AN EXPERIMENTAL INVESTIGATION ON PERFORMANCE OF A MODEL GEOTHERMAL PILE IN SAND

A Thesis in
Civil Engineering
by
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ABSTRACT

The use of geothermal piles as foundation elements of residential and office buildings is an innovative and sustainable method of energy conservation. Heat exchange through piles may have significant impact on their geotechnical performance due to thermally induced mechanical stresses and additional settlements caused by constrained thermal strains. Thermal behavior of geothermal piles are often assessed based on available idealized heat transfer models which rely on over-simplified assumptions and neglect critical operational parameters such as variable heat flux and circulation rate of heat carrier fluid. The physics and mechanics of geothermal piles subjected to thermal and mechanical loadings need to be properly understood in order for the development of an efficient design methodology.

An experimental setup is developed as part of this research to investigate performance of a model geothermal pile under a series of thermal and mechanical loading. Such controlled laboratory experiments in a soil chamber allow isolation of different variables and their effects on system performance. Temperature was measured at several locations within the pile-soil system to identify the nature of heat transfer through the model heat exchanger pile and to quantify energy output, which is a function of temperature gained from or rejected to the soil, from the pile. Axial load-displacement behavior of the model pile was studied through a series of mechanical load tests performed under different thermal conditions. Load was measured at the pile head using a load cell, and pile head and base displacements were measured using a pair of Linear Variable Differential Transformers (LVDTs). Such displacement measurements were critical in studying how the load-displacement behavior changed with thermal loads.

Radial heat transfer was observed along most part of the pile length. Circulation velocity of the heat carrier fluid and thermal conductivity of soil were found to be important parameters that affect heat transfer through a geothermal pile. Results obtained from this study may further
be used to validate numerical and analytical models that predict heat transfer through vertical heat exchangers. Although some changes in mechanical load-displacement behavior due to thermal loading were observed, the role of thermal loading in changing mechanical capacity of the model geothermal pile could not be precisely quantified. Nevertheless, the observed changes in pile load-displacement behavior warrants further research that would quantify strains (and thus stresses) at different pile cross-sections. Simulation of different in situ stress levels in the soil tank should be an important aspect to consider in such a future work.
# TABLE OF CONTENTS

LIST OF FIGURES ................................................................. vii

LIST OF TABLES .................................................................. x

ACKNOWLEDGEMENTS ....................................................... xi

Chapter 1 Introduction .......................................................... 1
  1.1. Motivation .................................................................. 1
  1.2. Objectives ................................................................ 3
  1.3. Thesis Outline ......................................................... 4

Chapter 2 Geothermal Piles – Background and Present Practice ............... 5
  2.1. Ground Coupled Heat Exchangers .................................. 5
      2.1.1. Open-loop GSHP Systems ..................................... 7
      2.1.2. Closed-loop GSHP Systems .................................. 9
  2.2. Thermo-active geostructures .......................................... 10
  2.3. Present Practice with Geothermal Piles ......................... 12
  2.4. Literature Review ..................................................... 14
      2.4.1. Experimental Studies ........................................... 14
      2.4.2. Theoretical Studies .............................................. 20

Chapter 3 Experimental Setup for Present Research .............................. 26
  3.1. Laboratory Pile Load Test Setup .................................... 27
      3.1.1. Soil Tank ........................................................... 29
      3.1.2. Sand Pluviation Device ....................................... 31
      3.1.3. Model Geothermal Pile ....................................... 35
  3.2. Materials ................................................................... 37
      3.2.1. Sand .................................................................. 37
      3.2.2. Concrete ............................................................ 45
      3.2.3. Crushed Limestone .............................................. 47
  3.3. Instrumentation and Data Acquisition ................................ 48
      3.3.1. Thermocouples .................................................. 48
      3.3.2. Load Cell .......................................................... 51
      3.3.3. Displacement Sensor .......................................... 52
      3.3.4. Software .......................................................... 53

Chapter 4 Results and Discussion ................................................... 57
  4.1. Heat Transfer Behavior of the Model Pile .......................... 57
  4.2. Axial Load-displacement Behavior of the Model Pile ........... 70
  4.3. Comparative Load-Displacement Behavior ........................ 74

Chapter 5 Summary and Conclusions ................................................. 80
5.1. Research Summary........................................................................................................... 80
5.2. Conclusions..................................................................................................................... 81
5.3. Future Research.............................................................................................................. 82

References.................................................................................................................................. 84
LIST OF FIGURES

Figure 2-1. Measured ground temperature in Nicosia, Cyprus (data obtained from Florides and Kalogirou 2006) ................................................................. 6

Figure 2-2. Open-loop ground source heat pump system ................................................. 8

Figure 2-3. Closed-loop ground source heat pump system: (a) horizontal circulation loops and (b) vertical circulation loops.......................................................... 9

Figure 2-4. Pile-anchored ground source heat pump system ............................................ 10

Figure 2-5. Instrumented centrifuge test setup adapted from McCartney and Rosenberg (2011) ....................................................................................... 18

Figure 2-6. Laboratory test setup after Wang et al. (2011) ................................................ 19

Figure 2-7. Development of shaft and base resistance in an axially loaded pile ............... 22

Figure 2-8. Typical load-displacement curve for an axially loaded pile............................. 23

Figure 3-1. Schematic of the laboratory test setup designed for the present research ....... 29

Figure 3-2. Assembled soil tank with adjustable reaction frame attached ....................... 30

Figure 3-3. Sand pluviation device with #6, #10, and #12 sieves attached ...................... 31

Figure 3-4. Uniform deposition of sand as it passes through the pluviator system ............. 34

Figure 3-5. Relative density calibration curve ..................................................................... 34

Figure 3-6. Formwork used for pile casting and PVC circulation loop ............................. 35

Figure 3-7. Vertical and horizontal cross-sections of the pile ........................................... 36

Figure 3-8. Surface roughness profile for the concrete model pile .................................... 37

Figure 3-9. Particle size distribution curve for the F50 Ottawa sand ............................... 38

Figure 3-10. SEM image of F50 silica sand: (a) at 65x magnification and (b) at 159x magnification ..................................................................................... 39

Figure 3-11. EDS spectrum of F50 silica sand .................................................................. 40

Figure 3-12. Results from direct shear tests on F50 Ottawa sand .................................... 41

Figure 3-13. Custom-built thermal conductivity test apparatus used in this research ....... 42
Figure 3-14. Average temperature at each radial distance ................................................. 43
Figure 3-15. Hydraulic conductivity of F50 Ottawa sand as a function of void ratio ............ 44
Figure 3-16. Particle size distribution of fine aggregate used in concrete ..................... 46
Figure 3-17. Particle size distribution of #8 crushed stone used as a filter layer at the bottom of the sand bed ................................................................. 48
Figure 3-18. Thermocouple locations: (a) in the XZ plane (b) in the YZ plane ............... 51
Figure 3-19. Screenshot of GUI developed for real-time monitoring of temperature data .... 54
Figure 3-20. Screenshot of GUI developed for real-time monitoring of load and displacement data ................................................................................................. 55
Figure 4-1. Initial temperature distribution within the soil tank for the thermal tests in dry sand ........................................................................................................... 57
Figure 4-2. Variation of air temperature near soil surface ............................................. 58
Figure 4-3. Temperature contours at different times during thermal loading (heating) under dry condition with $\Delta \theta = 20^\circ\text{C}$ and $v = 0.11\text{ m/s}$ (a) $t = 0.25\text{ days}$ (b) $t = 0.5\text{ days}$ (c) $t = 1.0\text{ days}$ (d) $t = 4\text{ days}$, and (e) $t = 7\text{ days}$ ................................................ 61
Figure 4-4. Time-dependent variation of soil temperature $T_g$ (under dry condition) recorded at different thermocouple locations along the depth ............................. 62
Figure 4-5. Comparison between time-dependent variations of soil temperature $T_g$ recorded at different thermocouple locations along the depth under dry and saturated conditions ........................................................................................................ 63
Figure 4-6. Change in the temperature of dry sand at different radial distances from the pile during heat rejection (heating) and heat extraction (cooling) through the pile ......... 64
Figure 4-7. Change in soil temperature at different radial distances from the pile during heat rejection (heating) into dry and saturated sand deposits ..................................... 65
Figure 4-8. Evolution of dry soil temperature $T_g$ (at different radial distances r) with time .. 66
Figure 4-9. Fluid temperature difference $\Delta T_f$ between inlet and outlet points of the circulation tube embedded within the model geothermal pile installed in dry sand ......... 67
Figure 4-10. Geothermal energy output (for different fluid circulation rates) from the model geothermal pile installed in dry sand .................................................. 68
Figure 4-11. Effect of sequential heat extraction and rejection on energy output from the model geothermal pile installed in dry sand .................................................. 69
Figure 4-12. Constrained thermal expansion of the model pile during thermal loading under dry condition..........................................................70

Figure 4-13. Typical load steps applied to the pile head during axial loading of the model pile........................................................................................................71

Figure 4-14. Typical record of pile head and base displacements during axial loading ..........72

Figure 4-15. Typical load-displacement behavior of the model pile ........................................74

Figure 4-16. Load-displacement plots as obtained from several axial load tests on the model pile................................................................................................................75

Figure 4-17. Results of pile load tests plotted following the Chin's extrapolation method ....76

Figure 4-18. De Beer's method for pile load tests at ambient and elevated temperature .........77

Figure 4-19. Effect of mechanical loading on limit capacity of the pile.................................79
LIST OF TABLES

Table 2-1. Typical thermal properties for soil, water and air.................................................. 7
Table 3-1. Properties of F50 Ottawa sand................................................................................. 44
Table 3-2. Concrete mix design (per cubic meter) for model pile ............................................. 46
Table 3-3. Mechanical and thermal properties of concrete at 28 days....................................... 47
Table 4-1. Interpretation of pile load test results ....................................................................... 78
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Chapter 1

Introduction

1.1. Motivation

Present worldwide concerns with rising energy costs and global climate change have initiated a strong shift in Civil and Architectural Engineering practice towards designing more energy efficient buildings. The concept behind designing “zero-energy buildings” is motivated by the fact that these buildings will consume zero net energy, and therefore will be self-sufficient. The use of conventional energy sources can be minimized through the use of sustainable energy sources such as wind, solar, or geothermal, as well as by utilizing more efficient building insulation. The use of geothermal piles (also known as heat exchanger or energy piles) for the dual purpose of supporting superstructure load and harvesting shallow geothermal energy is an innovative and sustainable method of energy conservation. Geothermal piles operate in conjunction with a heat pump that collects heat from the building’s air and stores it in a circulation fluid. This fluid is then pumped through circulation tubes embedded within pile and heat exchange occurs between the pile and the ground.

Based on a recent study by the U.S. Department of Energy (DOE), heating and cooling constitute, respectively, about 54% and 65% of the operating costs of typical residential and office buildings (DOE 2010). According to the U.S. Environmental Protection Agency (EPA), ground source heat pumps (GSHP) are the most efficient, economical, and environmentally friendly building heating and cooling systems available (EPA 1993). The reason is that ground source heat pumps have a very high Coefficient of Performance (COP), which is the ratio of heat supplied by the system to the energy consumed by the heat pump. The most common method to regulate the temperature of a building is a standard air-source heat pump (ASHP), which usually
have a COP ranging from 1.46 to 2.75. In contrast, COP values for GSHP systems can range from 2.5 to 4.0 (Heinonen et al. 1996). Thus it is evident that even poorly performing GSHPs are more efficient than well performing ASHPs. A variety of GSHP systems is available in practice; however, their individual performance and efficiency depends greatly on the individual type. For an example, the efficiency of a borehole heat exchanger would differ significantly from a horizontal ground loop system. This is due to the embedment depth in the ground, material through which the heat conducts, and surface area available for heat exchange.

Geothermal piles have several advantages over other GSHP systems from a construction and economic perspective. The majority of the expenses associated with GSHPs occur due to up-front installation costs. For horizontal and vertical ground loops, as well as borehole heat exchangers, soil must be excavated for the sole purpose of installing the circulation tubing. In the case of geothermal piles, the circulation tubing is generally put in place during installation of a drilled shaft foundation. While the drilled shaft is installed following the standard procedure, the circulation tubing is tied to the reinforcement cage before it is lowered into the borehole. This drastically reduces the installation cost associated with the GSHP system since drilling a borehole is already a part of conventional drilled shaft installations.

