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# PARAMETERS FOR LOAD TRANSFER ANALYSIS OF ENERGY PILES IN UNIFORM NON-PLASTIC SOILS

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**Abstract:** This study focuses on the use of a thermo-mechanical soil-structure interaction (load transfer) analysis to assess the axial strains, stresses, and displacements during thermo-mechanical loading of energy piles in various soil deposits and having different end restraint boundary conditions. After providing details of the model and its novel features, this paper presents a parametric evaluation performed to understand the roles of the soil shear strength parameters, toe stiffness, head stiffness, side shear stress-displacement curve, and radial expansion, as well as the magnitude of temperature change. This evaluation showed that the end restraint boundary conditions play the most important role in controlling the magnitude and location of the maximum thermal axial stress. The soil type also causes changes in the nonlinearity of the axial stress distribution throughout the energy pile. The radial expansion did not affect the thermo-mechanical soil-structure interaction for the conditions investigated in this study. The thermo-mechanical load-transfer analysis was then calibrated to identify the parameters that match the observed soil-structure interaction responses from four case studies of energy piles in non-plastic soil or rock layers during monotonic heating, including one field study and three centrifuge studies. The ranges of calibrated parameters provide insight into the behavior of energy piles in non-plastic soils, and can be used for preliminary design guidance.

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## 20 INTRODUCTION

21 Energy piles are a dual-purpose structural element built underground to exchange heat  
22 between a building and the subsurface while also transferring loads from the structure to the  
23 ground. Different from classical deep foundations, energy piles incorporate closed-loop, flexible,  
24 high density polyethylene (HDPE) tubing within the reinforcing cage, through which a heat  
25 exchange fluid (i.e., typically water mixed with propylene glycol) is circulated to transfer heat to  
26 or from the subsurface. The temperature of the fluid is controlled using a heat pump within the  
27 building. The relatively steady temperature of subsurface soil and rock below 3 to 5 m is  
28 approximately equal to the mean annual air temperature (Burger et al. 1985), which makes it a  
29 stable heat source for efficient heat exchange needed to cover the base heating and cooling thermal  
30 loads for a built structure (Brandl 2006).

31 Geotechnical design of energy piles requires consideration of the impact of temperature on  
32 the induced stresses and strains in the pile, which may affect building performance. Specifically,  
33 heating and cooling of the pile during heat exchange will lead to expansion and contraction of the  
34 pile and soil. This may lead to deformations and changes in the stress state (Brandl 2006; Laloui  
35 and Nuth 2006; Bourne-Webb et al. 2009). Therefore, it is important for designers to understand  
36 the thermo-mechanical behavior of energy piles in various soil or rock deposits expected for  
37 different temperature changes.

38 There are many means to simulate the thermo-mechanical behavior of energy piles. For  
39 example, the mechanisms of thermo-mechanical soil-structure interaction have been documented  
40 in several full-scale case histories in the field (Laloui et al. 2003; Brandl 2006; Laloui et al. 2006;  
41 Bourne-Webb et al. 2009; Amatya et al. 2012; McCartney and Murphy 2012; Akrouch et al. 2014;  
42 Sutman et al. 2014; Murphy et al. 2015; Murphy and McCartney 2015; Olgun et al. 2014a; Wang

43 et al. 2014). Although these studies provide insight into the mechanisms governing energy pile  
44 behavior, this behavior must be synthesized into the form of a model to provide quantitative  
45 predictions of energy pile behavior in different settings. Thermoelastic finite-element (FE)  
46 analyses have also been used to predict the changes in axial displacement, strain, and stress in  
47 energy piles during heating and cooling (Laloui et al. 2006; Ouyang et al. 2011; Wang et al. 2012,  
48 2015; Rotta Loria et al. 2015a, 2015b). However, FE analyses are complicated to perform for  
49 energy pile design due to the large number of parameters potentially needed that may require  
50 advanced testing to obtain, especially when nonlinear soil behavior is considered. Alternatively, a  
51 comparably simpler method referred to as the thermo-mechanical load transfer analyses can be  
52 used to predict the behavior of energy piles under temperature changes (Knellwolf et al. 2011;  
53 Plaseied 2012; Suryatriyastuti et al. 2013). This method combines a known or assumed shape of  
54 the mobilized side shear resistance and end bearing resistance curves together with knowledge of  
55 the ultimate side shear and end bearing capacities to estimate the distribution in mobilized axial  
56 stress, strain and displacement with depth. Although this approach is simpler and requires fewer  
57 parameters to consider nonlinear soil-pile interaction, there is limited information on the range of  
58 parameters that describe the shapes of the mobilized side shear resistance and end bearing  
59 resistance curves of energy piles needed to perform thermo-mechanical load transfer analyses.

60 The main objective of this study is to understand the effects of model parameters on the output  
61 of the load transfer analyses to understand the relative importance of the different variables.  
62 Another equally important objective is to understand typical ranges of model parameters calibrated  
63 using experimental data associated with the thermo-mechanical behavior of energy piles having  
64 different end-restraint boundary conditions in non-plastic soils where soil thermal volume changes  
65 are not expected. To reach these objectives, the thermo-mechanical load transfer analysis described

66 by Plaseied (2012) and McCartney (2015) was updated to better identify the null point location  
67 (the location of zero thermal displacement in an energy pile undergoing a uniform temperature  
68 change) and to incorporate different models for the side shear resistance of soils under drained and  
69 undrained conditions. Next, a parametric evaluation was performed to understand the effects of  
70 different parameters on the stress-strain response. Then, the model was fitted to the experimental  
71 results from four different studies to calibrate the different model parameters. Finally, ranges and  
72 trends in the calibrated model parameters were synthesized to provide design guidance.

### 73 **BACKGROUND**

74 Subsurface geothermal resources represent a great potential of directly usable energy,  
75 especially in connection with deep foundations and heat pumps. It is already common to utilize  
76 the geothermal energy in providing thermal needs of buildings. To utilize subsurface geothermal  
77 energy, a heat exchanger is commonly incorporated into drilled shaft foundations for circulating  
78 heat exchange fluid between the subsurface ground and a structure. However, it also presents new  
79 challenges for the broader geotechnical engineering profession in terms of technical issues  
80 associated with soil-structure interaction (Laloui et al. 2006; Bourne-Webb et al. 2009; Amatya et  
81 al. 2012; Murphy et al. 2015).

82 Observations from several case histories involving full-scale energy piles indicate that heating  
83 and cooling will lead to movements associated with thermal expansion and contraction of the pile  
84 element and surrounding soil (Laloui et al. 2003; Brandl 2006; Laloui et al. 2006; Bourne-Webb  
85 et al. 2009; Amatya et al. 2012; McCartney and Murphy 2012; Akrouch et al. 2014; Sutman et al.  
86 2014; Olgun et al. 2014a; Wang et al. 2014; Murphy et al. 2015; Murphy and McCartney 2015).  
87 These thermally-induced movements may lead to the generation of axial stresses due to the  
88 restraint of the pile provided by soil-structure interaction and end-restraint boundary conditions

89 (i.e., the stiffness of the overlying structure and the underlying bearing layer). Lateral movements  
90 of energy piles during heating and cooling has been proposed as a mechanism of changing soil  
91 structure interaction (McCartney and Rosenberg 2011; Mimouni and Laloui 2014), although cavity  
92 expansion analyses indicate that the amount of lateral expansion may not be sufficient to change  
93 the lateral stress state in all soils profiles (Olgun et al. 2014b). The end-restraint boundary  
94 conditions play an important role in design guidelines being proposed for energy piles  
95 (Suryatriyastuti et al. 2013; Mimouni and Laloui 2014). As it is often difficult to vary the end-  
96 restraint boundary conditions in full-scale energy pile systems to investigate their impact on soil-  
97 structure interaction mechanisms, an alternate modeling approach involves the use of centrifuge-  
98 scale energy (McCartney and Rosenberg 2011; Stewart and McCartney 2014; Goode et al. 2014;  
99 Goode and McCartney 2014; Goode and McCartney 2015; Ng et al. 2014, 2015). Although  
100 centrifuge tests represent an idealized situation compared to field tests and may not properly  
101 consider the role of construction effects, they have been shown to be useful for calibration or  
102 validation of numerical simulations using thermo-elasto-plastic finite element models or load  
103 transfer analyses.

104       One of the first studies to modify the conventional load transfer analysis for mechanical  
105 loading to consider thermo-mechanical load transfer analysis was performed by Knellwolf et al.  
106 (2011), where the energy pile was assumed to consist of elastic pile elements connected to the soil  
107 by elastic perfectly-plastic springs. Plaseied (2012) developed a load transfer analysis by  
108 considering nonlinear springs, where the mobilized side shear and end bearing resistance springs  
109 were represented by hyperbolic curves. Plaseied (2012) also considered the role of radial  
110 expansion of the pile elements, but did not perform a through parametric evaluation of this  
111 parameter. The algorithm in the model of Plaseied (2012) also fails to capture the exact location

112 of the null point, and requires the user to choose the null-point, which leads to potentially  
113 inaccurate results.

114 Thermo-mechanical load transfer analyses have been validated on the basis of in-situ  
115 measurements of the loads and deformations experienced by heat exchanger test piles (Knellwolf  
116 et al. 2011; Plaseied 2012), but the choice of parameters for this method needs to be further studied  
117 in order to put this method to practical use. Energy pile design guidelines to account for thermal  
118 soil-structure interaction effects are available in different countries (Burlon et al. 2013; Mimouni  
119 and Laloui 2014; Bourne-Webb et al. 2014), but there is still a need for consistent soil-structure  
120 guidance to ensure implementation in practice worldwide.

## 121 **LOAD TRANSFER ANALYSIS DESCRIPTION**

### 122 **Theory**

123 An axial load transfer analysis is developed in this study to predict the behavior of energy  
124 piles subject to combined mechanical and thermal loading. The thermo-mechanical load transfer  
125 analysis is based on the following assumptions:

- 126 1. The properties of the pile such as the Young's modulus ( $E$ ) and coefficient of thermal  
127 expansion ( $\alpha_T$ ) remain constant along the pile.
- 128 2. Downward and upward movements are taken as positive and negative respectively.  
129 Compressional stresses and forces are also taken to be positive.
- 130 3. The pile expands and contracts about a point referred to as the null point when it is heated or  
131 cooled (Bourne-Webb et al. 2009). The location of the null point depends on the upper and  
132 lower axial boundary conditions and side shear distribution, and will be defined later.  
133 Expansion strains are assumed to be negative.

134 4. Depending on the particular details of the soil profile, the ultimate side shear resistance can be  
 135 assumed to be constant with depth in a soil layer (i.e., the  $\alpha$  method) (Tomlinson 1957) or it  
 136 can be assumed to increase linearly with depth in a soil profile (i.e., the  $\beta$  method) (Rollins et  
 137 al. 1997). Both approaches are used in the parametric analysis.

138 The following notations are used in the thermo-mechanical load transfer analysis:  $Q$  is used  
 139 to represent axial forces within the pile or on the pile boundaries; the letter  $\rho$  stands for the axial  
 140 displacement of the pile,  $K_f$ ,  $K_s$  and  $K_{base}$  are the stiffness values of the reinforced concrete pile  
 141 spring, the side shear spring, and the base spring, respectively; the indices “ $b$ ”, “ $t$ ” and “ $s$ ” represent  
 142 the bottom, top and side of an element; the indices  $M$ ,  $T$ ,  $MT$  stand for mechanical, thermal loading  
 143 and thermo-mechanical loading, respectively; the superscript “ $i$ ” represents the element number  
 144 within the pile; and the variable “ $L^i$ ” represents the length of each element along the pile.

145 Considering a pile under mechanical loading, the pile is firstly discretized into  $n$  elements, as  
 146 shown in Figure 1. The value of the displacement at the bottom of the pile  $\rho_b^n$  is assumed for  
 147 initiating the T-z analysis. Then the behavior of pile can be obtained by iterating the following  
 148 calculations for each pile element starting from the  $n^{\text{th}}$  to the 1<sup>st</sup>, and reaching convergence one by  
 149 one. The axial forces at the base, middle and top of the  $i^{\text{th}}$  element are defined as follow:

$$(1) \quad Q_{b,M}^i = \begin{cases} Q_{t,M}^{i+1} & (i \neq n) \\ Q_{b,max} \cdot f_{Q-z}(\rho_b) & (i = n) \end{cases}$$

$$(2) \quad Q_{t,M}^i = 0$$

$$(3) \quad Q_{ave,M}^i = (Q_{b,M}^i + Q_{t,M}^i)/2$$

150 Next, the elastic compression of element  $n$  ( $\Delta_M^n$ ) can be calculated by dividing the average  
 151 force  $Q_{ave}^n$  by the stiffness  $K_f^n$ , as follows:



$$(4) \quad \Delta_M^i = Q_{ave,M}^i / K_f^i$$

152 Next, the displacement at the base, middle and top of  $i^{\text{th}}$  element are defined as follow:

$$(5) \quad \rho_{b,M}^i = \begin{cases} \rho_{t,M}^{i+1} & (i \neq n) \\ \rho_{b,M} & (i = n) \end{cases}$$

$$(6) \quad \rho_{s,M}^i = \rho_{b,M}^i + 0.5\Delta_M^i$$

$$(7) \quad \rho_{t,M}^i = \rho_{b,M}^i + \Delta_M^i$$

153 Next, the mobilized side shear force ( $Q_{s,M}^n$ ) and new axial force ( $Q_{t,M}^i$ ) at top can be defined  
154 as follows:

$$(8) \quad Q_{s,M}^i = Q_{s,max}^i \cdot f_{T-z}^{loading}(\rho_{s,M}^i)$$

$$(9) \quad Q_{t,M}^i = Q_{b,M}^n + Q_{s,M}^n$$

155 Then, Equations (3) to (9) are repeated in sequence until the absolute value of the change in  
156  $Q_{t,M}^i$  between different iterations becomes less than a user-specified criterion (a value of  $10^{-10}$  is  
157 used in this study). After the  $i^{\text{th}}$  element converges, the same process used for the  $i^{\text{th}}$  element is  
158 used for the  $(i-1)^{\text{th}}$  element, and so on. Once the 1<sup>st</sup> element converges, Newton's method with a  
159 secant stiffness ( $k_{sec}$ ) that passes through the origin is used to find the value of the base  
160 displacement ( $\rho_{b,M}^i$ ) that causes the corresponding load on the top of the pile  $Q_{t,M}^1$  to equal the  
161 applied mechanical load ( $P$ ).

162 To extend the mechanical load transfer analysis to thermo-mechanical loading conditions, a  
163 spring should be added to the top of the pile that represents the pile head-structure stiffness  
164 (Knellwolf et al. 2011). When an energy pile is heated or cooled, it begins to expand or contract  
165 about its null point (Bourne-Webb et al. 2009) defined as the location in the pile where there is no  
166 thermal expansion or contraction, assuming that the temperature change occurs uniformly

167 throughout the pile. The complexity of the algorithm for solving the behavior of energy pile is  
 168 largely alleviated once the null point is identified, and the behavior of pile can be analyzed by  
 169 separately investigating the parts of the pile below and above the null point.

170 To compute the settlement and the stress distribution of the energy pile under thermo-  
 171 mechanical loading (restricted to heating in this study), the first step is to assume a null point  
 172 location, NP. Then, the pile is evenly divided into  $n_1$  elements for the part of the pile above NP  
 173 and  $n_2$  elements for the part of the pile below NP into, as shown in Figure 1. The mechanical load  
 174 transfer analysis is performed and the displacements and forces on each element of the pile under  
 175 mechanical loading are used as the initial condition for heating of the energy pile. After this  
 176 preprocessing is done, the energy pile is initially assumed to be totally free to move similar to the  
 177 approach of Knellwolf et al. (2011), in which case an initial estimate of the thermal elongations of  
 178 each element can be obtained from the following expression:

$$(10) \quad \Delta_T^i = L^i \alpha_T \Delta T$$

179 For the part of energy pile below the null point, these elements move downward. Thus, the  
 180 thermal displacement at the top, middle and base of element  $(n_1 + 1)$  downward to element  $n$  can  
 181 be calculated as follows:

$$(11) \quad \rho_{t,T}^i = \begin{cases} 0 & \text{for } i = n_1 + 1 \\ \rho_{b,T}^{i-1} & \text{for } i \neq NP \end{cases}$$

$$(12) \quad \rho_{s,T}^i = \rho_{t,T}^i + \frac{\Delta_T^i}{2}$$

$$(13) \quad \rho_{b,T}^i = \rho_{s,T}^i + \Delta_T^i$$

182 and the thermo-mechanical displacements can be calculating as follows:

$$(14) \quad \rho_{b,MT}^i = \rho_{b,M}^i + \rho_{b,T}^i$$

183 where the subscripts “T” and “MT” represent the thermal and combined thermal and mechanical  
 184 loading, respectively.

185 Next, the thermo-mechanical force at base, side and top of elements from  $n$  upward to the  
 186 element  $(n_1 + 1)$  can be calculated using Equations (15) to (17), as follows:

$$(15) \quad Q_{b,MT}^i = \begin{cases} Q_{b,max} \cdot f_{Q-z}^{loading}(\rho_{b,MT}^i) & \text{for } i = N \\ Q_{t,MT}^{i+1} & \text{for } i \neq N \end{cases}$$

$$(16) \quad Q_{s,MT}^i = Q_{s,T,max}^i \cdot f_{T-z}^{loading}(\rho_{s,MT}^i)$$

$$(17) \quad Q_{t,MT}^i = Q_{s,MT}^i + Q_{b,MT}^i$$

187 and the thermally induced axial stress at the middle of each element can be calculated as follows:

$$(18) \quad \sigma_T^i = \frac{Q_{t,MT}^i + Q_{b,MT}^i}{2A_b} - \sigma_M^i$$

188 where  $\sigma_M^i$  is calculated as follows:

$$(19) \quad \sigma_M^i = \frac{Q_{ave,M}^i}{A_b}$$

189 After the forces acting on these elements are defined, the actual thermal elongation of each element  
 190 is calculated as follows:

$$(20) \quad \Delta_{Tactual}^i = \Delta_T^i - \frac{\sigma_T^i \cdot L_i}{E}$$

191 The actual thermal elongation of each element will be lower than that present when the energy pile  
 192 is free to move. Equations (11) to (19) can then be recalculated using the thermal elongation from  
 193 Equation (20) for the pile elements below the null point, and this process should be repeated until  
 194 the values of restrained thermal elongation converge to a desired tolerance (i.e., when the  
 195 difference between the new and old elongations calculated with Equation (20) is less than  $10^{-10}$ ).

196 A similar process for solving for the behavior of the part of the energy pile below the null point

197 can be applied for the part above the null point but instead considering the boundary conditions at  
 198 the head of the pile. It should be noted that in the region of the pile above the null point, the pile  
 199 will move upward and the side shear stress will decrease following the unloading path shown in  
 200 Figure 2.

201 When both parts of the pile reach convergence, the unbalanced force at the null point can be  
 202 defined as follows:

$$(21) \quad F_{unb} = |Q_{t,MT}^{NP+1}| - |Q_{b,MT}^{NP}|$$

203 If the unbalanced force  $F_{unb}$  is not less than a user-defined tolerance, this means that the assumed  
 204 null point is not the actual one. If  $F_{unb}$  is positive, the actual null point is located at a lower point  
 205 than the currently assumed null point, and a new null point needs to be assumed at a lower location.  
 206 If  $F_{unb}$  is negative, the opposite is true. After a new null point is selected based on the sign of the  
 207 unbalanced force, the process starting from pile discretization needs to be repeated. When the  
 208 unbalanced force  $F_{unb}$  is less than the user-defined tolerance, the selected null point is  
 209 approximately at the actual null point and the pile can be assumed to be in thermo-mechanical  
 210 equilibrium.

## 211 Discussion of the T-z and Q-z Curves

212 The relationships between displacement and mobilized side shear resistance and toe  
 213 resistance are defined using Equations (22) and (23), respectively:

$$(22) \quad Q_s^i = Q_{s,max}^i \cdot f_{T-z}(\rho_s^i)$$

$$(23) \quad Q_b = Q_{b,max} \cdot f_{Q-z}(\rho_b^n)$$

214 where  $Q_{s,max}^i$  and  $Q_{b,max}$  are ultimate side shear force and ultimate end bearing, respectively, and  
 215  $f_{T-z}$  and  $f_{Q-z}$  are normalized force values that depend on the displacement of the pile element and

216 toe elements  $\rho_s^i$  and  $\rho_b^n$ , respectively. The relationships between  $f_{T-z}$  and  $f_{Q-z}$  and displacement  
217 are typically referred to as the T-z and Q-z curves, respectively, in axial load transfer analyses.

