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Authors

Murphy, Kyle D
McCartney, John S

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Thermal Borehole Shear Device

Kyle D. Murphy, M.S., E.I.T.

Engineer,
Shannon and Wilson, Denver, CO, USA,
kdm@shanwil.com.

John S. McCartney, Ph.D., P.E.

Associate Professor and Lyall Faculty Fellow,
Department of Civil, Environmental, and Architectural Engineering,
University of Colorado Boulder, Boulder, CO, USA,
john.mccartney@colorado.edu.

12ABSTRACT

13 This paper presents the details of a new modified borehole shear device that has the
14 capability of measuring the impact of temperature on the in-situ shear stress-displacement curves
15 for soil-concrete interfaces. The thermal borehole shear device incorporates concrete shoes with
16 embedded heaters, a pneumatic loading device for application of horizontal normal stresses, and
17 an automated loading system with local vertical displacement and load measurement systems
18 that permits either displacement-control or load-control testing. A methodology for measurement
19 of the soil-concrete shear stress-displacement curves and for evaluation of the drained interface
20 shear strength failure envelopes at different temperatures is presented in this study. Typical
21 results from proof-of-concept tests performed in a clay layer compacted in a laboratory tank in a
22 borehole in a silty sand deposit in the field are presented in this paper. The results are synthesized
23 to show how the impacts of temperature and normal stress on the normalized shear stress-
24 displacement curves can be evaluated. These normalized curves can be measured on a site-
25 specific basis for the calibration of thermo-mechanical load transfer analyses or finite element
26 analyses, which are often used to design and evaluate soil-structure interaction in drilled shaft
27 foundations with geothermal heat exchangers (energy foundations).

28 **Keywords:** Modified borehole shear test; load transfer; energy foundations, soil-concrete
29 interface; temperature effects

30

31INTRODUCTION

32 An emerging field of study within geotechnical engineering is the use of geotechnical
33infrastructure like deep foundations as geothermal heat exchangers to transfer thermal energy
34from the subsurface for heating and cooling of buildings (Brandl 2006). A side effect of the heat
35transfer process is the potential for foundation movements due to thermal expansion and
36contraction of the foundation element or surrounding soil. Further, soil-structure interaction may
37restrain foundation movements, leading to generation of thermally induced stresses within the
38reinforced concrete. The mechanisms of thermo-mechanical soil-structure interaction in energy
39foundations have been documented in several full-scale case histories in the field (Laloui et al.
402006; Bourne-Webb et al. 2009; Laloui 2011; Bouazza et al. 2011; Amatya et al. 2012;
41McCartney and Murphy 2012; Murphy et al. 2014) and in centrifuge-scale experimental studies
42(McCartney and Rosenberg 2011; Stewart and McCartney 2013). Although these studies
43generally indicated that adverse conditions to foundation performance will not be encountered
44during heat exchange processes, they indicated that it is important to consider the role of thermo-
45mechanical soil-structure interaction in the design of energy foundations. This is especially the
46case when the energy foundations are being installed in challenging stratigraphy, such as
47expansive soils, unsaturated soils, soft, normally-consolidated soils, or layered soil deposits.

48 Thermo-mechanical soil-structure interaction phenomena have been predicted using modified
49load transfer analyses (McCartney and Rosenberg 2011; Knellwolf et al. 2011) and thermo-
50elastic finite element analyses (Laloui et al. 2006; Regueiro et al. 2012). Although these analyses
51are capable of predicting changes in axial displacement, strain, and stress in energy foundations
52during heating and cooling, they rely upon the accurate characterization of the effects of
53temperature and stress state on the shear stress-displacement behavior of the soil-concrete

54interface. At the moment, most load-transfer analyses assume that temperature does not have a
55significant impact on the shear stress-displacement response of the soil-concrete interface
56(McCartney and Rosenberg 2011; Knellwolf et al. 2011). Although one approach to characterize
57the soil-concrete interface behavior is to perform nonisothermal direct shear tests in the
58laboratory (Xiao et al. 2014), the role of in-situ boundary conditions and soil structure cannot be
59completely considered in these tests. Accordingly, the objective of this study is to develop a new
60field test to characterize the soil-concrete interface shear stress-displacement curves under
61different temperatures and normal stresses so that they may be used to define relationships that
62can be used to analyze soil-structure interaction phenomena in energy foundations. Specifically,
63a modified borehole shear device was developed with concrete shearing plates (shoes) to
64simulate the interface between soil and a cast-in-place reinforced concrete deep foundation. The
65approach to use a modified borehole shear device has been used by Audibert and Aggarwal
66(1982) to define the isothermal shear-stress displacement curves for drilled shafts with concrete
67shoes and by AbdelSalam et al. (2012) to define curves for steel friction piles using steel shoes.
68Different from the concrete shoes developed by Audibert and Aggarwal (1982), the shoes
69developed in this study incorporate embedded copper tubing heat exchangers that can be used to
70control the temperature of the interface. This paper presents the results and analyses of a series of
71borehole shear tests conducted in soil layers at different temperatures and normal stresses in
72laboratory and field settings.

73**BACKGROUND**

74*Soil-Structure Interaction*

75 Soil-structure interaction phenomena are expected during axial loading of deep foundations
76due to the differences in stiffness between the soil and foundation and mobilization of interface

77shear stresses. Coyle and Reese (1966) recognized the need for an analytical method to predict
78the load carrying capacity and load-settlement response for axially-loaded deep foundations.
79Their predictive technique, referred to as load-transfer analysis, divides a deep foundation into
80several rigid elements connected by elastic springs. A load transfer analysis is performed by
81ensuring equilibrium and compatibility of displacements between the foundation and the soil,
82which is represented by soil springs with nonlinear stress-displacement curves. Specifically, the
83curves representing the interface shear stress-displacement curve at different depths are referred
84to as T-z curves, while the curve representing the end bearing stress-displacement curve at the
85toe is referred to as a Q-z curve. Thermo-mechanical load transfer analyses such as the one
86developed by Knellwolf et al. (2011) use the same concepts as those developed by Coyle and
87Reese (1966), but they also consider soil-structure interaction in the axial direction arising from
88the constrained thermal expansion and contraction of the reinforced concrete during heating or
89cooling, respectively. The radial expansion and contraction of energy foundations during heating
90and cooling, respectively, may change the normal stress at the soil-foundation interface.
91McCartney and Rosenberg (2011) proposed an empirical equation to consider the impact of
92temperature on the normal stress on the interface, and assumed that the normalized T-z curve did
93not depend on temperature. However, this assumption has not been verified using experimental
94results. Regardless if the foundation is heated or not, the T-z curve is a critical input that must be
95defined using either a database of normalized curves or experimentally using nonisothermal
96interface direct shear tests or in-situ tests.