Although some empirical methods and software are available to design ground-sourced heat exchanger systems, those cannot be directly used for design of geothermal piles (Hellström & Sanner 2001). The main reason is that the primary purpose of geothermal piles is to support the superstructure load, so this must be the initial focus for design. However, the use of these piles for heat exchange generates complicated thermo-mechanical behaviors that have yet to be fully understood. For this reason, much of the geothermal pile design is still based on experience and empirical methods. These methods have been derived from field tests which have inherent uncertainties and are often site-specific.
The purpose of performing controlled laboratory experiments is to be able to isolate and study important variables while minimizing uncertainties. In this research, the soil test bed was prepared in a controlled manner, and the soil and pile material (concrete) was characterized extensively. Moreover, the test setup was carefully designed to avoid thermal and mechanical scale effects which might limit the application and interpretation of test results. Instrumentation was implemented to monitor thermal and mechanical behavior of the pile-soil system, which is particularly useful in understanding the physics governing the complex heat and load transfer mechanism for geothermal piles. Data gathered in this research will also be used in validating analytical models aimed at performance evaluation and design geothermal piles.

1.2. Objectives

The dual use of geothermal piles as foundation elements as well as heat exchangers is a sustainable and promising method of reducing primary energy consumption in residential and commercial buildings. Although worldwide installation and use of geothermal piles is increasing at a considerable rate, significant technical challenges are yet to be addressed. Regarding thermal efficiency, there is a clear need for a comprehensive model that can determine optimal circulation parameters based on building energy demand and soil properties. Additionally, the present practice lacks proper understanding of the geotechnical behavior (load transfer mechanisms) of geothermal piles under combined thermo-mechanical loading. Without precise understanding of the mechanics of load transfer under thermo-mechanical loading, a pile that would otherwise be safe for supporting a superstructure load may be inadequate to support the same structure under thermo-mechanical loading. In the absence of adequate knowledge of the above scenarios, the design and analysis of geothermal piles are still in their infancy.
The primary focus of this research is to study the mechanical, thermal, and thermo-mechanical behavior of a model geothermal pile under laboratory controlled conditions. Two major components of this research are: (i) to investigate heat transfer through a model geothermal pile installed in sand and (ii) to explore any potential changes in pile load transfer behavior under thermo-mechanical loading. Results obtained from a series of well-designed laboratory tests will assist in understanding the key effects of some operational parameters (e.g., flow rate, fluid circulation velocity, and temperature of circulation fluid at inlet) on the design of pile anchored GSHP systems. Moreover, results obtained from this research will be used to validate numerical models targeted at predicting the thermal performance of full-scale geothermal piles under variable site conditions. Results from pile load tests under different thermal conditions will assist in understanding the potential effects of thermal loading on ultimate- and limit-state capacities of geothermal piles.

1.3. Thesis Outline

The content of this thesis is distributed over five chapters. Following the introduction to the problem, as presented in the present chapter (Chapter 1), Chapter 2 provides background information in context to this research and a brief discussion of the current body of literature in this area. Chapter 3 describes the experimental setup, material characterization, and data acquisition system used in this research. Analysis and discussion of results obtained from a series of experiments are presented in Chapter 4. Chapter 5 summarizes key findings of this research; specific conclusions drawn from the laboratory tests and recommendations for future research are also outlined in this chapter.
Chapter 2
Geothermal Piles – Background and Present Practice

2.1. Ground Coupled Heat Exchangers

In ground source heat pump (GSHP) systems, the ground is utilized as either a heat source or sink, respectively, during winter and summer. During summer months heat is collected from the air inside the building using a heat pump and stored in a fluid. This fluid is then passed through either a closed or open loop of circulation tubes embedded in ground. Heat is partially dissipated from the circulation fluid to the ground surrounding the circulation tubes. In winter months the reverse process takes place; heat is extracted from the surrounding soil into the circulation fluid. This warm fluid then returns to the heat pump where it is used to heat air that is then circulated through the building using blowers.

The use of GSHP systems results in a higher coefficient of performance (i.e., the ratio of energy utilized by a system to that required to run the system) compared with that of traditional air-source heat pump (ASHP) systems. This is because heat exchange with soil is much more efficient than heat exchange with air. The temperature of the ground (used as a heat source or sink) is relatively stable year-round and very stable compared with air temperature. Air temperature fluctuates quite substantially due to both short term diurnal variations as well as long term seasonal variations. Such variations of temperature are also observed at shallow depth within the ground, but are less noticeable at increasing depth. Beyond a certain depth, the soil can be considered to be at a fairly constant temperature year round; this zone with ‘constant’ temperature is referred to as the “deep zone”.

Based on a field study, Popiel et al. (2001) proposed that the boundary between the ‘shallow zone’ where ground temperature fluctuates seasonally and the ‘deep zone’ in which the
ground temperature is relatively constant lies at a depth ranging from 1 to 8 m below the ground surface. Ground temperature recorded by Florides and Kalogirou (2006) reveals that the boundary between these zones lies approximately at a depth of 5 m below the ground (Figure 2-1).

![Graph showing ground temperature profile](image)

**Figure 2-1. Measured ground temperature in Nicosia, Cyprus (data obtained from Florides and Kalogirou 2006)**

From both numerical and field studies, it is apparent that the starting depth of the deep zone and the temperature that is maintained in that zone vary geographically. This is due to the local climate as well as the soil type. Soil type governs various thermal properties of the soil such as heat capacity, thermal conductivity, thermal diffusivity, and hydraulic conductivity. Specifically, the mineral composition, particle size distribution, and void ratio of the soil are important parameters. These properties eventually dictate heat transfer to and from the soil. A moderate constant temperature in the ground ensures a sufficient temperature gradient between the circulation fluid and the surrounding soil and thus, such condition is beneficial for ground-
coupled heat exchangers to function effectively irrespective of the seasonal variation of air temperature. A larger temperature gradient will result in a more efficient heat exchange system, as it increases the heat dissipation due to conduction.

The presence of a groundwater table further facilitates heat transfer to and from the ground. As summarized in Table 2-1, the thermal conductivities for both water and soil particles are several orders of magnitude higher than that of air (Côté and Konrad 2005). Water and many soil particles have higher heat capacities than air so they can store much more heat (Waples and Waples 2004, Michopoulos and Kyriakis 2009). Thus saturated soil is a more efficient medium for heat transfer when compared to dry soil. Typical values of thermal properties for soil particles, air and water, as obtained from literature, are given in Table 2-1.

Table 2-1. Typical thermal properties for soil, water and air

<table>
<thead>
<tr>
<th></th>
<th>Thermal Conductivity (W m⁻¹ K⁻¹)</th>
<th>Specific Heat Capacity (J kg⁻¹ K⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Particles</td>
<td>2.0 to 20.0</td>
<td>300 to 1500</td>
</tr>
<tr>
<td>Water</td>
<td>0.56</td>
<td>4179</td>
</tr>
<tr>
<td>Air</td>
<td>0.024</td>
<td>1006</td>
</tr>
</tbody>
</table>

2.1.1. Open-loop GSHP Systems

Open-loop GSHP systems refer to configurations where the circulation fluid is allowed to mix with shallow groundwater. The most common type of open system design is to construct two wells at a certain distance away from one another such that they do not interact with each other. However, specific recommendation about a minimum distance that should be maintained between two wells is not readily available in literature. While water discharged from heat pump, at elevated or reduced temperature compared to groundwater, is rejected into the shallow aquifer through the injection well, groundwater is drawn from the extraction well and fed to the heat pump through a circulation pump (Figure 2-2).
Figure 2-2. Open-loop ground source heat pump system

The benefit of using open systems is that warm water will be discharged into the ground, and will be replaced by cool water via the extraction well. Provided the wells are spaced far enough apart, the influence of the injection well on the extraction well can be assumed negligible. In the presence of natural groundwater flow, if the injection well is downstream of the extraction well this will also minimize the influence between the two wells.

While open GSHP system can be very effective in zones with high ground water table (GWT) and in the presence of ground water flow, there are two major concerns that must be addressed when considering such systems. Due to the fact that the circulation fluid is discharged directly into the ground, for environmental and practical reasons, the only fluid that can be used is water. In closed-loop systems, any heat carrier fluid (e.g., alcohol or brine solutions), which are more efficient for heat transport, can be used as long as the fluid does not mix with ground aquifer. Moreover, open systems cannot be used in frozen ground or in sites with very deep GWT; if the extraction well is unable to draw water from the ground aquifer, the system cannot operate.
2.1.2. Closed-loop GSHP Systems

Closed systems do not allow the circulation (heat carrier) fluid to leave the circulation loop. In some cases, the circulation loops are laid directly within the soil in either a vertical or horizontal orientation, thus the heat exchange occurs through the contact between the walls of circulation tubes and ground (Figure 2-3). These tubes are often made of durable plastics such as polyvinyl chloride (PVC) or high-density polyethylene (HDPE). If enough open land is available, the ground loops can be laid in horizontal loops at a relatively shallow depth. Care should be taken to ensure that the loops are below the frost depth and embedded below the GWT, if feasible.

![Figure 2-3. Closed-loop ground source heat pump system: (a) horizontal circulation loops and (b) vertical circulation loops](image)

If a large area is not available due to property boundaries or nearby structures, the ground loops can be placed vertically in the ground. The benefit of a vertical ground loop configuration is that the loops are able to reach greater depths where the soil temperature is more stable and have more contact with shallow groundwater. One common method of installing vertical ground loops is to embed the loops within a borehole which is then backfilled with grout. Such a vertical heat
exchange system is commonly known as borehole heat exchanger or geothermal borehole. Installing vertical ground loops is typically more expensive than horizontal or trench loops due to the fact that often boreholes need to be drilled to a depth up to 100 m, and is often unfeasible in urban environments due to existing subsurface infrastructure.

2.2. **Thermo-active geostructures**

Rather than placing the circulation tubes directly within the ground or in a geothermal borehole, the tubes can also be embedded within various geostructural elements such as retaining walls, footings, floor slabs, and drilled shafts. Much of the cost associated with geothermal systems is in up-front capital costs. A major advantage of embedding the circulation tubes within geostructures is significant savings in the installation cost because embedment of circulation pipes within an already planned geostructure or foundation adds only little to the cost of construction. When circulation tubing is embedded in drilled shafts (also referred to as bored piles), the special type of pile foundation is called geothermal pile, heat exchanger pile or energy pile. A GSHP system connected to geothermal piles is known as pile anchored GSHP system (Figure 2-4).

![Figure 2-4. Pile-anchored ground source heat pump system](image)
Geothermal piles are economically advantageous due to the fact that the drilled shaft would already be constructed for the purpose of supporting the superstructure load. After the borehole is drilled, the circulation tubing is attached to the reinforcement cage and lowered into the borehole. Concrete is then poured into the borehole to complete the installation of the most common type of geothermal piles. Since the drilled shaft installation cost does not change due to the insertion of the circulation loops, the only additional cost for installing geothermal pile is from the material cost for the circulation tubes.

The use of geothermal piles for harvesting geothermal energy in addition to their primary purpose of supporting superstructure load raises two primary geotechnical concerns that must be addressed adequately. Past studies indicate that capacity for axially loaded piles may change under thermal loading; however, there is no clear consensus in literature about the nature and magnitude of such a change. Such a change in pile capacity may occur due to multiple reasons such as a change in fundamental soil properties at elevated or reduced temperature, change in pile-soil interface behavior, and change in load displacement behavior of the pile due to constrained thermal loading and accumulation of irreversible thermo-plastic strains along the length of the pile and at pile base. While all of the above scenario may not be equally applicable for piles installed in clays and sand; however, all of the above conditions are worth for systematic exploration.

Another concern for geothermal piles is regarding tolerable pile settlement. Axially loaded piles are designed such that when loaded to an ultimate state, pile head settlement should not violate an allowable threshold. Due to the imposed thermal cycles on geothermal piles, additional mechanical stresses are expected at different pile cross sections. Such additional induced mechanical stresses will compound with the mechanical stresses imposed by the superstructure load and may cause additional settlement that would not be expected in the absence of the thermal loading.
2.3. Present Practice with Geothermal Piles

Over the last two decades, geothermal piles have successfully been used in different countries in Europe (e.g., Germany, Austria, Switzerland), U.K., Australia and Asia (e.g., China, Japan). In recent years, the use of these piles has drawn significant attention of the construction industry in the U.S. as well. Nevertheless, the use of geothermal piles does present technical concerns which are not relevant for ground loops or borehole heat exchangers. The imposed thermal loading on the piles tend to cause thermal expansion and contraction. However, the thermal expansion (or contraction) of the pile is confined due to frictional resistance along the shaft, as well as due to soil resistance at the pile base. Such confinement would induce mechanical stresses that may compound with the stresses already present from the superstructure load. These additional stresses can be significant and must be addressed in the geotechnical design of the geothermal piles. In spite of the growing recognition of the benefits of geothermal piles, actual geotechnical behavior of these piles under thermo-mechanical loading is not well understood yet. Additionally, amount of heat exchange through these piles are still calculated based on ad-hoc rules. Consequently, design of geothermal piles relies mostly on experience and empirical rules.