218 The ultimate side shear force for undrained or drained soils can be estimated using different  
219 methods depending on the soil type and the assumed effects of temperature on the surrounding  
220 soil. The former subject has been well-studied, and includes methods that assume a constant side  
221 shear with depth (typical for soil layers that are undrained during pile loading) or methods that  
222 assume the side shear increases with depth (typical for soil layers that are drained during pile  
223 loading, but can also be applied to soil layers that are undrained during pile loading). Consideration  
224 of the effects of temperature on the surrounding soil in a load transfer analysis is complex, and  
225 ideally requires a combination of heat transfer analyses (Loveridge and Powrie 2013), thermo-  
226 hydro-mechanical constitutive modeling (Laloui et al. 2015), and potential changes in stress state  
227 imposed on the soil by the differential thermal expansion of the pile and soil during heating  
228 (McCartney and Rosenberg 2011; Olgun et al. 2014b). Except in the case of normally consolidated  
229 clays, heating is expected to lead to thermo-elastic expansion of the soil in the zone of influence  
230 of the changes in temperature. Heating of the soils surrounding energy piles is likely to occur  
231 slowly. For example, Murphy and McCartney (2015) found that the temperature of a full-scale  
232 energy pile during building heating and cooling changed by approximately 20 °C over the course  
233 of 6 months. This rate of heating is expected to be slow enough to permit drainage of thermally-  
234 induced excess pore water pressures in the case that they occur, so drained heating analyses can  
235 be assumed. Nonetheless, due to a lack of experimental evidence on the impact of the thermo-  
236 hydro-mechanical response of soils surrounding energy piles, and because this study focuses on  
237 relatively stiff, non-plastic soils that are expected to behave thermo-elastically during heating, the

238 impact of temperature change on the soil properties is neglected in the thermo-mechanical load-  
239 transfer analysis.

240 The following equation can be used to calculate the side shear capacity of soils that have  
241 constant undrained shear strength with depth (referred to generally here as undrained soils):

$$(24) \quad Q_{s,max}^i = \alpha A_s^i c_u$$

242 where  $\alpha$  is an empirical reduction factor representing soil-interface behavior, and  $A_s^i$  is the pile  
243 surface area for the element of interest. The value of  $\alpha$  in this equation varies with the undrained  
244 shear strength and soil layering (e.g., Tomlinson 1971), but may also vary with temperature  
245 depending on the impact of temperature on the soil. However, temperature effects on  $\alpha$  were not  
246 considered in this study as more experimental evidence is required to support this and to ensure  
247 that temperature effects on  $\alpha$  and  $c_u$  are appropriately isolated.

248 Alternatively, the  $\beta$  method can be used to estimate the side shear capacity of soils that  
249 have a shear strength increasing with depth (referred to generally here as drained soils, even though  
250 undrained soils may also have an increasing undrained shear strength with depth). The drained  
251 shear strength analysis here includes a feature to account for the potential change in lateral stress  
252 due to differential expansion of the pile and soil similar to the approach of McCartney and  
253 Rosenberg (2011), as follows:

$$(25) \quad Q_{s,T,max}^i = [c' + \beta_T \sigma'_v(z)] A_s^i$$

254 where  $c'$  is an apparent cohesion that can be used to evaluate the impact of unsaturated conditions  
255 on the interface shear strength ( $c'$  is assumed equal to zero for dry or saturated soils), and  $\beta_T$  is a  
256 temperature-dependent side shear factor that can be defined as follows:

$$(26) \quad \beta_T = \chi_T [K + (K_p - K) K_T] \tan \phi'$$

257 where  $\chi_T$  is a fitting parameter,  $K$  is the initial coefficient of lateral earth pressure before heating,  
 258  $K_p$  is the coefficient of passive lateral earth pressure, and  $K_T$  is a reduction factor representing the  
 259 mobilization of passive earth pressure with thermal-induced strain, equal to:

$$(27) \quad K_T = \kappa \alpha_T \Delta T \left( \frac{D/2}{0.02L} \right)$$

260 where  $\kappa$  is an empirical coefficient representing the soil resistance to expansion of the pile,  $\alpha_T$  is  
 261 the coefficient of thermal expansion of reinforced concrete, and the geometric normalizing factor  
 262  $[(D/2)/0.02L]$  was proposed by Reese et al. (2006). The value of  $\kappa$  may be a stress-dependent  
 263 variable, but was assumed to be constant and equal to 65 based on centrifuge tests on Bonny silt  
 264 performed by McCartney and Rosenberg (2011). The value of  $K$  assumed in the analysis depends  
 265 on the soil type and drilled shaft construction method for the energy pile. It can be assumed that  
 266  $K = K_0$  for piles in stiff soils or rock where the open-hole method is used,  $K$  is closer to  $K_a$  for  
 267 piles in soft clay or sand. The value of  $K$  may be greater than  $K_0$  in model experiments where the  
 268 soil was placed using pluviation or compaction (e.g., Goode and McCartney 2015).

269 The ultimate end bearing  $Q_{b,max}$  for the pile can be defined for different soils as follows:

$$(28) \quad Q_{b,max} = \begin{cases} A_b c_{u,b} N_c s_c d_c & \text{for undrained soil} \\ A_b \sigma'_{zD} N_q & \text{for drained soil} \\ A_b q_u & \text{for rock or rigid materials} \end{cases}$$

270 where  $c_{u,b}$  is the undrained shear strength of the soil or rock at the pile tip,  $A_b$  is the cross sectional  
 271 area of the shaft toe,  $s_c$  is the shape factor (i.e., equal to 1.2 for a pile with a circular or square  
 272 cross-section),  $d_c$  is the depth factor (i.e., equal to 1.5 for a pile with depth over diameter ratio  
 273 larger than 2.5),  $N_c$  is the undrained bearing capacity factor for deep foundations (i.e., equal to 5  
 274 for a pile with a circular or square cross-section and a tip depth greater than 2 pile diameters),  $\sigma'_{zD}$

275 is vertical effective overburden,  $N_q$  is the bearing capacity factor related to effective friction angle  
 276 obtained from Vesic (1975), and  $q_u$  is uniaxial compressive strength of rock. The last equation in  
 277 Equation (28) is also used when simulating the behavior of model tests by Stewart and McCartney  
 278 (2013) where an energy pile was placed on a rigid aluminum plate which can be assumed to have  
 279 an extremely high value of  $q_u$ . Similar to the side shear capacity, heating of energy piles may lead  
 280 to a change in end bearing capacity. Specifically, an energy pile is heated under a mechanical load  
 281 (e.g., a building load), it is able to react against the building causing the soil at the toe to  
 282 consolidate. This may lead to a higher end bearing capacity than a pile. Further research is needed  
 283 to quantify this effect, and it is neglected in this study.

284 Examples of the Q-z and T-z curves for drilled shafts reported by O'Neill and Reese (1988)  
 285 shown in Figure 2 are nonlinear, and have a shape that is approximately hyperbolic. Accordingly,  
 286 they are represented in this study using a hyperbolic model for simplicity. The normalized side  
 287 shear resistance ( $f_{T-z}$ ) and normalized base reaction ( $f_{Q-z}$ ) at any relative displacement are given  
 288 by the following equations:

$$(29) \quad f_{Q-z}(\rho_b^n) = \frac{\rho_b^n}{a_b + b_b \rho_b^n}$$

$$(30) \quad f_{T-z}(\rho_s^i) = \begin{cases} \frac{\rho_s^i}{a_s + b_s \rho_s^i} & \text{for loading} \\ \frac{\rho_s^i}{a_s} + \frac{Q_{s,M}^i}{Q_{s,M,max}^i} - \left( \frac{1}{\frac{Q_{s,M,max}^i}{Q_{s,M}^i} - b_s} \right) & \text{for unloading} \end{cases}$$

289 where  $a_b$  and  $b_b$  are the parameters that determine the shape of the Q-z curve,  $a_s$  and  $b_s$  are  
 290 parameters that determine the shape of the T-z curve, and  $Q_{s,M}^i$  represents the side shear resistance



291 force at a given depth in the pile after the mechanical load is applied (which is the initial condition  
292 for heating). Because Murphy and McCartney (2014) found that the T-z curves are not sensitive  
293 to temperature, the T-z curves evaluated in this study are assumed to be independent of  
294 temperature. The mobilized values of base reaction ( $Q_b^n$ ) and the side shear resistance ( $Q_s^i$ ) can be  
295 obtained from Equations (29) and (30) by multiplying them by the ultimate end bearing force at  
296 the tip or the ultimate side shear resistance force at a given depth in the pile, respectively.

297 This study uses the thermo-mechanical load transfer analysis described in the previous section  
298 to evaluate two scenarios: (i) a parametric analysis in which the side shear stress is assumed to be  
299 constant with depth (typical of a soil that is undrained during shearing) or in which it is assumed  
300 to increase with depth (typical of a soil that is drained during shearing, although this could also be  
301 applied to soils that are undrained during shearing if desired), and (ii) a calibration of the model  
302 with the information from real case studies. In the latter scenario, all of the soils investigated are  
303 assumed to have the case of an increasing shear strength with depth.

## 304 **MODEL PARAMETRIC EVALUATION**

305 Using the improved thermo-mechanical load transfer analysis, a parametric evaluation was  
306 performed to understand the roles of the different model parameters (i.e., the soil shear strength  
307 parameters, toe stiffness, head stiffness, side shear stress-displacement curve, and radial  
308 expansion) and other issues (i.e., magnitude of the temperature change). In order to assess the  
309 impact of each parameter, it was first important to come up with a baseline set of soil and pile  
310 properties and loading conditions. The pile was assumed to consist of the typical concrete mixture  
311 used in drilled shaft foundations in the field. The energy pile used for the baseline case has a length  
312  $L$  of 13.1 m, a diameter  $D$  of 1.2 m, a unit weight of  $24 \text{ kN/m}^3$ , a Young's modulus  $E$  of 30 GPa,  
313 a head stiffness of  $500 \text{ kN/mm}$  and a coefficient of thermal expansion  $\alpha_T$  of  $10 \times 10^{-6} \text{ m/(m}^\circ\text{C)}$ . As

314 semi-floating energy piles are the most common type of deep foundations encountered in practice,  
315 soil-structure interaction parameters representative of this type of pile are used for understanding  
316 the effect of each parameter, where  $a_s$  is 0.0035,  $a_b$  is 0.002,  $b_s$  and  $b_b$  are 0.9. These parameters  
317 are used in Equations (29) and (30), which are used to represent the smooth Q-z and T-z curves,  
318 respectively. A load of 500 kN is applied to the pile head then a uniform change in temperature of  
319 20 °C with depth is applied. In all of the comparisons, it is assumed that the pile temperature is  
320 constant with depth, an assumption that is approximately valid based on field data (Murphy et al.  
321 2015; Murphy and McCartney 2015).

322 The thermo-mechanical axial stresses and strains, mobilized side shear stresses, and thermo-  
323 mechanical displacements for energy pile under varying friction angle for energy piles in drained  
324 soils with an ultimate side shear capacity characterized by Equation (25) are shown in Figures 3(a)  
325 through 3(d). It is clear from the results in Figure 3(a) that the thermo-mechanical axial stresses  
326 increase with increasing friction angle. The results in Figure 3(b) show that the axial thermo-  
327 mechanical strains at the head decrease with increasing friction angle, while the results in Figure  
328 3(c) show that the friction angle does not affect the mobilized side shear stresses at the head of pile.  
329 The total and maximum negative side shear stress induced by heating increases and is applied to  
330 longer part of pile with an increase of friction angle as the tip capacity increases much more  
331 significant than side shear capacity. The results in Figure 3(d) indicate that the thermo-mechanical  
332 displacement at the tip decreases and the head of the pile moves upwards with an increase in  
333 drained friction angle, and the influence of drained friction angle on displacement diminishes for  
334 soils with a higher friction angle.

335 The maximum axial thermo-mechanical, thermal, mechanical stresses and pile thermo-  
336 mechanical, thermal, mechanical total axial strains (which is defined as the difference between the

337 displacements of the pile head and pile tip, normalized by pile length) for varying friction angle  
338 values for energy pile in drained soils are shown in Figures 4(a) and 4(b). It can be observed from  
339 the results in Figure 4(a) that the thermo-mechanical axial stresses increase approximately linearly  
340 with an increase of friction angle ranging from 20 to 35°, and the largest thermo-mechanical axial  
341 stress is close to the head of pile. The results in Figure 4(b) show that the total pile expansion due  
342 to thermal and thermo-mechanical loading decreases with increasing friction angle. This is because  
343 the tip and side shear resistances increase with friction angle, leading to larger axial stresses to  
344 resist the pile expansion caused by heating.

345         The thermo-mechanical axial stresses and strains, mobilized side shear stresses, and thermo-  
346 mechanical displacements for an energy pile under varying undrained shear strength for energy  
347 pile in undrained soils with an ultimate side shear capacity described by Equation (24) are shown  
348 in Figures 5(a) through 5(d). It is clear from the results in Figure 5(a) that the axial thermo-  
349 mechanical stresses increase with increasing undrained shear strength, as expected. Figure 5(b)  
350 shows that the thermo-mechanical strains decrease with increasing undrained shear strength. As  
351 the undrained shear strength increases, the profiles of thermo-mechanical axial stresses and strains  
352 become increasingly nonlinear with depth. The axial thermo-mechanical strain decrease with  
353 increasing undrained shear strength, and the increase in undrained shear strength causes the null  
354 point to move downward and the thermo-mechanical stresses and strains to increase. The results  
355 in Figure 5(c) show a similar effect as that of the friction angle, where the total downward  
356 mobilized side shear stresses increase, and the total upward mobilized side shear stresses decrease  
357 with increasing undrained shear strength. The difference is that the increasing undrained shear  
358 strength leads to a greater increase in the downward mobilized side shear stresses at the head of

359 the pile. The results in Figure 5(d) show that the thermo-mechanical displacement decreases with  
360 increasing undrained shear strength.

361 The maximum thermo-mechanical, thermal and mechanical axial stresses and the thermo-  
362 mechanical, thermal, mechanical total axial strains in undrained soils are shown in Figure 6. The  
363 maximum thermo-mechanical stresses in the pile in Figure 6(a) are observed to increase with an  
364 increase in undrained shear strength of soil, while the maximum mechanical stress is not sensitive  
365 to the undrained shear strength of the soil. The results in Figure 6(b) shows that the thermo-  
366 mechanical expansion decreases with increasing undrained shear strength, which occurs because  
367 an increase in undrained shear strength leads to an increase in both mobilized side shear stress and  
368 toe resistance, which serve to resist thermal expansion.

369 The effect of the magnitude of the change in temperature on the maximum values of axial  
370 stresses and total axial strains in terms of mechanical, thermal, and thermo-mechanical axial  
371 loading are shown in Figures 7(a) and 7(b), respectively. The results in both figures show that the  
372 maximum thermal and thermo-mechanical axial stresses and total axial strains increase  
373 approximately linearly with an increase of temperature. This is because the thermal expansion of  
374 the pile is linear and thermo-elastic, and the changes in temperature are not sufficient to reach the  
375 plastic portion of the T-z or Q-z curves for the axial loading case considered.

376 The effect of toe stiffness on maximum values of axial stresses and pile total strain in terms  
377 of mechanical, thermal, and thermo-mechanical values are shown in Figures 8(a) and 8(b),  
378 respectively. It can be observed from Figure 8(a) that the influence of  $a_b$  on the maximum thermal  
379 and thermo-mechanical axial stresses becomes less significant as  $a_b$  increases. This occurs because  
380 the initial stiffness of the mobilized base resistance Q-z curve decreases with increasing  $a_b$ , which  
381 leads to a softer base and allows more downward displacement. When there is less base resistance,

382 the maximum axial stress decreases. A similar influence of toe stiffness on the trends in total axial  
383 strains can be observed in Figure 8(b).

384 The effect of head stiffness on maximum values of axial stresses and total axial strains in  
385 terms of mechanical, thermal, and thermo-mechanical values are shown in Figures 9(a) and 9(b).  
386 The results in both figures show that the maximum stresses and total axial strains increase with an  
387 increase in head stiffness, and the influence of  $K_h$  on the thermo-mechanical stresses and pile  
388 thermal expansions become less significant as  $K_h$  increases. This is because an increase in  $K_h$   
389 lowers the uplift displacement by increasing the head resistance and larger side shear upward stress  
390 and toe resistance will be generated in order to achieve equilibrium, which leads to increases in  
391 axial stresses. As the head resistance stress increases, the null point moves upward and a larger  
392 part of the energy pile experiences a downward movement, which leads to an increase in the  
393 maximum displacements. Slightly larger axial stress on the top cause a lower maximum  
394 displacement by leading to a decrease in thermal expansion. When  $K_h$  is very large, the null point  
395 is very close to the head of pile. The uplift displacement at the pile head increases slightly despite  
396 the large changes in  $K_h$ . As the slope of the T-z curve and Q-z curve depends on the magnitude of  
397 displacement, in this case the axial displacements are insignificant so there is not a major effect of  
398  $K_h$  on the axial stresses and strains.

399 The effect of the parameters of the side shear stress-displacement curve on the maximum  
400 values of axial stresses and total axial strains in terms of mechanical, thermal, and thermo-  
401 mechanical values are shown in Figures 10(a) and 10(b). The results in Figure 10(a) indicate that  
402 the maximum thermal and thermo-mechanical axial stress decreases with increases in  $a_s$ , and the  
403 influence of parameter  $a_s$  become insignificant when  $a_s$  is large. The results in Figure 10(b) indicate  
404 that an increase of parameter  $a_s$  leads to a slight increase of the total axial strains.

405 The effect of radial expansion on maximum values of axial stresses and total axial strains in  
406 terms of mechanical, thermal, and thermo-mechanical values are shown in Figures 11(a) and 11(b).  
407 The results in both Figures 11(a) and 11(b) indicate that radial expansion has a negligible influence  
408 on axial stresses and total axial strains. The results from this evaluation indicate that the effect of  
409 radial displacement is relatively small and can be neglected in the load transfer analysis, indicating  
410 that the trends in the load-settlement curves of McCartney and Rosenberg (2009) are more likely  
411 due to the effects of changing unsaturated conditions in the soil rather than radial expansion effects,  
412 although it is possible that the high initial lateral stresses induced by compaction in their tests may  
413 have had some effect on the stresses induced by lateral expansion.

#### 414 **MODEL CALIBRATION WITH FIELD/CENTRIFUGE DATA**

415 The updated load-transfer model was calibrated to evaluate the expected soil-structure  
416 interaction response of four case studies, including one field study (Murphy et al. 2015) and three  
417 centrifuge studies (Stewart and McCartney 2013; Goode and McCartney 2015; Ng et al. 2014).  
418 The calibration of the model to these studies permits evaluation of the typical ranges of values of  
419 the different model parameters that are difficult to measure in the field, including the parameters  
420 of the T-z and Q-z curves fitting parameter  $\chi_T$  and the head stiffness  $K_h$ . All of these studies  
421 involve non-plastic soils with a drained loading response and negligible temperature effects on the  
422 soil properties.

#### 423 **Case #1 Murphy et al. (2015)**

424 In this study, several energy piles were constructed beneath a one-story, shower-shave  
425 building constructed at the US Air Force Academy (USAFA) beginning in March 2012. A site  
426 investigation was performed in September 2011, which consisted of two 102 mm-diameter borings  
427 located within the building footprint, extending 12 and 7 m below the ground surface. At selected

428 intervals, disturbed samples were obtained by driving a split-spoon sampler. Exploration results  
429 from both boreholes were similar and showed three prominent strata, and relevant data are  
430 presented in Murphy et al. (2015). One of the end bearing concrete energy piles with a length of  
431 15.2 m and a diameter of 0.61 m was considered in this evaluation (Foundation 4). The pile has a  
432 unit weight of  $24 \text{ kN/m}^3$ , a Young's modulus of 30 GPa, and a coefficient of thermal expansion is  
433  $12 \times 10^{-6} \text{ m/m}^\circ\text{C}$ , and the empirical coefficient for radial expansion is assumed to be 65. An axial  
434 mechanical load of 833 kN was applied to the pile before changes of temperature of  $\Delta T = 6, 12,$   
435  $19^\circ\text{C}$  were applied to heat the pile. For simplicity, the subsurface stiff gravel and sandstone layers  
436 are assumed to be one equivalent layer with a friction angle of  $43.6^\circ$  and a unit weight of  $19.2$   
437  $\text{kN/m}^3$ . An uniaxial compressive strength at the toe  $q_u$  of 12000 kPa was assumed for the intact  
438 sandstone at the toe. The calibrated load transfer parameters that provided the best fit to the data  
439 are:  $a_b = 0.002$  and  $b_b = 0.9$  for Q-z curve,  $a_s = 0.0003$ ,  $b_s = 0.9$ ,  $\beta = 0.9$  for the T-z curve, fitting  
440 parameter  $\chi_T = 2.5$  and head stiffness  $K_h$  of 900 kN/mm.

441 The processed field data and simulation results for Foundation 4 are shown in Figure 12. The  
442 calibrated results in Figure 12 indicate a good estimate of the energy pile response in terms of the  
443 axial compressive stresses and strains and displacements induced by heating. Although the overall  
444 trend from the model is consistent with the field data, inconsistencies are observed for depths  
445 between 0 to 3 m, possibly because the model assumes a homogeneous layer of soil.

#### 446 **Case #2 Goode and McCartney (2015)**

447 In this study, a scale-model semi-floating concrete energy pile having a diameter of 63.5 mm  
448 and a length of 342.9 mm (short pile) was heated in Bonny silt and Nevada sand, respectively, at  
449 a centrifuge g-level of 24 under an applied axial stress of 360 kPa. The corresponding prototype-  
450 scale energy pile has a diameter of 1.5 m and a length of 8.2 m, a unit weight of  $24 \text{ kN/m}^3$ , a

451 Young's modulus of 33 GPa, a coefficient of thermal expansion  $16 \times 10^{-6}$  m/m°C, and no head-  
452 structure restraint. The friction angles for Nevada sand and Bonny silt tested in this study are  $35^\circ$   
453 and  $32.4^\circ$ , respectively, and the unit weight for Nevada sand and for Bonny silt are 14.8 and 17.9  
454 kN/m<sup>3</sup>, respectively. Because the Bonny silt was tested in unsaturated conditions, an apparent  
455 cohesion  $c'$  of 30 kPa was assumed for the particular compaction conditions. The empirical  
456 coefficient for radial expansion is assumed to be 65 based on the study of McCartney and  
457 Rosenberg (2010), although this parameter is not expected to have a major effect on the  
458 simulations. The calibrated parameters that provided the best fit to the data are:  $a_b = 0.006$  and  $b_b$   
459  $= 0.9$  for Q-z curve,  $a_s = 0.0002$ ,  $b_s = 0.9$ , for the T-z curve and fitting parameter  $\chi_T = 2.5$  for  
460 Nevada sand; and  $a_b = 0.009$  and  $b_b = 0.9$  for Q-z curve,  $a_s = 0.0002$ ,  $b_s = 0.9$ , for the T-z curve,  
461 fitting parameter  $\chi_T = 2.5$ .