97*Temperature Effects on the Volume Change and Shear Strength of Soils*

98 As soil is heated under drained conditions, elastic and plastic volume changes may occur due
99to the expansion and contraction of the soil particles and pore water. Campanella and Mitchell

100(1968) and Paaswell (1967) found that the primary mechanism for plastic volumetric strains
101during heating of saturated soils is the differential expansion of the water and soil particles.
102Specifically, the coefficient of thermal expansion of water is approximately seven to ten times
103greater than that of most soil minerals (McKinstry 1965; Mitchell and Soga 2005). For normally
104consolidated soils, the differential expansion between the soil and water during heating will lead
105to positive excess pore water pressures, which will subsequently drain and cause plastic
106volumetric contraction (Graham et al. 2001; Cekeravac and Laloui 2004). If the soil were
107sheared without allowing drainage of the thermally-induced excess pore water pressure, the
108undrained shear strength would decrease due to the lower effective stress. However, if the soil
109were sheared after drainage of the thermally-induced excess pore water pressure the shear
110strength should increase due to the decrease in void ratio. The stress state also plays an important
111role in the thermal volume change of soils. For example, in saturated soils with
112overconsolidation ratios (OCRs) greater than 3, elastic expansion is observed during heating and
113excess pore water pressures are not as significant as in normally consolidated conditions, leading
114to a negligible change in undrained shear strength (Paaswell 1967; Plum and Esrig 1969; Demars
115and Charles 1982; Baldi et al. 1988; Hueckel and Baldi 1990; Hueckel and Pellegrini 1992;
116Towhata et al. 1993; Delage et al. 2000). Uchaipichat and Khalili (2009) observed that
117unsaturated soils have similar volume change behavior to saturated soils when heated under
118normally consolidated or overconsolidated conditions. The tests performed in this study focus on
119measurement of the soil-concrete interface shear stress-displacement curve after reaching
120thermal equilibrium and do not consider the role of thermally induced volume change during
121heating. However, this is a subject that could be investigated in the future with this device.

122 Although the dissipation of excess pore water pressure after heating may have a considerable
123 impact on the volume change and undrained shear strength of soils, temperature changes are not
124 expected to have a major effect on the drained frictional properties of soils. Laloui (2001)
125 summarized the results from several studies on different soils and observed that the friction angle
126 at critical state is relatively insensitive to changes in temperature, except for a few studies on
127 saturated Kaolinite which showed observed a decreasing trend in friction angle with increasing
128 temperature. Uchaipichat and Khalili (2009) observed that temperature had an effect on the peak
129 shear strength of unsaturated silt, but not on the shear strength at critical state conditions or the
130 friction angle at critical state. They also observed that suction has a greater effect on the
131 undrained shear strength than temperature.

132 *Shear Stress-Displacement Behavior of Interfaces*

133 The behavior of interfaces between soil and concrete, metal, or geosynthetics is an important
134 subject in geotechnical engineering. In this study, the interface between soil and the concrete of a
135 drilled shaft foundation is of interest. If the interface is very rough, it is expected that the
136 interface friction angle will be greater than that of the soil, which will force the failure to occur
137 in the soil (Kulhawy and Peterson 1979). However, for the case of a relatively smooth drilled
138 shaft foundation, it is likely that the interface friction angle is less than that of the soil, making
139 the interface a plane of weakness that should be carefully characterized. Although it is expected
140 that the interface will only have a frictional component in drained conditions (no adhesion), it is
141 also possible to define a failure envelope defined over a certain normal stress range that has an
142 adhesion value due to nonlinear trends in the failure envelope.

143 Several laboratory studies have been conducted to quantify the parameters affecting the soil-
144 pile interface shear strength. Early studies used a modified direct shear apparatus in which

145concrete or steel occupied one of the halves of the shear box (Potyondy 1961; Peterson et al.
1461976; Kulhawy and Peterson 1979). These studies involved tests that were performed over a
147range of normal stresses and soil conditions to determine factors relating to the shear strength of
148a given interface. They observed that the interface friction angle depends on grain size and
149distribution, soil particle angularity, soil water content, type of interface material and its texture,
150and the normal stress on the interface. The peak interface friction angle increases as the
151roughness of the artificial material increases, and may potentially move into the surrounding soil
152for very rough interfaces. Jardine et al. (1993) indicated that constrained conditions may occur
153during drained axial loading of foundations in sand and silt. The constrained dilation that occurs
154in this case may lead to a greater peak shear strength than in constant normal stress conditions.
155However, the displacements that occur during thermal expansion and contraction of energy
156foundations are relatively small (Murphy and McCartney 2014), and may not be sufficient for
157dilation to occur.

158 Although temperature effects on the undrained shear strength of soils have been widely
159characterized, only a few studies had been conducted on the thermal effects on the interface shear
160strength. Xiao et al. (2014) used a direct shear device with a concrete lower box with an
161embedded heat exchanger, and observed that heating led to a slight increase in the shear strength
162of the interface with a compacted silt. However, it also led to a slight decrease in the water
163content, potentially to thermally induced water flow away from the interface.

164 Depending on the drilled shaft construction process and the soil type in the field, it is possible
165for drilled shafts to have smooth, rough or even irregular surfaces. As the shoes in this study
166consist of uniform concrete roughened with coarse sand paper, the results from the modified
167borehole shear device are expected to be most representative of drilled shafts in stiff soil deposits

168constructed using the dry method (O'Neill and Reese 1999). In this case, the sides of the hole are
169expected to be relatively smooth and can be inspected visually.

170***Borehole Shear Device***

171 The borehole shear test (BST) device was developed by Professor R.L. Handy in the 1960's
172and was originally used to measure the in-situ drained shear strength properties of soils (Handy
173and Fox 1967). The test involves insertion of a bi-lateral expanding shear head into a borehole at
174a specified depth, after which a piston extends outward resulting in contact between two grooved
175semi-circular steel plates and the soil. The grooved plates engage the soil so the shear plane is
176fully in the soil and not at the soil-plate interfaces. Because the test must be performed in an
177open borehole, this device is best suited for soil strata that have sufficient stiffness such that the
178borehole will remain open through the duration of the test (at least eight hours). Boreholes in
179loose sands or soft clays will usually not stay open without the use of drilling mud, so they are
180not well suited for this test.