Due to the complex nature of heat transfer and thermo-mechanical behavior of geothermal piles, design of pile anchored GSHP systems still relies heavily on empirical data and regional experience. The Ground Source Heat Pump Association (GSHPA) in U.K. has created a standard that details geothermal pile design and installation (GSHPA 2012). However, this guideline lacks specific calculation steps to quantify the effects of different design parameters on geothermal pile performance. For example, the GSHPA standard qualitatively addresses the notion that thermal loading will induce mechanical loads in addition to the existing superstructure loads, however, it does not outlines any calculation steps to quantify these loads. The physics of geothermal pile anchored GSHP systems crosses into a variety of different scientific disciplines,
therefore, competent professionals are required to ensure the systems are designed efficiently and safely. GSHPA standard addresses this issue by requiring collaboration among mechanical and electrical (M&E) engineer, geotechnical engineer, geologist and contractors for design and construction of geothermal piles. In the United States, design and installation of residential and light commercial GSHP systems follows the International Ground Source Heat Pump Association (IGSHPA) manual, which is primarily focused on vertical ground loops and borehole heat exchangers.

To properly design and construct GSHP systems, along with standard geotechnical site exploration for conventional pile design, additional site properties such as soil thermal conductivity, heat capacity, undisturbed temperature, and thermal diffusivity should be obtained. Some commercial software such as GLHEPRO, GchpCalc, GS2000, and EED (Hellström & Sanner 2001) are available to assist with the design of various GSHP configurations if the above parameters are known. Note that these software packages were primarily developed for design of geothermal boreholes and thus cannot directly be used for geothermal piles.

Simple idealized models are often used for analysis of heat transfer through vertical geothermal heat exchangers. Among such models, the most practical one is a finite solid cylindrical heat source model discussed in Section 2.4.2. However, the idealized models suffer from the approximation of a constant heat flux along the entire length of a heat exchanger, which is far from reality and may introduce significant errors in the heat transfer calculation (Ghasemi-Fare and Basu 2013a, 2013b). While the available software packages and idealized heat source models can suggest a circulation tubing length, none of these can determine circulation velocity to optimize heat transfer. Furthermore, additional design considerations must be adopted for geothermal piles. A design method for geothermal piles must address the effects of heat exchange on the thermo-mechanical behavior of the geothermal piles.
2.4. Literature Review

Several case studies examine the state of geothermal piles in terms of design and current usage worldwide (Sanner 2003, Florides and Kalogirou 2007, Preene and Powrie 2009, Self et al. 2013, Bahadori et al. 2013, Park et al. 2013). Though complete failure of geothermal piles is not anticipated in the sense of structural collapse or limit state conditions, Preene and Powrie (2009) anticipates that serviceability issues might be a concern for these piles under thermo-mechanical loading. Such concern essentially points to the fact that the pile-anchored GSHP systems may not either meet the thermal demand of the building or pass serviceability conditions while operating under their full thermal capacity.

2.4.1. Experimental Studies

2.4.1.1. Field Tests

There have been a number of field tests within the past decade that study the performance and feasibility of geothermal piles as an effective and energy efficient foundation alternative (Brandl 2006, Laloui et al. 2006, Hamada et al. 2007, Gao et al. 2008, Bourne-Webb et al. 2009, Li et al. 2009, Jalaluddin et al. 2011, Amatya et al. 2012, Wang et al. 2012a). Most of these studies tend to focus on the heat transfer behavior of the geothermal pile systems, though there has been some effort to understand the effect of induced thermal gradients on the thermo-mechanical behavior of geothermal piles (Laloui et al. 2006, Knellwolf et al. 2011).

An instrumented field study was performed by Laloui et al. (2006) at the Swiss Federal Institute of Technology campus. A U-shaped polyethylene circulation tube was embedded in one of the 88-cm-diameter and 25.8-m-long piles used to support a four story building. Throughout building construction, strain was monitored along the pile and load was measured at the pile head.
and base. Several thermal load cycles with initial temperature difference (between ground and inlet fluid) \( \Delta \theta = 21^\circ C \) was applied at various stages of construction. Initially, thermal loading was applied to the pile before the building was constructed so the pile head was free to move, while there was still resistance at the pile base and along the shaft due to the soil. After construction of the building, thermal loading was again applied to the pile. In this case, the pile head movement was partially restricted due to the superstructure load and thus, additional mechanical stresses were generated due to thermal loading. This study found that the additional thermally induced mechanical stresses were as much as twice the mechanical stress caused solely by the superstructure load. Such additional induced stress was manifested most significantly at the base of the pile. Heating or cooling of the pile cause the pile to expand or contract respectively. This thermal expansion will be resisted by shear stress mobilized at the pile-soil interface. The mobilization of interface shear stress depends mostly on the surrounding soil layer. Stiffer soil layers will generate more shear resistance to the expanding pile than soft clays or loose sands.

Hamada et al. (2007) studied heat transfer performance of a number of instrumented geothermal piles used as foundation of a two-story office building in Hokkaido, Japan. Four different configurations were used for the embedded circulation tubes: U-shaped, double U-shaped, indirect double-pipe, and direct double-pipe. Research concluded that as compared to the U-shaped circulation tube, the other configurations did not provide enough additional efficiency to warrant benefit with respect to the additional construction issues and cost. For the circulation parameters used in this study (flow rate \( Q = 5 \) L/min; 40% propylene glycol mixture for circulation fluid), the average COP was calculated to be equal to 3.9, which lead to a seasonal primary energy reduction of 23% as compared to tradition air source heat exchangers.

In a similar field study by Gao et al. (2008), a series of cast-in-place geothermal piles were installed in Shanghai, China with various circulation tube configurations. The piles were 600 mm in diameter, 25 m in length and were installed in a sandy silt deposit. Water was used as
the heat carrier fluid circulating through embedded high density polyethylene (HDPE) tubes. Four circulation tube configurations were studied: W-shaped, single U-shaped, double U-shaped, and triple U-shaped. The circulation fluid entered the piles with an imposed temperature difference about 17°C above ambient. The heat carrier fluid (water) was circulated at three different flow rates (= 2.84, 5.68 and 11.36 L/min) in order to study the effect on the thermal efficiency of each system under varying flow conditions. It was observed that an increase in the flow rate of the circulation fluid reduced the temperature change (a measure of heat transfer to the ground) between the fluid inlet and outlet points. However the overall heat dissipation, and therefore efficiency, of the pile increased with increase in circulation flow rate. It was also observed that the configurations with multiple loops, which results in longer fluid travel length, dissipated more heat than the single looped systems. Such increase is not linearly proportional to the number of loops. The study concluded that the W-shaped circulation loop with the reference flow rate was the most efficient configuration.

Jalaluddin et al. (2011) performed a field study in Saga City, Japan on steel geothermal piles embedded in a stratified soil deposit including soft Ariake clay, sand, and sandy clay. Various circulation tube geometries were studied including U-tube, double-tube, and multi-tube. To study the effect of circulation rate on the efficiency of the geothermal pile, the carrier fluid was pumped at 2, 4 and 8 L/min. Temperature was monitored at a number of locations along the pile and within the ground. Data recorded during this field test showed for the double-tube and multi-tube configurations, increasing the flow rate of the circulation fluid increased the efficiency of the pile. For the single U-tube configuration, increasing the flow rate had a negligible effect on the thermal efficiency of the pile. With the circulation parameters and soil properties reported by Jalaluddin et al. (2011), heat exchange rates ranged from 30.4 W/m to 49.6 W/m for the various geothermal piles.
A case study by Amatya et al. (2012) examined three published field studies performed at Lambeth College, London, U.K. (Bourne-Webb et al. 2009), Lausanne, Switzerland (Laloui et al. 2006), and Bad Schallerbach, Austria (Brandl 2006). The purpose of this comparative case study was to understand the thermo-mechanical behavior of geothermal piles, specifically with respect to additional mechanical stresses generated due to imposed thermal loads. The research concluded that the effect of thermo-mechanical loading on geothermal piles depend on end conditions of the pile (e.g., fixed base, fixed head, end bearing). Thermal expansion or contraction of the pile will be resisted by shaft resistance along the pile, as well as base resistance provided by the soil. The shear stresses along the pile shaft will act in the direction opposite to that of the pile movement. The thermally induced mechanical stresses measured in this study were significant and compounded with the stresses induced by the superstructure.

2.4.1.2. Laboratory Tests

Laboratory-scale investigations on model geothermal piles are rather few. There are only two such attempts reported in literature: a series of centrifuge tests in the University of Colorado at Boulder, U.S. (McCartney and Rosenberg 2011) and laboratory tests performed in the Monash University, Australia on a miniature steel pile (Wang et al. 2011). Interestingly, current studies have reported conflicting results regarding the change in shaft capacity of the piles after thermal loading. In the centrifuge tests by McCartney and Rosenberg (2011) a model-scale geothermal pile was embedded in compacted Bonny silt (Figure 2-5). The precast concrete pile had a diameter of 76.2 mm and length of 38.1 mm; under a 24g centrifuge environment, this would be roughly equivalent to a prototype diameter of 1.8 m and prototype length of 9.1 m. The cylindrical soil tank had a diameter of 0.6 m and a height of 0.54 m.
The model pile was subjected to a series of axial load tests using a vertical worm drive. Thermal loads were induced using a heat pump that circulated silicone fluid through the embedded aluminum circulation tubing within the pile. Note that the use of silicone fluid and aluminum circulation tubes deviate from the practice with real geothermal piles. To study thermo-mechanical behavior, load tests were first performed at ambient conditions and then after the pile was subjected to a thermal gradient of $\Delta \theta = 29^\circ C$ and $41^\circ C$, respectively.

As the thermal gradient increased, the model pile showed a stiffer load-displacement behavior and had a higher limit capacity than at ambient conditions. For the imposed thermal gradients, an increase in shaft resistance of up to 40% was observed. It was suggested that this may be due to increased lateral stresses induced by the thermal expansion of the soil surrounding the pile (McCartney and Rosenberg 2011).

A laboratory study on a steel pile replica was performed by Wang et al. (2011) at the Monash University, Australia. A 25-mm-diameter pipe pile with an internal heating element was
embedded in N50 fine silica sand as well as 300WQ flour (Figure 2-6). The test was performed in a cylindrical steel tank with a diameter of 272 mm. Mechanical load was imposed axially from the bottom of the pile. Thermal loads were imposed using the heating element contained within the pipe pile.

![Laboratory test setup after Wang et al. (2011)](image)

**Figure 2-6. Laboratory test setup after Wang et al. (2011)**

Before any thermal gradient was imposed on the pile, loading-unloading cycles were performed to determine the load-displacement behavior of the pile at ambient conditions. The steel pile was then heated to 40°C ($\Delta\theta = 19^\circ\text{C}$) for 20 hours and loading-unloading cycles were repeated to study any potential change in mechanical behavior of the pile. In contrast to the findings from the centrifuge tests performed by McCartney and Rosenberg (2011), Wang et al.
(2011) reported a reduction of shaft resistance between 11 and 50% after thermal loading. Since the pile was loaded from the bottom, and the top of the pile extended above the soil surface, the effect on the soil resistance at the base of the pile was not studied.

2.4.2. Theoretical Studies

2.4.2.1. Heat Exchange Behavior

The analysis of heat exchange through geothermal piles is currently an active area of research. An extensive overview of the development from simple models to more comprehensive recent models is provided by Ghasemi-Fare and Basu (2013a). The heat transfer through a vertical heat exchanger was first approximated using infinite line heat source with constant heat flux (Carslaw and Jager 1947, Ingersoll et al. 1954). This solution is the simplest one since it reduces the dimensionality of the problem; due to radial and vertical symmetry, this results in a solution for one-dimensional (1-D) conductive heat flow.

