains	
Materials	Ottawa F-110 sand	Table salt	
D <sub>50</sub>	0.12	0.3	
$G_S$	2.65	2.165	
$e_{\min}$	0.535	0.45	
$e_{\rm max}$	0.848	0.78	
Roundness	0.7	Cubical	
Sphericity	0.7	—	

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The zero lateral strain calibration chamber used for this study is spring loaded to impose a constant vertical stiffness condition analogous to field situations (inner diameter = 191 mm; see Fig. 1). The spring length (152 mm) and stiffness (58 N/mm) are selected to accommodate significant deformations with minor changes in vertical

 Table 2. Specimens: Parametric Study

		Before dissolution		After dissolution	
Grain properties		Dry density $\rho_d (g/cm^3)$	Void ratio	Dry density $\rho_d (g/cm^3)$	Void ratio
No salt	Loosest	1.46	0.81	Same	Same
	Interm. 1	1.47	0.80	Same	Same
	Interm. 2	1.55	0.71	Same	Same
	Interm. 3	1.60	0.66	Same	Same
	Interm. 4	1.63	0.62	Same	Same
	Interm. 5	1.65	0.60	Same	Same
	Densest	1.66	0.59	Same	Same
5% salt	Loosest	1.48	0.77	1.48	0.79
	Interm. 1	1.54	0.70	1.50	0.76
	Interm. 2	1.63	0.61	1.56	0.70
	Densest	1.69	0.55	1.61	0.65
10% salt	Loosest	1.49	0.74	1.48	0.79
	Interm. 1	1.57	0.65	1.50	0.76
	Interm. 2	1.65	0.57	1.53	0.73
	Densest	1.70	0.53	1.55	0.70
20% salt	Loosest	1.52	0.67	1.46	0.81
	Interm. 1	1.59	0.59	1.48	0.79
	Interm. 2	1.66	0.53	1.49	0.78
	Densest	1.70	0.49	1.48	0.79

Note: Interm. = intermediate.

stress for a target vertical effective stress of 100 kPa. The bottom plate is densely grooved to ensure one-dimensional fluid flow.

The small-diameter electrical cone (outer diameter = 7.8 mm) satisfies a 24-to-1 chamber-to-cone diameter ratio to minimize boundary effects. The cone tip (apex angle =  $60^{\circ}$ ) is mounted onto a force-sensing stud to effectively determine the tip resistance (force transducer range is 0–1,200 N). The cone side resistance is mechanically removed by using an outer sleeve so that the tip resistance can be independently measured.

#### Sediment Preparation

The sediment is formed by mixing Ottawa F-110 sand (insoluble grains) and table salt (dissolvable grains). Tables 1 and 2 list grain properties and mixture proportions. Mixtures are packed at various densities. First, the mixture is placed inside the chamber by funneling, followed by successive impacts on the chamber walls to achieve the target densities. Table 2 lists mixture dry densities and initial void ratios. The specific gravities of insoluble ( $G_U = 2.65$ ) and dissolvable grains ( $G_D = 2.165$ ) are different; their weighted average is used to calculate the initial global void ratio

$$e = \frac{\rho_W}{\rho_{\rm dry}} \frac{G_U G_D}{(1 - m)G_D + mG_U} - 1$$
(1)

where m = mass fraction of dissolvable grains; and densities  $\rho_w$  and  $\rho_{dry}$  correspond to water and the dry mixture, respectively.

