GEOTECHNICAL DESIGN STUDY
TUNDRA WOMEN'S COALITION
BETHEL, ALASKA

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GEOTECHNICAL DESIGN STUDY
TUNDRA WOMEN'S COALITION
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1.0 INTRODUCTION

This report presents the results of our geotechnical design study for the proposed replacement of the Tundra Women's Coalition facility in Bethel, Alaska. The purpose of our studies was to develop foundation design and construction recommendations for a passively cooled deep-foundation system based on the results of previous geotechnical investigations conducted by our firm in 2007, which presented foundation concepts.

2.0 PROJECT UNDERSTANDING AND SITE CONDITIONS

The project is on an approximately 0.6-acre parcel near the intersection of Sixth and Willow Streets (Lots 18, 19 and 20, Block 2, U.S. Survey 3770, Northwest Addition to the Townsite of Bethel). The site generally slopes gently to the north and west except for the northwest corner, which slopes more steeply to the north and west. The existing Tundra Women's facility is on the eastern portion of the site; two small structures on the western portion of the site at the time of our explorations were to be demolished or moved (Figure 1). Roughly 1/3 of the site is vegetated with brush consisting primarily of willows. During our initial investigation, we observed construction debris, drums, and automotive parts scattered across the site.

The Tundra Women's Coalition Project originated with a concept to upgrade and expand the existing facility. Due to numerous deficiencies in the existing structure, a new facility is planned. The existing facility is founded on a post-and-pad foundation system with an open air space of approximately 1 foot beneath the structure. The floor reportedly slopes in many different directions throughout the structure; numerous cracks have occurred in the walls, suggesting differential settlement of the structure due to permafrost degradation, possibly aggravated by frost heaving.

The new two-story, 120-foot by 80-foot structure is planned on the western half of the site. We assume the structure will be continuously heated and connected to city utilities.

3.0 SUBSURFACE EXPLORATIONS

3.1 Subsurface Conditions

We conducted a geotechnical exploration of the site in mid-May 2007. Three borings were advanced to depths ranging from 41.5 feet to 50.5 feet. Logs of the borings are presented in Appendix A. The general subsurface conditions encountered during exploration consisted of a 2.5-foot to 4.5-foot thick layer of silty, sand fill overlying up to 2.5 feet of peat underlain by silty sand and sandy silt to the depths explored. We noted zones with thin peat seams in the natural soils throughout the depths explored and observed seasonal frost to depths of 5 feet to 7 feet. Ice-rich potentially thaw-unstable permafrost was encountered below depths of 12 feet and 16 feet. The most notable ice was a layer of nearly pure ice between depths of 33.5 feet and 38 feet in boring B-4.

We placed 1-inch Schedule 40 PVC in boreholes B-3 and B-4 and backfilled the holes with drill cuttings to allow the measurement of ground temperatures using a thermistor string. Initial temperature measurements made 24 to 48 hours after drilling were likely disturbed; therefore, we took another set of ground temperature measurements on April 29, 2008, nearly one year later. The new temperature profiles are shown in Figure 2.

The new measurements confirm the presence of permafrost and an unfrozen zone between the base of the seasonal frost and the top of the permafrost, as logged in the exploratory borings. The measurements show the permafrost is relatively warm, with temperatures ranging from near 32 °F at the top and cooling to around 31.3 °F at a depth of 50 feet.

4.0 CONCLUSIONS

Exploratory drilling of the site shows an area underlain with frost-susceptible soils and relatively warm thaw-unstable permafrost within the depths explored. We anticipate the permafrost at the site may continue to slowly degrade, resulting in potential foundation settlement in the future; although future climate change, ground warming, and rate of permafrost degradation are uncertain and difficult to predict. Foundation options that control permafrost warming and degradation and the risk of long-term settlement include:

1. Shallow footings or a thickened-slab bearing on an insulated pad passively refrigerated with flat-loop thermosyphon devices;

2. A hybrid system of driven H piles bearing in the underlying permafrost fitted with passively refrigerated vertical thermosyphon devices in the flange; and/or

3. Passively refrigerated, slurry-backfilled pipe piles bearing in permafrost.

Option 1 is not suitable for the site, because frost-heaving can occur as the talik beneath the pad refreezes in response to the refrigeration in the pad. We anticipate more than one freezing season would likely be required to refreeze the talik soils, unless additional cooling capacity (either passive or active) is added.

In our opinion, either of Option 2 or 3 would be suitable for the site. The most desirable system would be a function of the allowable capacity, fabrication schedule, shipping, and construction costs. For this study, we prepared recommendations for passively refrigerated, slurry-backfilled pipe piles.

We also considered and prepared recommendations for an alternative, nonrefrigerated driven-pile foundation system bearing in the permafrost, although there is more risk of unacceptable foundation movement with this type of system over the life of the structure due to uncertainty about future climate and permafrost conditions. Non-refrigerated driven-pile foundations have been used in Bethel with mixed success; problems with non-refrigerated piles have included frost-jacking, excessive creep-settlement (particularly on south-facing sides of structures), and settlement of the ground beneath structures. Passively cooled foundation systems would reduce the risk of these problems occurring.

5.0 THERMAL ANALYSES

To assist us in the design of a passively cooled pile foundation system, we performed two-dimensional thermal simulations, using Temp/W, a finite element simulation package developed and sold by Geoslope International Inc., Calgary, Alberta. The simulation program was written to solve two-dimensional, nonsteady-state, heat-transfer problems with phase change. An apparent heat capacity method is used to treat latent heat, which requires unfrozen moisture-content curves for materials undergoing phase change. Iteration is performed at every time-step to improve accuracy in establishing the phase-change boundary. An axisymetric analysis was

used to simulate a three-dimensional space with symmetry about a vertical axis of rotation at the location of the refrigerated pile.

The simulations were conducted for passively refrigerated 12-inch steel pipe piles (Thermo-Piles), such as those manufactured by Arctic Foundations, Inc. in Anchorage. We used the simulation to estimate the amount of cooling, insulation requirements, and long-term permafrost temperatures used in developing pile capacities. The cooling capacity of a passively cooled pile is a function of the temperature differential between the outside air and the ground around the buried portion of the pile and the conductance or performance of the pile.