462 The comparisons of the calibrated model results with the experimental data for Nevada sand  
463 and Bonny silt are shown in Figures 13 and 14, respectively. Goode and McCartney (2015) plotted  
464 the thermal axial displacement values against the location at the midpoint between two strain  
465 gages. However, this should be plotted against the location of the upper gage so the experimental  
466 data from Goode and McCartney (2015) were re-analyzed so that the calculated thermal axial  
467 displacement values correspond to the location of the upper gage. The results shown in these  
468 figures indicate a good match between the overall trend of axial stresses and strains induced by  
469 thermal loading in this case study. The difference between the trends in the thermal strain and  
470 stress are probably due to the small differences between the average temperatures applied in the  
471 model from the actual temperature profile in the experiment, which was not completely uniform.



472 **Case #3 Stewart and McCartney (2014)**

473 In this study, a small-scale, end-bearing concrete model energy pile with dimensions of 50.8  
474 mm in diameter and 533.4 mm in length was heated in a layer of unsaturated Bonny silt in the  
475 centrifuge at a g-level of 24.6 under an applied axial stress of 443 kPa. The corresponding  
476 prototype-scale energy pile has a length of 12.8 m and a diameter of 1.22 m, with a unit weight of  
477 24 kN/m<sup>3</sup>, a Young's modulus of 7.17 GPa, a coefficient of thermal expansion  $7.5 \times 10^{-6}$  m/m°C,  
478 and no head-structure restraint. The empirical coefficient for radial expansion is assumed to be 65.  
479 The friction angle used in this case is 32.4° and dry weight of soil is 16.9 kN/m<sup>3</sup>. For this case, the  
480 unsaturated Bonny silt was found to have an apparent cohesion  $c'$  of 30 kPa. A mechanical load of  
481 443 kN was applied to the pile head before heating in load-control conditions. A uniaxial  
482 compressive strength at the toe  $q_u$  of 25000 kPa was used in the analysis as the toe of the pile is  
483 supported by a relatively rigid aluminum plate. The calibrated parameters that provided the best  
484 fit for the data are:  $a_b = 0.002$  and  $b_b = 0.9$  for Q-z curve,  $a_s = 0.0008$ ,  $b_s = 0.9$ , for the T-z curve,  
485 fitting parameter  $\chi_T = 2.5$ .

486 The predicted axial compressive stress profiles using load-transfer model analysis along with  
487 the centrifuge data for each temperature change condition are shown in Figures 15(a) through  
488 15(c). Stewart and McCartney (2014) plotted the thermal axial displacement values against the  
489 location at the midpoint between two strain gages. However, this should be plotted against the  
490 location of the upper gage so the experimental data from Stewart and McCartney were re-analyzed  
491 so that the calculated thermal axial displacement values correspond to the location of the upper  
492 gage. A good match was observed between the calibrated and measured data in Figures 15(a)  
493 through 15(c). The worst fit was observed near the head of the pile for small changes in

494 temperature, likely due to differences between the average pile temperature and the actual pile  
495 temperature at that depth.

#### 496 **Case #4 Ng et al. (2015)**

497 In this study, three instrumented, semi-floating aluminum alloy model pipe piles having an  
498 external diameter of 22 mm and length of 600 mm were tested in saturated sand at a centrifuge g-  
499 level of 40. The corresponding prototype-scale energy pile has a length of 19.6 m, a diameter of  
500 0.88 m, a Young's modulus of 70 GPa and a coefficient of thermal expansion of  $22.2 \times 10^{-6}$   
501  $\text{m}/(\text{m} \cdot \text{K})$ . No head-structure restraint or mechanical load was applied at the head of piles. The  
502 empirical coefficient for radial expansion is assumed to be 65, despite the difference in materials  
503 from the previous studies, due to the lack of strong effect of this variable. All the tests were carried  
504 out in saturated Toyoura sand with a buoyant unit weight  $\gamma = 9.4 \text{ kN}/\text{m}^3$  and a friction angle of  $31^\circ$ .

505 The calibrated parameters that provided the best fit for the data are:  $a_b = 0.07$  and  $b_b = 0.9$  for  
506 the Q-z curve, and  $a_s = 0.006$ ,  $b_s = 0.9$  for the T-z curve. The fitting parameter  $\chi_T$  is assumed to  
507 be 1, since the coefficient of friction of aluminum with sand is more than half that of concrete and  
508 sand. The predicted thermal induced axial force profiles obtained using load-transfer model  
509 analysis along with the experimental data for different changes in temperature are shown in Figure  
510 16. The load transfer results are generally consistent with the centrifuge data, although there are  
511 discrepancies in the region around the null point. The trends in the experimental data with depth  
512 in this study were more nonlinear than those observed in the other studies, with a lower null point.  
513 This could be due to the characteristics of the pile or uncertainties in the nonlinearity of the pile-  
514 sand interface shear resistance.

## 515 **Case Study Summary**

516 A summary of the model parameters from the four different studies is presented in Table 1.  
517 The sands and sandstone were both represented using drained shear strength analyses, while the  
518 silt was represented using an undrained analysis with a linear distribution with depth, with the  
519 ultimate side shear distribution with depth described by Equation (25) for all of the soils. The end  
520 bearing resistance was modeled depending on the corresponding end bearing condition. The  
521 differences in the model parameters for the Bonny silt layers tested by Stewart and McCartney  
522 (2014) and Goode and McCartney (2015) can be attributed to the different compaction conditions  
523 for the soil layers tested. The value  $\chi_T$  was found to be highly affected by the type of material and  
524 roughness on the side of the pile. The energy piles in the centrifuge were installed by placing the  
525 soil around the piles, which may have led to different interface shear strengths from those expected  
526 of a full-scale pile in the field.

## 527 **DESIGN EVALUATION CHARTS**

### 528 **Soil-Structure Interaction Curves**

529 The Q-z curves and T-z curves used in the simulation of the different case histories with the  
530 parameters listed in Table 1 are compared in Figures 17(a) and 17(b), respectively. Although there  
531 is some spread in the curves depending on the stiffness of the soil and the constraint provided at  
532 the base of the pile, the curves all fall within a reasonable band. The data points for the mobilized  
533 toe resistance and displacement values for changes in temperature of 10 and 20 °C are shown on  
534 top of the curves in Figure 17(a) for comparison. In most of the cases the normalized tip resistance  
535 value does not exceed 0.3 and falls in the linear range of the Q-z curve. Accordingly, it is fair to  
536 consider the Q-z curve to be linear for simplifying the computation in practice. It can be observed  
537 from Figure 17(b) that the mobilized side shear stress reaches an ultimate value at low

538 displacements. Accordingly, it is important to carefully consider ultimate side shear capacity for a  
539 given energy pile in design as this can affect the nonlinearity of trends with depth. The curves used  
540 to represent the behavior of the aluminum pile in sand tested by Ng et al. (2015) are softer than the  
541 other curves, potentially due to the loose sand and the smoother interface of the aluminum pile.

## 542 **Ratios of the Mobilized Resistance to the Ultimate Resistance**

543 A plot of the variations in the ratio of the total thermo-mechanical mobilized resistance to the  
544 ultimate resistance for the pile base resistance  $Q_{b,MT}/Q_{b,max}$  and the side shear resistance  
545  $Q_{s,MT}/Q_{s,max}$  are shown in Figure 18. For each of the case histories,  $Q_{b,MT}$  is obtained by performing  
546 the thermo-mechanical load transfer analysis using calibrated parameters presented in Table 1,  
547 except the head stiffness  $K_h$  are varying from 0 to 1000 MN/m. The value of  $Q_{s,MT}$  was obtained  
548 by summing the side shear resistances of each pile element from the simulation results, while the  
549 value of  $Q_{b,max}$  was calculated using Equation (28). The value of  $Q_{s,max}$  is calculated by summing  
550 the ultimate side shear force for each elements using Equation (25). Upward and downward  
551 resistances are taken as positive and negative, respectively.

552 During heating, the base resistance systematically increases. As observed in Figure 18, the  
553 value of  $Q_{b,MT}/Q_{b,max}$  increases by an insignificant compared to increase in the value of  $Q_{s,MT}/Q_{s,max}$   
554 as the head stiffness  $K_h$  is varied from 0 to 1000 MN/m. The value of  $Q_{s,T}/Q_{s,max}$  increases linearly  
555 with the value of  $Q_{b,T}/Q_{b,max}$  for each case with a slope in the range of 8 to 50 depending on the  
556 stiffness of the soil-pile interface. The head stiffness has a significant influence on the side shear  
557 resistance caused by heating. It is clear that the aluminum pile tested by Ng et al. (2015) showed  
558 the most nonlinearity and widest mobilization ratios in all of the cases studied due to the higher  
559 coefficient of thermal expansion of the pile and the comparably softer load transfer curves. The  
560 thermo-mechanical stresses are closer to failure (i.e., ratios closer to 1.0). More nonlinear behavior

561 in the side shear resistance is observed as the head stiffness increases, leading to larger downward  
562 displacements of the pile. It should be noted that it is increasingly possible that creep strains or  
563 cyclic effects may be encountered for piles loaded closer to failure (Pasten and Santamarina 2014).

#### 564 **Load-Settlement Relationships for the Head of the Energy Pile**

565 It is important to understand the variation of head displacement for design because if it is  
566 more than an allowable limit for the structure, then the structure may fail by the “Serviceability  
567 Limit State”. The variations in head settlement ( $\rho_{i,T}$ ) and head load ( $Q_{i,T}$ ) induced by temperature  
568 variations for different head stiffness values are presented in Figure 19. Downward head load and  
569 head settlement are taken as positive. The values of  $\rho_{i,T}$  and  $Q_{i,T}$  in Figure 19 are obtained from  
570 thermo-mechanical load transfer analysis using parameters presented in Table 1 with stiffness  $K_h$   
571 varying from 0 to 1000 MN/m. An interesting observation is that most of the curves for the piles  
572 made of concrete have a similar slope, which may be affected by the magnitudes of the Young’s  
573 modulus and coefficient of thermal expansion of the pile, as well as the shapes of the T-z and Q-z  
574 curves. This plot is useful to assess the maximum thermal axial stress and displacement at the head  
575 of an energy pile made from a given material. It also emphasizes that very stiff pile elements with  
576 high thermal expansion (i.e., aluminum) may have different behavior than softer pile elements with  
577 lower thermal expansion (i.e., concrete).

#### 578 **CONCLUSIONS**

579 This study involved the development of a thermo-mechanical load-transfer analysis for  
580 capturing the behavior of energy piles during mechanical loading and monotonic heating. The key  
581 for successful simulation was found to be an accurate identification of the null point location. Once  
582 the null point was identified, the status of axial strain and stress in the energy pile were iteratively  
583 computed to reach equilibrium in the upper and lower parts of the pile considering compatibility

584 of displacements between the soil and pile. A parametric evaluation was performed to identify the  
585 effects of different parameters on the axial stress and strain distributions within the pile and the  
586 total axial strain computed from the displacements at the head and toe of the pile. The model for  
587 the ultimate side shear capacity used in the analysis was found to affect the behavior of the energy  
588 pile by affecting the ultimate capacity and the mobilized side shear stress distribution at different  
589 depth. The increasing changes in temperature caused the energy pile to expand resulting in larger  
590 resistance to movement at the heat, toe and sides of the pile, leading to increases in axial stress in  
591 the pile. The temperature had a linear effect on the maximum axial stress in the pile for the range  
592 of temperatures investigated. The toe stiffness mainly affects the displacements at the ends of the  
593 pile and the magnitude of the axial stress near the toe of the pile. The head stiffness leads to greater  
594 downward toe displacements as well as greater stresses along the pile due to the greater  
595 mobilization of the end bearing resistance. The impact of the side shear stress-displacement curve  
596 mainly lies in the thermally-induced stress distribution along the pile. The radial expansion was  
597 found to cause a slight increase in side shear capacity, but this effect was small enough to be  
598 neglected.

599 The thermo-mechanical load-transfer model was calibrated using the results from four case  
600 studies involving energy piles in non-plastic soils or rock in order to evaluate the typical ranges of  
601 values of the model parameters. The calibrated results were synthesized to provide preliminary  
602 design charts in terms of the ratio of the mobilized resistance to the ultimate resistance and the  
603 head load and head settlement. However, the evaluation of more case studies need to be evaluated  
604 using the load transfer analysis to fully delineate these trends.

605 Issues that require further evaluation using the load transfer method include the impacts of  
606 thermal volume change of the surrounding soil or rock on the changes in ultimate side shear and

607 end bearing capacities and shapes of the T-z and Q-z curves. As this study was focused on non-  
608 plastic soils, the effect of this issue was assumed to be insignificant. However, energy piles may  
609 be installed in soft clays or expansive clays where appreciable volume changes or cyclic effects may  
610 be encountered in the soil leading to changes in the ultimate side shear or end bearing values.  
611 Another issue that deserves further study is the role of heating and cooling superimposed atop an  
612 initial force-displacement condition induced by mechanical loading. This requires modifying the  
613 shapes for the nonlinear Q-z and T-z curves to capture the hysteretic heating and cooling trends.  
614 Although this has been considered in other studies for elastic-perfectly plastic models, this  
615 modification is still required for the hyperbolic curves used in this study. Although the hyperbolic  
616 model can be used to form a backbone curve in the directions of monotonic heating and cooling,  
617 a transition function is required to extend the unloading path from the point of zero stress to the  
618 backbone curve in the other direction of displacement, which will require additional experimental  
619 validation.

## 620 **ACKNOWLEDGMENTS**

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622 of this paper reflect the views of the authors and do not necessarily reflect the views of the sponsor.

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740 **LIST OF FIGURE CAPTIONS**

741 **FIG. 1:** Typical foundation schematic highlighting the location of the null point

742 **FIG. 2:** Typical nonlinear spring inputs and hyperbolic approximation for the load transfer  
743 analysis: (a) Q-z curve; (b) T-z curve

744 **FIG. 3:** Soil-structure interaction behavior of a semi-floating energy pile in drained soils:  
745 (a) Thermo-mechanical axial stresses; (b) Thermo-mechanical axial strains; (c) Mobilized side  
746 shear stresses; (d) Thermo-mechanical displacements

747 **FIG. 4:** Soil-structure interaction behavior of a semi-floating energy pile in drained soils:  
748 (a) Maximum stresses vs.  $\phi$ ; (b) Pile total axial strains vs.  $\phi$

749 **FIG. 5:** Soil-structure interaction behavior of a semi-floating energy pile in undrained soils:  
750 (a) Thermo-mechanical axial stresses; (b) Thermo-mechanical axial strains; (c) Thermal axial  
751 stresses; (d) Thermal axial strains

752 **FIG. 6:** Soil-structure interaction behavior of a semi-floating energy pile in undrained soils:  
753 (a) Maximum stresses vs.  $c_u$ ; (b) Pile total axial strains vs.  $c_u$

754 **FIG. 7:** Comparison of the impact of average pile temperature on soil-structure interaction  
755 behavior: (a) Maximum stresses vs. temperature; (b) Pile total axial strains vs. temperature

756 **FIG. 8:** Comparison of the impact of toe stiffness on soil-structure interaction behavior:  
757 (a) Maximum stresses vs.  $a_b$ ; (b) Pile total axial strains vs.  $a_b$

758 **FIG. 9:** Comparison of the impact of head stiffness on soil-structure interaction behavior:  
759 (a) Maximum stresses vs.  $K_h$ ; (b) Pile total axial strains vs.  $K_h$

760 **FIG. 10:** Comparison of the impact of T-z curve on soil-structure interaction: (a) Maximum  
761 stresses vs.  $a_s$ ; (b) Pile total axial strains vs.  $a_s$

762 **FIG. 11:** Comparison of the impact of radial expansion on soil-structure interaction: (a) Maximum  
763 stresses vs.  $\kappa$ ; (b) Pile total axial strains vs.  $\kappa$

764 **FIG. 12:** Simulated profiles of end bearing energy pile #4 with field test data by Murphy et al.  
765 (2015): (a) Thermal axial strains; (b) Thermal axial stresses; (c) Thermal axial displacements

766 **FIG. 13:** Simulated profiles of semi-floating energy pile with centrifuge model test data in Nevada  
767 sand by Goode and McCartney (2015): (b) Thermal axial strains; (d) Thermal axial  
768 displacements; (f) Thermal axial stress

769 **FIG. 14:** Simulated profiles of semi-floating energy pile with centrifuge model test data in Bonny  
770 silt by Goode and McCartney (2015): (a) Thermal axial strains; (b) Thermal axial  
771 displacements; (c) Thermal axial stress

772 **FIG. 15:** Simulated profiles of end-bearing pile with centrifuge model test data by Stewart and  
773 McCartney (2014): (a) Thermal axial strains; (b) Thermal axial displacements; (c) Thermal  
774 axial stress

775 **FIG. 16:** Simulated profiles of thermal axial forces for the semi-floating pile tested by Ng et al.  
776 (2015)

777 **FIG. 17:** Ranges in nonlinear spring inputs for the load transfer analysis obtained from the  
778 evaluation of the case studies: (a) Q-z curves; (b) T-z curves

779 **FIG. 18:** Ratios of  $Q_{b,MT}/Q_{b,max}$  and  $Q_{s,MT}/Q_{s,max}$  for different head restraint conditions and changes  
780 in temperature of  $\Delta T = 20^\circ\text{C}$

781 **FIG. 19:** Head resistance vs. head displacement plots at equilibrium for piles under a change in  
782 temperature of  $\Delta T = 20^\circ\text{C}$