181 General borehole shear testing procedures are provided by Lutenecker (1987) and Handy
182(2002). In most tests, the normal stress is applied to the interface and 5-15 minutes are permitted
183for dissipation of excess pore pressure so that drained conditions are obtained. This is assumed to
184be a reasonable time period for consolidation of stiff soils that do not change significantly in
185volume during application of stresses close to the in-situ horizontal stress. After consolidation,
186an upward force is applied to the shear head in displacement-control conditions, which causes
187shearing within the soil due to the grooved shoes. A displacement rate of 0.05 mm/s is typically
188used during shearing, which is assumed to correspond to drained pullout conditions. The value of
189peak shear stress is typically the main outcome of the test, although some studies have reported
190the shear stress-displacement curves. After failure, the shear head is then lowered to relax the

191shear stress back to zero, after which a higher normal stress is applied to the device and the
192process is repeated. A succession of tests over a range of normal forces allows definition of a
193drained failure envelope. Several improvements have been made to the borehole shear test over
194time, and it has been used in many applications to study different aspects of the in-situ shear
195strength and compressibility behavior of soils (Lutenegger et al. 1978; Demartinencourt and
196Bauer 1983; Lutenegger and Tierney 1986; Handy 1986; White and Handy 2001; Ashlock and
197Lu 2012).

198 Borehole shear testing has also been used to interpret the uplift behavior of drilled shaft
199foundations (Lutenegger and Miller 1994). Although Lutenegger and Miller (1994) did not
200create concrete shoes to evaluate the soil-concrete interface shear strength, they found that using
201the BST approach to characterize the soil shear strength has the added advantage that the drilling
202process of a hole for borehole shear testing involves a similar excavation method as for a drilled
203shaft and may possess similar irregularities in texture and roughness in both cases. The
204mechanism and location of the failure surface of a concrete BST shear head and a drilled shaft
205may be similar assuming the drilled shaft fails as a result of side shear mobilization. The device
206can also be used to investigate the behavior of driven piles using shoes that simulate the material
207type and roughness of the steel or concrete used in the piles. Suleiman et al. (2011) developed a
208modified borehole shear device to measure the shear stress-displacement behavior of a steel pile
209interface with soil. The shear head of the BST utilized smooth steel plates instead of plates with
210ridges, and was instrumented such that shear stress and displacement of the shear head were
211recorded during the testing process. Results from this study indicated that T-z curves measured
212using a borehole shear device are comparable to data obtained from modified direct shear testing

213and are much more reliable than load-displacement curves developed using data from cone
214penetration tests (AbdelSalam et al. 2012).

215**EXPERIMENTAL SETUP**

216*Overview*

217 A schematic of the overall thermal borehole shear device setup is shown in Figure 1. The
218system includes a loading frame that is placed above a borehole, which is used to pull vertically
219upward on the borehole shear device. The loading frame developed in this study incorporates
220many of the advances in automation of the modified borehole shear device developed by
221Bechtum (2012). The thermal borehole shear head consists of a pair of concrete shoes with
222embedded heat exchangers that are sandwiched around a pneumatic piston that is used to control
223the normal stress between the shoes and the sides of the borehole. The shear head is connected to
224the load frame through a series of steel extension rods. The system also includes a heat pump
225used to control the temperature of the shoes, a data acquisition system, and instrumentation to
226measure the vertical displacement of the shoes, the temperature of the shoes, and the vertical
227force.

228*Thermal Borehole Shear Head*

229 The components of the thermal borehole shear head include the two concrete shoes that are
230sandwiched around a pneumatic piston from Fabco, Inc., as shown in the plan-view schematic in
231Figure 2. Normal stresses can be applied by supplying a controlled air pressure to the pneumatic
232piston when the shoes are in contact with the side of the borehole. The pneumatic piston has a
233stroke of 25 mm. This stroke is sufficient to attain contact with the sides of a borehole having a
234diameter of 105-115 mm, while still permitting additional displacement for consolidation. Shear
235stresses are applied to the shoes by pulling vertically on the extension rods attached to the load

236frame. These extension rods are connected to a U-shaped yoke, which is connected to each of the
237shoes through steel loading straps, as shown in Figure 3. This device is best suited for relatively
238stiff soils as large horizontal displacements (i.e., during application of high normal stresses or in
239soft soils) may cause undesirable moments within the device during application of vertical
240stresses to the device.

241 The two concrete shoes were fabricated using a 1:2:3 water:cement:sand mortar mix ratio.
242The mortar mix was poured into a modified concrete cylinder mold, shown in Figure 4, so that
243the shoes form a 101.6 mm-diameter circular segment having a thickness of approximately
24425 mm. The concrete is reinforced using a 19-gauge steel wire hardware cloth with 12.7 mm
245openings. A copper heat exchange tubing is bent into a U-shape and is centered within the
246reinforcing cage. A K-type thermocouple was also attached to the reinforcing cage near the
247surface of the shoe facing the soil to monitor the temperature of the shoes. Bolts were pre-
248installed through the concrete shoes so that they could be attached to the pneumatic piston and
249loading strap after the concrete cured. A greased wooden form was used with the modified
250concrete cylinder to create the concrete shoe with the desired shape, as shown in Figure 4. The
251top edges of the shoes were rounded down using sand paper to avoid a sharp edge that may
252penetrate into the soil and lead to formation of a passive wedge. It was preferred to maintain a
253known, uniform contact area, so the top edge of the shoes was not tapered. Inspection of the
254shoes after shearing in the relatively stiff soils tested in this study showed that no soil
255accumulated on the top of the shoes. However, the fact that there is no taper on the top edge may
256cause passive bearing problems when testing in soft clays. After curing the surface of the
257concrete shoes were roughened using 40-grit sandpaper to avoid having a smooth interface from

the plastic concrete cylinder. As mentioned, this surface treatment is expected to represent the interface between a stiff soil and a drilled shaft constructed using the dry method.

After assembly of the shoe-strap-pneumatic system assembly (referred to as the shear head) and when the piston is fully retracted, the diameter of the shear head is 101.6 mm. Perfluoroalkoxy (PFA) tubing with an inside diameter of 6.35 mm was used to supply the air to the pneumatic piston from a pressure regulator at the ground surface, with one line used for extending the piston and the other used for retracting the piston. The same type of tubing was also used to supply the heat exchange fluid to the copper tubing to control the temperature of the concrete shoes.

Load Frame and Instrumentation

The load frame is constructed out of two horizontal aluminum plates separated by three threaded rods. The dimensions and layout of the plates are shown in Figure 6. The threaded rods separating the two plates are useful because they permit the top plate to be positioned at various heights, and also act as a leveling mechanism to ensure the extension rods are vertical. A Haydon Kerk size 34 linear actuator and motor controller is used to apply the vertical load to the extension rods, while the loads applied to the extension rods are measured using a 4.5 kN load cell obtained from Futek, Inc. The bottom plate has a large cut-out on one side. This permits the thermal borehole shear head and extension rods to be placed into the borehole first, after which the loading frame can be moved over the hole to connect the linear actuator to the extension rods. The motor and data collection system are both connected to an onsite laptop for automatic control using National Instruments LabView software.