#### **Dissolution Procedure**

The sediment is loaded to 100 kPa. Then, while keeping the vertical stress constant, i.e., continuous adjustment to maintain the spring



**Fig. 2.** Normalized settlement (change in specimen height divided by initial specimen height) during dissolution: (a) fraction of soluble particles SF = 5%; (b) fraction of soluble particles SF = 20%



1

0.8

0.6

0.4

0.8

0.6

0.4

1.4

1.4

(c)

e<sub>max</sub>

e<sub>min</sub>

(a) SF=5%

Before

dissolution

1.5

SF=20%

1.6

Initial dry density [g/cm<sup>3</sup>]

Before

1.5

dissolution

1.6

Initial dry density [g/cm<sup>3</sup>]

After

dissolution

Q A

1.7

After

1.7

Ò

dissolution

e<sub>max</sub>

e<sub>min</sub>

# Results

#### Settlement during Dissolution

The sediment settles as it dissolves at constant vertical stress (Fig. 2). Clearly, sediments that experience more extensive dissolution settle more, where the soluble fraction (SF) is 20% compared with SF = 5%. Furthermore, specimens with a lower initial density experience larger settlement for the same fraction of soluble particles (Fig. 2). These observations hint to internal changes in the void ratio.

#### Void Ratio

1

0.8

0.6

0.4

0.9

0.8

0.7

0.6

0.5

0.4

0.4

Void ratio after dissolution

1.4

(d)

Void ratio

1.8

(b) SF=10%

Before

dissolution

1.5

SF=5%

0.5

0.6

0.7

Initial void ratio

1.6

Initial dry density [g/cm<sup>3</sup>]

After

1.7

e<sub>max</sub>

e<sub>min</sub>

0.9

0.8

1.8

dissolution

e<sub>max</sub>

 $e_{min}$ 

Changes in the void ratio and dry density after dissolution are summarized in Table 2 and plotted in Fig. 3. The void ratio increases and the dry density decreases after dissolution. Denser sediments experience a more pronounced change in density, and the increase in void ratio is higher in mixtures with the higher fraction of soluble grains. However, postdissolution void ratios do not exceed the maximum void ratio  $e_{\text{max}}$  for this sand. Note that the sand-salt mixture is a binary mixture with two different particle sizes (Table 1). Thus, the mixture can have a smaller  $e_{\min}$  than that of the sand-only sediment, as seen in Figs. 3(b and c). At a very high salt fraction, e.g., SF = 20%, the sand void ratios after dissolution are independent of the initial density or void ratio [Fig. 3(c)]. This is the terminal void ratio for dissolution under zero lateral strain; the postdissolution void ratio cannot be higher than this terminal void ratio regardless of the initial soluble fraction (Narsilio and Santamarina 2008).

#### Tip Resistance

Fig. 4 shows the cone tip resistance profiles for all 18 specimens. Note that the initial high gradient at shallow depth z < 30 mm results from upper boundary effects around the central orifice. In general, the tip resistance decreases after dissolution (see Fig. 4). The tip resistance increases with sediment density (see Fig. 4). The drop in

void ratio versus void ratio after dissolution for all cases

1.8

flushed with water before penetration as well.

The instrumented cone is driven into the sediment at a constant speed of 12.5 mm/min using a multipurpose loading frame (Humboldt Manufacturing Company, Schiller Park, IL). This low penetration velocity guarantees drained conditions. The vertical stress is kept constant during penetration.

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**Fig. 4.** Characteristic penetration profiles; tip resistance versus depth for different degrees of dissolution and different densities; initial soluble fraction: (a) SF = 0%; (b) SF = 5%; (c) SF = 10%; (d) SF = 20%

tip resistance increases with the extent of dissolution, and it is most pronounced when comparing the penetration resistance in initially dense soils to the penetration resistance in the same soil after dissolution [see Figs. 4(b–d) compared with Fig. 4(a)]. No appreciable changes in tip resistance are observed after dissolution in initially loose sediments.

#### Analyses and Discussion

#### Tip Resistance versus Final Void Ratio

The mean values of penetration resistance in the steady lower 2/3 of the profile are plotted together in Fig. 5(a). A single trend is observed

when the tip resistance is plotted versus void ratio at the time of penetration [Fig. 5(b)]. This suggests that penetration resistance is primarily a measure of density at the time of penetration.