In principal, the passively refrigerated pile consists of a pressure vessel filled with a two-phase fluid (liquid/gas). The aboveground portion of the vessel acts as a condenser section. Energy is transferred from the buried portion of the unit (evaporator section), where relatively warm soil causes pooled, liquid-phase fluid to evaporate and rise as a gas to the condenser section exposed to cold outside air, where the gas-phase fluid then condenses and returns to the evaporator section. Insulation may be used to reduce the transfer of heat from the surface to subgrade soils. The thermosyphons function during periods when outside air temperatures are lower than ground temperatures, typically during the freezing season.

5.1 Simulation Description

Thermal simulations were performed in two areas of the foundation: one in the interior of the structure, and one near the outside edge of the structure. Both areas were represented with a rectangular, finite-element mesh. In the interior of the structure, the mesh was formed having a width of one-half of the anticipated pile-group spacing, assumed to be approximately 16 feet (Figures 8 and 9). The mesh representing an outside pile group had a width of 50 feet (Figures 10 and 11). The bottom boundary in both meshes was set at a depth of 70 feet below the ground surface (bgs). The vertical sides of the meshes were no-heat-flow boundaries, except along the left edge, where Thermo-Pile boundary conditions were specified over the depth of the pile. Air temperatures used to drive the Thermo-Pile were developed from air-temperature histories for Bethel. Ground-surface temperatures specified at the top of the mesh were developed by modifying air-temperature histories for site-surface conditions in Bethel.

The boundary condition at the base of both meshes was set to a constant temperature of 31.5 °F, consistent with data collected from the thermistor strings.

5.2 Ground Surface Temperatures

Air and ground-surface temperatures used in the models were developed from daily maximum and minimum temperatures for the Bethel area provided by the Alaska Climate Research Center (ACRC) for the period 1923 to 2008. The record is a compilation of temperatures from the first-order weather station at the Bethel airport, BETHEL AP (PABE).

From the mean daily temperatures, we calculated annual air freezing (AFI) and thawing indices (ATI), plotted in Figures 3 through 6. Design air-freezing and thawing indices were selected, from which air and ground surface temperatures were generated for use in the simulation (Figure 7).

The temperature record shows a relatively abrupt warming or decrease in the mean AFI since 1977, though it is uncertain whether the warming is the result of climate change or variability due to large-scale oscillations in atmospheric circulation patterns. Currently, the increase is being attributed to the Pacific Decadal Oscillation (PDO), a long-lived El Niño-like pattern of Pacific climate variability.

A linear regression through the whole record shows a decreasing trend AFI at an annual rate of approximately 5 ºF-days per year; however, since 1977, an increasing trend is indicated in a linear regression of the data.

The record shows an overall increasing trend in the ATI of approximately 3 ºF-days per year based on a linear regression; however, since 1977 the increase has been about 16 ºF-days per year.

Given the uncertainty in whether long-term climate change is indicated by the temperature record, we selected the mean AFI and ATI for the period of warmer winters from 1977-2008 to develop mean daily air and ground-surface temperatures in the model. We used ann AFI of 1 standard deviation below its mean and the mean ATI to develop mean daily design-air temperatures used to drive the Thermo-Piles. These indices are summarized in the following table:

SUMMARY OF AFI AND ATI (1977-2008)

Parameter	AFI	ATI
Mean	3339	2931
Standard Deviation	564	300
Mean -1σ (design value)	2775	---

The selected design indices were entered into the Alaska Department of Transportation & Public Facilities (ADOT&PF) Berg2 computer program to generate the following sinusoidal relationship for daily air temperatures used to drive the Thermo-Piles:

$$T_{Condensor} = 32.4 - 24.6\cos\left(\frac{2\pi t}{365}\right) \text{ where "t" is in days}$$

Similarly, the mean AFI and ATI were used to generate the following sinusoidal relationship for mean daily air temperatures from which ground surface temperatures were developed:

$$T_{Mean_Air} = 30.9 - 27.0\cos\left(\frac{2\pi t}{365}\right) \text{ where "t" is in days}$$

The mean air temperature function was then modified for various surface conditions in Temp/W using n-factors for freezing and thawing (n_f and n_t) to describe the surface-boundary conditions in the model. The following n-factors, based on published values, were selected to represent the two surfaces considered in the simulations:

SUMMARY OF N FACTORS

Surface	n-freezing	n-thawing
Ground surface outside of building footprint	0.3	1.6
Ground surface beneath the structure kept free of snow	0.9	0.9

The ground surface beneath the building is expected to be cooler than outside the building footprint due to shading in the summer and a relatively snow-free surface in the winter. The

ground surface outside the building footprint is assumed to be surfaced with gravel and covered with snow in the winter.

5.3 Thermo-Pile Performance

Heat transfer through a Thermo-Pile from the evaporator section (the face of the pipe in contact with the ground) to the aboveground section is described by the following relationship:

$$Q = C_{hp}(T_{evap} - T_{air}) \text{ and } C_{hp}^{*} = \frac{C_{hp}}{L_{evap}}$$

where Q is the heat flow rate, BTU/hr
C_{hp} is the total heat pipe conductance, BTU/hr-^{o}F
C_{hp}^{*} is the unit length conductance, BTU/hr-ft-^{o}F
T_{evap} is the evaporator surface temperature,
T_{air} is the ambient air temperature
L_{evap} is the evaporator or embedded length of heat pipe

The following relationship (Johnson, 1971) was used to describe the thermosyphon conductance in Btu/hr-°F:

$$\text{Aeh} = .69 + 1.23\text{V}.42$$

where A is the area of the vessel exposed to the air, ft^2
e is the efficiency factor ($e = 1.0$ for unfinned pipe)
h is the surface heat transfer coefficient BTU/hr-ft^2
V is the mean wind speed in miles per hour

We assume CO_2-filled Thermo-Piles manufactured by Arctic Foundations, Inc., will be used in construction.

Based on the wind-speed data from the Western Regional Climate Center (WRCC), shown below, we selected a design wind speed of 8 miles per hour (mph) for use in the simulation.