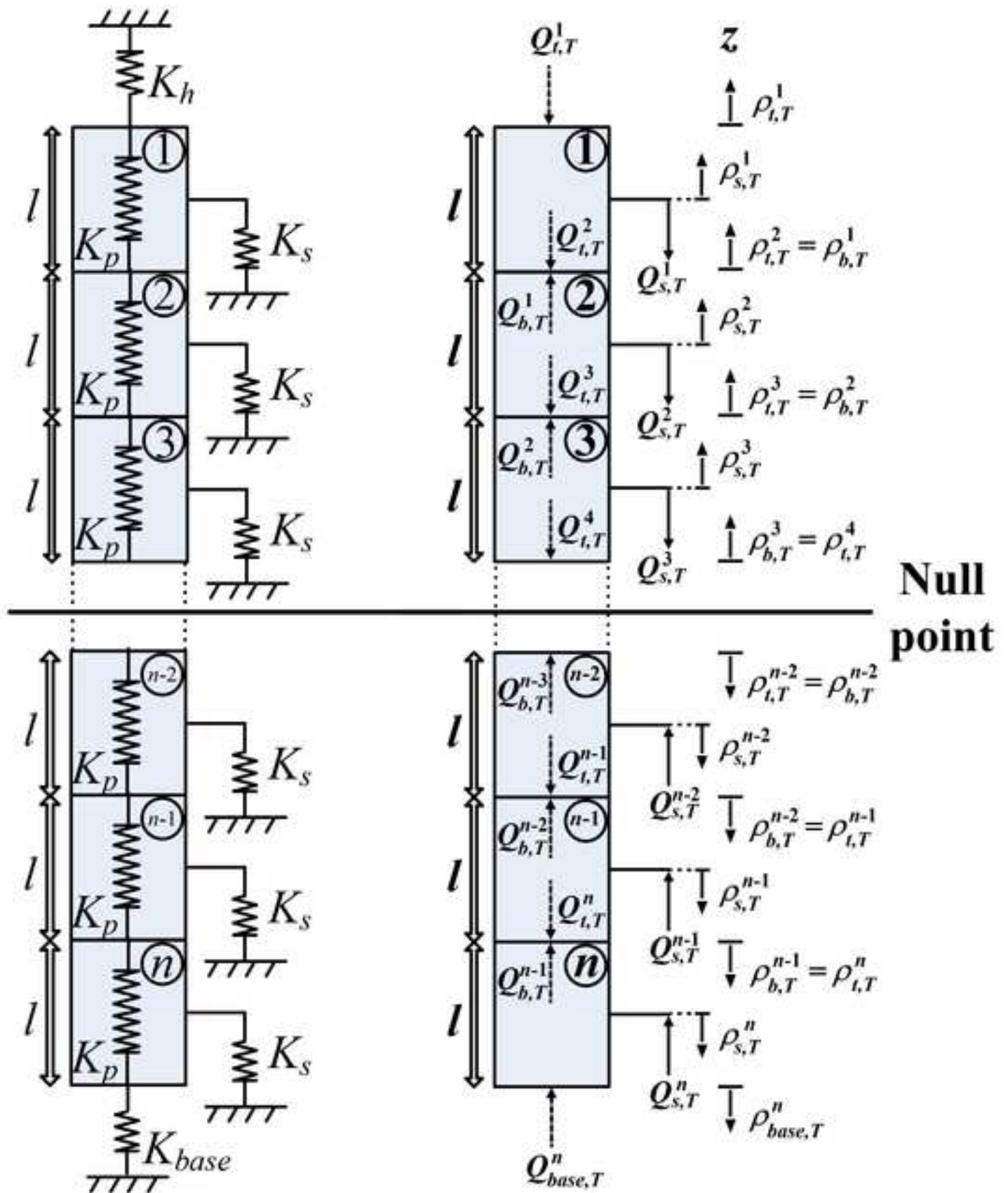
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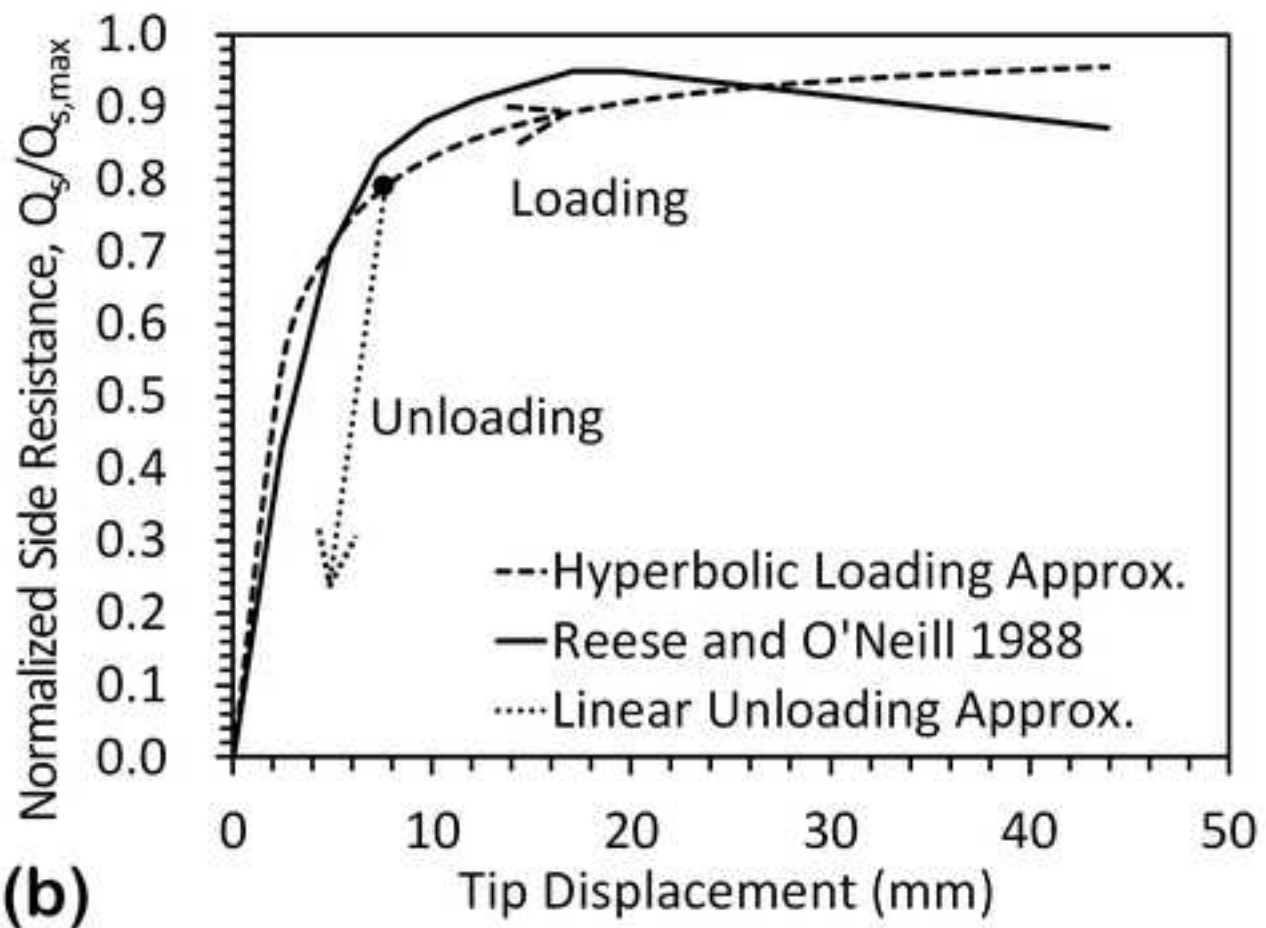
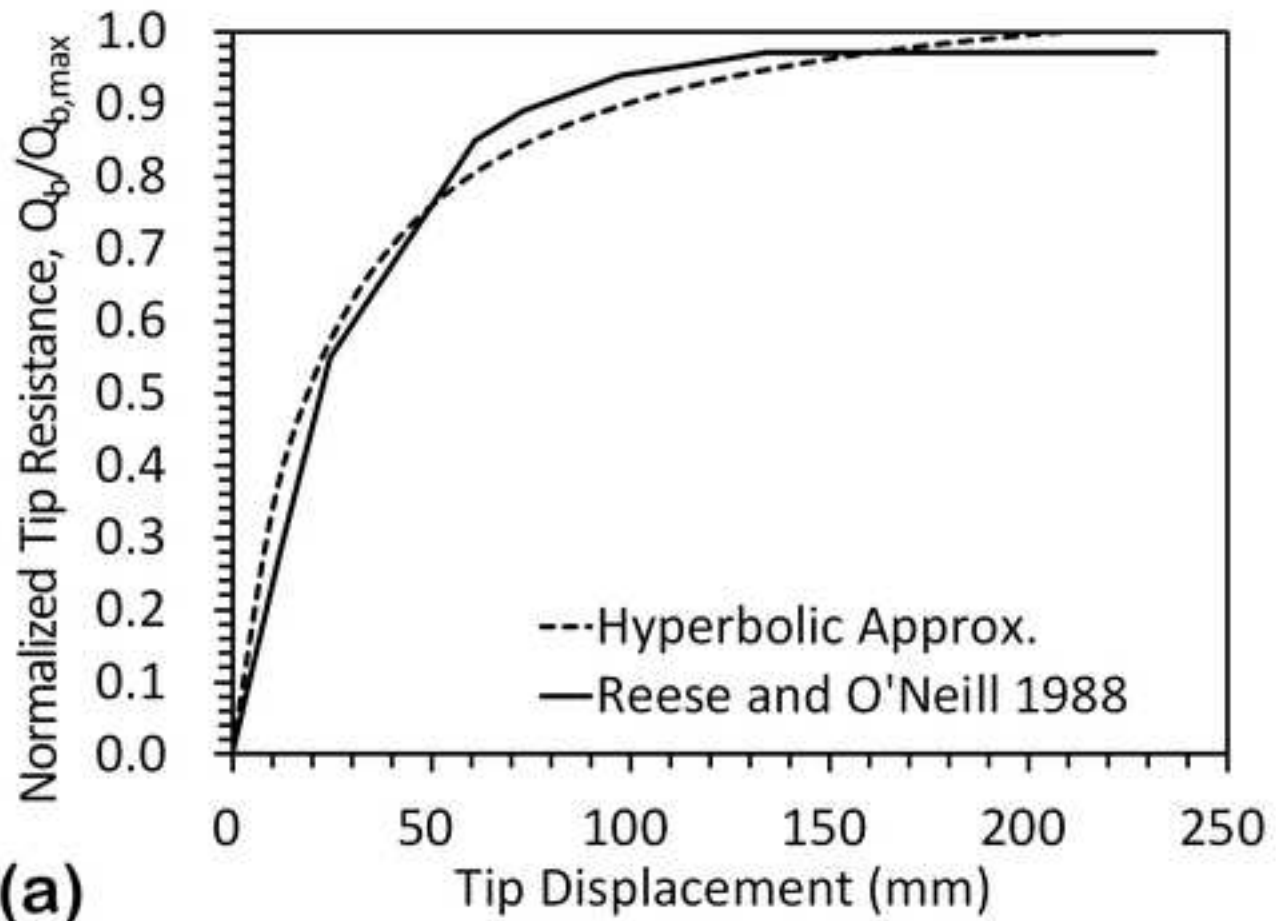
785 **TABLE 1:** Summary of model parameters for different energy piles reported in the literature

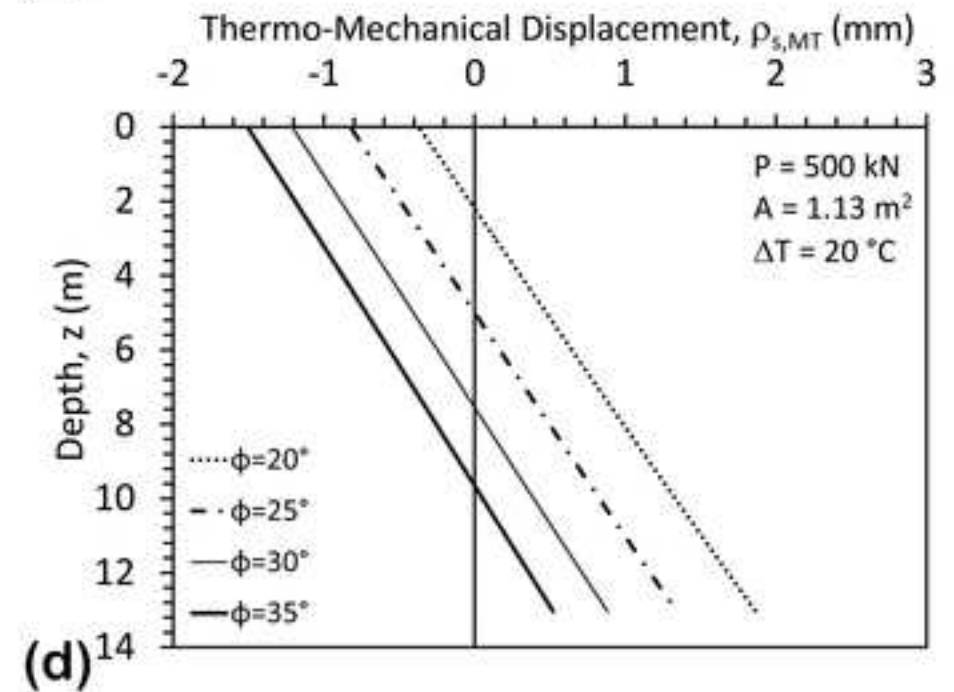
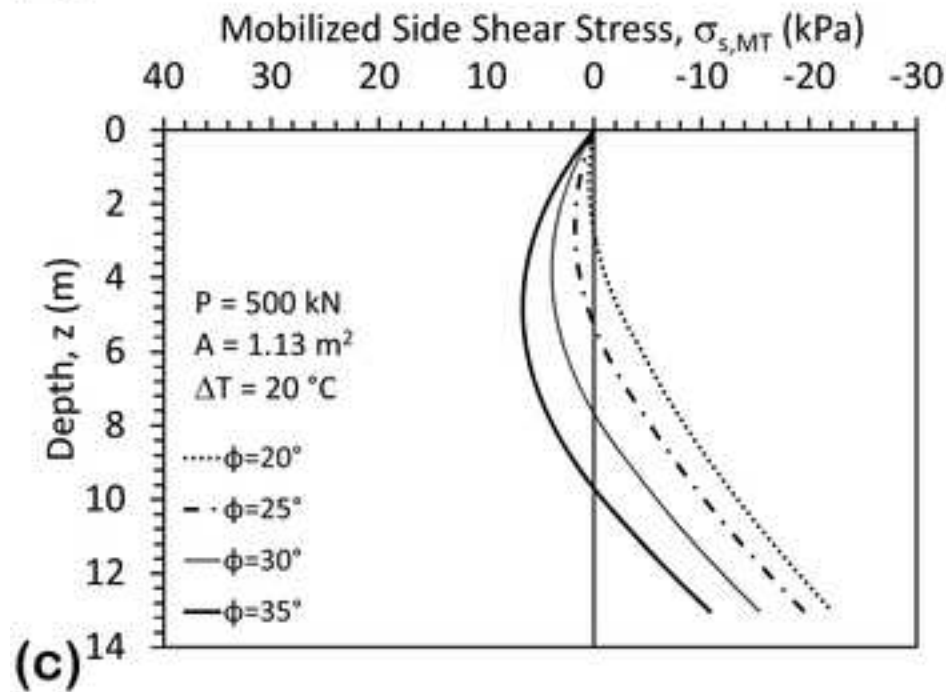
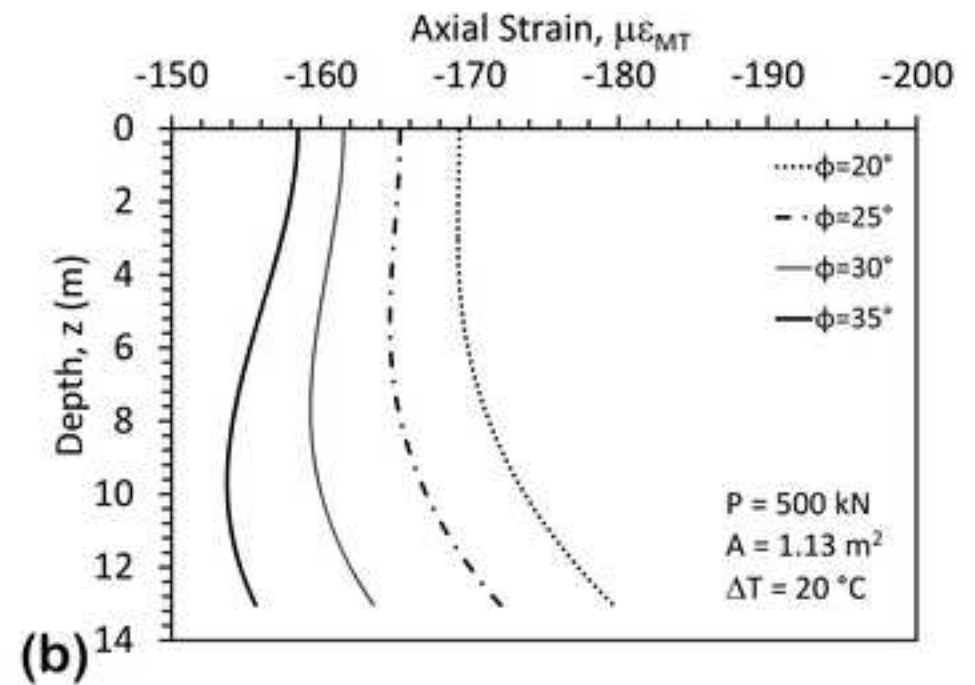
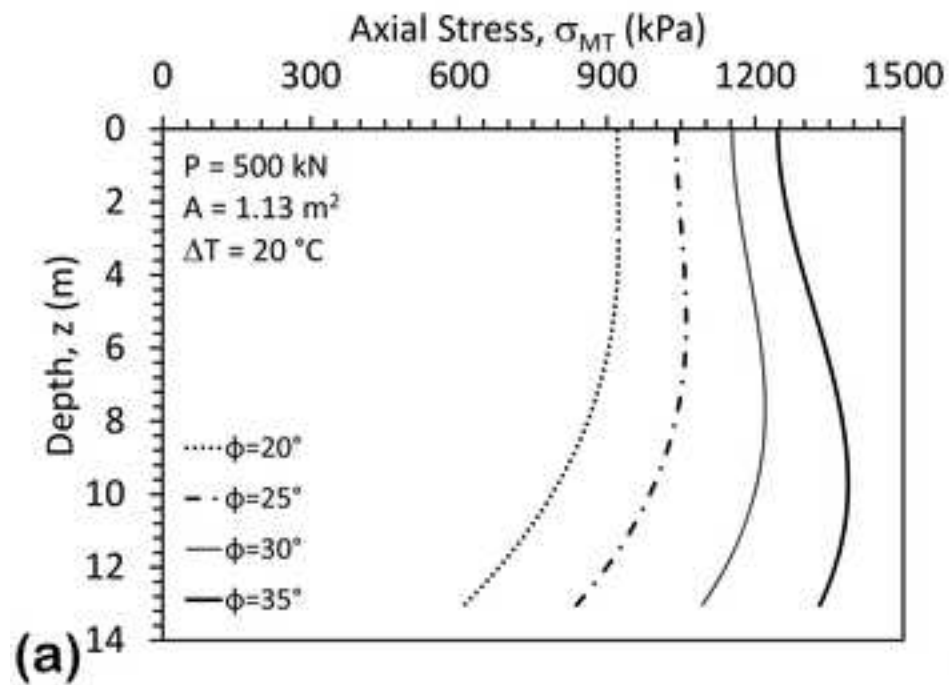
	Murphy et al. (2015)	Goode and McCartney (2015)	Stewart and McCartney (2014)	786 Ng et al. (2015)	
Soil Type	Sand	Nevada Sand	Bonny Silt	Bonny silt	Toyoura Sand
Foundation Type	Concrete	Concrete	Concrete	Concrete	Aluminum
$L$ (m)	15.2	8.2	8.2	12.8	19.6
$D$ (m)	0.61	1.5	1.5	1.22	0.88
$\alpha_T$ ( $\mu\epsilon/^\circ\text{C}$ )	-12	-16	-16	-7.5	-22
$E$ (GPa)	30	33	33	7.17	70
$K_h$ (kN/mm)	900	-	-	-	-
$P$ (kN)	833	360	360	443	-
$\gamma$ (kN/m <sup>3</sup> )	19.2	14.2	17.9	17.9	9.4
$\phi'$ ( $^\circ$ )	43.6	35.0	32.4	32.4	31
$N_q$	-	33	24	-	21
$q_u$ (kPa)	12000	-	-	25000	
$c'$ (kPa)	-	-	30	30	-
$\chi_T$	2.5	2.5	2.5	2.5	1
$a_s$	0.0003	0.0002	0.0002	0.0008	0.006
$a_b$	0.002	0.006	0.009	0.002	0.07
$b_s$	0.9	0.9	0.9	0.9	0.9
$b_b$	0.9	0.9	0.9	0.9	0.9

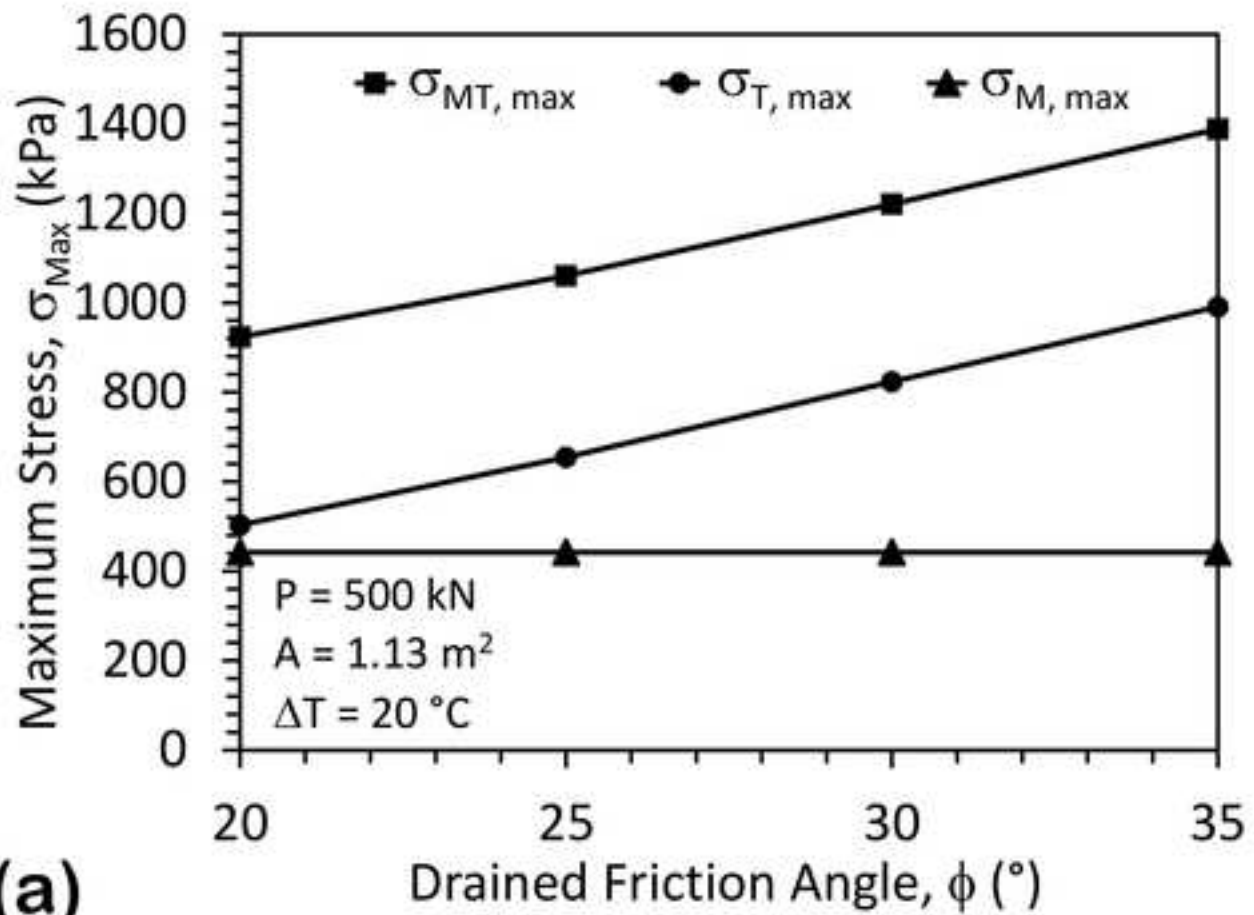
787 \*This model pile was end bearing on a rigid aluminum plate, so this was modeled as rock