Previously proposed equations for penetration resistance  $q_t$  as a function of relative density  $D_R$  are superimposed on Fig. 5(b)

$$D_R = 100 \cdot \left[ 0.268 \cdot \ln\left(\frac{q_t/\sigma_{\text{atm}}}{\sqrt{\sigma'_{vo}/\sigma_{\text{atm}}}}\right) - 0.675 \right]$$
(2)

$$D_R = 100 \cdot \sqrt{\frac{q_t/\sigma_{\rm atm}}{300 \cdot \text{OCR}^{0.2} \sqrt{\sigma'_{vo}/\sigma_{\rm atm}}}}$$
(3)

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**Fig. 5.** Void ratio versus tip resistance (lower 2/3-depth profile in Fig. 4); tip resistance plotted versus: (a) initial void ratio; (b) void ratio after dissolution, i.e., at time of insertion

$$D_R = -98 + 66 \cdot \log_{10} \frac{q_c}{(\sigma'_{vo})^{0.5}} \quad (q_c \text{ and } \sigma'_{vo} \text{ in } t/\text{m}^2)$$
 (4)

where  $\sigma_{atm}$  = atmospheric pressure used here for normalization. These trends agree with postdissolution experimental results obtained in this study and confirm the prevalent effect of void ratio on penetration resistance (Jamiolkowski et al. 2003, 1985; Kulhawy and Mayne 1990).

The single trend between cone resistance and void ratio at the time of penetration indicate that cone resistance is insensitive to other dissolution effects such as changes in soil fabric and horizontal stress. In part, this confirms that the initial fabric does not significantly affect large-strain shear strength. High postdissolution void ratios minimize the effect of horizontal stress on postdissolution cone resistance even as the horizontal stress may change between  $k_0 = 0.3$  and  $k_0 = 0.65$  during dissolution. This range applies to normally consolidated soils; changes depend on the initial soluble fraction and grain size reduction (Shin et al. 2008; Shin and Santamarina 2009; Cha 2012).

# **Terminal Void Ratio**

Soils reach a characteristic terminal density or void ratio for every repetitive or large-strain process (Narsilio and Santamarina 2008). The results in Fig. 3 suggest that dissolution has its own associated terminal density. Void ratios at the end of dissolution are plotted versus the normalized settlement (change in specimen height divided by initial specimen height) experienced during dissolution in



**Fig. 6.** Final void ratio versus normalized settlement: terminal void ratio; extreme and terminal void ratios  $e_{\min}$ ,  $e_{\max}$ , and  $e_{term}$  correspond to 100% sand

Fig. 6. Extreme void ratios,  $e_{\text{max}}$  and  $e_{\text{min}}$ , are shown for reference; the terminal void ratio  $e_{\text{term}} \approx 0.80 \pm 0.01$  ( $Dr \approx 15\%$ ) for dissolution under the test conditions imposed in this study. Furthermore, the data suggest that SF > 10% is needed to achieve the terminal void ratio; the initial density does not affect the final void ratio in sediments with soluble fractions above this threshold. As corollaries, (1) sediments with a soluble fraction higher than the threshold fraction eventually settle at a constant void ratio and proportional to the extent of dissolution; and (2) most of the mass loss leads to a gain in porosity, and the global settlement is small when the soluble fraction is smaller than the threshold fraction.

# Conclusions

Mineral dissolution is a ubiquitous diagenetic process. The analysis of predissolution and postdissolution penetration resistance in normally loaded and uncemented clean sands shows that

- Void ratio increases because of mineral dissolution. The change in void ratio is more pronounced in initially dense sediments. There is a terminal density or void ratio for dissolution, and the void ratio after dissolution will not exceed this terminal void ratio regardless of the extent of dissolution. For the conditions of this study, the terminal void ratio for dissolution corresponds to a relative density of  $Dr \approx 15\%$  and is attained when dissolution exceeds a mass fraction loss of SF > 10% (probably closer to 20%).
- Mineral dissolution decreases the cone tip resistance. The drop in tip resistance increases with the extent of dissolution, is most pronounced in dense sands, and is minimal for initially loose sands.
- There is a single trend in penetration resistance versus void ratio at the time of penetration. Fabric and changes in lateral stress that accompany mineral dissolution do not seem to affect this trend.