SUMMARY OF AFI AND ATI (1977-2008)

Month	Mean Wind Speed (mph)	Prevailing Direction
September	11.7	SSW
October	12.6	NNE
November	13.5	NE
December	14.2	NNE
January	14.6	NE
February	15.2	NNE
March	14	NNE
April	12.9	NW
May	11.6	S

5.4 Material Properties, Initial Conditions, and Analyses

We developed a generalized soil profile based on our exploration of the site; material thermal properties were developed based on soil type and laboratory testing. The generalized soil profile used in the simulation is as follows:

GENERALIZED SOIL PROFILE

Depth (feet)	Material
0 to 4	Silty Sand Fill
>4	Natural Silty Sand

Material properties selected for use in the simulations are presented below. Thermal conductivities for soils were determined by a method after Johansen (1975).

SUMMARY OF MATERIAL PROPERTIES

Material	Dry Unit Weight (lb/ft^3)	Moisture Content (%)	Thermal Conductivity (unfrozen) (Btu/h-ft-°F)	Thermal Conductivity (frozen) (Btu/h-ft-°F)	Heat Capacity (unfrozen) (Btu/ft^3-°F)	Heat Capacity (frozen) (Btu/ft^3-°F)
Silty Sand Fill	100	20	1.00	1.49	37.0	27.0
Natural Silty Sand	96	25	1.01	1.79	40.3	28.3
Insulation	-	0	0.02	0.02	0.65	0.65

The phase change and change in thermal conductivity were spread over a temperature range of 0.2 °F, from 31.8 °F to 32 °F.

For calibration of the model, we selected an initial temperature profile and surface-boundary conditions that would simulate a slowly degrading warm permafrost profile similar to the one observed in the exploratory borings. The following initial temperature profile was specified:

INTIAL TEMPERATURE PROFILE

Depth (ft)	Temperature (°F)
0-5	40
5-10	35
10 to 16	33
>16	31.5

Using the boundary conditions, material properties, and initial conditions discussed above, we conducted simulations using an axisymmetric analysis with 42-foot-deep Thermo-Piles. The simulations were run for a period of 10 years, using a one-day time-step.

The axisymmetric analysis lends itself to the analysis of a single Thermo-Pile that has symmetry about a vertical axis of rotation with the heat pipe along the axis. Effectively, a two-dimensional problem is rotated about vertical central axis 2π radians.

The exterior pile was simulated with 4 inches of insulation at the ground surface extending out horizontally 10 feet from the pile. An interior pile was simulated without insulation at the surface.

Example results of the simulations are shown in Figures 8 through 11. Over the 10-year simulation period, the Thermo-Piles maintain permafrost beneath the structure and insulation, but the thaw-front slowly advances outside the limits of the foundation system.

The simulations show cooler permafrost temperatures are maintained at the interior piles than at the exterior piles. Based on the simulations, we selected warm-season soil-design temperatures of 31 °F for interior piles and 31.5 °F for exterior piles.

We note that simulation results should be used as a guide, since there are uncertainties in our analyses. Additionally, there are uncertainties about future climatic conditions at this site. As we stated, the climate data used in our analysis includes a noticeable increase in the mean annual air temperature across the State since 1977. There is great uncertainty over whether temperature trends will continue to increase over the life of the structure, remain at post-1977 levels, or cool; for the purposes of our analyses, we have assumed they will remain at post-1977 levels.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Discussion and Conclusions

Based on the results of our field and laboratory studies, we recommend founding the new Tundra Women's Coalition facility on steel Thermo Helix-Piles manufactured by Arctic Foundations, Inc., in Anchorage. The Thermo Helix-Piles should be placed in preaugered borings and backfilled with freshwater sand slurry. The passively refrigerated piling should be capable of maintaining mean ground temperatures, within the permafrost below a depth of 16 feet, at or below the design soil temperature during the summer months. Based on the results of the thermal simulations, the design permafrost temperatures assumed for exterior pile groups is 31.5 °F; design permafrost temperatures assumed for interior pile groups is 31 °F.

The following recommendations for support of the proposed structure assume the conditions observed in the exploratory borings are representative of the subsurface conditions throughout the site and within and below the embedment depths of the foundations. The presence of deep thawed zones, massive ice, or other anomalies may impact the performance of the foundation

system. For these reasons, it is important we be retained to monitor the drilling, installation, and backfilling of the piles; this will allow us, if necessary, to modify our foundation recommendations.

6.2 Pile Load Capacities and Movement

The foundation design discussed below presents a synthesis of soil and ground temperature data provided from the exploration program, along with available data on adfreeze and creep-deformation in frozen soil. The pile-design calculations assume the warm end of the summer design temperatures will exist near the piles for a period of six months. For the remainder of the year, the soil temperatures near the pile are assumed to be colder due to the operation of the passive-refrigeration devices on the selected exterior piles.

Recommended sustained design loads for 12-inch passively refrigerated piles are presented in Figures 12 and 13. The design curves presented in these figures are only valid for piles installed as discussed in the following sections. The design capacities presented have a factor of safety of 2.25 applied to the creep rate, and a factor of safety of greater than 3 applied to adfreeze. We recommend a minimum embedment of 30 feet below original grade. Due to the limited depth of the exploratory borings, we do not recommend piles deeper than 50 feet below grade. We performed our thermal simulations assuming a pile-embedment depth of 42 feet. For the proposed structure, an allowable long-term settlement of 3 inches in 20 years (in addition to initial construction settlements of less than ½ inch) was used in the calculation.

The sustained loads presented are defined as loads equal to the dead load plus that portion of the live load expected to act on the structure for 50 percent of a year or longer. Support of the sustained loads is based primarily on the creep deformation of ice-rich frozen soil, and less on the adfreeze strengths of the pile/soil interface. For the piles with protuberances, such as the recommended helix piles, the loads are based on creep deformation. In our opinion, this limiting analysis is applicable if the piling has rings or protuberances.

Short-term loads (for this study) are defined as the maximum live plus dead loads expected to act on the structure for short duration periods, generally less than 24 hours. The short-term loads consider the adfreeze strengths of the pile/soil interface and the shear-failure capacity of the soil slurry, soil slurry/*in situ* soil interface, and *in situ* soil, as well as the creep deformation of ice-rich frozen soil. For piles designed and installed as recommended, the maximum allowable,

short-term design loads, as defined above, are 1.5 times the allowable sustained loads, as presented in Figures 12 and 13, for both uplift and bearing.