A linear string potentiometer with a 100-gram tension spring was used to measure the vertical (shear) displacement of the concrete shoes during application of axial loads. Although it

281is possible to infer the vertical displacements using the linear actuator motor counts, in a similar
282manner to the manual counter in the original borehole shear device (Handy 2002), preliminary
283tests indicated that the displacements inferred using this approach were affected by slack in the
284connections between the extension rods and elastic expansion of the rods. The potentiometer is
285connected to the exposed hardware cloth of one concrete shoe with braided fishing line, so it
286provides an accurate local measurement of shear displacement. The fishing line is passed over a
287low friction pulley connected to a sliding bar, which permits the string to be vertically oriented in
288the borehole. Pipe-plug thermocouples are used to monitor the inlet and outlet heat exchange
289fluid temperatures, which is useful to evaluate the heat flux into the system. However, the
290temperature of the shoes were also monitored using a thermocouple embedded in the concrete
291shoes.

292*Temperature Control System*

293 The temperature of the concrete shoes is controlled by circulating a heat exchange fluid
294having a constant temperature through the copper tubing embedded in the shoes. Water is used as
295the heat exchange fluid in this study as only heating was investigated. A Julabo F25-ME heat
296pump was used to control the temperature of the heat exchange fluid, and also contains a flow
297pump to circulate the fluid through the closed-loop heat exchange tubing. The heat pump
298contains a 4.5 liter bath with automated temperature control system for heating or cooling. The
299heat pump is capable of controlling the temperature of the heat exchange fluid within a range of -
30028 to 200 °C, although a range of 5 to 50 °C was used in this study. Because heat is lost through
301the tubing and through the concrete shoe, the temperature set on the heat pump is not necessarily
302the same as that of the shoes. Accordingly, the temperature measured in the shoes near the
303interface at steady-state conditions was assumed to represent the temperature at the soil-concrete

304interface. A picture of the thermal borehole shear device during a test in the field is shown in
305Figure 7. This picture shows all of the components of the system, including the loading frame,
306data acquisition board, pneumatic pressure control board, and the heat pump. Long lengths of
307tubing for the pneumatic and heat exchange systems were incorporated so that the thermal
308borehole shear test could be performed at different depths without having to connect different
309sections of tubing at each depth. The wooden stakes observed below the bottom bearing pad in
310this picture were used for leveling in this preliminary test. During the main series of experiments,
311the full area of the bearing pad was on the soil surface and the system was plumbed by leveling
312the top plate of the loading frame.

313 As the potential for surface settlement is a common issue borehole shear devices, the area of
314the bearing plate in this device (0.14 m^2) was increased to be greater than the original borehole
315shear device described by Handy and Fox (1967). It is important to have the full area of the
316bearing pad in contact with the soil to minimize errors in the displacement calculations due to
317mobilization of the bearing capacity of the bearing pad. As the borehole shear device is most
318commonly used in stiff soils, the elastic settlement of the plate on the soil surface is assumed to
319be much less than the movement of the shoe during shearing. If settlement is observed to be an
320issue at some sites, a gravel bearing pad may be placed around the borehole to improve the
321stiffness of the surface.

322EXPERIMENTAL PROCEDURES

323 Following excavation of a standard 101.6 mm-diameter borehole, the shear head device is
324lowered into the hole to the desired depth by connecting several 1 m-long extension rods. During
325this time, the shear head device is suspended within the hole using a clamp device. Next, the load
326frame is moved over the hole. The shear head is then coupled to the extension rods and the load

cell while keeping the shear head in the desired angular orientation (e.g., North-South, East-West, etc.). It is important to pretension the string potentiometer to a length greater than the expected total upward displacement, and attach the fishing line running from the concrete shoe using two reusable split shot fishing sinkers. The tubing connected to the borehole shear device is then connected to the pneumatic control board and the heat pump using quick-connect valves. The top plate of the load frame should now be leveled using the height adjustment nuts on the all-thread rods.

After providing at least 15 minutes for consolidation under the applied normal stress as recommended by Lutenecker (1987), the heat pump was started to reach a target temperature. Although a horizontal displacement sensor between the two shoes, as proposed by Suleiman and Xiao (2014), would have been useful to assess the transient consolidation process, one was not incorporated in the device described in this study in case groundwater was encountered. Further, horizontal displacement is not necessary if the purpose of testing is to infer the impact of temperature on the shape of the normalized T-z curve during drained pullout. Next, the heat pump was turned on to apply a desired temperature to the shoe, measured using the thermocouple embedded near the surface of the concrete shoes. Accordingly, heating is applied in load-control conditions, and is not expected to represent the radial expansion of an energy foundation during heating. Even though the shoe reached a constant temperature after a few minutes, a time period of 45 minutes was provided so that the surrounding soil could react to the applied temperature. This time period was selected based on practical time constraints in the field. As the concrete shoe rapidly reached thermal equilibrium, it is expected that the soil in the close vicinity of the interface would also respond quickly. Longer periods of time may be needed when testing low permeability materials, as it may take longer for any thermally-induced excess

350pore water pressures to drain. This is a subject for further study in the future, and would be a
351useful application of a horizontal displacement sensor. After 45 minutes under the applied
352temperature, the upward vertical load was applied to the extension rods under displacement-
353controlled conditions at a rate of 0.05 mm/min. After reaching a stable post-peak shear stress
354(typically 1 to 2 mm in stiff soils), shearing was stopped. The vertical load applied to the shear
355head was then returned to the original load. The horizontal stress was then increased to the next
356normal stress increment, and 15 minutes were provided for further consolidation to occur. This
357approach, which is consistent with the staged testing approach described by Handy (2002),
358permits the shear head to remain in thermal equilibrium and minimizes uncertainty related with
359uncertain variations in soil stratigraphy with depth. The borehole shear device was removed from
360the hole and inspected after each series of tests under different normal stresses at a given
361temperature. The shear head was then returned to the same depth but rotated to a different
362location before application of a different temperature. AbdelSalam et al. (2012) performed each
363interface test at a different location to minimize the chances of particle alignment into the
364direction of shear, which will minimize the chances that a test at one normal stress will affect the
365results at another normal stress. This is likely most important in soils that show a strain-softening
366behavior during shear. All of the tests in this study involved a constant normal stress during
367shearing. As proposed by Suleiman and Xiao (2014), a horizontal displacement sensor may be
368useful in performing constant normal stiffness tests by varying the normal stress to maintain a
369constant displacement to evaluate the effects of constrained dilation (Jardine et al. 1993), which
370is a subject that can be evaluated in future studies.