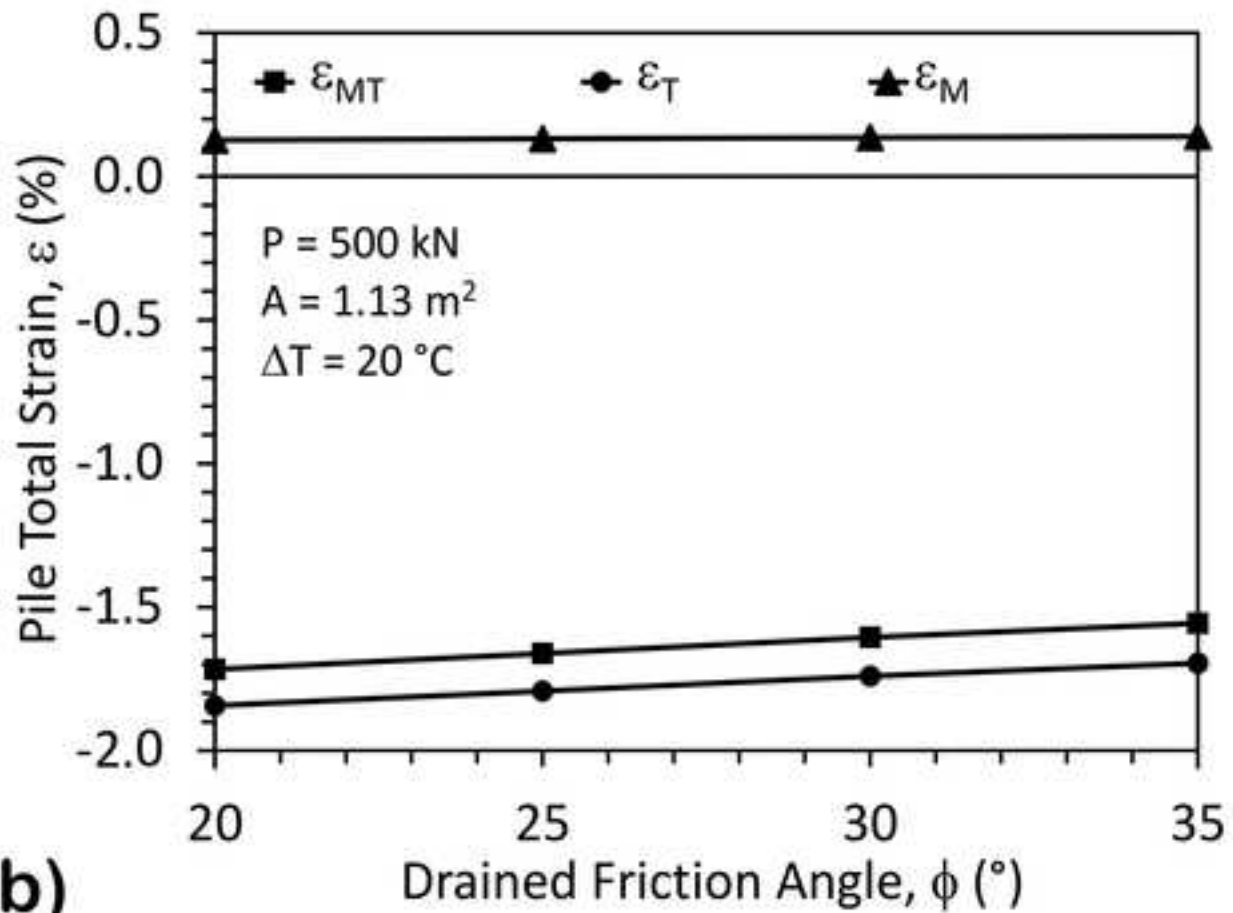




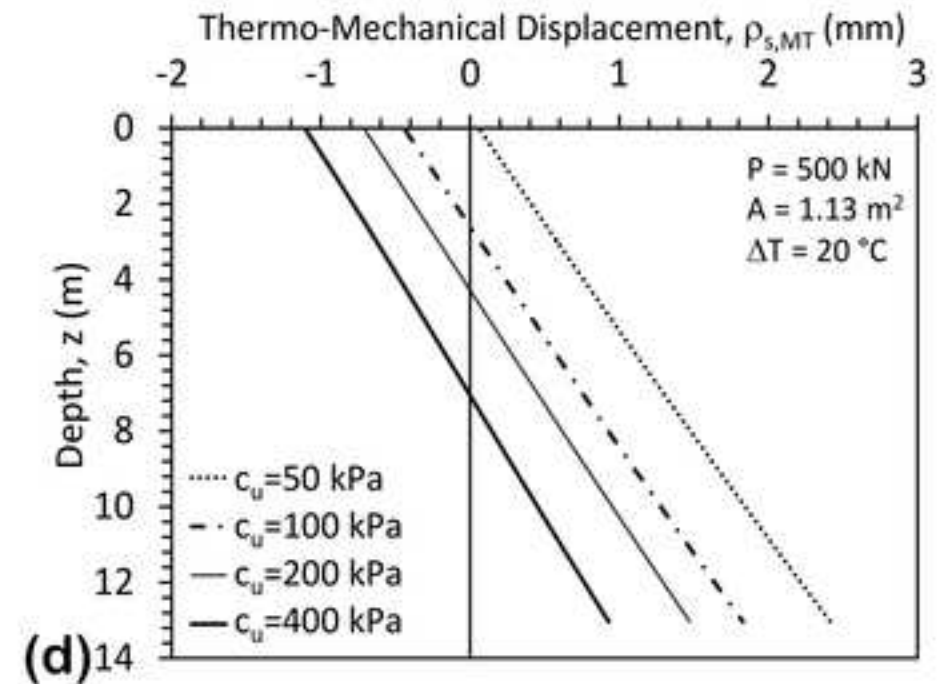
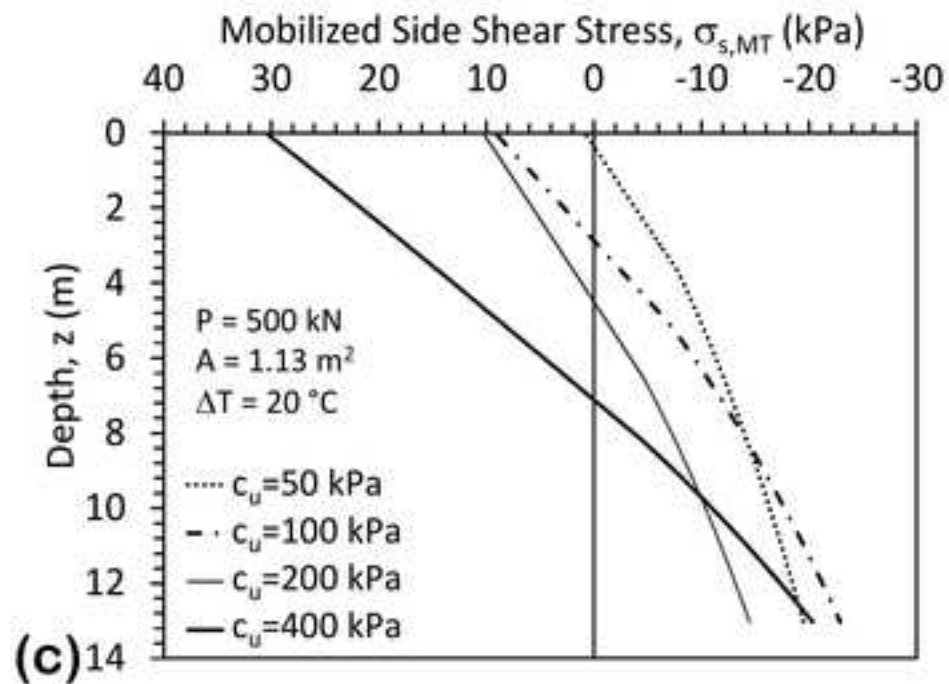
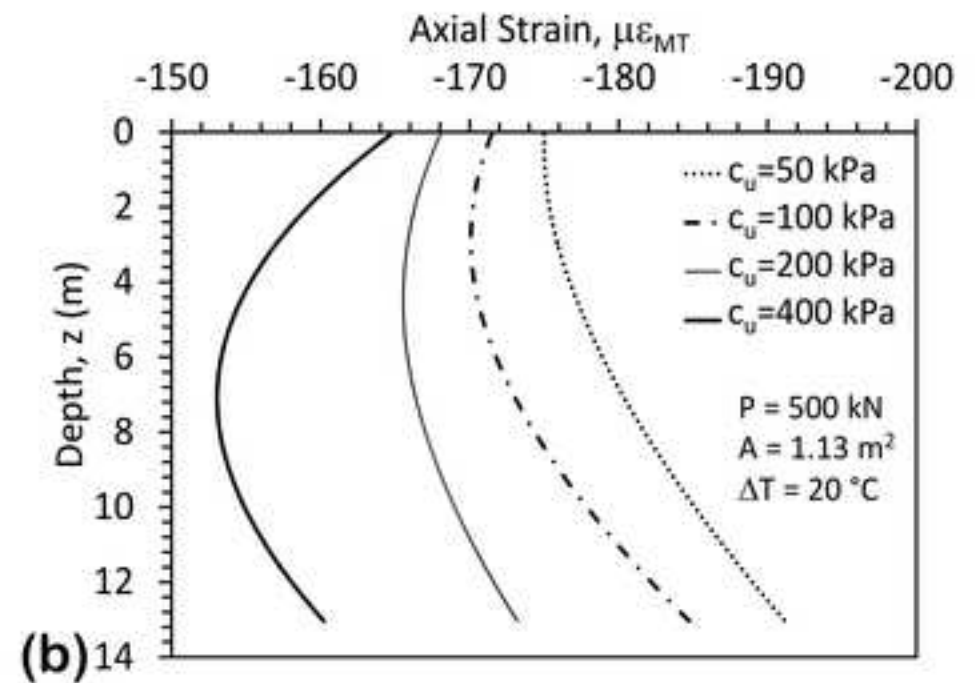
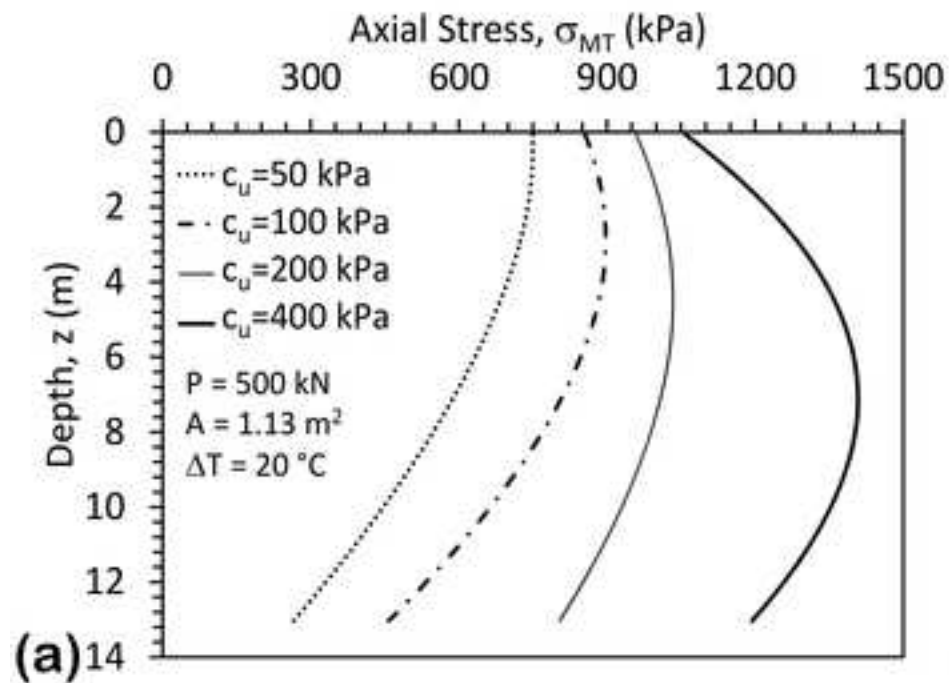


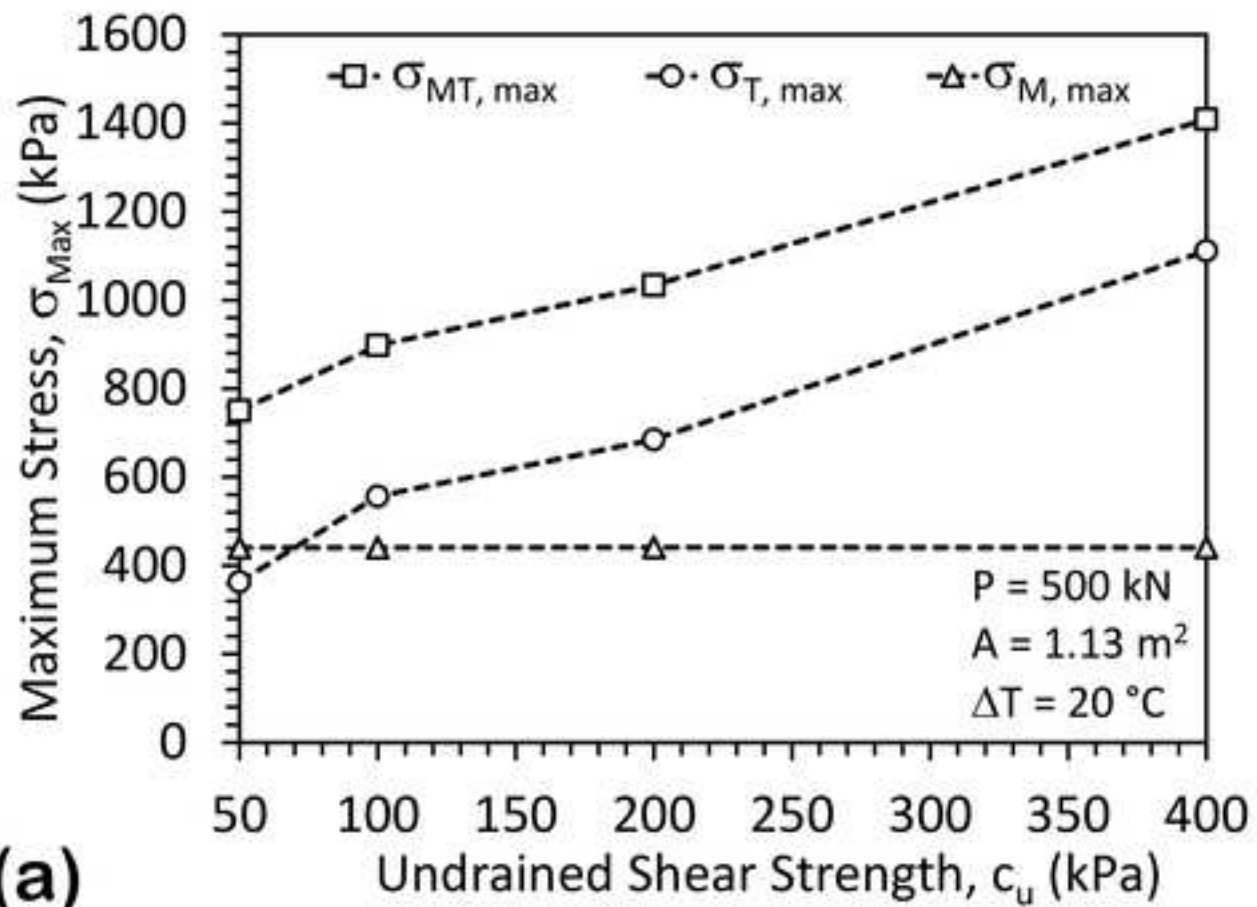


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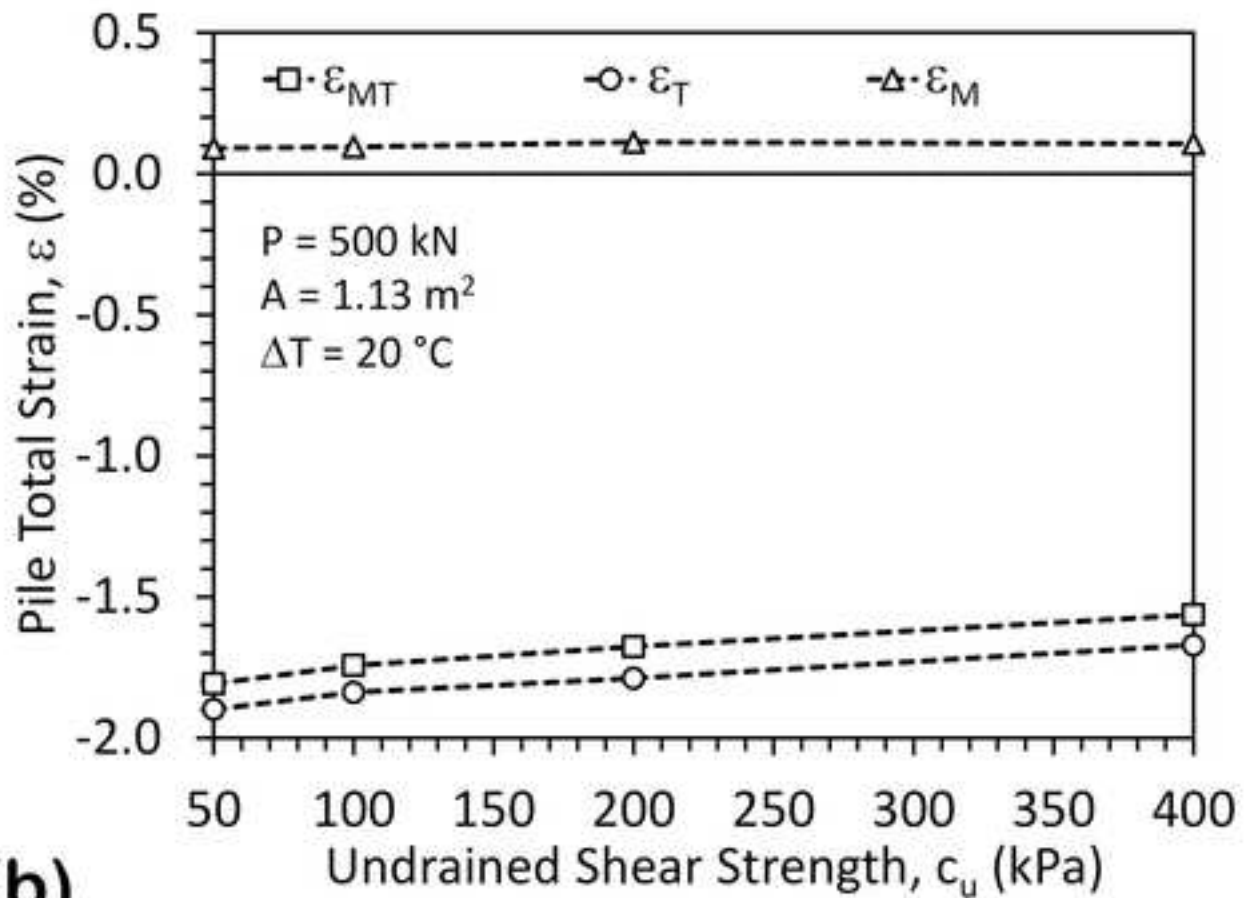


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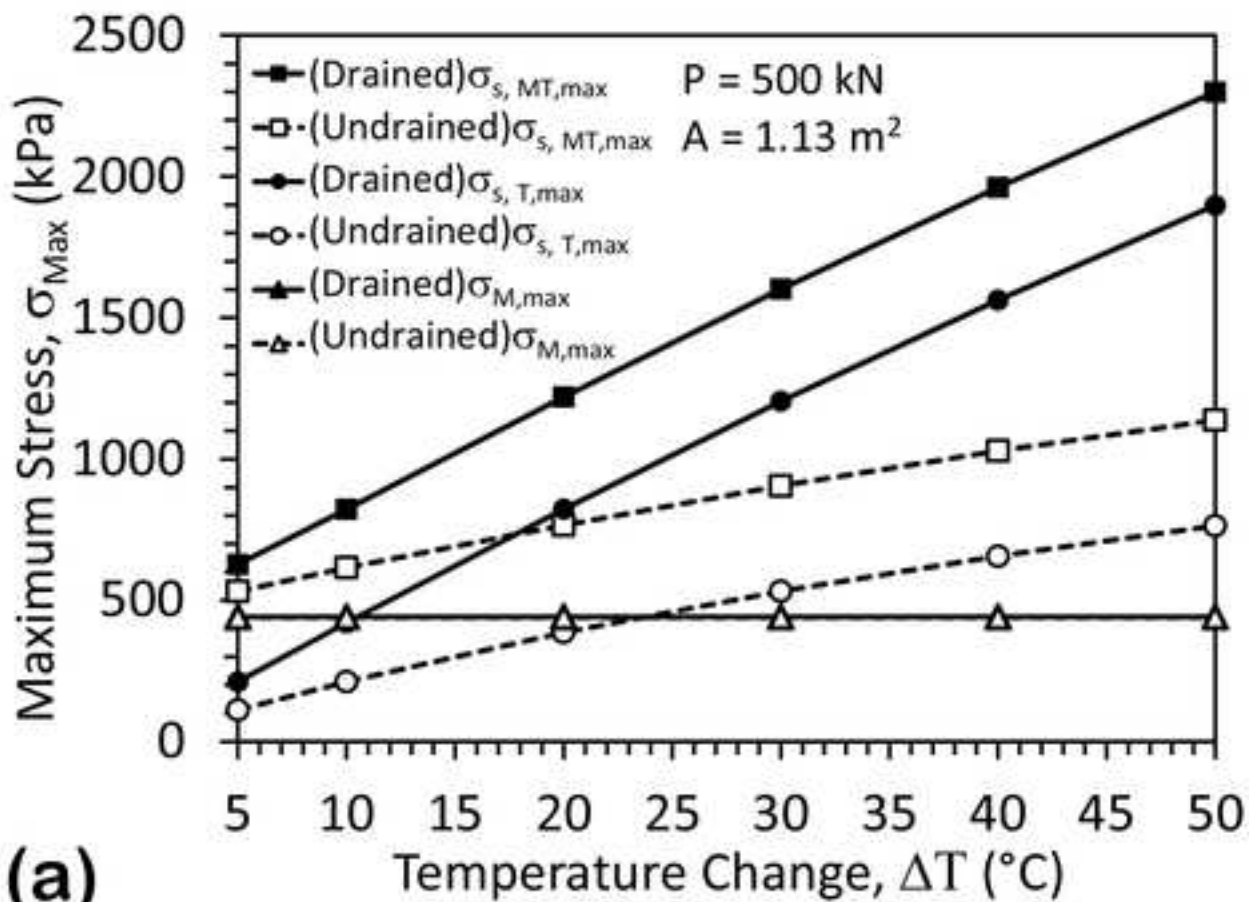