As a precaution against loss of bond strength at the pile soil interface, the steel piling should be designed with protuberances (rings, fins, or plates) along that portion of the pile below a depth of 8 feet). The use of rings on a pile reduces the effect of loss of adfreeze bond between pile surface and soil by transferring the load through the rings, and developing load-carrying capacity dependent on the shear strength and creep characteristics of the slurry backfill and natural soils. We have assumed 2-inch rings will be welded to the piles, and the rings will extend from 6 feet bgs to the bottom of the piles. For the piles, we recommend helical rings be used with a pitch ranging from 12 inches to 15 inches. It has been our experience that helical rings are less likely to bridge during slurry backfilling, creating voids. Horizontally welded rings can trap voids beneath them during slurry backfilling, resulting in decreased pile capacities.

The sustained loads presented assume clean, nonsaline slurry sand and gravel are used and placed as recommended, thereby forcing creep deformation to occur within the *in situ* soils at the slurry/soil *in situ* boundary.

Prior to installation, the piling below the seasonal thaw penetration depth should be free from scale, rust, paint, or any other surface treatment. The maximum thickness of the active layer for the site is estimated to be approximately 8 feet, if the temperature is maintained at the ambient air temperature, a natural ground surface vegetative layer is maintained, and insulation is not used. The use of passively refrigerated piles with, and without, insulation is anticipated to reduce the thaw depth, lower the temperature around the pile significantly during operation, and reduce the period when significant creep is anticipated to occur.

If localized high-bearing loads are present for this Tundra Women's Coalition, we anticipate groups of piles will be required to support the loads imposed. The recommended spacing for pile groups is discussed in Section 6.5 below.

6.3 Pile Loading

Pile loading should not begin until all slurry backfill has been frozen back and cooled to design temperatures. The design permafrost temperatures assumed for exterior pile groups is 31.5 °F below a depth of 16 feet; the design permafrost temperatures assumed for interior pile groups is

31 °F below a depth of 16 feet. We recommend actual temperatures at the pile surface be monitored by temperature measuring devices (thermistors) as discussed in Section 6.6.

If the piles are installed in summer or late spring, the natural freezeback may not remove enough heat to freeze back and maintain the slurry and surrounding soil temperatures below or at the design temperature throughout the summer. Under these circumstances, active mechanical cooling may be required to obtain freezeback.

6.4 Pile Fabrication and Construction

We recommend all steel piles exposed to solar radiation be painted white in the exposed sections, to change their emissivity and reflectivity characteristics so heat conduction along the pile into the ground is minimized. A white vinyl or fusion-bonded epoxy coating, approximately 4 mils thick, is recommended for this purpose. Under no circumstances should piles be coated in the load-bearing sections of the pile.

With a passively refrigerated pile foundation design, we recommend a minimum 4-foot free airspace be maintained between the bottom of the floor and the final grade to minimize the possible exchange of heat from the building to the underlying ground, and to provide a space for the operation of pile-condenser sections or heat exchangers when passive refrigeration devices are used. The condenser (i.e., pressure vessel) section of the pile should stick up a minimum of 3 feet above grade. Open latticework or screening is typically used to minimize the ventilation space from being used as a storage area; however, the screening or lattice could aggravate snow drifting beneath the structure. The ground around the structure should be graded to allow free air flow beneath structure and prevent drifting of snow.

The free airspace would also provide room for active-refrigeration piping or additional passive-refrigeration condenser sections if required. All piping, utilities, or structures placed within the ventilation space should be designed and located to minimize snow drifting and/or airflow disturbance below the structure.

Passively refrigerated piles must be welded to pressure-vessel standards.

6.5 Pile Installation

The diameter of the preaugered hole should be at least 6 inches greater than the maximum diameter of the rings or the piles. If pilings are installed during the winter, it is recommended the piling be free of ice or snow prior to installation.

The annular space between the soil and the pile should be backfilled with machine-mixed sand and gravel slurry, with a recommended salinity of less than 1 ppt. The slurry sand and gravel backfill should be a clean, well-graded mixture of sand and gravel with a maximum particle size of less than 1 inch. It should have sufficiently high water content to freely flow around the pile (about the consistency of 6-inch slump concrete). During placement, a puddle of water should form on the surface of the slurry. The water content of the slurry should be adjusted to maintain the depth of this puddle of water at about 1 inch to 4 inches. The water used for mixing should be fresh and no warmer than 40 °F. We recommend the slurry be placed at a rate of approximately 2 to 4 cubic feet per minute and carefully vibrated with a concrete vibrator to prevent bridging or trapping of water and to displace air voids. A schematic of the recommended pile installation is shown in Figure 14.

The minimum spacing between piles placed by the dry-auger slurry-backfilled method should be such that the resulting thermal disturbance will not thaw the soil or the slurry between piles, or disturb the position or integrity of previously installed piles. We recommend a minimum spacing of 3 feet from side of preaugered hole to side of preaugered hole. Based on our experience in the Bethel area, the thawed soils above the permafrost will likely not stand open. The contractor should plan to set surface casing for each pile installation, which should extend down into the top of the permafrost to form a seal and prevent groundwater from entering the preaugered hole.

We recommend slurry backfilled piles be installed in early winter to allow time for the pile to freeze back, cool, and stabilize the *in situ* soil temperatures. If a late winter or spring installation is performed, passive cooling may be insufficient to refreeze and cool the subgrade soils, and active refrigeration may be required.

We strongly recommend a geotechnical engineer from our firm be retained to continuously observe all piling operations. Potential contractors should be made aware that the embedment depth or the number of piles required might change, depending on observed field conditions. The recommended allowable loads presented later in this report assume bonded permafrost is

encountered throughout the section. If nonbonded or large massive ice zones are observed in the borings, the pile capacities will be reduced. We recommend the depth of embedment of the pile be increased to a distance equal to the thickness of the undesirable zones in order for the piles to achieve the full design loading.

As we noted above, potential contractors should be aware that surface casing will likely be required to seal off near-surface groundwater flow and caving soils.