Solutions were later generated using a finite line heat source model with steady-state (Eskilson 1987) and transient conditions (Zeng et al. 2002, Lamarche and Beauchamp 2007a). An analytical solution was developed by Lamarch and Beauchamp (2007b) using an infinite solid cylinder model. This was an advancement towards analyzing borehole heat exchangers, as well as geothermal piles, because the solution now included two media where the solid cylinder was the heat source, and the second medium was the soil surrounding the heat source.

Man et al. (2010) developed an analytical solution for quantifying heat flow through one- and two-dimensional solid cylinder heat sources. Analytical solutions were also found for heat exchanger piles with spiral heat sources (Cui et al. 2011, Li and Lia 2012). This was an attempt to
better approximate the geometry of the heat source as it would occur in spiral borehole heat exchangers.

The available analytical solutions are all helpful in approximating the heat flow behavior of geothermal pile systems, but they are all limited in that they all assume a constant heat flux from the heat sources. This is not representative of the heat flux from a geothermal pile which utilizes the circulation fluid to carry heat, and therefore will have non-uniform heat flux along the length and over time of operation as the fluid dissipates heat.

An annular heat source model was developed by Ghasemi-Fare and Basu (2013a, 2013b) to study heat flow through geothermal piles. This model considers heat transport through the heat carrier fluid. Consequently, the heat flux down the length of the pile varies over time, rather than being constant as assumed by idealized heat source models. The finite difference code developed using this model is capable of predicting the fluid, pile, and soil temperature spatially and temporally.

### 2.4.2.2. Load Transfer Behavior for Geothermal Piles

For an axially loaded pile, shaft resistance starts mobilizing from the top of the pile and as the load at the pile head increases more resistance is developed along the pile to balance the load at the pile head. Shaft resistance is developed due to the friction mobilized along the pile-soil interface. When a vertical load is applied to the top of the pile (i.e., at pile head), the load experienced by a certain pile cross-section decreases along the length of the pile. After full mobilization of the shaft resistance, any remaining load is accounted for through the base resistance of the pile, which is developed due to the compressive resistance of the soil under the pile base (Figure 2-7).
Figure 2-7. Development of shaft and base resistance in an axially loaded pile

For case (a) shown in Figure 2-7, a vertical load $Q_a$ applied to the pile is resisted entirely by the surrounding soil within a depth $z_a$. In this case, the portion of the pile below depth $z_a$ does not experience any vertical stress since the load is carried entirely by the shaft resistance mobilized within depth $z_a$. As the vertical load increases, the curve will shift from case (a) to case (b), where shaft resistance is mobilized along the entire surface of the pile. At this point, the entire load is still being carried via shaft resistance and the base of the pile does not carry any load. As vertical load continues to increase, the curve will continue to shift from (b) to (c). In this case since the shaft resistance has been fully utilized, the remaining load will be transferred to the base of the pile. Following this logic, the total capacity of the pile can be resolved into the limit shaft resistance $Q_{SL}$, and the limit base resistance $Q_{BL}$. For all practical cases, $Q_{SL}$ will be reached far before $Q_{BL}$.

When quantifying the axial capacity of a pile, it is important to make a distinction between the limit capacity $Q_L$ and the ultimate capacity $Q_U$. Note that under the limit load $Q_L$, an
axially loaded pile would undergo indefinite settlement (or plunging); this would mean that the pile will continuously penetrate into the soil without any further increase in loading. Thus, limit loading is not a practical pile design criterion to consider. It is in fact very difficult, particularly in competent soils, to achieve the limit load for an axially loaded pile. Design of axially loaded piles are often controlled by an ultimate load $Q_U$, which is mostly determined based on different settlement thresholds. Several ultimate capacity criteria are proposed in literature; among those the mostly used ones are Van der Veen (1953), DeBeer (1968), Chin (1970) and Davisson (1972, 1975). The limit shaft and ultimate base resistance of a pile can also be estimated if the field conditions are known; for drilled shafts such equations are proposed by Fleming et al. (1992), Lee and Salgado (1999), Salgado (2006), and Salgado and Prezzi (2006).

![Typical load-displacement curve for an axially loaded pile](image)

**Figure 2-8. Typical load-displacement curve for an axially loaded pile**

The reason for defining ultimate capacity criteria is that excessive settlement of a pile will cause structural and functional damage (i.e., lack of serviceability), even if the limit capacity
of the pile has not yet been reached. Small settlements reaching serviceability limit states may cause cosmetic damage to a building such as cracked windows, jammed doors, and other problems. However, at settlements exceeding the serviceability limit state structural members would start showing significant signs of distress. If settlement continues, key structural members may completely fail leading to the complete collapse of the structure. Up until this point, the piles supporting the structure may still not have reached their limit load capacity, albeit a plethora of serious issues have occurred.

Rigorous theoretical research on energy piles is very limited in number. A theoretical-experimental study performed at École polytechnique fédérale de Lausanne (EPFL), Switzerland investigated the thermal influence on the geotechnical capacity of energy piles (Laloui et al. 2003, 2006; Laloui and Nuth 2006). The results from field load tests performed on an instrumented energy pile were used to examine the capabilities of an axisymmetric Finite Element (FE) model that simulated the thermal and mechanical loading on a single pile. A Drucker-Prager, thermo-elastic soil constitutive model following associated flow rule was used for the FE simulation. The study concluded that the mechanical and thermal stresses induced along the length of the pile were different depending on the displacement boundary conditions at the pile head and pile tip. Also, the thermal stress distribution along the pile length was different during heating and cooling cycles. Knellwolf et al. (2011) proposed a simplified analytical model to assess the effect of thermal loading on the stress-strain response of a pile. However, the limitation of this method lies in the over-simplified assumptions made regarding (i) the zero radial displacement for both pile and soil and (ii) the approximate load-displacement relations along pile shaft and at the pile head. Additionally, the study neglects the temperature changes in the soil and its potential effect on soil deformation and stresses. McCartney and Rosenberg (2011) performed a simple analysis to fit theoretical hyperbolic load transfer curves to the load-displacement response obtained from centrifuge tests on model energy piles. Recently, Wang et al. (2012b) pointed out the need for and
the challenges involved in coupled thermo-hydro-mechanical continuum-based analysis of energy piles, and presented initial results from a small-strain, coupled FE formulation for linear isotropic elastic soil skeleton.
Chapter 3

Experimental Setup for Present Research

Results from full-scale pile load tests can help in understanding pile-soil interaction; however, from a practical point of view, large-scale laboratory load tests on model piles can be advantageous over full-scale pile load tests because of the high cost and uncertain field conditions associated with field-scale tests. In addition, soil properties can be controlled and measured more easily in laboratory than in the field (Turner and Kulhawy 1994). The same is also true for temperature measurement for geothermal piles. One of the main advantages of model pile tests, like the one detailed in this thesis, is that multiple load tests under varying conditions can easily be performed under fully controlled testing conditions, avoiding uncertainties of natural soil profiles. The laboratory pile load test program is designed carefully such that scale effects arising from the ratio of pile diameter B to mean particle size $D_{50}$ and that from the distance of the chamber boundary from the pile can be avoided. With the test program described in this study, the ratio of pile diameter to tank width is 1:18, and that of pile diameter to soil height under the pile base is 1:6. For a nondisplacement pile, the pile to tank diameter ratio should be at least 1:8 to avoid mechanical boundary effects (Kraft 1991, Schnaid and Houlsby 1991). Scale effects arising from the ratio of mean sand grain size $D_{50}$ (which is directly related to shear band thickness $t_s$ in sand) to pile diameter B is avoided in this study by choosing a very small value of $t_s/B$ ($= 0.03$) such that sand particle size no longer affects the shear stress mobilization along the pile shaft (Loukidis and Salgado 2008). A soil depth of $6B$ was kept under the pile base so that both thermal and mechanical boundary effects could be avoided below the pile base.

An inherent limitation of laboratory model tests at earth gravity ($1 \text{ g}$) in soil chambers or tanks is that these tests cannot simulate the conditions of full-scale field tests perfectly because stress similitude cannot be maintained, such as with centrifuge tests. This disadvantage of having
a test at low confining stress is present only if one expects to scale the results up to prototype scale without proper theoretical intervention. However, results from model pile load tests can be used to validate the results from numerical analyses and to gain additional insight into the nature of pile-soil interaction under thermo-mechanical loading. Thus validated numerical models can further be used to obtain desired output under field stress conditions.

3.1. Laboratory Pile Load Test Setup

The geothermal pile load test setup, built and used for this research, is located at the Civil Infrastructure Testing and Evaluation Laboratory (CITEL) facility of The Pennsylvania State University (PSU), University Park. A model-scale precast geothermal pile was embedded in sand bed prepared within a large soil tank (Figure 3-1). The model pile was subjected to several thermal and mechanical load cycles under varying conditions.

The test bed was prepared using conventional ‘sand raining’ technique (Bieganousky and Marcason 1976, Rad and Tumay 1987, Cresswell et al. 1999). A sand pluviation device was designed and fabricated for this purpose. By varying the size of the sieves used in the sand pluviator and sand drop height, a desired relative density can be achieved for the sand deposit (Gandhi and Selvam 1997, Cho et al. 2002, Lings and Dietz 2005, Kim and Yoon 2010, Iskander 2010, Lee et al. 2011). Sand deposition was temporarily halted when the desired level of pile base was reached. The precast model pile was then held vertically in place and sand deposition was resumed without disturbing the pile. The model pile installation (placement) process adopted for this research closely simulates the in situ stress condition that would exist around nondisplacement piles, i.e., piles that produce minimal or zero soil displacement during its installation and thus, the in situ stress condition in the vicinity of the pile is not significantly disturbed by pile installation (Salgado 2008). Before depositing the sand into the tank, a 6 cm
thick layer of crushed stone was placed at the bottom of the tank and the stone layer was covered with a felt fabric. The stone layer was used to facilitate with the tank saturation. The free draining stone layer allowed bottom-up saturation of the sand bed in a uniform manner. Also, this layer helped to avoid any possibility of piping that would arise if water was forced directly into the sand bed from the bottom. The fabric separating the stone from the sand acts as a filter to ensure the sand does not clog the stone layer.

Heat exchange performance and load-displacement behavior of the model geothermal pile was monitored using proper instrumentation. A well-controlled test environment was maintained for all mechanical and thermal load tests performed on the model pile. In contrast to the previous laboratory tests reported by Wang et al. (2011, 2012a), in which a heating element was used to induce thermal load on a steel pipe pile, the present research follows the exact heating mechanism used in real geothermal piles. This allows variables such as circulation rate and imposed thermal gradient to be isolated and studied.
3.1.1. Soil Tank

The model geothermal pile is installed in a sand bed prepared within a custom-designed steel tank. The soil tank has a 1.83 m × 1.83 m (6 ft × 6 ft) square cross-section and is composed of a 1.22-m-tall base portion and a top portion with height equal to 0.91 m (Figure 3-2). The upper half of the tank fits directly on top of the lower portion of the tank, and has bolted connections around the circumference of the tank. The advantage of having two separate sections for the tank is that the lower part of the tank is more easily accessible (without the top half placed on it) during preparation and instrumentation of the soil bed. An adjustable reaction frame is attached to the tank, position of the cross beam in this reaction frame can be changed to attain a desired height during pile load tests.
The width of the soil tank is equal to 18 times pile diameter B. For nondisplacement piles, the distance of the free-field boundary is dictated mostly by loading condition (i.e., lateral versus axial); however, for displacement piles such boundary is mostly governed by installation process (i.e., driving or jacking). Literature suggests that for axial load tests on nondisplacement piles in sand, the use of a tank that is at least 8 to 10 pile diameters wide is sufficient to avoid mechanical boundary effects; however, such a distance should be significantly higher for full displacement piles (Kraft 1991, Parkin et al. 1980, Schnaid and Houlsby 1991, Salgado et al. 1998). The distance between the pile base and the bottom boundary is kept equal to 6B, which is greater than the expected zone of influence (around 1.5B to 3B) below the pile base when the pile is subjected to axial loading (Salgado 2008). In addition to the mechanical boundary effects, the tank is designed to avoid immediate thermal boundary effects. Preliminary finite element simulations of heat transfer through the model pile suggested that thermal loading can be applied
for almost 7 days without affecting the ambient temperature at the boundary of the tank. Actual thermal tests in the tank later validated such initial calculation.