371DETAILS OF THE PROOF OF CONCEPT TESTS

372*Overview*

373 Two testing series were performed to obtain proof-of-concept results from the thermal
374borehole shear device in terms of the shear stress-displacement curves and the peak shear
375strength failure envelopes. The first series of tests were performed in a soil tank in the laboratory
376to evaluate the results of the device under controlled conditions, while the second series of tests
377were performed in a borehole at a field site to consider the effects of actual in-situ boundary
378conditions and soil structure. More complex testing programs can be performed with this device
379in the future to assess different geotechnical topics related to energy foundations, such as the
380variation in temperature effects on the T-z curves with depth in the same soil deposit, the role of
381temperature cycles, or time-dependent mechanical and thermal consolidation curves at different
382depths. The latter issue would require the incorporation of a horizontal displacement sensor
383between the two shoes.

384*Laboratory Test Details*

385 Before going to the field, a preliminary test was performed in a laboratory tank to refine the
386testing procedures and instrumentation configuration. A 1 m-deep layer of Boulder clay was
387compacted within a cylindrical tank having a diameter of 500 mm. Relevant geotechnical
388properties of Boulder clay measured by Svoboda (2013) are presented in Table 1. The clay was
389compacted in lifts to reach a dry density of 15.6 kN/m^3 at a gravimetric water content of 15%.
390This corresponds to a compaction effort lower than the standard Proctor compaction effort, and
391corresponds to dry of optimum conditions. After compaction of the clay layer, a hand auger
392device was used to create a 114 mm diameter borehole to a depth of 680 mm in the center of the
393clay layer. Borehole shear tests was then performed at a depth of 500 mm (to the center of the

394shear head), for temperatures of 10 and 35 °C, respectively. The shear head was rotated 90° for
395each temperature but the tests were at the same depth. The bearing plate was mounted on the top
396of the steel tank. The normal stresses used in these tests were much greater than K_0 conditions for
397this shallow depth. It is expected that the results from these laboratory tests are likely affected by
398the surface boundary conditions and to the radial stress field in the rigid-wall tank. Nonetheless,
399the laboratory tests provide useful information on the trends with normal stress and temperature
400and highlight the capabilities of the device.

401*Field Test Details*

402 The field test was performed on the property of Entech Engineering in Colorado Springs,
403Colorado. A truck mounted drill rig with continuous flight auger was used to provide a 5 m deep
404101 mm-diameter borehole. The method for drilling was comparable to a traditional exploratory
405boring, with standard penetration testing performed at approximately every 1.5 m using a
406California sampler. After the borehole had reached its desired depth, a new drill bit with an extra
407bead of hard facing was used to clean out the hole to a slightly larger diameter (approximately
408110 mm) with a smooth surface. The location of the boring was on a relatively flat gravel lot and
409drilled on the morning of testing to minimize the chances of the borehole collapsing. The results
410of the site investigation are summarized in Table 2. The soils encountered at the Entech test site
411were stiff, compacted sandy gravel fill up to 2 m, with underlying native medium dense silty
412sand. All of the tests were performed in a day, during which time the borehole remained open
413without visual changes in diameter. The borehole shear tests were performed at a depth of 2.74
414m for multiple normal stresses and temperatures of 10, 25, and 45 °C. The shear head was
415cleaned after each testing series, and the borehole shear device was reset at a different angular
416location before the start of each normal stress series.

417 The maximum normal stress applied in the field of 22.6 kPa was approximately equal to the
418 horizontal stress at rest, calculated using the coefficient of lateral earth pressure at rest K_0
419 estimated using Jaky's (1944) equation ($K_0 = 1 - \sin\phi$). Two lower normal stresses of 13.6 and
420 18.1 kPa were also used to evaluate the frictional response of the soil under the stress range
421 expected for drilled shaft foundations. The soil around a hole constructed using the dry method
422 typically experiences stress relief and active earth pressure conditions during excavation.

423 RESULTS

424 The shear stress-displacement curves from the two testing series in the Boulder clay layer are
425 shown in Figures 8(a) and 8(b) for temperatures of 10 and 35 °C, respectively. As expected, the
426 interface shear strength increases with increasing normal stress. The shapes of the curves are
427 similar for the tests at the two temperatures, showing a hardening behavior during shear. The
428 strain at failure defined as the onset of plastic displacement is slightly greater for the tests
429 performed at 35 °C, indicating a slight softening with increasing temperature.

430 The shear stress-displacement curves from the three testing series in the silty sand layer in
431 the field are shown in Figures 9(a), 9(b), and 9(c) for temperatures of 10, 25, and 45 °C,
432 respectively. Although the normal stresses are lower in the field tests, the shapes of the curves
433 are similar to those measured in the laboratory, with a hardening behavior during shearing. The
434 field curves also show a slight increase in the strain at failure with increasing temperature from
435 10 to 25 °C, indicating a slight thermal softening. However, the behavior at the highest
436 temperature of 45 °C does not appear to follow the same trend as at the lower temperatures, and
437 the reasons for this will be discussed later.

438 ANALYSIS

439 The peak shear strength values from the laboratory tests at different normal stresses are
440 shown in Figure 10(a). The failure envelopes were defined so that they have a zero adhesion
441 intercept, which provides a good fit to both sets of data. The application of temperature to the
442 compacted clay soil in the laboratory was observed to lead to a negligible change in peak shear
443 strength at the same normal stress. Further the interface friction angle was not affected
444 significantly by temperature. The interface friction angle of 29° was lower than the internal
445 friction angle of the soil of 34.5° reported by Svoboda (2013).