6.6 Instrumentation and Pile Performance Monitoring

We recommend 1-inch PVC casing be installed at a minimum of three locations in the footprint of the structure: one near the center of a pile group, the remaining near a pile around the perimeter of a pile group. We recommend, at a minimum, thermistors be installed approximately 2 feet above the bottom of the pile, near the middle of the pile, 3 feet below the original ground surface, and 3 feet above the ground surface.

We recommend instrumentation be installed and designed so as to be protected from disturbance, weathering, and vandalism. In addition, it would be convenient if the instrumentation leads from the piles could be grouped at locations where they can be accessed and read on a regular schedule. The pile temperatures should be measured in January, April, July, and October for at least the first two years of structure operation, with readings taken every October and January thereafter. The most critical time to monitor the temperatures is in late summer and early fall, when the soils can be expected to be at their warmest.

In addition, the functioning of the exposed Thermo-Pile condenser sections can be readily checked with handheld infrared-heat detectors. These devices can be used to quickly survey the operation of Thermo-Piles or thermosyphon devices. An operating Thermo-Pile or thermosyphon condenser section will show up as a warm object with the infrared devices. If improper condenser operation is detected, pile charge pressure could be checked, and the pile could be recharged or repaired as required.

Throughout the life of the proposed structure, visual inspections of the open crawl-space area should be conducted periodically, especially after wind or snow events. Any snow blocking the clear passage of air around the exposed portions of the piles should be removed.

6.7 Construction Schedule

Installation of the Thermo Helix-Piles should be done in the late fall or early winter after the ground surface has been frozen, to allow for vehicle traffic. Installation of the piles during this time of year should reduce the potential requirement for active refrigeration, unless the freezeback time needs to be accelerated in order to meet the construction schedule. Many factors will influence the rate of freezeback, including: ambient air temperature, diameter of boring, exposed condenser area, initial ground temperature, and wind speed. We recommend the construction schedule contain some flexibility to allow for weather-related slurry freezeback delays. The instrumentation recommended in Section 6.6 can provide information on slurry freezeback rates.

If the piles are installed in summer or late spring, the natural freezeback may not remove enough heat to freeze back and maintain the slurry and surrounding soil temperatures below or at the design temperature throughout the summer. Under these circumstances, active mechanical cooling may be required to obtain freezeback.

6.8 Environmental Concerns and Assumptions

The recommendations above are based on historical environmental conditions in the area, such as wind speed, air and permafrost temperature, and soil and ice conditions. If environmental conditions, such as average annual air temperature, increase over the life of the structure due to global warming, or over a period of several years during the life of the structure, or if snow drifting is or becomes a problem, the performance of the passive refrigeration devices could be adversely affected. The foundation recommendations presented should be reevaluated in light of changed conditions. Due to the potential for varying or changing thermal conditions, we recommend a pile performance-monitoring program be used throughout the life of the structure. Details of the recommended monitoring program were discussed in Section 6.6 above. We also recommend incorporating releveling capability in the foundation system to contend with the uncertainty in future climate and performance.

6.9 Excavation and Site Preparation

The site should be prepared by constructing a pad across the building footprint that extends at least 12 feet out beyond the edge of the footprint. The surface of the pad should be sloped to drain, preventing the ponding of water beneath the structure. The perimeter of the pad should be

insulated with a minimum of 4 inches of insulation suitable for direct burial, overlain by a minimum of 2 feet of cover material described in this report. The insulation should extend out horizontally 12 feet beyond the edge of the building and 8 feet beneath the building. Along the north and west sides of the building, the width of the insulation extending out horizontally beyond the edge building can be reduced to 8 feet if necessary to avoid crossing the property line. We recommend a minimum of 1 foot of cover over the outside edge of the insulation.

Begin site preparation by clearing and grubbing the existing surface. Before placement of fill and insulation, the base of the excavation should be proof-rolled and compacted to achieve a density suitable for the placement of fill. If the subgrade is not compacted adequately, it will be difficult for the contractor to achieve the required density with the first lift of fill. Any soft soils exposed in the base of the excavation should be excavated and replaced with compacted classified fill. Proof-rolling of the *in situ* soils will allow the observing geotechnical engineer to identify soft spots, which may be present near the base of the excavation.

Following compaction of the base, the excavation should be backfilled with classified fill as necessary, according to the recommendations given in this report. We recommend placing a geotextile separator between subgrade soils and the pad fill.

A 6-inch bedding layer should be provided for the insulation. The bedding should be placed and compacted to at least 90 percent of the Modified Proctor moisture-density relationship (American Society for Testing & Materials [ASTM] D1557). The insulation should be placed as described in Section 6.13 and covered with a minimum of 2 feet of classified material. The thickness of the cover should be increased as necessary to prevent damage to the insulation by construction loads. The cover should extend out beyond the edge of the insulation a minimum of 2 feet.

Existing fill excavated from the site that meet the gradation requirements for classified material can be reused as fill, provided it is placed and compacted in controlled lifts as described under the requirements for classified material.

All excavations should be sufficiently sloped or shored to provide a stable bank. We recommend the stability of the excavated slopes be made the responsibility of the contractor, as they will be most familiar with the conditions encountered in the excavations and have direct control of working conditions in the field. The work should be accomplished in general accordance with

the applicable local, state, and federal standards. For planning purposes, we recommend you assume unsupported excavation slopes will be no steeper than 1 vertical to 1 horizontal. It is also important to note that temporary excavation slopes may initially stand steep, but slough and cave as they dry out, particularly when equipment is operated nearby. Similarly, steep cuts made in seasonally frozen ground can become unstable upon thawing.

6.10 Play Deck Post and Pad Foundations

We considered both deep driven piles and post-and-pad foundation systems for the play decks, ramps, and elevated walkways. Although deep, driven piles would be more stable, they would require different construction equipment than that used to install passively refrigerated piles, increasing the cost of the project. A more cost-effective alternative would be to support elevated play decks, ramps, and walkways on a post-and-pad foundation system. Several inches of seasonal and long-term movement would be expected, but the foundation could easily be designed with releveling capabilities. The posts supporting the structures would bear on a pad founded on compacted classified fill a minimum of 2 feet thick. The three borings drilled on the site suggest the site is overlain by 2.5 feet to 4.5 feet of silty fine sand. The fill was underlain by 1.5 feet to 2.5 feet of peat in two of the borings. Our recommendation for construction of the pad foundations are as follows.