#### Acknowledgments

Support for this research was provided by the Department of Energy Savannah River Operations Office and the Goizueta Foundation.

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# References

- Abduljauwad, S. N., and Al-Amoudi, O. S. B. (1995). "Geotechnical behaviour of saline sabkha soils." *Geotechnique*, 45(3), 425–445.
- Ahmadi, M. M., Byrne, P. M., and Campanella, R. G. (2005). "Cone tip resistance in sand: Modeling, verification, and applications." *Can. Geotech. J.*, 42(4), 977–993.
- Ahmadi, M. M., and Robertson, P. K. (2008). "A numerical study of chamber size and boundary effects on cpt tip resistance in NC sand." *Sci. Iranica*, 15(5), 541–553.
- Baxter, C. D. P., and Mitchell, J. K. (2004). "Experimental study on the aging of sands." J. Geotech. Geoenviron. Eng., 130(10), 1051–1062.
- Been, K., Crooks, J. H. A., and Rothenburg, L. (1988). "A critical appraisal of CPT calibration chamber tests." *Proc.*, *1st Int. Symp. on Penetration Testing, ISOPT-1*, Vol. 1, Elsevier, New York, 651–661.
- Castellanza, R., and Nova, R. (2004). "Oedometric tests on artificially weathered carbonatic soft rocks." J. Geotech. Geoenviron. Eng., 130(7), 728–739.
- Cha, M. (2012). "Mineral dissolution in sediments." Ph.D. thesis, Georgia Institute of Technology, Atlanta.
- Chang, M. F., Yu, G., Na, Y. M., and Choa, V. (2006). "Evaluation of relative density profiles of sand fill at a reclaimed site." *Can. Geotech. J.*, 43(9), 903–914.
- Chen, J. W., and Juang, C. H. (1996). "Determination of drained friction angle of sands from CPT." J. Geotech. Engrg., 122(5), 374–381.
- Eggleton, R. A., Foudoulis, C., and Varkevisser, D. (1987). "Weathering of basalt - Changes in rock chemistry and mineralogy." *Clays Clay Miner.*, 35(3), 161–169.
- Espinoza, D. N., Kim, S. H., and Santamarina, J. C. (2011). "CO<sub>2</sub> geological storage - Geotechnical implications." *KSCE J. Civ. Eng.*, 15(4), 707–719.
- Espinoza, D. N., and Santamarina, J. C. (2011). "P-wave monitoring of hydrate-bearing sand during CH<sub>4</sub>-CO<sub>2</sub> replacement." *Int. J. Greenh. Gas Control*, 5(4), 1031–1038.
- Fam, M. A., Cascante, G., and Dusseault, M. B. (2002). "Large and small strain properties of sands subjected to local void increase." *J. Geotech. Geoenviron. Eng.*, 128(12), 1018–1025.
- Ghafghazi, M., and Shuttle, D. (2008). "Interpretation of sand state from cone penetration resistance." *Geotechnique*, 58(8), 623–634.
- Goldich, S. S. (1938). "A study in rock-weathering." J. Geol., 46(1), 17-58.
- Hansen, J. B. (1970). "A revised and extended formula for bearing capacity." *Bulletin No.* 28, Danish Geotechnical Institute, Copenhagen, Denmark.
- Holtzman, R., Silin, D. B., and Patzek, T. W. (2008). "Micromechanics of hydrate dissociation in marine sediments by grain-scale simulations." *SPE Western Regional and Pacific Section AAPG Joint Meeting*, Society of Petroleum Engineers, Richardson, TX.
- Houlsby, G. T., and Hitchman, R. (1988). "Calibration chamber tests of a cone penetrometer in sand." *Geotechnique*, 38(1), 39–44.
- Hunt, R. E. (2007). Geologic hazards: A field guide for geotechnical engineers, CRC, Boca Raton, FL.
- Ismael, N. F., and Mollah, M. A. (1998). "Leaching effects on properties of cemented sands in Kuwait." J. Geotech. Geoenviron. Eng., 124(10), 997–1004.
- Iwasaki, K., Tanizawa, F., Zhou, S., and Tatsuoka, F. (1988). "Cone resistance and liquefaction strength of sand." *Proc., 1st Int. Symp. on Penetration Testing.*, A. A. Balkema, Rotterdam, Netherlands, 785–791.
- Jamiolkowski, M., Ladd, C. C., Germaine, J. T., and Lancellotta, R. (1985). "New developments in field and laboratory testing of soils." *Proc.*, 11th Int. Conf. on Soil Mechanics and Foundation Engineering, Vol. 1, A. A. Balkema, Rotterdam, Netherlands, 57–153.
- Jamiolkowski, M., Lo Presti, D. C. F., and Manassero, M. (2003). "Evaluation of relative density and shear strength of sands from CPT and DMT." *Soil behavior and soft ground construction*, J. T. Germaine, T. C. Sheahan, and R. V. Whitman, eds., ASCE, New York, 201–238.