446 The peak shear strength values from the field tests at different normal stresses are shown in
447 Figure 10(b). The failure envelopes were also defined so that they have a zero adhesion intercept,
448 which provides a good fit to all of the data except the field test at 10°C . Heating leads to an
449 inconsistent trend in the peak shear strength in the field. As mentioned above, the results from
450 the tests at 45°C do not appear to be consistent with the others. It is possible that the time
451 permitted for thermal equilibration and dissipation of thermally induced excess pore water
452 pressures was not sufficient. In this case, the effective stress at the interface may have been lower
453 for the tests at 45°C than at 25°C . Although it appears that heating leads to an increase in peak
454 shear strength from 10 to 25°C , along with a corresponding increase in friction angle, the peak
455 shear strength in the tests at the two lower normal stress values at a temperature of 10°C appear
456 to be too low. Specifically, if they were considered alone without the test at a normal stress of
457 22.6 kPa , the apparent adhesion would be negative. It is possible that the pneumatic piston did
458 not fully expand for these two tests or that consolidation was still occurring under the mechanical
459 load. If the test at a temperature of 10°C and a normal stress of 22.6 kPa were compared to the
460 tests at the other temperatures, the increase in peak shear strength and friction angle is not

461significant. Another reason for the difference in the peak shear strength values is that thermally
462induced water flow in the unsaturated silty sand deposit led to an increase in suction at the
463interface and a corresponding increase in apparent adhesion. As all three tests at each
464temperature were performed sequentially in the same location, it is possible that the thermally
465induced water flow progressed over time, which may lead to an overestimate of the peak shear
466strength at each subsequent normal stress. This possibility indicates that it may be better to
467follow the approach of AbdelSalam et al. (2012) by performing each test for a given temperature
468and normal stress at a different location.

469 A summary of the friction angles from the different drained failure envelopes in the
470laboratory and field is shown in Figure 11. The friction angle of the Boulder clay tested in the
471laboratory is insensitive to temperature, which is consistent with the observations of Laloui
472(2001). The inconsistent variations in friction angle of the silty clay in the field could have been
473due to the low peak shear strength values from the tests at 10 °C and/or from the possibly
474incomplete thermal consolidation in the tests at 45 °C. The tests on the silty clay in the field at a
475temperature of 25 °C showed the most consistent behavior between the tests, so it is likely that
476this higher interface friction angle of 35° is representative of the soil layer. However, the
477inconsistencies in the results does not permit a strong conclusion to be drawn regarding the effect
478of temperature on the friction angle of the silty soil in the field.

479 Normalized T-z curves were defined for the laboratory and field data by dividing the shear
480stresses by the peak shear strength, and are shown in Figures 12 and 13, respectively. In most
481cases the peak and large displacement shear strength values are the same as the curves exhibit
482hardening during shear. The normalized T-z curves for Boulder clay obtained from the
483laboratory tests shown in Figure 12 indicate that the normal stress leads to a decrease in the

484initial slope of the normalized curves for both temperatures. The effect of normal stress on the
485normalized curves has been observed in T-z curves obtained by O'Neill and Reese (1999), and is
486consistent with the predictions from the hyperbolic model of Duncan and Chang (197). The
487normalized curves at higher temperature show a slightly lower initial slope than those at ambient
488temperature, except at the two lower normal stresses. It is possible that a transition behavior is
489noted with increasing normal stress. The shapes of the normalized T-z curves for the silty sand
490obtained from the field tests shown in Figure 13 indicate the same effect of normal stress as
491observed in the laboratory tests, although the magnitude of the effect is not as significant.

492 The initial slope was defined for each of the normalized T-z curves using the data at shear
493stress levels ($\tau/\tau_{\max}$) between 30 and 70%. In this range of shear stress levels, the slopes of the T-
494z curves are relatively consistent. Below a shear stress level of 30% it is possible that the shoes
495do not start moving even though the shear stress is increasing. The normalized T-z curves
496typically start to exhibit nonlinearity at stress levels greater than 80 to 90%. The impacts of
497normal stress and temperature on the initial slopes of the normalized T-z curves from the
498laboratory and field are summarized in Figure 14(a) and 14(b), respectively. The results in Figure
49914(a) indicate that the initial slopes of the normalized T-z curves decrease nonlinearly with
500normal stress in both the laboratory and field, except for the field test at a temperature of 45 °C,
501which could have been affected by incomplete thermal consolidation. The results in Figure 14(b)
502indicate that temperature does not have a major impact on the initial slope of the normalized T-z
503curves, except for the tests in the laboratory and field at the lowest normal stresses of 12.4 and
50413.6 kPa where inconsistent behavior was noted. Although the effects of temperature on the
505normalized T-z curve were not significant for the stiff, unsaturated soil layers investigated in this

study, the modified thermal borehole shear device described in this study permits definition of site-specific trends in the these curves for different soils.

CONCLUSIONS

This study presents the details of a new thermal borehole shear test that can be used to measure the in-situ shear stress-displacement curve of soil-concrete interface under different temperatures. The new device incorporates concrete shoes with embedded heat exchangers whose temperature is controlled using a heat pump at the ground surface, a loading frame for shearing under displacement- or load-control conditions, and an approach to measure local interface displacements. The device is best suited for measurement of the effects of temperature on the in-situ shear stress-displacement curves and the drained failure envelope for saturated, stiff soils. In these soils, it is expected that the horizontal displacements will be small during mechanical loading, which will minimize the chances for moments within the horizontal loading system or the possibility for plowing. Further, the roughened surface of the concrete shoes best represents the interface between a stiff soil and a drilled shaft constructed using the dry method. In unsaturated soil layers, thermally induced water flow may cause an increase in the peak shear strength that would not occur in saturated soil layers. Finally, the elastic settlement of the surface loading plate is expected to negligible in stiff soil layers when compared to the movement of the soil-concrete interface.

The results from the laboratory tests indicate that temperature does not have a major impact on the peak shear strength, friction angle, or shape of the normalized T-z curves. These results support the assumption in load transfer analyses on energy foundations that temperature does not have a major effect on the shape of the T-z curve. Further, the laboratory tests confirm the importance of the horizontal normal stress on the shape of the normalized T-z curve. The results

529from the field tests indicate that temperature does not have a major impact on the shape of the
530normalized T-z curves, but inconsistent behavior was observed regarding the effects of
531temperature on the peak shear strength and friction angle. These inconsistencies can be
532minimized in future tests by moving the shoe to a new location after each test to minimize the
533effects of thermally induced water flow, and waiting for a sufficient time to permit consolidation
534due to the application of the normal stress or thermally induced excess pore water pressures.
535Incorporating a horizontal displacement sensor between the two shoes may be useful in
536evaluating transient changes horizontal displacement of the shoes during mechanical loading or
537heating, even though this feature may not be necessary for defining the impact of temperature on
538the interface shear stress-displacement curve. A method for suction measurement at the interface
539may also help evaluate the in-situ stress state as well as to confirm that there are no shear-
540induced pore water pressures when performing drained shearing tests.

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545**REFERENCES**

546AbdelSalam, S.S., Suleiman, M.T., and Sritharan, S., 2012, "A Load Transfer Analysis for
547 Axially Loaded Piles Using a Modified Borehole Shear Test," *Geotechnical Testing Journal*,
548 Vol. 35, No. 6, 1-11. DOI: 10.1520/GTJ20120071.
549Amatya, B.L., Soga, K., Bourne-Webb, P.J., Amis, T. and Laloui, L., 2012, "Thermo-
550 Mechanical Behaviour of Energy Piles," *Géotechnique*. 62(6), 503–519.Brandl, H., 2006,

551 “Energy Foundations and other Thermo-Active Ground Structures,” *Géotechnique*, Vol.
552 56, No. 2, p. 81–122.