Site preparation for the pad footings should include the removal and replacement of peat underlying the footing areas with classified fill, extending a minimum of 2 feet beyond the outside edge of the pad. During construction, the location of each pad should be explored to a minimum depth of 5 feet to check for and remove peat. The excavation to remove the peat should be backfilled with classified fill.

Classified fill should consist of unfrozen sand, free of organics, containing less than 25 percent passing the No. 200 sieve. In general, classified fill should be placed in layers not exceeding 8 inches in loose height. Except for the first lift over the top of the insulation, the material in each layer should be compacted to achieve a density of at least 92 percent (or higher, as recommended) of the Modified Proctor. ASTM D6938 should be used to determine in-place densities. The water content of the fill should be altered by wetting or drying as necessary to achieve the desired compaction.

We recommend pads be buried a minimum of 1.5 feet deep, and be a minimum of 2 feet by 2 feet but no larger than 5 feet by 5 feet. We understand these pads will be constructed near the proposed new building in an area where an insulated sand-and-gravel-pad fill will be constructed. We recommend placing the footings below the insulation. Pad footings constructed as recommended and bearing on a minimum of 2 feet of compacted classified fill can be sized for an allowable bearing pressure of 1,500 pounds per square foot.

Posts supporting the deck must be designed to be adjustable and tolerate several inches of movement. We expect pads constructed as recommended will experience several inches of movement, including seasonal heave and long-term thaw-settlement. In addition, if peaty soils remain below the footings, additional compression and consolidation of the underlying soils may occur, resulting in additional settlement.

6.11 Water Tanks

Site preparation for the water tanks should include removal and replacement of peat underlying the areas with classified fill. The fill should extend a minimum of 4 feet beyond the outside edge of the tank-skid footings. During construction, the location of each tank should be explored to a minimum depth of 5 feet to check for and remove peat. The excavation to remove the peat should be backfilled with classified fill. Skid footings constructed as recommended and bearing on a minimum of 2 feet of compacted classified fill can be sized for an allowable bearing pressure of 800 pounds per square foot.

We expect pads constructed as recommended will experience several inches of movement, including seasonal heave and long-term thaw-settlement. In addition, if peaty soils remain below the skid footings, additional compression and consolidation of the underlying soils may occur, resulting in additional settlement.

To reduce seasonal movement, an alternative is to insulate the pad prepared for the water tank. We recommend 4 inches of rigid, extruded board insulation suitable for direct burial. Insulation will help reduce seasonal frost-penetration and heaving. It should be placed a minimum of 1 foot below grade. We recommend a minimum of 1 foot of cover over the outside edge of the insulation.

6.12 Parking Areas

Due to the presence of compressible and frost-susceptible soils, and the potential for long-term settlement due to permafrost degradation, we recommend gravel-surfaced access roads and parking areas that can be periodically re-graded when excessive settlement occurs. The pavement section for roads and parking areas subject to light vehicular traffic should be constructed with a minimum of 24 inches of compacted, classified fill overlain by a 6-inch crushed-surface course.

Begin site preparation by clearing and grubbing the existing surface. Before placement of fill and insulation, the base of the excavation should be proof-rolled and compacted to achieve a density suitable for fill placement. If the subgrade is not compacted adequately, it will be difficult for the contractor to achieve the required density with the first lift of fill. Any soft soils exposed in the base of the excavation should be excavated and replaced with compacted classified fill.

We recommend placing a geotextile separator between silty subgrade soils and the cleaner materials used to construct the road and parking area. The subbase should be constructed using classified fill as described in Section 6.12.

The surface course should meet the general requirements for E-1 Surface Course given in ADOT&PF Standard Specifications for Highway Construction. The surface course should be compacted to achieve a density of at least 98 percent of the maximum dry density based on the Modified Proctor moisture-density relationship (ASTM D1557). The surface course should consist of unfrozen materials and be placed at above-freezing air temperatures.

6.13 Classified Fill and Insulation Bedding

Classified fill should consist of unfrozen sand, free of organics, containing less than 25 percent passing the No. 200 sieve. Classified fill can be used as insulation bedding, provided the maximum particle size is less than 1 inch. Soils meeting this gradation criterion are commonly available from local sources as pit-run sand.

Bedding material below the insulation should be placed in layers not exceeding 6 inches in loose height and compacted to 90 percent of the maximum dry density based on the Modified Proctor moisture-density relationship (ASTM D1557). In general, classified fill should be placed in layers not exceeding 8 inches in loose height, except for the first layer over the top of the insulation, which should be 12 inches in loose height. Except for the first lift over the top of the

insulation, the material in each layer should be compacted to achieve a density of at least 92 percent (or higher, as recommended) of the Modified Proctor. ASTM D6938 should be used to determine in-place densities. The water-content of the fill should be altered by wetting or drying as necessary to achieve the desired compaction.

Compaction over the top of the insulation should be conducted so as not to damage the insulation and adequate for obtaining the required density in subsequent lifts.

The fill should consist of unfrozen materials placed at above-freezing air temperatures. If previously placed fill freezes, for instance overnight, the frozen material should be excavated and wasted or allowed to thaw and recompacted prior to the placement of additional fill.

6.14 Insulation

The insulation used in construction should have a maximum thermal conductivity of 0.02 BTU/hr-ft-°F and exhibit no greater conductivity with time. The insulation should be suitable for direct burial and possess adequate compressive strength and suitable long-term creep characteristics for the loads, imposed by the structure, fill, and construction loads.

Prepare the site as described in Section 6.9. A leveling course of relatively clean sand or gravel, or classified fill should be placed over the ground surface prior to the placement of the insulation. By overlapping two layers of 2-inch insulation, 4 inches of insulation can then be effectively placed. In our opinion, at least 2 feet of classified fill should be placed over the insulation to protect the insulation as well as provide a nonflammable material under the facility. Silt is not recommended as a fill material over the insulation, as it may be blown away during the drier summer months. If larger or heavier equipment than is typically used when installing slurry backfilled piles is used on the site, an even thicker cover may be required over the insulation. The insulation should be placed in late spring or early summer, when ground temperatures are coldest. In our opinion, the optimum time for placement of insulation over the ground surface in Bethel is May.