- Jung, J. W., Santamarina, J. C., and Soga, K. (2012). "Stress-strain response of hydrate-bearing sands: Numerical study using discrete element method simulations." J. Geophys. Res. Solid Earth, 117(B4).
- Kulhawy, F. H., and Mayne, P. W. (1990). "Manual on estimating soil properties for foundation design." *Rep. EPRI EL-6800*, Electric Power Research Institute, Palo Alto, CA.
- Le Guen, Y., et al. (2007). "Enhanced deformation of limestone and sandstone in the presence of high P<sub>CO2</sub> fluids." J. Geophys. Res. Solid Earth, 112(B5), 1–21.
- Lee, M.-J., Choo, H., Kim, J., and Lee, W. (2010). "Effect of artificial cementation on cone tip resistance and small strain shear modulus of sand." *Bull. Eng. Geol. Environ.*, 70(2), 193–201.
- Leon, E., Gassman, S. L., and Talwani, P. (2006). "Accounting for soil aging when assessing liquefaction potential." J. Geotech. Geoenviron. Eng., 132(3), 363–377.
- Mansour, Z., Chik, Z., and Taha, M. R. (2008). "On soil collapse potential evaluation." *Proc., Int. Conf. on Construction and Building Technology*, International Association for Concrete Technology, Kuala Lumpur, Malaysia, 21–32.
- Mansour, Z. M., Taha, M. R., Chik, Z., and Kasa, A. (2009). "Leaching of Lisan Marl and its consequences on the engineering behavior of dikes, studied by finite elements modeling." *Environ. Earth Sci.*, 59(4), 891–900.
- Mayne, P. W., and Kulhawy, F. H. (1991). "Calibration chamber database and boundary effects correction for CPT data." *Proc.*, 1st Int. Symp. on Calibration Chamber Testing (ISOCCTI), Elsevier, New York, 257– 264.
- Mayne, P. W., Peuchen, J., and Bouwmeester, D. (2010). "Estimation of soil unit weight from CPTs." *Proc.*, 2nd Int. Symp. on Cone Penetration Testing, Vol. 2, Taylor & Francis, London, 371–376.
- Mesri, G., Feng, T. W., and Benak, J. M. (1990). "Postdensification penetration resistance of clean sands." J. Geotech. Engrg., 116(7), 1095–1115.
- Meyerhof, G. G. (1963). "Some recent research on the bearing capacity of foundations." *Can. Geotech. J.*, 1(1), 16–26.
- Mitchell, J. K. (1986). "Practical problems from surprising soil behavior." J. Geotech. Engrg., 112(3), 255–289.
- Narsilio, G. A., and Santamarina, J. C. (2008). "Terminal densities." Geotechnique, 58(8), 669–674.
- Parkin, A. K., and Lunne, T. (1982). "Boundary effects in the laboratory calibration of a cone penetrometer for sand." *Proc., 2nd European Symp. of Penetration Testing*, Norwegian Geotechnical Institute, Vol. 2, Oslo, Norway, 761–768.
- Polyak, V. J., and Güven, N. (2000). "Clays in caves of the Guadalupe Mountains, New Mexico." J. Caves Karst Stud., 62(2), 120–126.
- Puppala, A. J., Acar, Y. B., and Tumay, M. T. (1995). "Cone penetration in very weakly cemented sand." J. Geotech. Engrg., 121(8), 589–600.
- Rad, N. S., and Tumay, M. T. (1986). "Effect of cementation on the cone penetration resistance of sand: A model study." *Geotech. Test. J.*, 9(3), 117–125.
- Rinaldi, V. A., Redolfi, E. R., and Santamarina, J. C. (1998). "Characterization of collapsible soils with combined geophysical and penetration testing." *Proc., 1st Int. Conf. on Site Characterization* (*ISC* 98), A. A. Balkema, Rotterdam, Netherlands, 581–588.
- Robertson, P. K. (2009). "Interpretation of cone penetration tests A unified approach." *Can. Geotech. J.*, 46(11), 1337–1355.
- Robertson, P. K., and Wride, C. E. (1998). "Evaluating cyclic liquefaction potential using the cone penetration test." *Can. Geotech. J.*, 35(3), 442–459.
- Salgado, R., Mitchell, J. K., and Jamiolkowski, M. (1997). "Cavity expansion and penetration resistance in sand." J. Geotech. Geoenviron. Eng., 123(4), 344–354.
- Salgado, R., Mitchell, J. K., and Jamiolkowski, M. (1998). "Calibration chamber size effects on penetration resistance in sand." *J. Geotech. Geoenviron. Eng.*, 124(9), 878–888.
- Schlesinger, W. H. (1991). Biogeochemistry: An analysis of global change, Academic, San Diego, 443.
- Schmertmann, J. H. (1991). "The mechanical aging of soils." J. Geotech. Engrg., 117(9), 1288–1330.