### 3.1.2. Sand Pluviation Device

A pluviation system (0.76 m × 0.76 m) was designed for ‘raining’ sand into the soil tank. This system contains a perforated steel box with an attached shutter plate on its bottom (to stop sand raining when desired) and up to four layers of sieves underneath. The large sieves which were fabricated for use in the pluviation device include #6, #10, #12, and #16 standard size meshes (corresponding to sieve opening sizes 3.36, 2.00, 1.68 and 1.19 mm, respectively). The assembled pluviator with three of the four sieves attached is shown in Figure 3-3.

![Sand pluviation device with #6, #10, and #12 sieves attached](image)

**Figure 3-3.** Sand pluviation device with #6, #10, and #12 sieves attached

The pluviator assembly was lifted using a rolling crane so that it could be moved to any point over the soil tank. The relative density $D_R$ achieved by using sand raining technique
depends primarily on the free fall height of sand particles and the number and size of sieves obstructing the sand’s descent.

The relationship between drop height and relative density was studied by Vaid and Negussey (1984). It was found that the final relative density of the sand was a function of the drop height, since that would determine the velocity of the sand particles immediately before impact. Mathematically,

\[ ma = mg - V \rho_m g - C_d \rho_m A \frac{v^2}{2} \]  

(1)

where \( m, a, V \) and \( A \) are, respectively, mass, acceleration, volume and projected area of the sand particle, \( \rho_m \) is the mass density of the medium through which the sand particles are deposited, \( g \) is the acceleration due to gravity, \( C_d = \text{drag coefficient} \) and \( v \) is the velocity of particle immediately preceding impact. Equation 1 describes the net acceleration of a spherical particle falling through a uniform medium – the first term is the downward gravitational force, the second term is the buoyancy force, and the last term in the drag force. As shown in Equation 1, the force of the particle at impact is related to the square of the velocity of the particle. The kinetic energy of the particle will increase with drop height, which increases the potential energy, due to the law of conservation of energy.

This trend will continue, though the net downward acceleration will slow, then eventually reach a constant velocity if the particle is provided enough height to reach terminal velocity. When at terminal velocity, the downward gravitational force is negated by the buoyancy and drag forces, which results in zero net force on the particle, which results in it falling at a constant velocity. At this point, increasing the drop height will not change the relative density of the test bed, since the particles cannot fall any faster than their terminal velocity. An interesting observation is that this phenomenon is very pronounced if sand is rained into standing water. Since water is much more viscous than air, the drag force is much greater on the sand particles.
As such, the particles will reach their terminal velocity after falling through just around 2 mm of standing water, and this terminal velocity will be about 3% of the terminal velocity in air (Vaid and Negussey 1984).

Along with achieving a desired relative density, the pluviator also helps with ensuring a relatively uniform test bed is prepared. After passing through the top shutter plate, the sand falls through a series of sieves, each of which assist in spreading the sand such that it falls uniformly under the footprint of the assembly (Figure 3-4). To ensure quality and repeatability, trial sand depositions were performed with varying fall heights (measured from the bottom of the shutter plate) and sieve combinations. Density calibration plots (Figure 3-5) were prepared to establish a relation between the fall height and sieve combinations. As expected, increasing the number of sieves increases relative density of the deposit. Interestingly, this trend discontinues after two sieves; from this point on, the relative density is independent of the number of additional sieves used. This is evident in Figure 3-5, which shows that $D_r$ obtained using three different sieve combinations with two or more sieves collapses onto a single curve. After this point, relative density of the sand deposit is only a function of drop height. For a target value of $D_r$, the sieve combination and drop height can be determined from Figure 3-5. Trial depositions made by using the density calibration curves revealed that the repeatability of the deposition process falls within a standard deviation of approximately 4%. The density calibration curve shown in Figure 3-5 was used to prepare the sand bed used for the tests reported in this thesis. The tank was filled using 7.5 cm lifts, with instrumentation placed at desired depths between lifts.
Figure 3-4. Uniform deposition of sand as it passes through the pluviator system.

Figure 3-5. Relative density calibration curve.
3.1.3. Model Geothermal Pile

A precast concrete model pile is used for this research. The concrete mix design was performed in accordance with the Federal Highway Administration guideline for drilled shafts (FHWA 2010). The mix design is discussed in further detail under section 3.2.2 of this thesis. The 100-mm-diameter and 1.38-mm-long pile was embedded 1.18 m into the sand bed. A poly-vinyl chloride (PVC) circulation tube with outer and inner diameters equal to 12.7 and 9.5 mm, respectively, was inserted into the pile. The U-shaped PVC circulation loop and the pile’s formwork are shown in Figure 3-6. The model pile also accommodates for a telltale rod that can pass through the entire pile length to rest on the pile base. The room for the telltale rod was made by positioning a 9.5-mm-diameter PVC tube within the formwork before casting the concrete. Figure 3-7 shows a schematic of the model pile with circulation tube in it. Following FHWA recommendation, a concrete cover of 25.4 mm was provided based on the maximum aggregate size used in the concrete mix.

![Figure 3-6. Formwork used for pile casting and PVC circulation loop](image-url)
Surface roughness measurements were made along several representative sections of the precast model pile to characterize the pile-soil interface. Due to size restrictions of the optical profilimeter, the profile was limited to approximately 1 cm sections. The interface between a pile and surrounding soil can be classified as either ‘perfectly smooth’ or ‘perfectly rough’ (Basu et al. 2011). For perfectly smooth interfaces, the dominant failure mode at the interface will be close to slip-failure along the surface of the pile. Conversely, perfectly rough interfaces will cause a shear band to form within the soil at the immediate vicinity of the pile interface. The failure mode at the pile-soil interface is governed by the normalized roughness $R_n$ which is calculated by dividing the maximum roughness of the pile surface $R_{\text{max}}$ by the mean particle size $D_{50}$ of soil. Using optical profilometry, the pile surface was determined to have $R_{\text{max}}$ equal to 20 µm (Figure 3-8). The $D_{50}$ of the F50 Ottawa sand used in this research is equal to 0.25 mm. This results in an $R_n$ value equal to 0.08, which indicates a perfectly rough pile-soil interface (Uesugi and Kishida 1986,
Uesugi et al. 1988, Lings and Dietz 2005, Basu et al. 2011). For perfectly rough pile-soil interface, the interface friction angle can be assumed to be equal to the critical state friction angle of the sand, which was determined to be equal to 31.8° for F50 Ottawa sand.

Figure 3-8. Surface roughness profile for the concrete model pile

3.2. Materials

3.2.1. Sand

Standard F50 Ottawa sand was chosen for this research due to its frequent use as laboratory test sand. This is silica sand with mean particle size $D_{50}$ equal to 0.25 mm and a value of coefficient of uniformity $C_u$ equal to 1.8. The particle size distribution curve for this sand, as obtained from sieve analysis in accordance with ASTM D6913-04, is shown in Figure 3-9.
The scanning electron microscope (SEM) images reveal a typical subangular shape for the sand particles (Figure 3-10). The sand is very uniform with regards to the chemical composition. The mineral composition was confirmed using energy-dispersive X-ray spectroscopy (EDS). The dominant mineral composition of the sand is quartz, with trace amounts of aluminum oxides and other metal oxides (Figure 3-11).
Figure 3-10. SEM image of F50 silica sand: (a) at 65x magnification and (b) at 159x magnification
The specific gravity $G_s$ of sand particles was determined using the pycnometer method as specified by ASTM D854-10. The value of $G_s$ was found to be equal to 2.65. The values of minimum and maximum void ratios are required for relative density calculations. Following ASTM D4254, the value of minimum void ratio $e_{\text{min}}$ was found to be equal to 0.48; similarly, the maximum void ratio $e_{\text{max}}$ was determined to be equal to 0.78 following ASTM D4253. Direct shear tests were performed (following ASTM D3080) under normal stresses of 100, 200, 300 and 400 kPa at a relative density of 75% (i.e., $e = 0.55$). The critical state friction angle for the sand was found to be equal to of 31.8° (Figure 3-12).

Figure 3-11. EDS spectrum of F50 silica sand
An apparatus was constructed to measure the thermal conductivity of sand and concrete (Figure 3-13a). The thermal conductivity test setup built as part of this research is similar to the standard test method for determining thermal conductivity in soil and soft rock (ASTM D5334); however, rather than using a heating probe, heat carrying fluid was used as the heat source. The sand was poured into a cylindrical mold with a diameter equal to 0.3 m and a height of 0.6 m. Through the center of the mold, a PVC tube (OD = 15.8 mm, ID = 12.4 mm) was placed so the heat transfer from the fluid was radially outward. The top and bottom of the mold were insulated such that all of the heat loss was in the radial direction. Thermocouples were placed in a 3×3 grid in the sand, as well as in the fluid at the inlet and outlet of the mold (Figure 3-13b).
The temperature of the fluid at the inlet was maintained at a constant temperature elevated $\Delta \theta$ above room temperature. The system was allowed to come to thermal equilibrium as shown in Figure 3-14, and then Fourier’s Law could be used to calculate the thermal conductivity since the geometry was known and the thermal gradients were measured. The total energy dissipated from the system could be calculated as:

$$E = \dot{m}C_p(T_{in} - T_{out})$$

where $E$ is the energy dissipated, $\dot{m}$ is mass flow rate of the fluid, $C_p$ is the specific heat capacity of the fluid, and $T_{in}$ and $T_{out}$ are fluid temperatures at the inlet and outlet, respectively. Once the system is at thermal equilibrium, it can be assumed to be at steady-state and, therefore, Fourier’s Law can be used to determine the thermal conductivity of the sand. Mathematically,
\[ E = -kA \frac{dT}{dX} \]  \hspace{1cm} (3)

where \( k \) is the material thermal conductivity, \( A \) is the area of heat flux, \( \frac{dT}{dX} \) is the thermal gradient across the aforementioned flux surface. Equation 3 can be integrated along the length of the cylinder, and can then be set equal to Equation 2 since both equations solve for total heat loss by the system. The solution can then be manipulated to solve for thermal conductivity, resulting in the expression:

\[
k = \frac{\dot{m}C_p(T_{in} - T_{out}) \ln \left( \frac{r_2}{r_1} \right)}{2\pi L(T_1 - T_2)}
\]  \hspace{1cm} (4)

where \( r \) is radial distance from the central axis, \( T \) is temperature at that location, and \( L \) is the length of the cylinder.

![Figure 3-14. Average temperature at each radial distance](image-url)
Values of hydraulic conductivity of the sand at different densities were determined using the constant head permeability test specified in ASTM D2434-68. The hydraulic conductivity values for the F50 sand range from 0.025 to 0.038 cm/s (Figure 3-15). Different properties for the F50 Ottawa sand, as determined as part of this research, are summarized in Table 3-1.

![Figure 3-15. Hydraulic conductivity of F50 Ottawa sand as a function of void ratio](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape</td>
<td>Subangular</td>
<td>Scanning Electron Microscope</td>
</tr>
<tr>
<td>Mineral composition</td>
<td>&gt; 99% Quartz</td>
<td>Energy Dispersive Spectroscopy</td>
</tr>
<tr>
<td>Mean particle size, $D_{50}$ (mm)</td>
<td>0.25</td>
<td>ASTM D6913</td>
</tr>
<tr>
<td>Coefficient of Uniformity, $C_u$</td>
<td>1.83</td>
<td>ASTM D6913</td>
</tr>
<tr>
<td>Coefficient of Curvature, $C_c$</td>
<td>0.95</td>
<td>ASTM D6913</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.65</td>
<td>ASTM D854</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{\text{min}}$</td>
<td>0.48</td>
<td>ASTM D4253</td>
</tr>
<tr>
<td>Maximum void ratio, $e_{\text{max}}$</td>
<td>0.78</td>
<td>ASTM D4254</td>
</tr>
<tr>
<td>Critical state friction angle, $\phi_c$</td>
<td>31.8°</td>
<td>ASTM D3080</td>
</tr>
<tr>
<td>Hydraulic conductivity (cm·s$^{-1}$)</td>
<td>0.025 to 0.038</td>
<td>ASTM D2434</td>
</tr>
<tr>
<td>Thermal conductivity, dry (W·m$^{-1}$·K$^{-1}$)</td>
<td>0.25</td>
<td>Cylindrical Heat Source</td>
</tr>
<tr>
<td>Thermal conductivity, moist (W·m$^{-1}$·K$^{-1}$)</td>
<td>2.65</td>
<td>Cylindrical Heat Source</td>
</tr>
</tbody>
</table>
3.2.2. Concrete

The pile used in this experiment was a precast nondisplacement pile. The concrete mix design was performed in accordance with the FHWA specifications for drilled shafts (FHWA 2010). Concrete used for foundations is very similar to structural concrete, although there are a few extra considerations.