553 Ashlock, J.C. and Lu, N., 2012, “Interpretation of Borehole Shear Strength Tests of Unsaturated
554 Loess by Suction Stress Characteristic Curves,” Proc. GeoCongress 2012. Oakland, CA,
555 March 25-29. ASCE. p. 2562-2571.

556 ASTM D2488. Standard Practice for Description and Identification of Soils (Visual-Manual
557 Procedure). ASTM International. West Conshohocken, PA.

558 Audibert, J. and Aggarwal, D. (1982). “Study to Investigate the Effects of Skin Friction on the
559 Performance of Drilled Shafts in Cohesive Soils, Vol. 1, Field Investigation,” Technical
560 Report GL-82-1, U.S. Army Waterways Experiment Station, Vicksburg, MS.

561 Bouazza, A., Singh, R.M., Wang, B., Barry-Macaulay, D., Haberfield, C., Chapman, G., Baycan,
562 S., and Carden, Y., 2011, “Harnessing on Site Renewable Energy through Pile
563 Foundations,” *Australian Geomechanics*. Vol. 46, No. 4, p. 79-90.

564 Bourne-Webb, P.J., Amatya, B., Soga, K., Amis, T., Davidson, C. and Payne, P., 2009, “Energy
565 Pile Test at Lambeth College, London: Geotechnical and Thermodynamic Aspects of Pile
566 Response to Heat Cycles.” *Géotechnique*. Vol. 59, No. 3, p. 237-248.

567 Bechtum, T., 2012, Automation and Further Development of the Borehole Shear Test. MS
568 Thesis. Iowa State University. 180 pg.

569 Burghignoli, A., Desideri, A., and Miliziano, S., 2000, “A Laboratory Study on the
570 Thermomechanical Behaviour of Clayey Soils,” *Canadian Geotechnical Journal*, Vol. 37, p.
571 764–780.

572 Campanella, R.G., and Mitchell, J.K., 1968, "Influence of Temperature Variations on Soil
573 Behavior," *Journal of Soil Mechanics and Foundation Engineering Division*, Vol. 94, No.
574 SM3, p. 709–734.

575 Cekerevac, C., and Laloui, L., 2004, "Experimental Study of Thermal Effects on the Mechanical
576 Behavior of a Clay," *International Journal for Numerical and Analytical Methods in*
577 *Geomechanics*, Vol. 28, p. 209–228.

578 Coyle, H.M. and Reese, L.C., 1966, "Load Transfer for Axially Loaded Piles in Clay," *Journal*
579 *of Soil Mechanics and Foundation Engineering Division*, Vol. 92, No. 2, p. 1-26.

580 Demartinencourt, J.P. and Bauer, G.E., 1983, "The Modified Borehole Shear Device,"
581 *Geotechnical Testing Journal*. Vol. 6, No. 1, p. 24-29.

582 Duncan, J.M. and Chang, C.-Y., 1970, "Nonlinear Analysis of Stress and Strain in Soils,"
583 *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 96, No. SM5, p. 1629-
584 1653.

585 Graham, J., Tanaka, N., Crilly, T., and Alfaro, M., 2001, "Modified Cam-Clay Modeling of
586 Temperature Effects in Clays," *Canadian Geotechnical Journal*, Vol. 38, No. 3, p. 608-621.

587 Handy, R.L. and Fox, N.S., 1967, "A Soil Bore-Hole Direct Shear Test Device," Highway
588 Research News, Highway Research Board, No. 27, p. 42-51.

589 Handy, R.L., 1986, "Borehole Shear Test and Slope Stability." *Use of In-Situ Tests in*
590 *Geotechnical Engineering*, ASCE, p. 161-175.

591 Handy, R.L., 2002, *Borehole Shear Test Manual*, Handy Geotechnical Instruments, Inc., Madrid,
592 IA.

593 Hueckel, T., and Borsetto, M., 1990, "Thermoplasticity of Saturated Soils and Shales:
 594 Constitutive Equations," *Journal of Geotechnical Engineering*, Vol. 116, No. 12, p. 1765-
 595 1777.

596 Jaky, J. 1944, "The Coefficient of Earth Pressure at Rest," *Journal of Society of Hungarian*
 597 *Architects and Engineers*, Vol. 7, p. 355-358.

598 Jardine, R.J., Lehane, B.M., and Everton, S.J., 1993, "Friction Coefficients for Piles in Sands and
 599 Silts," *Offshore Site Investigations and Foundation Behavior*. Vol. XXVIII Society for
 600 Underwater Technology, Netherlands, p. 661-677.

601 Knellwolf, C., Peron, H., and Laloui, L. 2011, "Geotechnical Analysis of Heat Exchanger Piles,"
 602 *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE. Vol. 137, No. 12, p.
 603 890-902.

604 Kulhawy, F.H., Peterson, M.S., 1979, "Behavior of Sand-Concrete Interfaces," *Proc. 6th Pan*
 605 *American Conference on Soil Mechanics And Foundation Engineering*, Vol. 2, Lima, Peru,
 606 p. 225-236.

607 Laloui, L., 2001, "Thermo-mechanical Behaviour of Soils," *Revue Française de Génie Civil*,
 608 Vol. 5, No. 6, p. 809-843.

609 Laloui, L., Nuth, M., and Vulliet, L., 2006, "Experimental and Numerical Investigations of the
 610 Behaviour of a Heat Exchanger Pile," *International Journal for Numerical and Analytical*
 611 *Methods in Geomechanics* Vol. 30, p. 763–781.

612 Lutenecker, A.J., Remmes, B.D., and Handy, R.L., 1978, "Borehole Shear Tests for Stiff Soils,"
 613 *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT11, p. 1403-
 614 1407.

615 Lutenegger, A.J., and Tierney, K.F., 1986, "Pore Pressure Effects in Borehole Shear Testing,"
616 *Use of In Situ Tests in Geotechnical Engineering*, ASCE, p. 752-764.

617 Lutenegger, A.J., 1987, "Suggested Method for Performing the Borehole Shear Test,"
618 *Geotechnical Testing Journal*, Vol. 10, No. 1, p. 19-25.

619 Lutenegger, A.J., and Timian, D.A., 1987, "Reproducibility of Borehole Shear Test Results in
620 Marine Clay," *Geotechnical Testing Journal*, Vol. 10, No. 1, p. 13-18.