- Schnaid, F., and Houlsby, G. T. (1991). "An assessment of chamber size effects in the calibration of in situ tests in sand." *Geotechnique*, 41(3), 437–445.
- Shin, H., and Santamarina, J. C. (2009). "Mineral dissolution and the evolution of k<sub>0</sub>." J. Geotech. Geoenviron. Eng., 135(8), 1141–1147.
- Shin, H., Santamarina, J. C., and Cartwright, J. A. (2008). "Contractiondriven shear failure in compacting uncemented sediments." *Geology*, 36(12), 931–934.
- Stumm, W., and Morgan, J. J. (1996). Aquatic chemistry: Chemical equilibria and rates in natural waters, Wiley, New York, 1022.

Terzaghi, K. (1943). Theoretical soil mechanics, Wiley, New York.

- Truong, Q. H., Eom, Y. H., and Lee, J. S. (2010). "Stiffness characteristics of soluble mixtures." *Geotechnique*, 60(4), 293–297.
- Waltham, T., Bell, F., and Culshaw, M. (2005). Sinkholes and subsidence: Karst and cavernous rocks in engineering and construction, Springer, Chichester, U.K., 382.
- Zhu, C., and Lu, P. (2009). "Alkali feldspar dissolution and secondary mineral precipitation in batch systems: 3. Saturation states of product minerals and reaction paths." *Geochim. Cosmochim. Acta*, 73(11), 3171–3200.