The main concern with concrete mix design foundation elements is that foundations are placed below ground and, therefore, are constantly exposed to groundwater and soil. This often facilitates chemical attacks on the concrete, as well as corrosion of the reinforcing steel. To protect against these durability issues, concrete mix used for foundation elements has low water-to-cement (w/c) ratio which helps to ensure the concrete is less porous. This typically results in the concrete being slightly stronger and stiffer than typical structural concrete.

Since drilled shafts extend to significant depths, the concrete has to be pumped or placed using a tremie. Concrete must also be designed such that it can easily flow around the reinforcing steel present within the drilled shaft. For such reasons, water-reducing admixtures are typically used to increase the slump and workability of the concrete.

The fine aggregate used in the concrete was river sand which met all ASTM C33 specifications for use in concrete. This sand had a specific gravity of 2.60 as determined by ASTM D854 and an absorption capacity of 0.96%. The fineness modulus was 2.93 as calculated based on the grain size distribution (Figure 3-16) determined following ASTM D6913. The coarse aggregate used in the concrete mix was the #8 crushed limestone, described in section 3.2.3.
Figure 3-16. Particle size distribution of fine aggregate used in concrete

Following the FHWA mix design recommendations along with ACI 211.1 mix design procedures, the concrete mix designed for the pile per cubic meter of concrete is summarized in Table 3-2. This results in a concrete with a w/c ratio equal to 0.42.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (kg)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>181</td>
<td>0.181</td>
</tr>
<tr>
<td>Cement</td>
<td>427</td>
<td>0.135</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>549</td>
<td>0.200</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>584</td>
<td>0.231</td>
</tr>
<tr>
<td>Air</td>
<td>0</td>
<td>0.023</td>
</tr>
</tbody>
</table>

To achieve the desired workability such that the concrete would flow into the form and around the circulation tubing, water-reducing admixture (Glenium 7710) was added at a ratio of 722 mL/m³. With this mix design, the fresh concrete had a measured slump of 140 mm and an air content of 2%.
Along with casting model piles into the forms illustrated in Figure 3-6, a series of cylinders were cast to determine various mechanical properties of the concrete. Two types of cylinders were cast: (1) concrete only and (2) concrete with embedded PVC tubing. This was to account for the loss of strength and stiffness of the pile when the tubing was embedded. Mechanical and thermal properties of the concrete are reported in Table 3-3. The embedded tubing resulted in a compressive strength reduction of 2.76 MPa (≈ 400 psi) on average.

<table>
<thead>
<tr>
<th>Table 3-3. Mechanical and thermal properties of concrete at 28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength without tubing in MPa (psi)</td>
</tr>
<tr>
<td>Compressive strength with tubing in MPa (psi)</td>
</tr>
<tr>
<td>Elastic modulus in MPa (psi)</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>Thermal conductivity (W/mK)</td>
</tr>
<tr>
<td>Specific Heat (J/Kg/C)</td>
</tr>
</tbody>
</table>

3.2.3. Crushed Limestone

In order to saturate the test bed, dry sand was first pluviated into the tank as detailed in 3.1.2 and then water was filled from the bottom of the tank. The tank had four spouts that allowed for water to be pumped into the tank using positive backpressure. To mitigate issues with piping or erosion of the test bed, a 7 cm layer of crushed limestone was placed at the base of the tank. The hydraulic conductivity of the limestone is an order of magnitude higher than that of the Ottawa sand, so it could be considered free draining. This allowed the water level to slowly rise through the tank, which is the preferred method of saturation, rather than filling the tank with water from the top. If water is introduced on the top surface of the soil, percolation effects will hamper the full saturation of the test bed since water will tend to travel through preferential pathways, thus leaving air pockets throughout the sand body (Iskander 2010). To ensure that the sand did not clog the crushed stone, thereby reducing its hydraulic conductivity, a permeable felt cloth was
placed between the sand and the crushed stone. While the cloth was water permeable, the typical aperture size was small enough to block sand particles from passing into the stone layer.

The material used as the stone layer at the base of the tank was a crushed limestone that met ASTM C33 criteria for #8 stone. This has a specific gravity of 2.80 following ASTM C127, $D_{50}$ equal to 6.8 mm and $C_u$ equal to 1.76 according to ASTM D6913 (Figure 3-17).

![Figure 3-17. Particle size distribution of #8 crushed stone used as a filter layer at the bottom of the sand bed](image)

3.3. Instrumentation and Data Acquisition

3.3.1. Thermocouples

Temperature was monitored at critical locations within soil and on pile to understand heat exchange within the pile-soil system. Type T thermocouples, composed of a pair of twisted wires: one copper and the other constantan (copper-nickel alloy), were selected for this research. The
selected thermocouples have a measureable temperature range of \(-200^\circ C\) to \(350^\circ C\) with an accuracy of \(\pm 0.5^\circ C\).

Thermocouples operate following the thermoelectric principle known as the Seebeck effect (Van Herwaarden and Sarro 1986). This physical phenomenon dictates that a thermal gradient between the two ends of a conductor will generate a measurable electric voltage. Thermocouples take advantage of this principle by using two wires of different metals that are twisted together on one end, which is referred to as the ‘hot end’. The other ends of the wires are left unpaired and this end is referred to as the ‘cold end’. The unpaired wires at the cold end are connected to a data acquisition (DAQ) system as a differential voltage input. For a given combination of metals, the measured differential voltage can be correlated to the temperature at the hot end.

The thermocouples were connected to a total of six NI 9213 modules, all of which were housed in a NI cDAQ 9178 chassis. Each NI 9213 module with built-in cold junction correction is capable of acquiring 16 differential voltage inputs at an aggregate data acquisition rate of 75 samples per second per channel (S/s/ch). Cold-junction correction is required when the cold end of the thermocouple is not at 0°C (standard ice-bath reference). In this case, the module has a built in temperature measurement at the cold end of the thermocouple to reference and correct the measured voltage differential. Temperature measurements were collected, displayed, and logged in real time at a rate of 0.1 Hz using data collection software written in LabVIEW 2011 (National Instruments 2013). The software development is further detailed in section 3.3.4.

For all thermal tests performed as part of this research a total of 94 thermocouples were placed within the sand bed, on pile surface, within the circulation tube and on tank boundaries. Layout of thermocouples is shown in Figure 3-18. The locations for temperature measurements were selected such that thermocouples were either on a plane passing through the pile and the circulation tube (hereafter referred to as XZ plane) or on a plane perpendicular to the plane
containing the circulation tube (hereafter referred to as YZ plane). With the selected layout, temperature data could be recorded on both the ‘warm side’ and the ‘cool side’ of the pile. Between these two sets of extreme temperature records, temperature could also be monitored and recorded at different points on the YZ plane. Thermocouples were also placed at 17 locations along the surface of the pile: eight thermocouples were placed along each side of the pile closest to the circulation tube (on the XZ plane), and one thermocouple was placed directly under the pile base. Two thermocouples were placed within the circulation fluid at points where the fluid entered and left the pile, respectively, referred hereafter as ‘inlet’ and ‘outlet’ points. Temperature readings at the inlet and outlet points enabled quantification of heat exchange through the pile. Six thermocouples were placed at the tank boundary and another six were placed just below the top of the sand bed.

![Figure 3-18 (a)](image-url)
3.3.2. Load Cell

Axial load on the pile was applied using a hydraulic cylinder (Enerpac RC-55) with a maximum capacity of 44.5 kN. The cylinder was operated using a manual jack fitted with a dial gage. To ensure accuracy of the load applied to the pile, a load cell (Omega LCM401-2.5K) was placed at the pile head. The load cell was placed directly between the steel pile helmet and the load cylinder. The load cell used in this experiment is a strain gage transducer based load cell. Essentially, the strain is measured within the load cell in the direction that the load is applied. The strain is then converted to load using a predetermined stress-strain curve for that particular strain gage. The load cell requires an excitation voltage of 10 VDC and has an output of 0-3 mV. To supply this excitation voltage, a process controller (Omega DP25B-E-A) was used. This controller is also able to provide signal amplification of the output signal from the load cell since the DAQ requires an input voltage within the range of 0-10 V. The load cell had to be configured
in a reference single end (RSE) mode since the voltage differential did not have a reference
ground and, therefore, acted as a floating signal. This is discussed in further detail in Section
3.3.4.

Once the signal was amplified, the output was passed to the data acquisition system. The
output leads from the load cell are connected to an NI 9205 module housed in the same NI
cDAQ-9178 chassis housing the thermocouple modules. This module is able to accommodate 16
channels with differential analog inputs, with 16-bit resolution, and a 250×10³ S/s aggregate
sampling rate. The data from the load cell was collected, logged, and displayed in real time at an
acquisition rate of 2 Hz through the data collection software written in LabVIEW 2011 (National
Instruments 2013) Section 3.3.4 of this thesis provides further details on this software
development.

3.3.3. Displacement Sensor

The displacement of the pile was measured at both the pile head, as well as the pile base. These
measurements were made using two linear variable differential transformers (LVDTs). As the
name infers, these instruments operate following transformer theory. An LVDT is composed of a
magnetic cylinder that passes through three coils: two secondary and a single primary coil
(Johnson 2006). The primary coil is located at the center of the three coil assembly, and is excited
with an AC voltage source. When the magnetic cylinder is initially centered in the neutral
position, the inductive voltage experienced in each of the secondary coils is equal in magnitude,
but opposite in charge, so when the differential is measured, it would read zero. As displacement
occurs, the magnetic cylinder moves through the coils, which causes the inductive voltages in the
secondary coils to become unbalanced. The differential voltage is linearly proportional to the
displacement of the magnetic cylinder.
The LVDTs used in this experiment were Omega LD621-100, which have a 10.6 cm travel length with greater than 0.2% linearity. The LVDTs required an excitation voltage of 24 Vdc and had an output voltage of 0-10 Vdc. To provide the excitation voltage, a variable DC power supply (GoldStar GP-4303D) was used. The differential output of the LVDTs was connected directly to the NI 9205 module, since no signal amplification was required. This is the same module as was used with the load cell, as described in 3.3.2.

To measure displacement at the pile head, the LVDT was fixed to the load cylinder, and the tip of the LVDT was placed directly on the pile cap. Measurement of the pile base settlement was performed using a ‘telltale’ system. This telltale system was composed of a 0.3175 cm steel rod that lies inside of a 0.476 cm PVC tube, both of which extend the full length of the pile. The purpose of the PVC tube is to separate the steel rod from the concrete so that there is no friction along the steel rod. This way, when displacement is measured at the top of the steel rod, only settlement at the pile base will cause a change in the LVDT reading.

The advantage of using a combination of these LVDTs is that more information can be determined about the behavior of the pile. The LVDT used with the telltale system will only read displacements due to settlement at the pile base. This is important because it is independent of the pile’s compression. The top LVDT will measure displacements due to both the base settlement as well as the pile compression, therefore this would reflect the total settlement experienced at the pile head. The combination of these two LVDTs helps decompose the total displacement into the base settlement component and the pile compression component.

3.3.4. Software

All of the measurements made throughout testing were recorded, displayed, and logged in real-time using custom software. As detailed in Section 3.3.1 temperature was measured at a total of
96 locations using type T thermocouples which were connected to a series of NI 9213 modules. Data logging software was developed in LabVIEW that included a graphical user interface (GUI) to display the data in real-time. The software was written using a producer-consumer architecture to allow data acquisition and data processing to occur in parallel. This allowed aggregate acquisition rates as fast as 75 samples per second per channel (S/s/ch), though typically samples were acquired at a rate of 2 S/min/ch due to the time scale of the tests. Intensity plots were displayed on the GUI to represent temperature along the XZ (Figure 3-18a) and YZ (Figure 3-18b) cross sectional planes of the tank (Figure 3-19). To provide a smooth thermal intensity plot, interpolation was required to reduce pixilation. Due to the limited number of measured temperatures, nearest neighbor linear approximations were required to generate a full temperature matrix from the sparsely populated matrix of measured temperatures.