Rigid-board insulation will degrade in the presence of hydrocarbon-based fuel or fuel vapors. We know of several instances where insulation placed beneath structures has been lost due to fuel spills, resulting in settlement. We do not know if the site is contaminated, as there were no

indications of contamination in our borings; however, the potential for contamination does exist due to the presence of heating-oil tanks and lines serving the existing structures.

To reduce the risks associated with a possible spill or contamination, an alternative is to encapsulate it with a membrane impervious to fuels and solvents ,to protect against petroleum vapor degradation, particularly in areas with potential exposure to fuels and solvents. In addition to protecting the insulation from known contamination, the membrane is intended to protect the insulation from future fuel and/or solvent spills. Furthermore, the membrane will also help to control moisture absorption by the insulation.

6.15 Geotextile Separator

The geotextile separator placed between the subgrade and the building pad and parking area should consist of a nonwoven needle-punched material with an apparent opening size (AOS) of less than the No. 70 U.S. Standard Sieve as determined by ASTM D4751-87, grab tensile strength in excess of 150 pounds as determined by ASTM D4632-91, or meet the approval of the geotechnical engineer. The fabric should be placed in accordance with the procedures outlined by the ADOT&PF Standard Specifications for Highway Construction.

6.16 Drainage and Grading

During construction of the project, the ground surface should be sloped away from open excavations to reduce water inflow into any excavations. The addition of water to the excavation may reduce the stability of the slopes and raise the moisture content of the soils in the base of the excavation to a point where they are difficult to compact to the required density. Additional moisture may also cause silty soils to become soft, difficult to work. and pump. The contractor should be prepared to remove soft, wet materials or work in other areas until the soils have dried enough to provide a suitable base.

Following construction, the site should be graded to provide positive drainage away from the new building, roadways, and parking areas. Water should not be allowed to pond on or adjacent to these structures. We anticipate settlement depressions may develop in the surface of the pad around the building, particularly where shallow, thaw-unstable permafrost is present. The surface should be periodically regraded to maintain positive drainage away from the new building.

6.17 Snow Clearing

The site should be designed to prevent snow from drifting around the structure. Snow should not be stockpiled near the building foundation.

6.18 Utility Connections

Due to the potential for long-term ground movement, we recommend flexible connections between buried utilities and the building.

7.0 ALTERNATIVE BUILDING FOUNDATION SYSTEM

This section presents recommendations for a driven nonrefrigerated pile foundation system. In our opinion, a driven-pipe pile foundation system will have acceptable performance under current permafrost conditions at the site; however, there is a greater risk of unacceptable foundation movement during the life of the structure, since ground temperatures will not be cooled with refrigeration devices. Connections between the structure and the foundation should therefore have adjustment capabilities in the event excessive settlement occurs.

7.1 Driven Pile Recommendations

Pipe piles for the main structure should bear in the permafrost below a depth of 45 feet at the site. We recommend driving the pipe piles open-ended, using high-strength, heavy-section pipe with cast-steel tip protection to reduce tip damage. Tip protection should be flush with the outside edge of the pile. We anticipate the piles will be advanced by driving into the top of permafrost and then by a combination of driving and drilling through the pipe-pile below the pile-tip to relieve/reduce driving stresses in the piles. The diameter of the predrilled hole should be 4 inches less than the outside diameter of the pile. The final 3 feet of embedment should not be predrilled.

The pile installation acceptance criteria should be based on a minimum design depth. The piles should not be overstressed during driving. We recommend using a wave equation analysis of pile driving to establish driving criteria, and a sequence of driving and relief drilling to prevent overstressing the piles during driving.

Two weeks before driving is scheduled to begin, the contractor should provide the engineer with the following information: pile-driving hammer, hammer cushion, drive head, pile cushion, and

other appurtenances to be furnished by the contractor for approval by the engineer. During pile-driving operations, the contractor should use the approved system. Any change in the driving system should be considered only after the contractor has submitted the revised pile-driving equipment to the engineer. After receiving the change, the contractor should allow seven days for approval of the revised equipment.

We strongly recommend a geotechnical engineer from our firm continuously observe all piling installation. Potential contractors should be made aware the embedment depth or the number of piles required might change, depending on observed field conditions. The recommended allowable loads assume bonded permafrost is encountered throughout the bearing section. If nonbonded or large massive-ice zones are observed in the borings, the pile capacities will be reduced. We recommend the depth of embedment of the pile be increased to a depth equal to the thickness of the undesirable zones to compensate for the potential reduction in pile capacity.

We recommend a complete installation record be kept for each pile. Each pile should be clearly marked in 1-foot increments for its full length prior to driving. Each pile record should include the number of blows required for each foot of pile penetration, ram stroke, type and energy rating of the hammer, and blows-per-minute near the estimated penetration depth.

Fixed-lead pile-driving equipment is recommended to install steel pipe-piles. The use of hanging or swinging leads is not recommended, unless constructed so they can be held in a fixed position during driving operations. Leads should be of sufficient length so the use of followers will not be necessary.

7.2 Axial Pile Capacity

Recommended sustained design loads for driven pipe piles are presented in Figure 15. The design capacities presented have a factor of safety of 2 applied to the creep rate, and a factor of safety of greater than 3 applied to adfreeze. For the proposed structure, an allowable long-term settlement of 2 inches in 20 years (in addition to initial construction settlements of less than ½ inch) was used in the calculation.

As previously described, the sustained loads presented are defined as loads equal to the dead load plus that portion of the live load expected to act on the structure for 50 percent of the year or longer. Support of the sustained loads is based primarily on the creep deformation of ice-rich frozen soil, and less on the adfreeze strengths of the pile/soil interface.