621 Lutenegger, A.J., and Powell, J.J.M., 2008, "Borehole Shear Tests in Stiff London and Gault
622 Clay," *Proc. 3rd International Symposium on Site Characterization*, Taipei. p. 719-723,

623 McCartney, J.S. and Murphy, K.D., 2012, "Strain Distributions in Full-Scale Energy
624 Foundations." *DFI Journal*. Vol. 6, No. 2, p. 28-36.

625 McCartney, J.S. and Rosenberg, J.E., 2011, "Impact of Heat Exchange on the Axial Capacity of
626 Thermo-Active Foundations," *Proc. Geo-Frontiers 2011 (GSP 211)*. J. Han and D.E.
627 Alzamora, eds. ASCE, Reston VA. p. 488-498.

628 McKinstry, H.A., 1965, "Thermal Expansion of Clay Minerals," *The American Mineralogist*,
629 Vol. 50, p. 212-222.

630 Miller, G.A. Azad, S. and Hassell, C.E., 1998, "Iowa Borehole Shear Testing in Unsaturated
631 Soil," *In: The Proceedings of the 2nd International Conference on Unsaturated Soils*,
632 *Geotechnical Site Characterization*, Vol. 1 International Academic Publishing House,
633 Beijing (1998), p. 1321-1326.

634 Murphy, K.D., McCartney, J.S., Henry, K.H., 2014, "Thermo-Mechanical Response Tests on
635 Energy Foundations with Different Heat Exchanger Configurations." *Acta Geotechnica*. p.
636 1-17. DOI: 10.1007/s11440-013-0298-4.

637 O'Neill, M.W. and Reese, L.C., 1999, "Drilled Shafts: Construction Procedures and Design
638 Methods," Report No. FHWA-IF-99-025, Federal Highway Administration, Washington
639 D.C., USA.

640 Paaswell, R.E., 1967, "Temperature Effects on Clay Soil Consolidation," *Journal of Soil*
641 *Mechanics and Foundations Division*, Vol. 93, No. SM3, pp. 9–22.

642 Peterson, M.S., Kulhawy, F.H., Nucci, L.R., and Wasil, B.A., 1976, "Stress-Deformation
643 Behavior of Soil-Concrete Interfaces," *Contract Report B-49 to Niagara Mohawk Power*
644 *Corporation*, Syracuse, NY.

645 Potyondy, J.G., 1961, "Skin Friction between Various Soils and Construction Materials,"
646 *Géotechnique*, Vol. 11, pp. 339-355.

647 Regueiro, R., Wang, W., Stewart, M.A., and McCartney, J.S., 2012, "Coupled Thermo-Poro-
648 Mechanical Finite Element Analysis of a Heated Single Pile Centrifuge Experiment in
649 Saturated Silt." GeoCongress 2012. Oakland, CA. March 25-29. ASCE. pp. 4406-4415.

650 Stewart, M.A. and McCartney, J.S., 2013, "Centrifuge Modeling of Soil-Structure Interaction in
651 Energy Foundations." ASCE Journal of Geotechnical and Geoenvironmental Engineering.
652 In press. DOI: 10.1061/(ASCE)GT.1943-5606.0001061.

653 Suleiman, M., AbdelSalam, S., and Sritharan, S., 2011, "Improving Prediction of the Load-
654 Displacement Response of Axially Loaded Friction Piles," *GeoFrontiers 2011*, ASCE,
655 Dallas, TX. pp. 36-45.

656 Suleiman, M. and Xiao, S. 2014, "Soil-Pile Interaction of Geothermal Deep Foundations," 27th
657 Central Pennsylvania Geotechnical Conference. ASCE. Reston, VA. p. 1-22.

658 Svoboda, J., 2013, Impact of Strain Rate on the Shear Strength and Pore Water Pressure
659 Generation of Clays and Sands. MS Thesis. University of Colorado Boulder. 181 pg.

660 Uchaipichat, A., and Khalili, N., 2009, "Experimental Investigation of Thermo-Hydro-
661 Mechanical Behavior of an Unsaturated Silt," *Géotechnique*, Vol. 59, No. 4, p. 339- 353.

662 van Genuchten, M.T., 1980, "A Closed-Form Equation for Predicting the Hydraulic
663 Conductivity of Unsaturated Soils." *Soil Science Society of America Journal*. Vol. 44, No.
664 5, p. 892-898.

665 White, D.J., and Handy, R.L., 2001, "Preconsolidation Pressures and Soil Moduli from Borehole
666 Shear Tests," *Proc. International Conference on In Situ Measurement of Soil Properties and*
667 *Case Histories*, Bali, Indonesia.

668 Xiao, S., Suleiman, M.T., McCartney, J.S. 2014, "Shear Behavior of Silty Soil and Soil-Structure
669 Interface under Thermal Loading," *Proc. GeoCongress 2014 (GSP 234)*, M. Abu-Farsak and
670 L. Hoyos, eds., ASCE, Reston, VA. pg. 4105-4114.

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696Table 1: Properties of Boulder clay (Svoboda 2013)

Parameter	Value
D_{10}	< 0.00015 mm
D_{30}	< 0.00015 mm
D_{50}	0.0012 mm
% Passing No. 200 Sieve	90 %
% Clay Size	60 %
% Silt Size	40 %
% Sand Size	0 %
G_s	2.7
Liquid Limit, LL	43
Plastic Limit, PL	21
Plasticity Index, PI	22
Effective friction angle, ϕ'	34.5°
Compression index, C_c	0.233
Recompression index, C_r	0.0041
Std. Proctor Max. Dry Unit Weight	17.1 kN/m ³
Std. Proctor Max. Opt. Water Content	18%
Drying-path soil water retention curve*	$\alpha_{VG} = 0.009 \text{ kPa}^{-1}$, $N_{VG} = 1.16$, $\theta_r = 0\%$
Initial void ratio, e_0	0.70
Initial water content, w_0	15.0%
Initial degree of saturation, S_0	0.58
Saturated hydraulic conductivity, k_s	$6.9 \times 10^{-10} \text{ m/s}$

*Defined using the van Genuchten (1980) model

697

698Table 2: Results from the site investigation at Entech Engineering, Colorado Springs, CO

Sample depth (m)	Material encountered*	Gravimetric water content (%)	California sampler SPT N-Value (blows/300 mm)
1.5	Dense sandy fill w/ gravel	15	28
3.0	Medium dense silty sand	9	17
4.6	Medium dense silty sand	8	19
6.1	Medium dense silty sand	12	22

699*Note: Soils were identified using the Visual Manual Procedure in the field. All fines were
700 deemed to be non-plastic

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