Figure 3-19. Screenshot of GUI developed for real-time monitoring of temperature data
Displacement measurements recorded from the LVDTs were logged and displayed on the same GUI as measurements recorded from the load cell since the relationships between these values were of interest. As detailed in Sections 3.3.2 and 3.3.3, the load cell and both LVDTs were connected to an NI 9205 module. While the thermocouples and LVDTs could be wired to the data acquisition modules using raw voltage differentials, the load cell had to be connected in a referenced-single end (RSE) configuration. This is required for instruments that do not have common mode voltage rejection, and therefore do not have a ground voltage to subtract from the signal. For the load and displacement measurements, the GUI contains three graphs that display the data in real time: (1) axial load vs. time, (2) pile head and pile base displacements vs. time, and (3) pile head displacement vs. axial load (Figure 3-20).

Figure 3-20. Screenshot of GUI developed for real-time monitoring of load and displacement data
The acquisition rate used during load-displacement tests was typically 1 S/s/ch. In an effort to reduce noise, aggregate sample averaging was used during acquisition. This was configured such that a given data point was actually the average of 100 measurements made at a rate of 1000 Hz. Due to the very high acquisition rates required to perform this task, a producer-consumer architecture was utilized to separate data acquisition from display and logging.

The GUI was very beneficial while performing pile load tests for a couple of reasons. The load vs. time plot would ensure that the loading rate was consistent and the pile was provided adequate time to compensate for each load increment. Using the load vs. displacement plot it was obvious when plunging began so the operator was aware of when to terminate the pile load test.
Chapter 4
Results and Discussion

4.1. Heat Transfer Behavior of the Model Pile

A temperature-controlled water bath was used to circulate heat carrier fluid through the tubing embedded within the model pile. Throughout the thermal testing, temperature was measured at a number of locations within the soil and along the pile as detailed in Section 3.3.1. Since the sand deposit did not change between tests, except for the dry and saturated conditions, the thermal conductivity and specific heat capacity of the sand, as well as for the concrete pile, are assumed to be constant for all tests under either dry or saturated condition. This allowed other variables, such as circulation velocity and imposed thermal gradient, to be isolated and studied individually. Initially, the soil and pile were at room temperature which was kept constant at 19°C. However, there was a slight temperature gradient in the tank due to the variation in room temperature as seen in the initial temperature contour shown in Figure 4-1.

Figure 4-1. Initial temperature distribution within the soil tank for the thermal tests in dry sand
The air temperature near the surface of the soil bed was measured throughout the testing period. Over the 7 days in which the test was performed, the mean air temperature was 19.4°C with a standard deviation of 0.304°C (Figure 4-2).

The first set of thermal tests was performed in dry sand with a set of typical parameters. For the base case under dry condition, the circulation fluid entered the pile at temperature $T_{in} = 39^\circ C$, which imposed an initial temperature difference $\Delta \theta = 20^\circ C$ above ambient. The fluid was circulated at a flow rate of $1.34 \times 10^{-5} \text{ m}^3/\text{s} \approx 0.8 \text{ L/min}$ which corresponds to a linear velocity $v = 0.11 \text{ m/s}$. The flow rate and entrance temperature were maintained throughout the test. Temperature was recorded at all thermocouple locations at a rate of 2 samples per minute. The test was terminated once a change in temperature at the tank boundary was recorded. The
temperature distribution within the tank at different times during 7 days of thermal loading (heating) is shown in Figure 4-3.

Figure 4-3 (a)

Figure 4-3 (b)
Figure 4-3 (c)

Figure 4-3 (d)
Figure 4-3. Temperature contours at different times during thermal loading (heating) under dry condition with $\Delta \theta = 20^\circ$C and $v = 0.11$ m/s (a) $t = 0.25$ days (b) $t = 0.5$ days (c) $t = 1.0$ days (d) $t = 4$ days, and (e) $t = 7$ days

During thermal loading, most of the heat flow occurred in the radial direction (Figure 4-4). Below the pile’s embedment depth ($z = 120$ cm) the temperature change is much more mild. For an example, after 7 days of thermal loading, the temperature at a depth of $2B$ (= 20 cm; $B$ is the diameter of the pile) below the pile base increased only by 1.2$^\circ$C. However, temperature increment at a point located at a radial distance $2B$ (and at a depth of 0.6 m, i.e., approximately at the mid-depth of the pile) was recorded to be equal to 11.3$^\circ$C. The magnitude of temperature change was damped within a depth $z < 0.2$ m measured from the surface of the sand deposit. This indicates significant convective heat transfer from the surface of the sand deposit.
Figure 4-4. Time-dependent variation of soil temperature $T_g$ (under dry condition) recorded at different thermocouple locations along the depth

For both dry and saturated sand, heat flow was primarily in the radial direction with very limited temperature increase observed below the pile. As seen in Figure 4-5, the magnitude of the temperature increase at any time was greater for the dry condition compared to that measured under saturated condition. This phenomenon can be explained due to the difference in specific heat capacity between the dry and saturated sand. Specific heat capacity is defined as the amount of energy to unit increase in temperature of a unit mass of material. Therefore it takes a larger amount of energy rejected from the geothermal pile to observe an identical change in temperature in the saturated sand.
As heat dissipated from the pile into the surrounding soil a zone of thermal influence was evident within the soil; an increase in temperature was recorded within this zone and soil temperature remained constant (at initial temperature) beyond this zone. The transient thermal influence zone increased with time. Change in soil temperature $\Delta T_g$ (under dry condition) at different radial distances with both real and normalized time (expressed as Fourier number $Fo = \alpha t/r_i^2$; $r_i$ is circulation tube radius and $\alpha$ is thermal diffusivity of soil) is plotted in Figure 4-6. As the radial distance from the pile increased, it took an increasing amount of time to experience a change in temperature (Figure 4-6). For dry sand and for the given test parameters, a point that was at a distance $0.5B$ ($= 5$ cm) away from the pile felt an increase in temperature in only 15 minutes after the heat transfer started. Whereas, for the same test, a point at a distance $10B$ ($= 50$ cm) away from the pile center did not experience temperature increase until 24 hours after the
start of the test. As expected, with all other parameters identical, heating ($\Delta \theta = 20^\circ C$) and cooling ($\Delta \theta = -20^\circ C$) stimulated identical but opposite thermal response in the soil (Figure 4-6).

![Diagram](image)

**Figure 4-6.** Change in the temperature of dry sand at different radial distances from the pile during heat rejection (heating) and heat extraction (cooling) through the pile

As shown in Figure 4-7, at every radial distance the saturated sand experienced an increase in temperature sooner than the dry sand. This is because saturated sand has a higher thermal conductivity than that of dry sand (see Table 3-1). However, the magnitude of the temperature increase was much larger in dry sand than in saturated sand. This is because saturated sand has a higher specific heat capacity, so it takes more energy per unit temperature increase.
For the duration of any thermal test, the pile-soil system was in a transient heat flow condition. Initially the tank was at a nearly uniform temperature $T_{\text{initial}} = 19^\circ\text{C}$, as shown by the $t = 0$ days curve in Figure 4-8. By the end of 7 days, the temperature at the tank boundary did not increase, so the initial thermal boundary conditions were still valid until 7 days after the thermal loading started. If thermal loading was continued beyond 7 days, the zone of influence would eventually reach the tank boundary and the system would reach equilibrium when the radiative heat flow from the tank walls was equal to the heat transfer through the pile. However at this point, the initial boundary conditions would have been violated. As shown by Figure 4-8, soil on the inlet side of the pile was slightly warmer than the outlet side of the pile at all points in time.
Throughout the thermal tests, the fluid temperature was recorded at the inlet and outlet points of the circulation tube embedded within the model pile. The temperature difference $\Delta T_f$ between these two points would indicate the amount of energy the pile was able to dissipate into the soil. To study the effect of circulation flow rate on the efficiency of the model geothermal pile, three thermal load tests were performed under dry condition. In these tests, all parameters were identical to those listed for the base case except the velocity was set to 0.11 m/s (the base case), 0.33 m/s and 0.66 m/s. Figure 4-9 shows the variation of $\Delta T_f$ for different circulation velocity $v$. It was observed that $\Delta T_f$ decreases with an increase in $v$.

Energy output $E$ over a certain period of heat transfer through a geothermal pile can be expressed as:

$$E = m C_p \Delta T_f$$  \hspace{1cm} (5)
where $\dot{m}$ is the mass flow rate of the circulation fluid, $C_p$ is the specific heat capacity of the circulation fluid, and $\Delta T_f$ is the fluid temperature difference between the inlet and outlet points. The mass flow rate $\dot{m}$ can be determined by multiplying volumetric flow rate by the density of the circulation fluid. Results from thermal tests with difference circulation velocity (or flow rate) indicated that an increase in the flow rate of the circulation fluid increases energy output from a geothermal pile (Figure 4-10).

![Figure 4-9. Fluid temperature difference $\Delta T_f$ between inlet and outlet points of the circulation tube embedded within the model geothermal pile installed in dry sand](image-url)
In practice, cyclic thermal loads are expected due to short-term diurnal variations as well as long-term seasonal variations in air temperature. This means that the soil may be subjected to heat rejection during the day followed by heat extraction during the night (or heat rejection during summer months followed by heat extraction during winter). To study the effect of thermal cycles on heat transfer performance of the model pile, a series of thermal load tests were run in sequence under dry condition. Heat was extracted from the soil ($\Delta \theta = -20^\circ$C) for 7 days; immediately following the conclusion of the heat extraction phase, heat was injected into the soil ($\Delta \theta = 20^\circ$C) for 4 days. Due to the additional thermal storage available in the soil following the heat extraction phase, the system experienced approximately 30% increase in initial efficiency (Figure 4-11). Once the initial storage was depleted, the system approached similar steady state efficiency as would be expected for a single thermal loading phase.
Throughout the thermal loading tests, the displacements at the pile head and base were recorded. Initially, the pile was at a nearly uniform temperature of 19°C since it was at equilibrium with the surrounding soil. After maintaining a thermal loading with $\Delta \theta = 20^\circ$C, the pile expanded as the temperature of the concrete increased. The pile quickly reached a steady-state temperature and therefore the majority of the expansion occurred within the first 2 hours of thermal loading (Figure 4-12). Since the base displacement was measured using a telltale system, although the pile base expanded downward, the steel telltale expanded and thus the net recorded movement was upward. This was corrected by subtracting the expansion of the steel telltale using an assumed coefficient of thermal expansion of $1.3 \times 10^{-5}$ m/mK. The difference in head and base displacements shown in Figure 4-12 is due to the fact that the thermal expansion of the pile was constrained by the resistance offered by the surrounding soil. Such a constrained expansion of the pile would cause residual mechanical stresses, which has to be balanced by mobilization of shear.
stress along the length of the pile. Assuming that mechanical properties of sand remain unaltered within the range of thermal loading applied to geothermal piles, the shear stress mobilized during constrained thermal expansion (or contraction) of the pile may lead to a change in mechanical capacity for a geothermal pile installed in sand.

4.2. Axial Load-displacement Behavior of the Model Pile

To study the mechanical behavior of the geothermal pile, a series of pile load tests were performed. A hydraulic jack was used to apply an axial load to the precast nondisplacement model pile. All mechanical load tests reported in this thesis were performed under dry condition. Throughout the pile loading process, displacements at the pile head and base, as well as load at the pile head, were recorded at a rate of 1Hz (data acquisition is described in detail in section 3.2). Axial load tests were performed adhering to the standard specified by ASTM D1143. To
achieve uniform stress distribution on the pile head, a steel cap (helmet) was placed on top of the
pile. The load cell was placed on the pile helmet and the hydraulic cylinder was placed above the
load cell. The hydraulic jack was fixed with the steel reaction frame. The displacement of the pile
was measured from a datum that was unaffected by pile deflection and therefore could be
considered stationary. This implementation is in accordance with the specified ASTM standard.

For all of the mechanical load tests, axial load was applied in 0.1 kN increments; after
each increase in load the pile was allowed to reach a mechanical equilibrium with the applied
load. This was evident when pile head displacement reached a constant value for a given load
(Figure 4-13). For all load tests performed in this study, the loading rate varied from 0.065 to 0.15
kN/min.

![Figure 4-13. Typical load steps applied to the pile head during axial loading of the model pile](image)

All load tests were continued until a limit load was reached. The limit load can be defined
as the axial load at which the piles show plunging behavior, i.e., pile head settlement continues to
increase without any increase in axial load. As shown in Figure 4-14, head displacement occurs first while the base displacement is not present. This occurs until approximately 10 minutes which corresponds to an axial load at the pile head of 1.7 kN; this indicates that to this point all of the load applied to the pile has been carried by shaft resistance and no load has been transferred to the pile base yet. As the load continues to increase, base displacement begins. From this point, pile head and base displacement occur at approximately the same rate indicating that the pile is no longer compressing but rather settling as a rigid body.