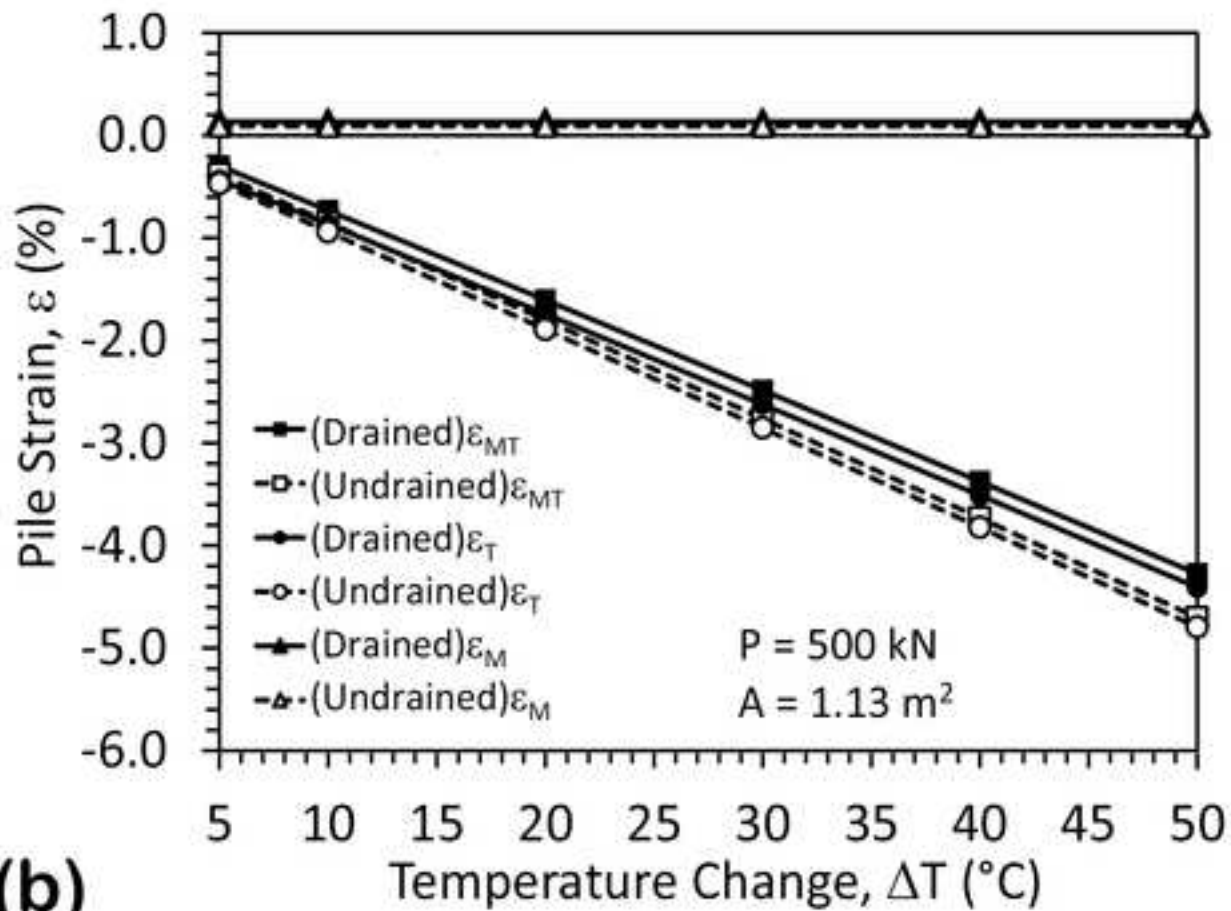
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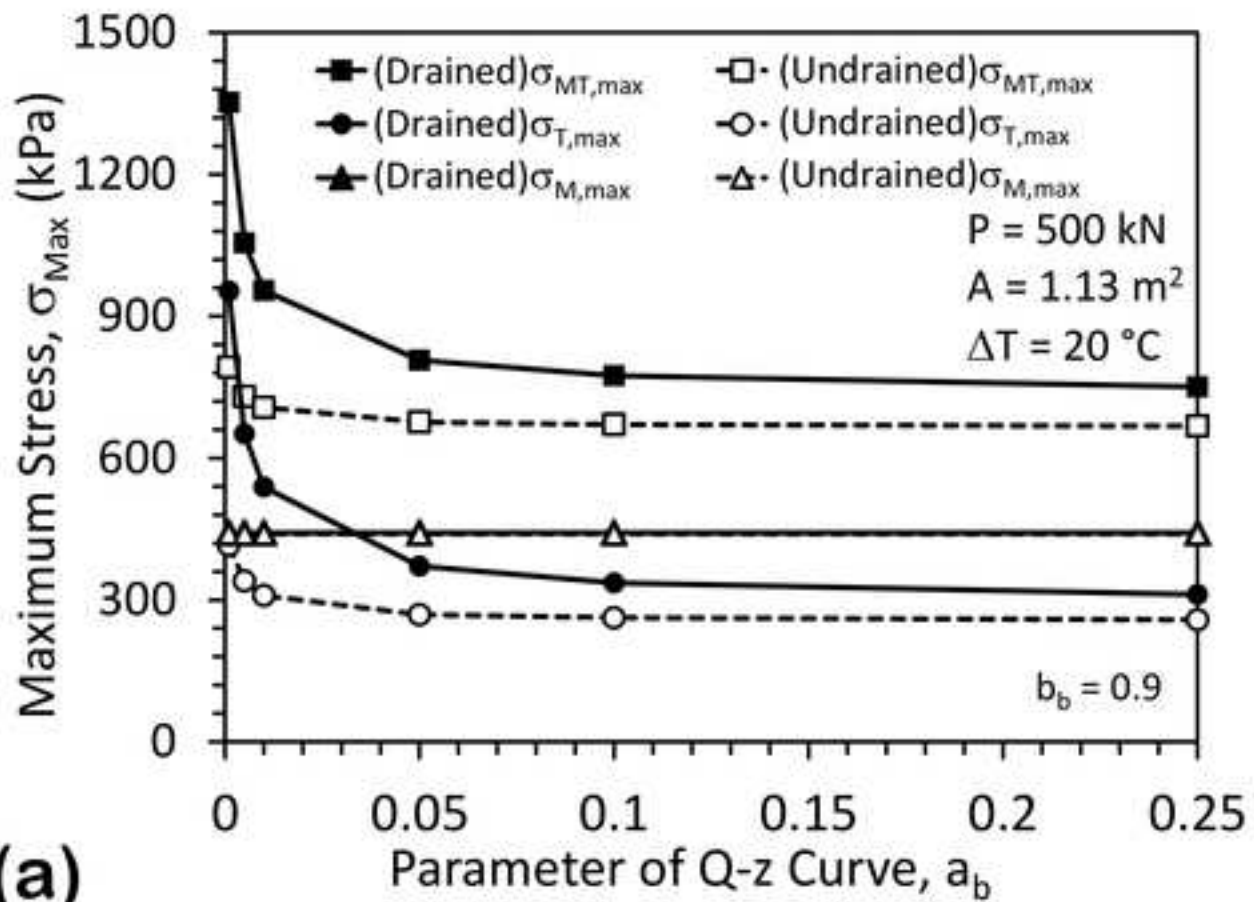
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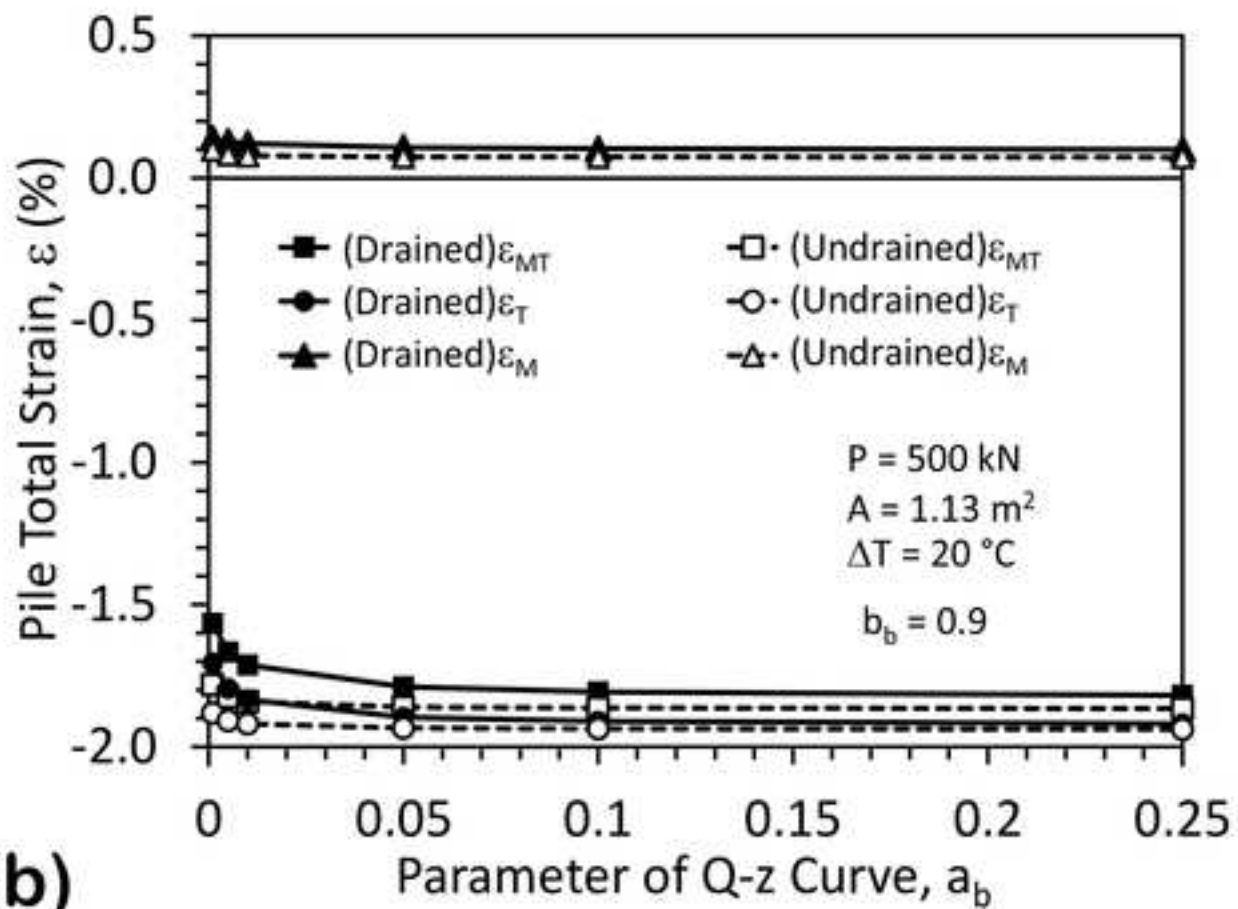
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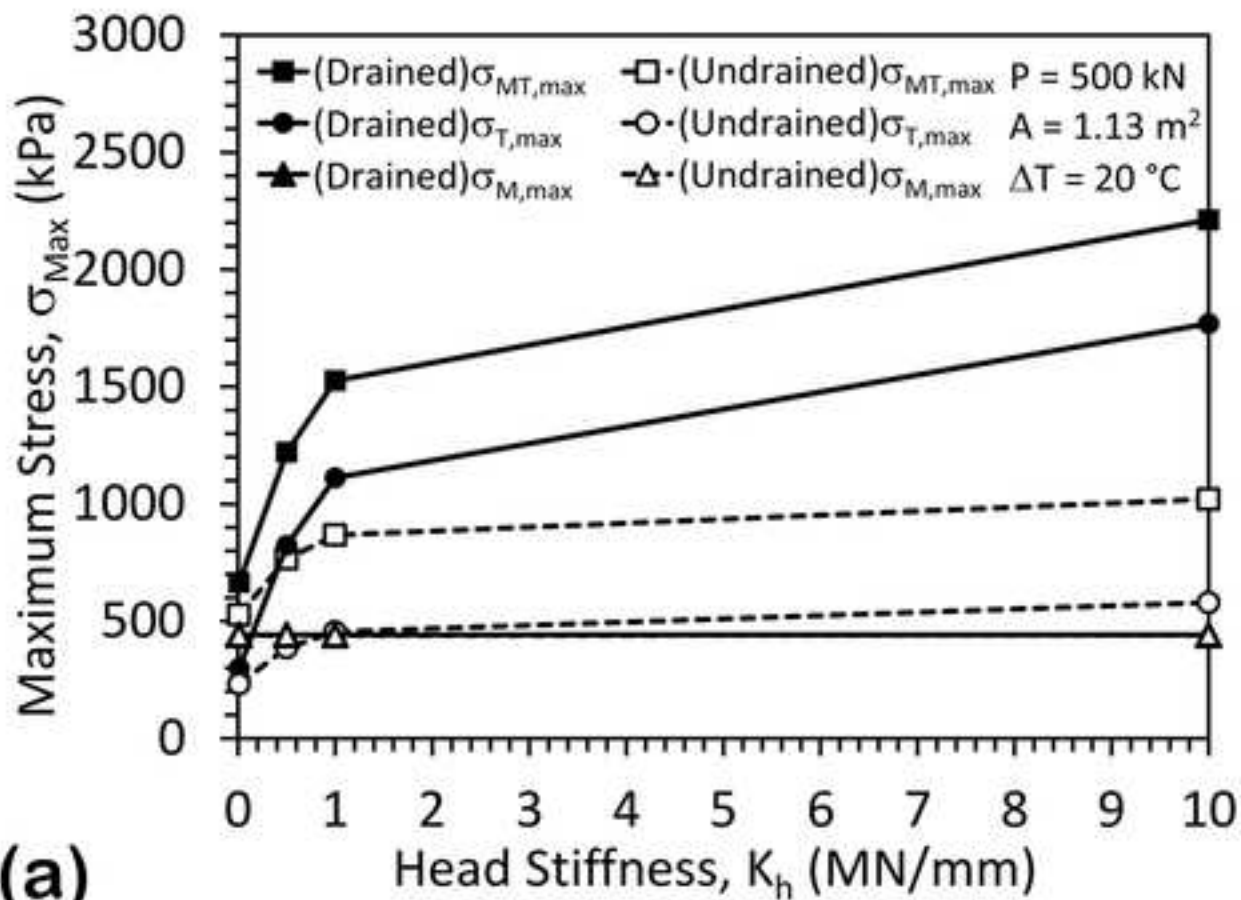
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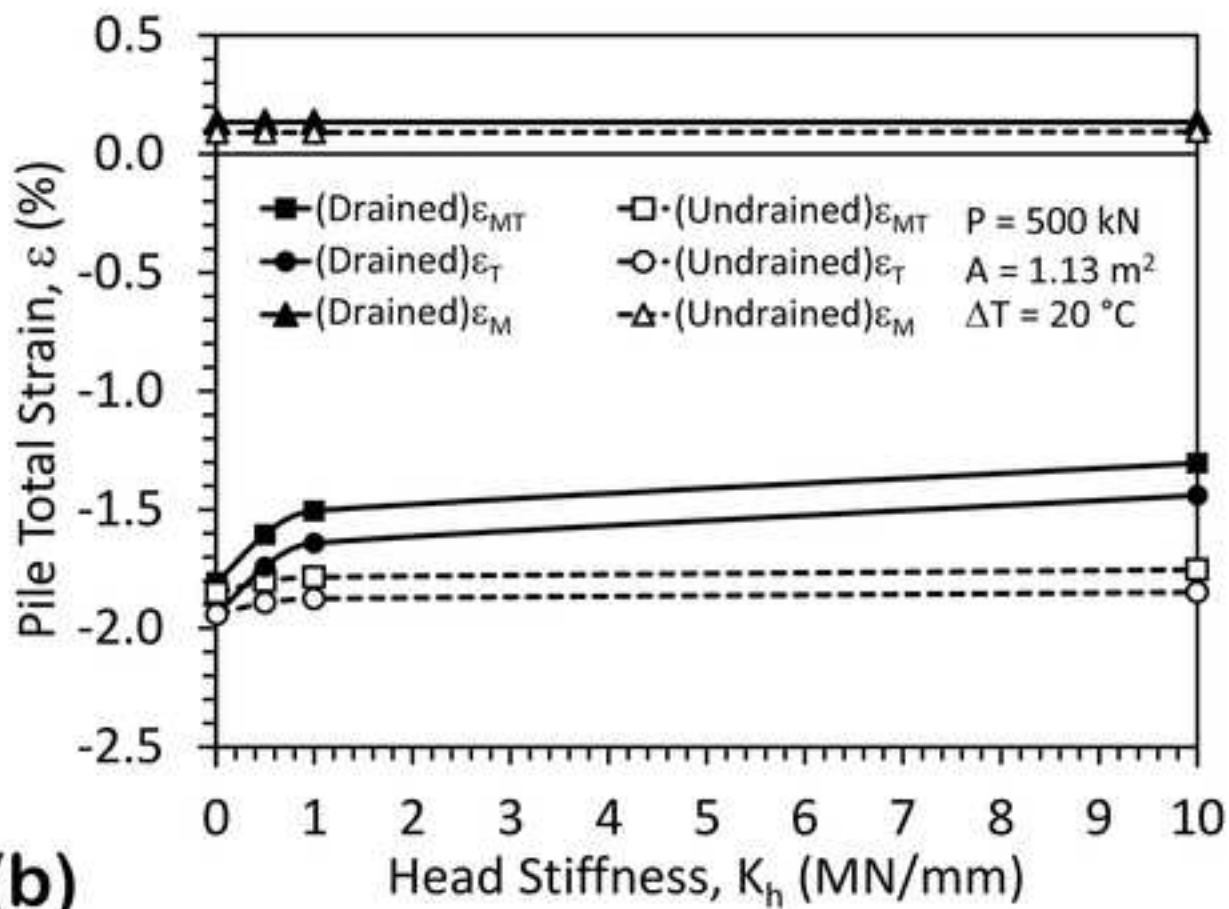
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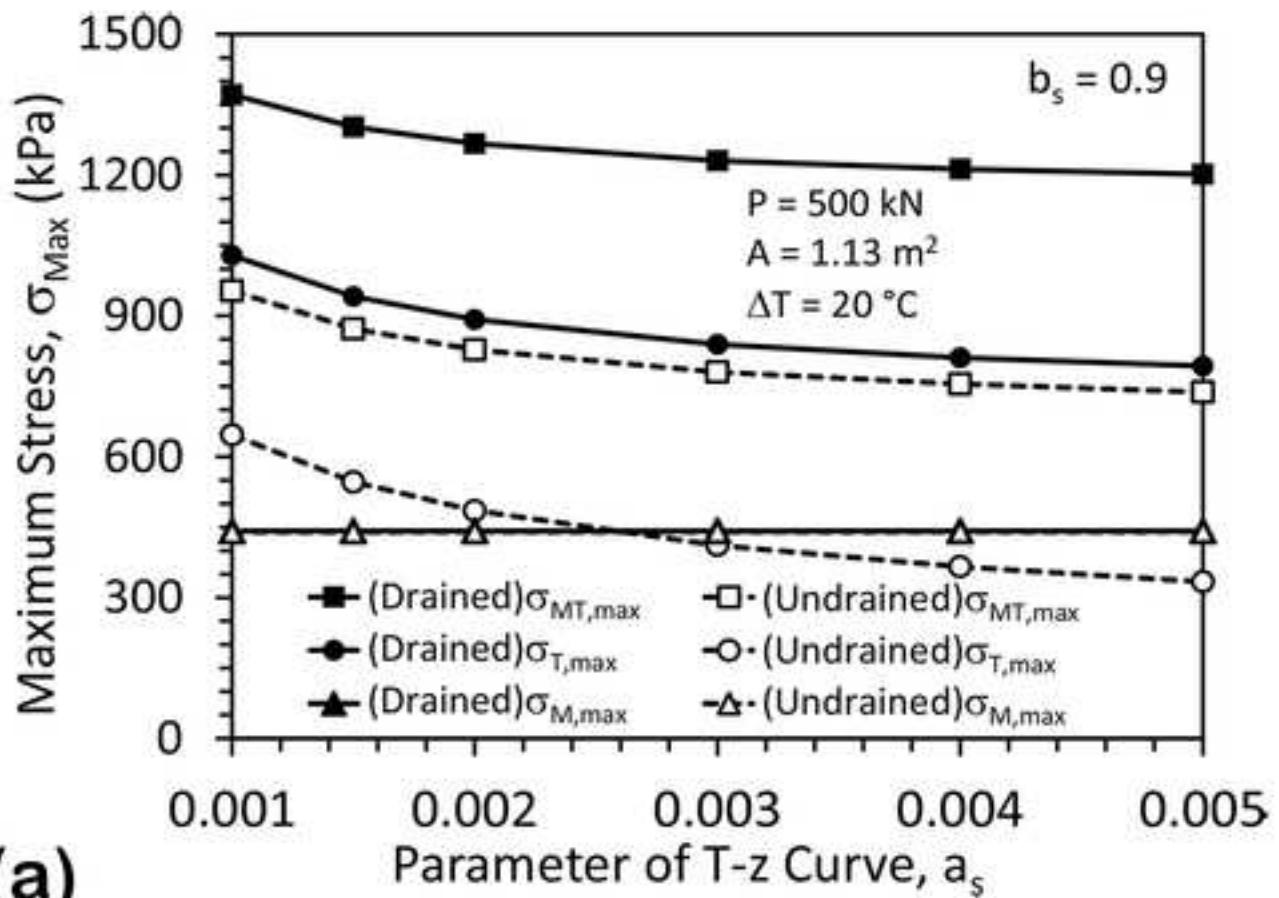