![Figure 4-14. Typical record of pile head and base displacements during axial loading](image)

When the pile reaches limit capacity and begins plunging, there is a sharp increase in displacement rate. For the load test shown in Figure 4-14, the limit state was reached at 37 minutes, which corresponds to an axial (limit) load (at the pile head) equal to 3.2 kN. After sufficient displacement has occurred to confirm plunging, the pile was unloaded while displacement was recorded. Most of the displacement was plastic and therefore the pile did not
return to its original position (before loading). However, as shown in Figure 4-14, approximately 0.3 mm of head displacement was recovered while the pile base displacement remained unchanged indicating that most of the pile compression was elastic. This is confirmed from the linear elastic portion of the load-displacement curve at small displacements.

Apart from pile capacity, the displacement at the pile head under an applied load is a primary concern during pile design. By combining the results given in Figure 4-13 and Figure 4-14, a load-displacement curve was generated as shown in Figure 4-15. From this curve, the typical phases of pile load-displacement behavior are evident: (a) linear elastic displacement during small displacements, (b) nonlinear displacement, (c) plunging, and (d) elastic recovery of pile head settlement after unloading. In the absence of thermal loading, the remainder of the pile load tests exhibited load-displacement behavior similar to that portrayed in Figure 4-15. Mechanical loading was performed at the end of every thermal load test as well as once the pile and sand returned to initial ambient temperature condition. These tests are described in further detail in Section 4.3.
4.3. Comparative Load-Displacement Behavior

Pile load tests were performed after each thermal loading discussed in Section 4.1. The results of these pile load tests were then theoretically interpreted to quantify the mechanical characteristics of the pile. One of the main concerns of using piles as heat exchangers is that the thermal loading may have an effect on the ultimate and limit state capacities of these piles. To study such effect pile load tests were performed at ambient temperature conditions and after imposing thermal loading. During each of the load tests at elevated temperature, the circulation fluid entered the pile at approximately 39.4°C and thereby imposing an initial temperature difference $\Delta \theta = 20^\circ$C (as detailed in Section 4.1). The pile was allowed to expand freely at the pile head during thermal loading, and the mechanical pile load test was performed immediately after the completion of thermal loading. The load tests at ambient conditions were performed either before thermal
loading or after the pile-soil system returned to room temperature at the end of thermal loading. Results from several pile load tests performed as part of this research are summarized in Figure 4-16. The pile load tests are listed in the sequence in which they were performed. The letter (H, R or C) in the name of the test is the temperature at which the test was performed; the letter H (from hot) signifies test at elevated temperature ($\Delta \theta = 20^\circ C$), the letter R signifies tests at room temperature ($\Delta \theta = 0^\circ C$), and the letter C (from cold) is used to designate tests performed at temperature below the initial ambient temperature ($\Delta \theta = -20^\circ C$). The number in the name of a load test signifies the sequence of a load tests performed at a certain thermal condition (i.e., at elevated, ambient or reduced temperature).

![Figure 4-16. Load-displacement plots as obtained from several axial load tests on the model pile](image)

The pile load test results were interpreted using Chin’s extrapolation method (Chin 1970). This method idealizes the load displacement behavior of a pile through a hyperbolic relation:
where $Q$ is axial load, $w$ is settlement at pile head, and $C_1$, and $C_2$ are fitting parameters. Following Chin’s method, when the data from a pile load test is plotted with head displacement $w$ on the horizontal axis and the ratio $w/Q$ on the vertical axis, the results should be linear (Figure 4-17). In the $w$ versus $w/Q$ space, the load-displacement curves can be fit using a linear regression where the slope and vertical axis intercept will provide the values of coefficients $C_1$ and $C_2$, respectively. Physically, $C_1$ represents the inverse of the limit capacity $Q_L$, whereas $C_2$ is the inverse of the initial pile head stiffness $K_t$. Substituting coefficients $C_1$ and $C_2$ in Equation 6 with variables $Q_L$ and $K_t$, the hyperbolic load-displacement behavior of an axially loaded pile can be expressed as:

$$
\frac{w}{Q} = \frac{1}{Q_L} + \frac{1}{K_{t=0}}
$$

(7)

Figure 4-17. Results of pile load tests plotted following the Chin's extrapolation method
De Beer’s method (De Beer 1968) was used to determine the ultimate capacity of the piles based on the results of the load tests. For this method the results of the pile load tests are plotted on a load-displacement curve, though the axes should be in log-log scale (Figure 4-18). The ultimate capacity is the load that corresponds to the point of maximum curvature. Using Chin’s and De Beer’s methods, the initial pile head stiffness, load at ultimate state, and load at limit state were approximated for each of the pile load tests (Table 4-1).
Based on the axial load tests performed under different imposed thermal loading, it appears that load-displacement behavior of geothermal piles in sand depends on thermal loading history. Increase in limit capacity \( Q_L \) and decrease in initial head stiffness were observed for all load tests performed at elevated temperature, which suggests thermal loading has an effect on the mechanical capacity of geothermal piles. However, the effect of reduced temperature (cooling) could not be captured decisively as the pile capacity continued to increase with number of load tests (Figure 4-19). It is anticipated that the soil immediately below the pile base was continually densified during several mechanical load tests and this mechanical effect could not be differentiated from the effect of thermal loading alone. Without strain gages installed along the length of the pile, it was not possible to uncouple the effect of thermal loading on the mechanical behavior of the model pile. Nonetheless, results presented in this thesis suggest that the mechanical behavior of geothermal piles installed in sand is likely to be affected in the presence of thermal loading. This may particularly be important for the determination of ultimate pile capacity, which is often based on a certain level of pile head displacement.

### Table 4-1. Interpretation of pile load test results

<table>
<thead>
<tr>
<th>Temperature</th>
<th>( \Delta \theta = -20^\circ C )</th>
<th>( \Delta \theta = 0^\circ C )</th>
<th>( \Delta \theta = 20^\circ C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>C1</td>
<td>C2</td>
<td>R1</td>
</tr>
<tr>
<td>( ^{\text{a}}Q_L ) (kN)</td>
<td>3.60</td>
<td>3.75</td>
<td>3.20</td>
</tr>
<tr>
<td>Average</td>
<td>3.68</td>
<td>3.41</td>
<td>3.81</td>
</tr>
<tr>
<td>( ^{\text{b}}Q_L ) (kN)</td>
<td>4.19</td>
<td>4.23</td>
<td>3.57</td>
</tr>
<tr>
<td>Average</td>
<td>4.21</td>
<td>3.77</td>
<td>4.26</td>
</tr>
<tr>
<td>( ^{\text{c}}K_t ) (mm/kN)</td>
<td>12.11</td>
<td>14.99</td>
<td>22.10</td>
</tr>
<tr>
<td>Average</td>
<td>13.55</td>
<td>18.55</td>
<td>17.10</td>
</tr>
<tr>
<td>( ^{\text{d}}Q_u ) (kN)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.0</td>
</tr>
<tr>
<td>Average</td>
<td>3.50</td>
<td>3.08</td>
<td>3.58</td>
</tr>
<tr>
<td>( ^{\text{e}}Q_{1%} ) (kN)</td>
<td>3.23</td>
<td>3.48</td>
<td>2.89</td>
</tr>
<tr>
<td>Average</td>
<td>3.36</td>
<td>3.09</td>
<td>3.53</td>
</tr>
</tbody>
</table>

Note: \(^{\text{a}}\) Measured values; \(^{\text{b}}\) Values predicted using Chin’s method of extrapolation; \(^{\text{c}}\) Values predicted using De Beer’s method
Figure 4-19. Effect of mechanical loading on limit capacity of the pile
Chapter 5
Summary and Conclusions

5.1. Research Summary

Geothermal piles are proven to be an efficient and sustainable alternative to partially meet the heating and cooling energy demand of buildings. Though the number of geothermal pile installations continues to increase worldwide, there are still a number of technical concerns that need to be addressed. The heat transfer models used in the present practice to design and assess the thermal performance of geothermal piles are over-simplified and neglect important operational parameters. Similarly, there are geotechnical concerns pertaining to thermally induced mechanical stresses and additional pile settlement due to thermally-induced strains. These mechanisms must be properly understood to ensure safe and efficient design of geothermal piles.

The focus of this research was to study the mechanical, thermal, and thermo-mechanical behavior of a model geothermal pile through a controlled experiment program. An experimental setup was built as part of this research to investigate the performance of a model geothermal pile installed in sand. The laboratory-scale geothermal pile was instrumented to monitor its behavior during various combinations of thermal and mechanical loading. Temperature throughout the pile-soil system was measured at a number of points using thermocouples. To study mechanical behavior of the pile, axial load was measured at the pile head using a load cell, and pile head and base displacements were measured using a pair of LVDTs. These measurements were critical in studying the change in load-displacement behavior of the model pile under thermal and mechanical loading.
5.2. Conclusions

A number of observations were made based on the data recorded during a series of thermal load tests on the model geothermal pile. It was observed that heat flow through geothermal piles occurs primarily in the radial direction. The heat dissipation raises the temperature of the soil in the vicinity of the pile within a ‘thermal influence zone’ that expands with time. When designing geothermal piles, particularly if they are to be installed as part of a pile group, the thermal efficiency of these piles will reduce if the zones of thermal influence overlap with one another.

Results show that increasing the flow rate of the heat carrier fluid will increase the energy output of the pile. The geothermal piles have high initial energy outputs that approach steady-state values as the soil surrounding the pile begins to change temperature. The steady-state energy output values should be used for design because such values represent long-term thermal behavior. However, running heating and cooling cycles in series does provide an increased short-term efficiency.

The model geothermal pile was subjected to a series of mechanical load tests at ambient conditions as well as during thermal loading. During all mechanical load tests, plunging of the pile, that signifies typical limit state behavior, was observed. Based on interpretation of the pile load tests performed, it appears that both limit- and ultimate-state capacities of geothermal piles are likely to be affected by thermal loading; however, this effect could not be precisely quantified. During heat dissipation through the pile, increase in limit-state pile capacity was observed for all tests. This is in agreement with behavior observed in centrifuge tests on geothermal piles in unsaturated silt (McCartney and Rosenberg 2011). The opposite behavior was reported by Wang et al. (2012), who performed jack-up tests on miniature metal piles in dry sand before and after heating but did not account for mobilization of shear resistance along the pile shaft during the heating process. Assuming that mechanical property of sand did not alter within
the range of applied thermal loading, the change in pile load-displacement behavior before and after thermal loading can be attributed to partially-restrained pile movement during the heat exchange operation. Measurement of pile head and base displacements revealed that thermal expansion of the pile was very close to free thermal expansion. Low confinement and free end conditions acted together resulting in such pile movement during thermal loading. Moreover, the observed change in pile head stiffness has particular relevance with respect to serviceability limit states for geothermal piles.

5.3. Future Research

A number of variables, many of which directly apply to design parameters, are yet to be explored to better understand the behavior of geothermal pile systems. The heat exchange through the pile-soil system is dependent on a number of material properties, namely the thermal conductivity and heat capacity of soil and pile material. The thermal conductivity of soil is known to be affected by density and, therefore, it will be of practical interest to explore if such a change would affect thermal performance of geothermal piles installed in soils with varying density. It is also useful to study the thermal performance of geothermal piles installed in stratified soil deposits with multiple layers with different thermal properties.

The thermo-mechanical behavior of geothermal piles is quite complicated and should be studied in further detail. In particular, thermal loads should be imposed under a variety of fixities at the ends of the pile. As was observed during thermal loading, temperature of a geothermal pile will approach to that of the heat carrier fluid and will therefore tend to expand or contract accordingly. If the pile movement is restricted by the superstructure load and by the resistance offered by the surrounding soil, the tendency of thermal expansion will induce additional mechanical stresses that will compound with the mechanical stresses already present due to the
superstructure load. The installation of strain gages along the length of the pile would certainly facilitate understanding of this behavior. The soil resistances mobilized along the pile shaft and that at the pile base can be uncoupled using strain measurements along the pile and thus, any change in either shaft or base resistance after thermal loading can be quantified. Moreover, application of different levels of surcharge at the soil surface would allow reproduction of stress similitude that are present at different depths along a real pile.
References


U.S. Department of Energy (DOE 2010), “U.S. residential and commercial buildings total primary energy consumption.” *Building Energy Data Book*. Table 1.1.1.