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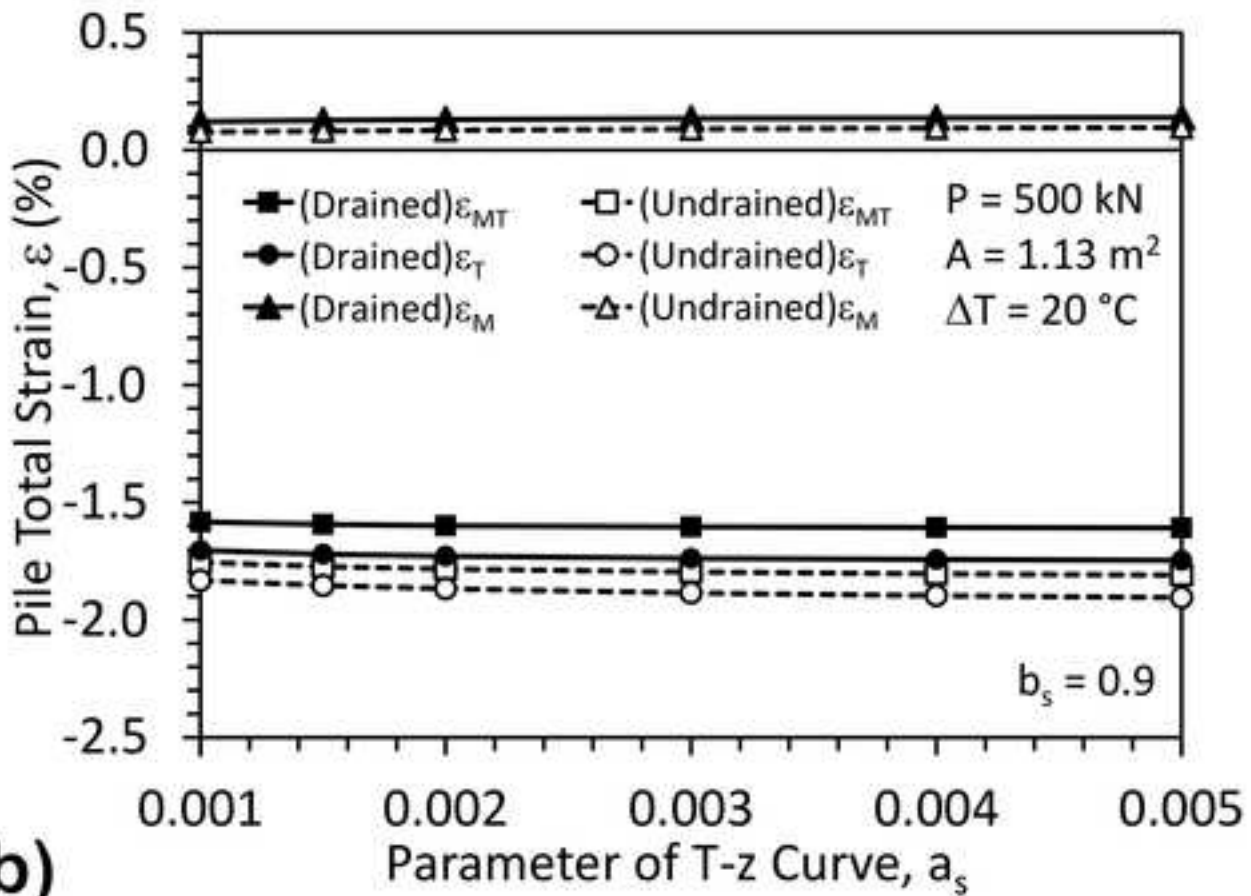


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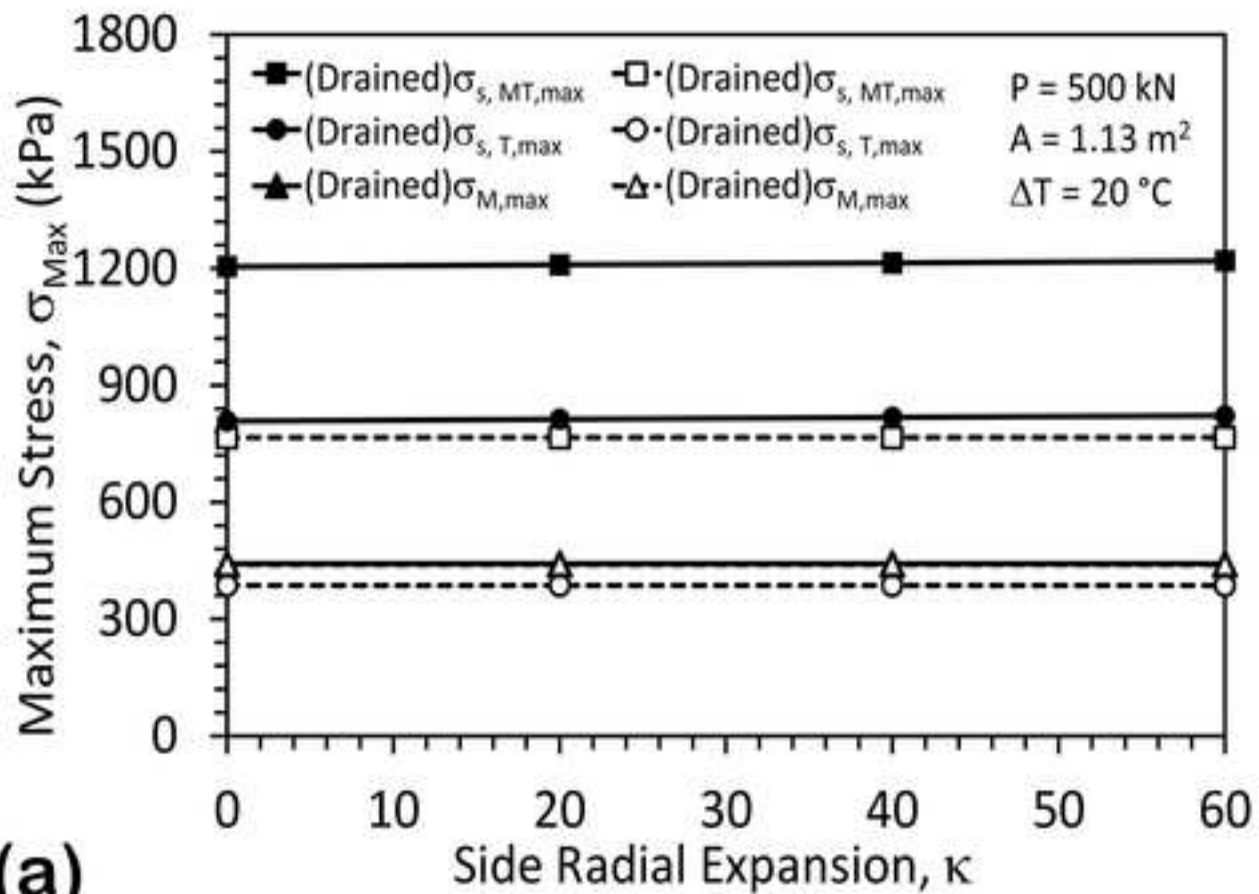




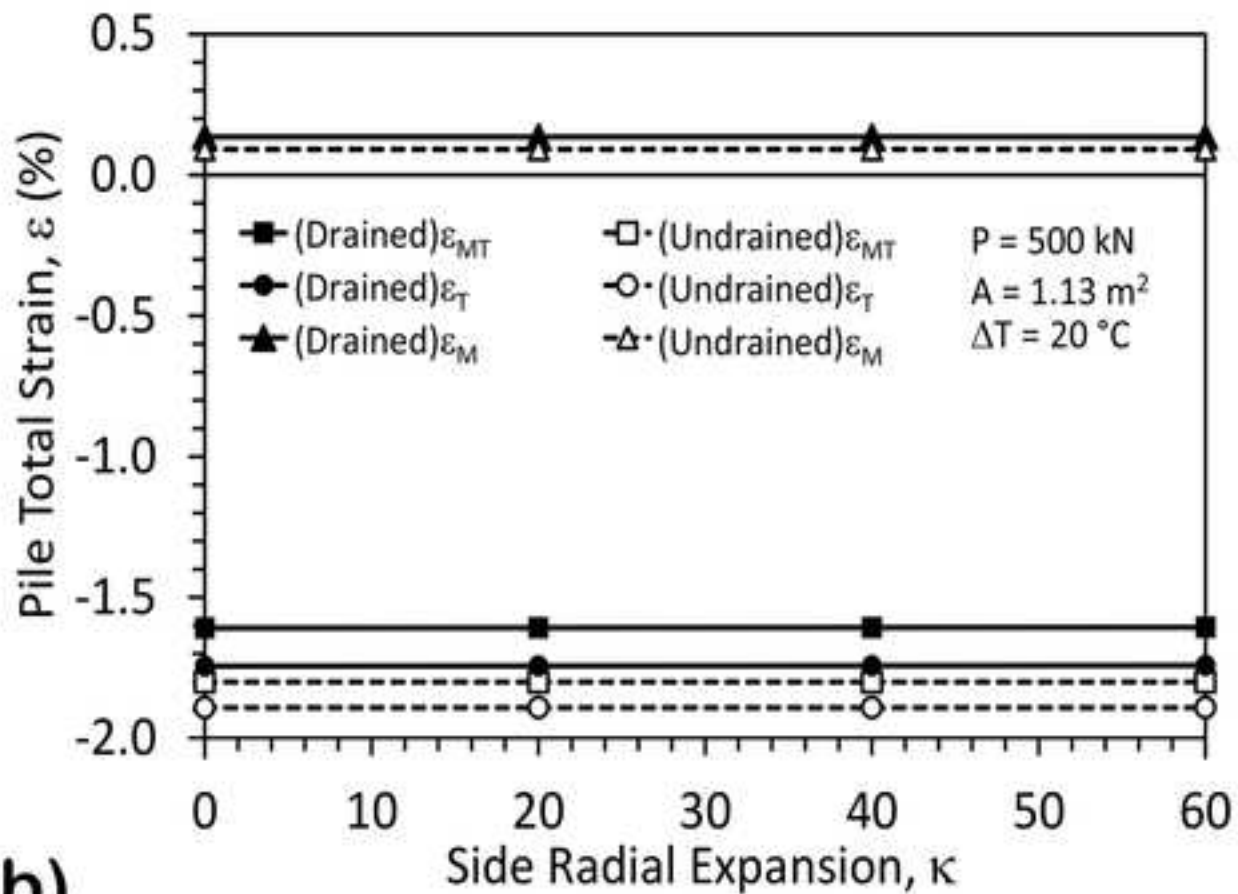
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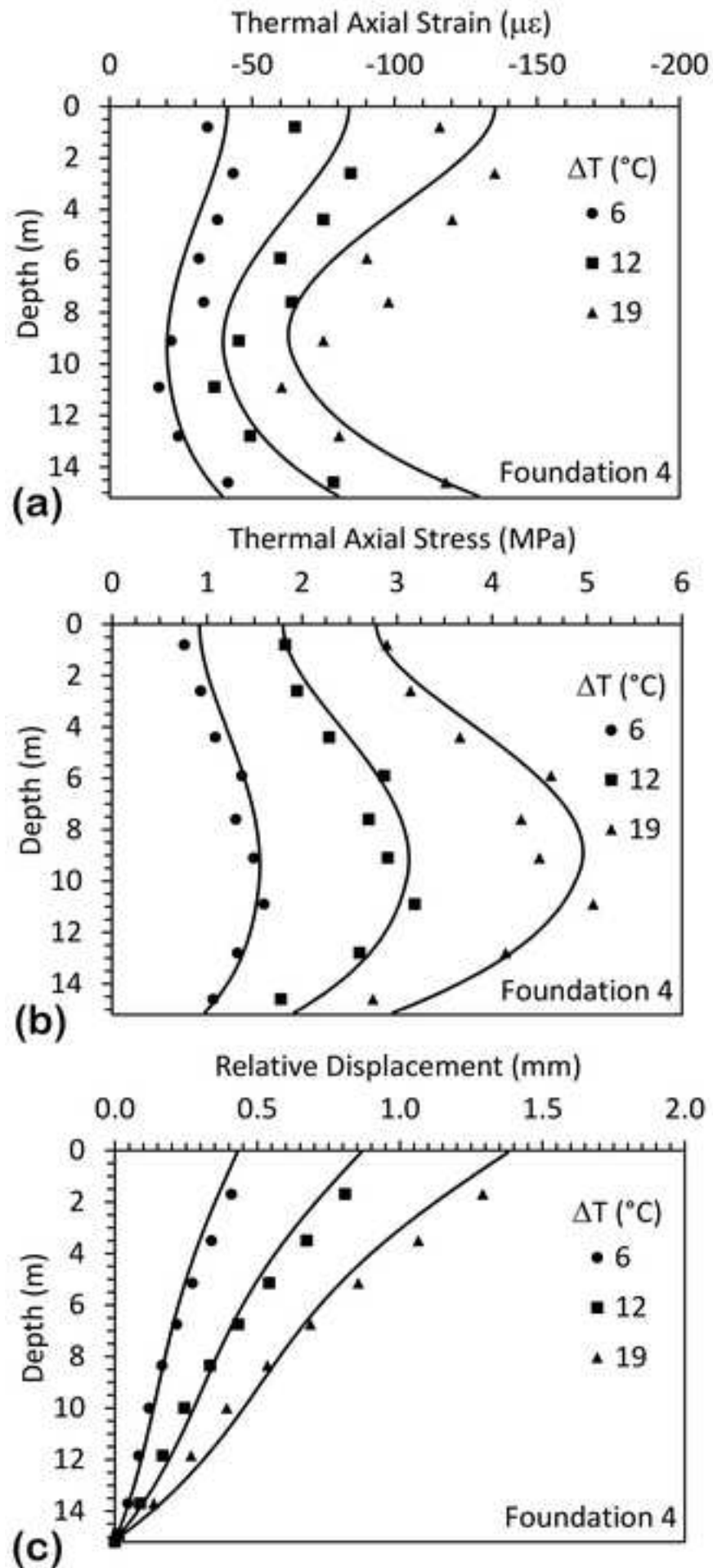
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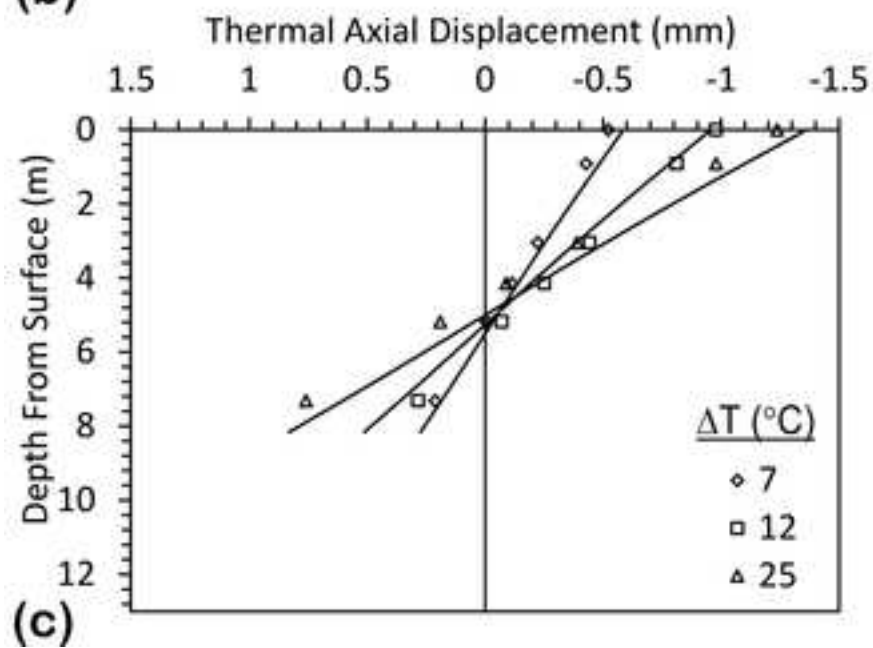
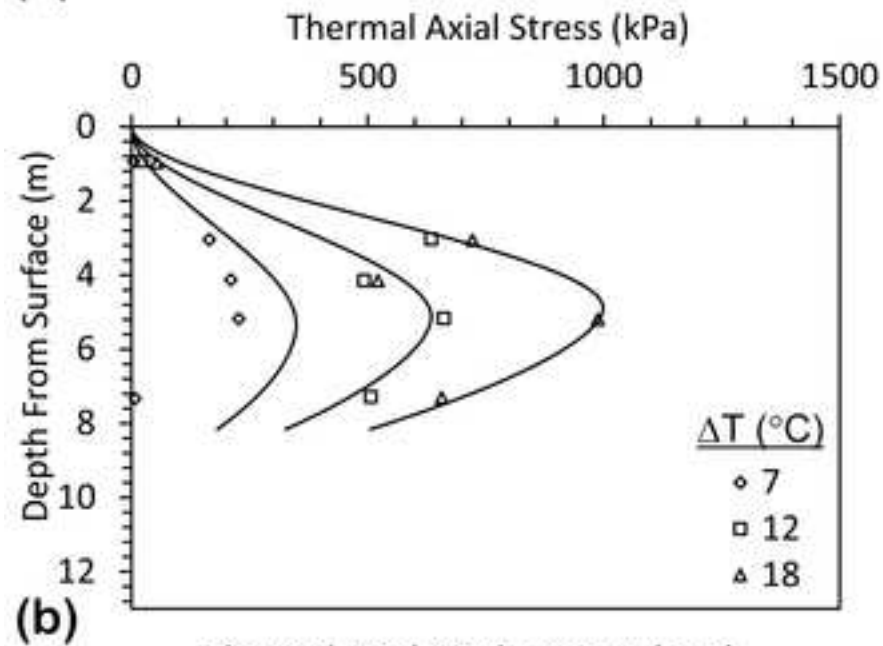
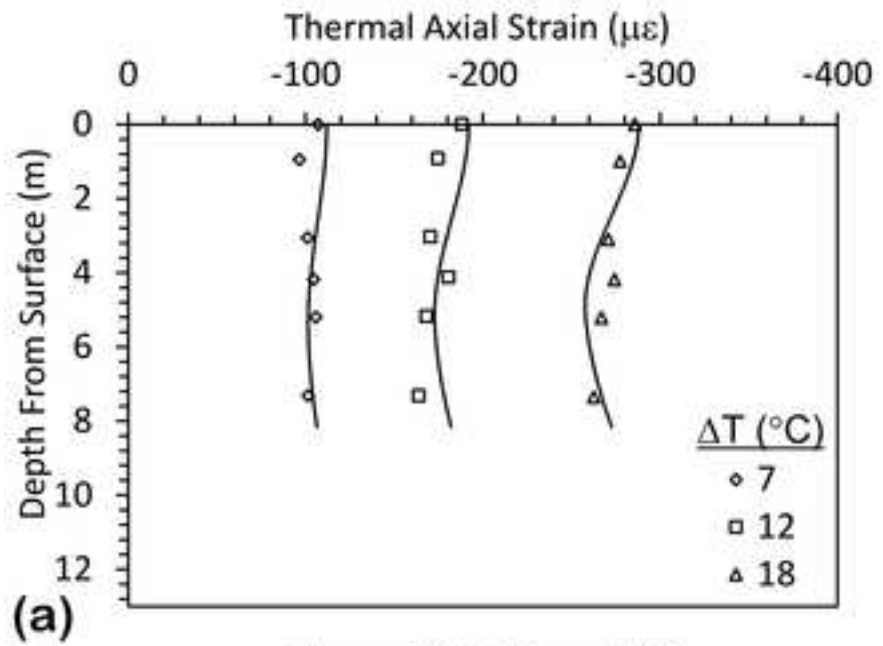


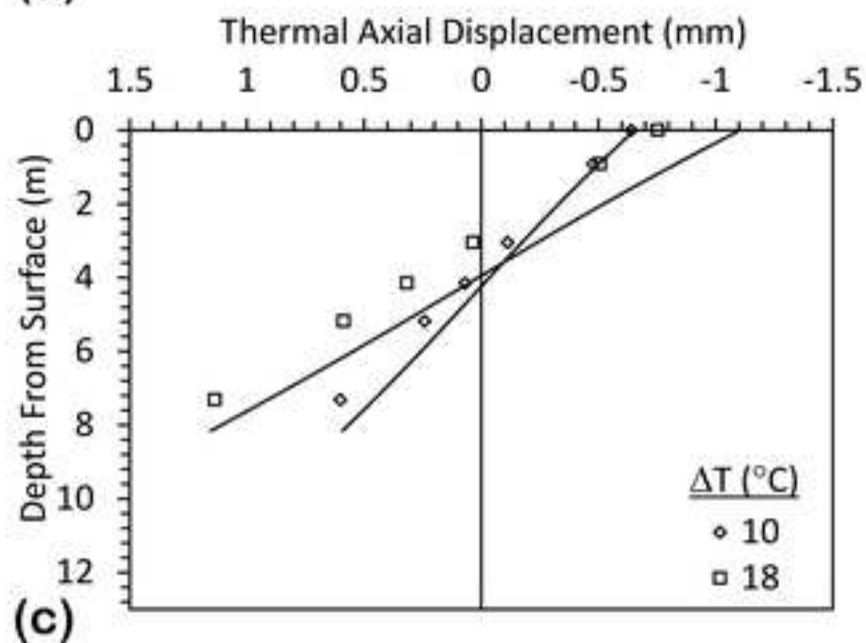
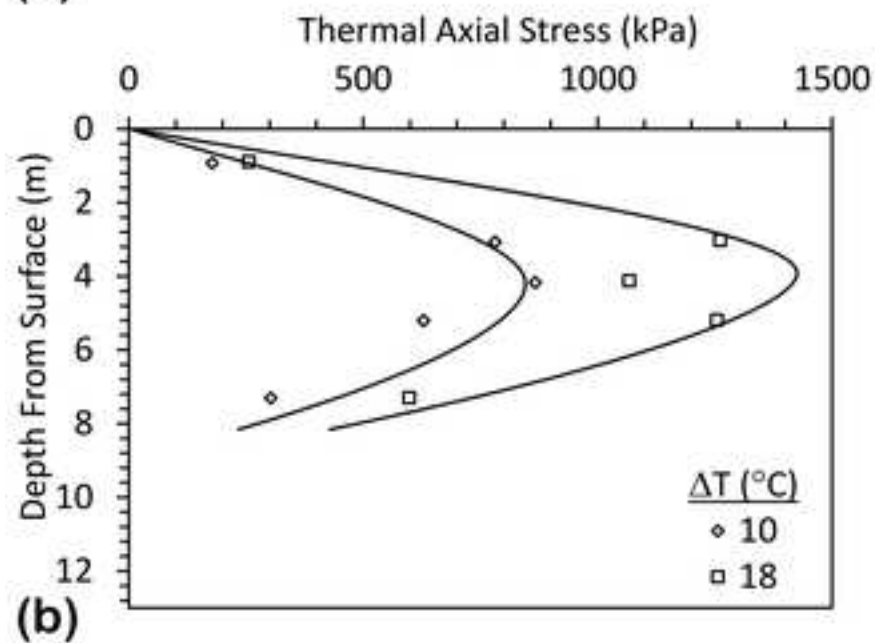
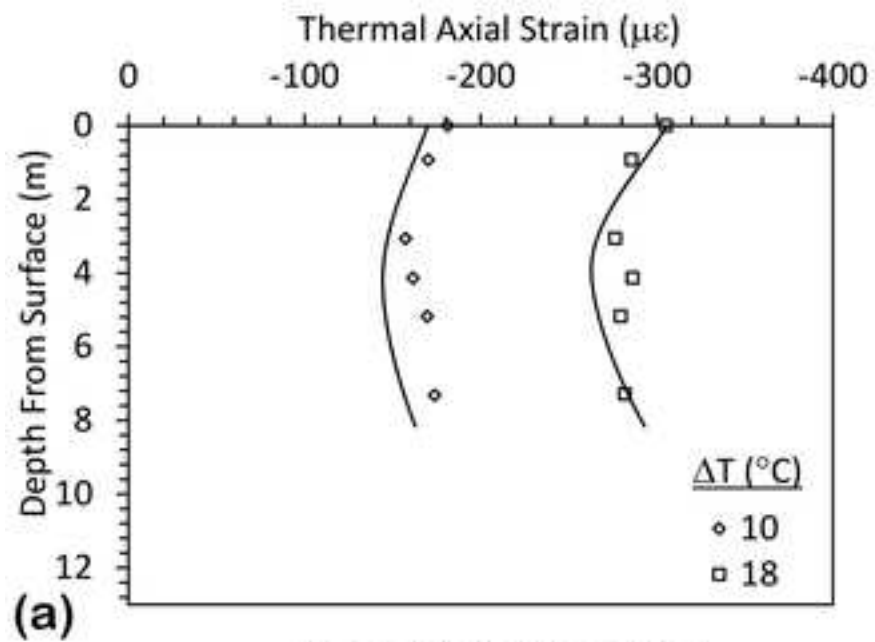
(a)

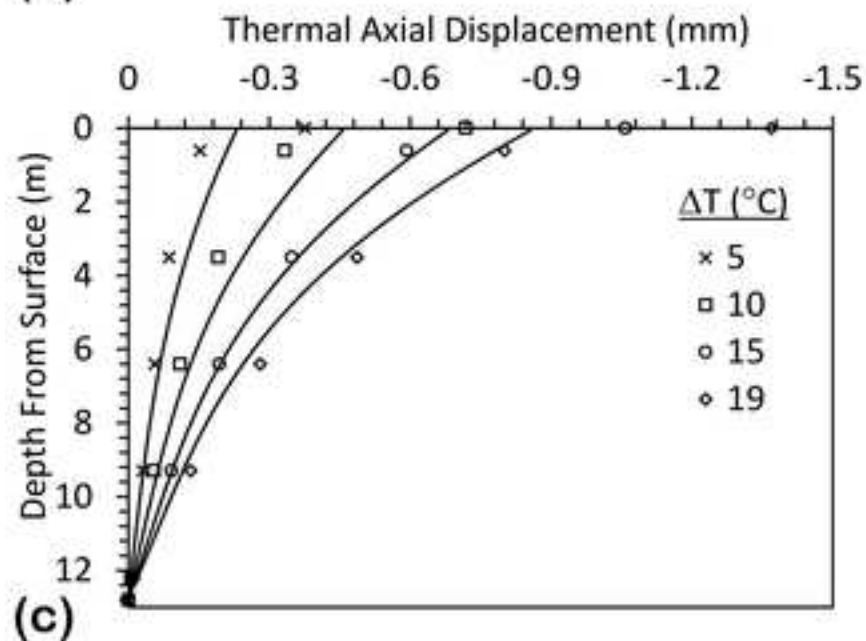
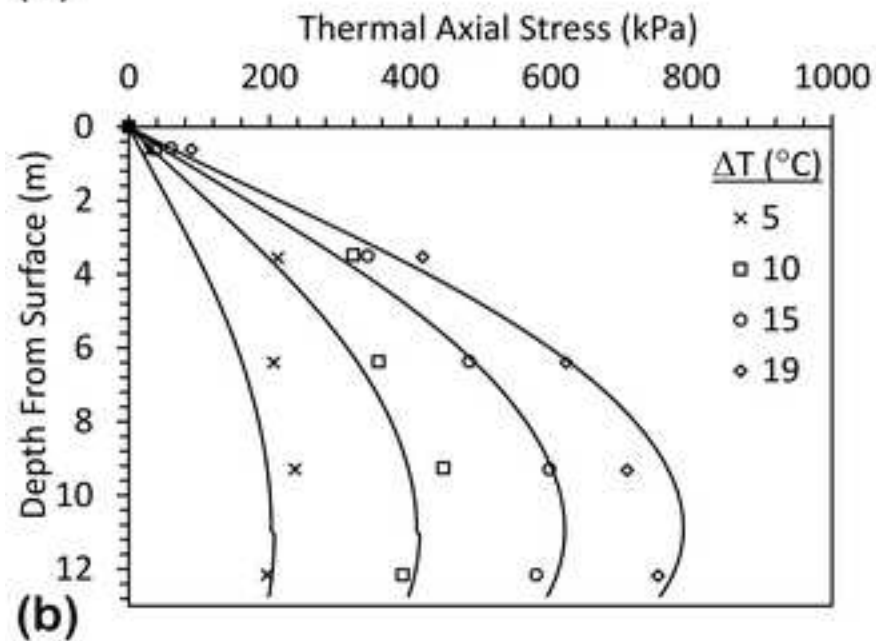
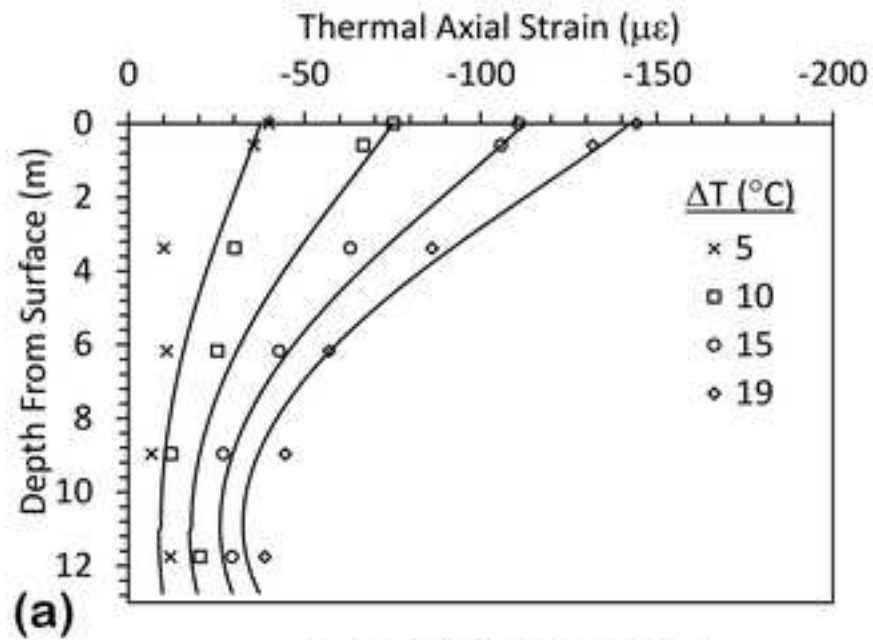


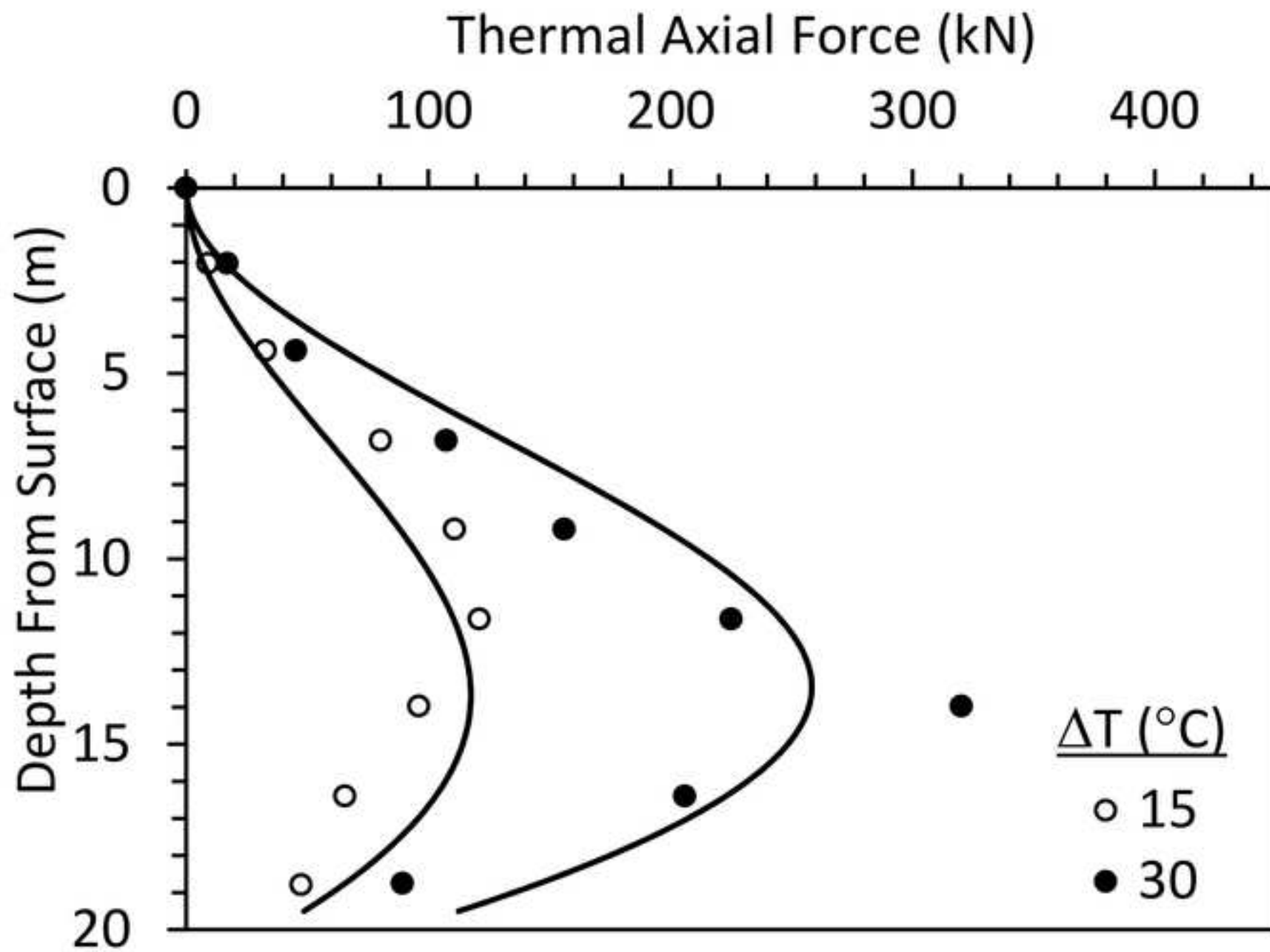
(b)

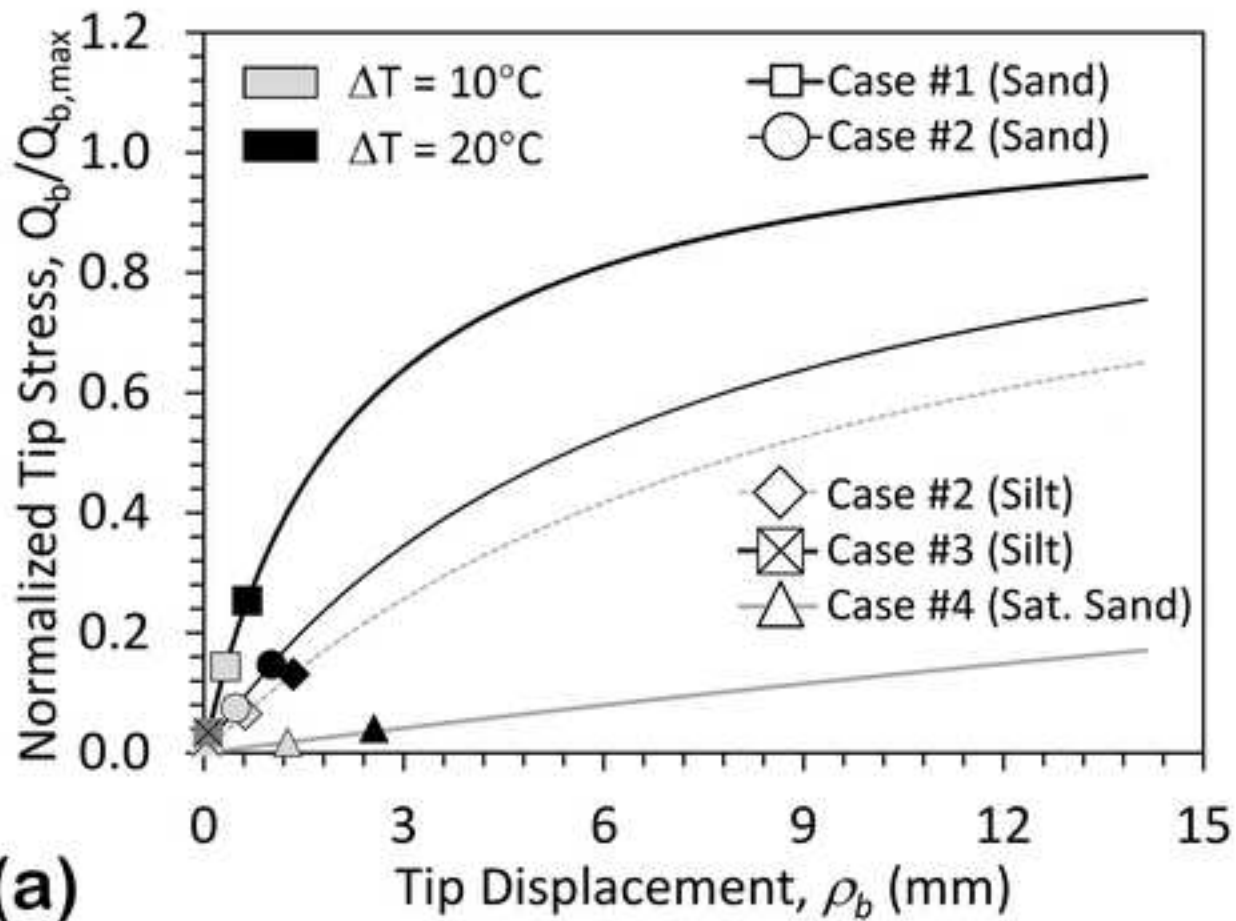
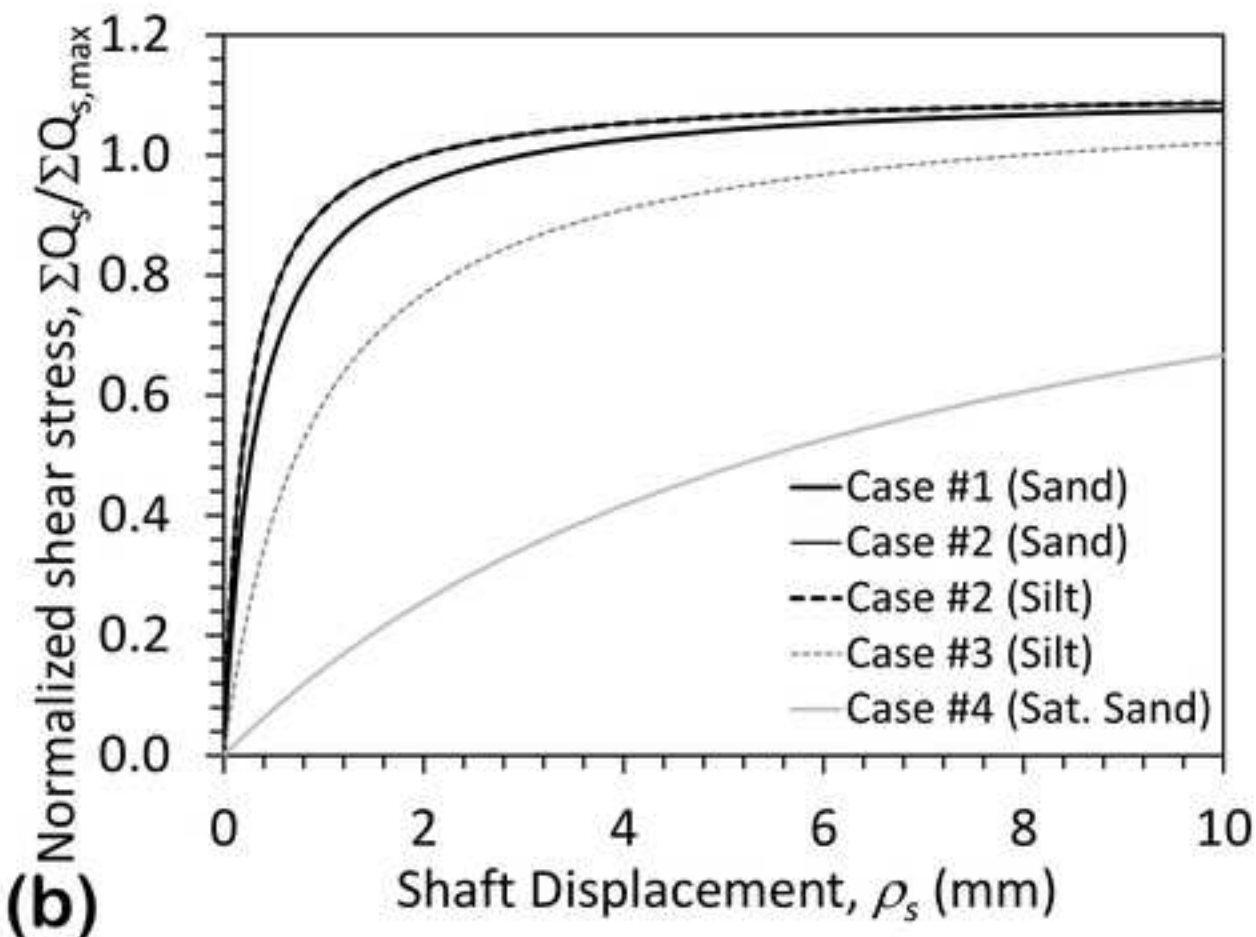










**(a)****(b)**