7.3 Excavation and Site Preparation

The site should be prepared by constructing a pad across the building footprint extending at least 12 feet beyond the edge of the footprint. The surface of the pad should be sloped to drain and prevent ponding of water beneath the structure. The perimeter of the pad should be insulated with a minimum of 4 inches of insulation suitable for direct burial, overlain by a minimum of 2 feet of cover material described in this report. The insulation should extend out horizontally 12 feet beyond the edge building and 8 feet beneath the building. Along the north and west sides of the building, the width of the insulation extending out horizontally beyond the building edge can be reduced to 8 feet if necessary, to avoid crossing the property line. We recommend a minimum of 1 foot of cover over the outside edge of the insulation.

Excavation and site preparation should be conducted as described in Section 6.9.

8.0 LIMITATIONS

The information and conclusions contained in this report are based on site conditions as they presently exist, and assume the exploratory borings are representative of the subsurface conditions throughout the site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or test borings. Such unexpected conditions frequently require additional expenditures to obtain a properly constructed project; therefore, some contingency fund is recommended to accommodate such potential extra costs.

If substantial time has elapsed between the submission of this report and the start of work at the site, or conditions have changed because of natural causes or construction operations at or adjacent to the site, we recommend this report be reviewed to determine the applicability of the conclusions considering the time lapse or changed conditions. If you desire, we can assist you with geotechnical design for the project and review those portions of the plans and specifications that pertain to earthwork and foundations.

This report was prepared for the exclusive use of the owner and architect and/or engineer in the design of the subject facility. It should be made available to prospective contractors and/or the contractor for information on factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the boring logs and presented in discussions of subsurface conditions in this report. The scope of our services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air at the site.

Shannon & Wilson, Inc., has prepared the attachment *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the uses and limitations of our reports.

We trust this information is sufficient for your needs at the present time. If you have any questions, please do not hesitate to call.

Sincerely,

SHANNON & WILSON, INC.

Frank Wuttig, C.P.G.
Associate

Reviewed by:

Stephen Adamczak, Jr., P.E
Vice President

STATE OF ALASKA
STEPHEN ADAMCZAK JR.
NO. CE 6481
REGISTERED CIVIL ENGINEER

Not to Scale

APPROXIMATE NORTH

Legend

07-1 - Approximate Boring Number and Location

Geotechnical Design Study
Tundra Women's Coalition
Bethel, Alaska

SITE PLAN AND BORING LOCATIONS

2008 31-1-02073-001

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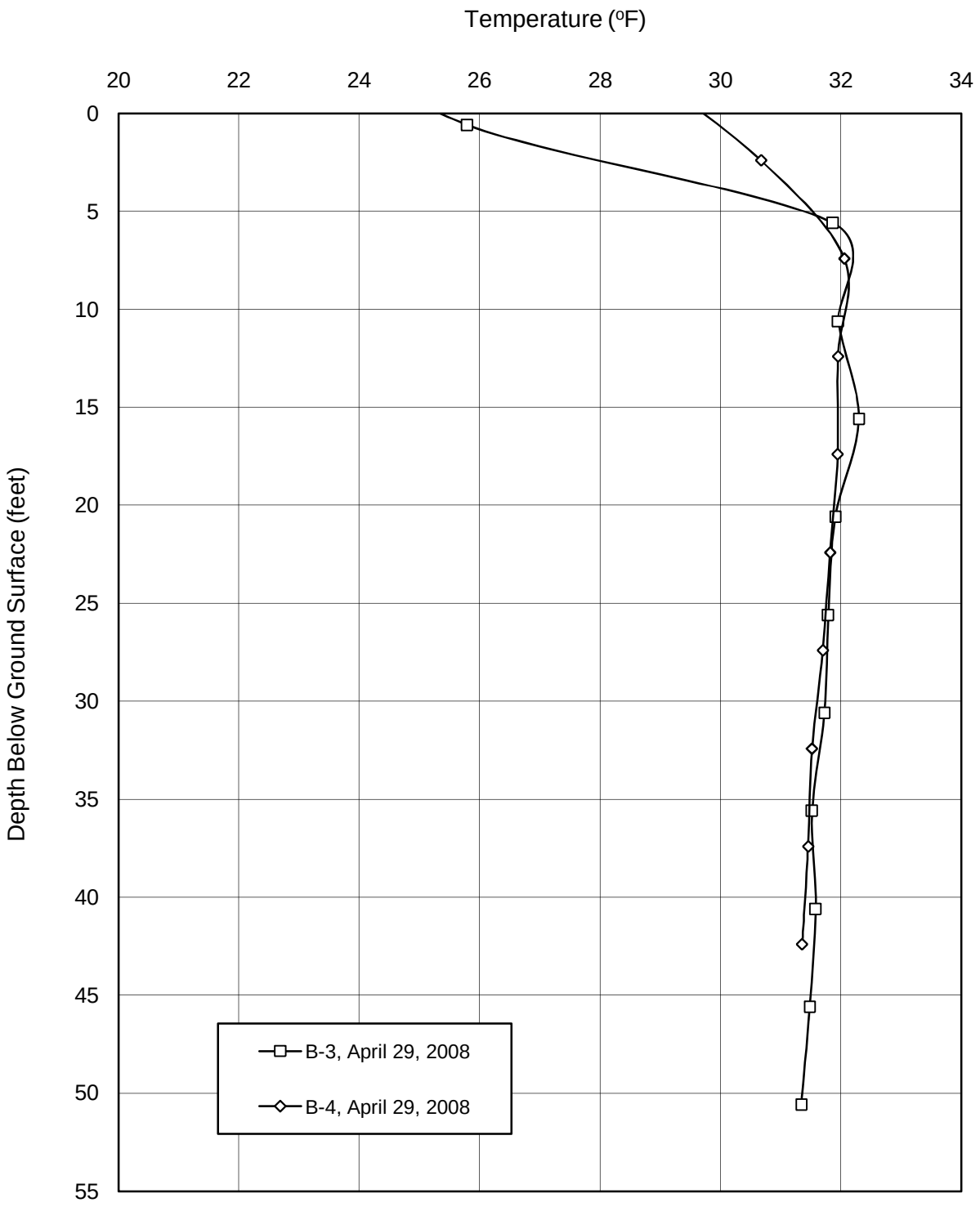
Figure 1



Geotechnical Design Study
Tundra Women's Coalition
Bethel, Alaska

GROUND TEMPERATURES:
BORINGS B-3 AND B-4

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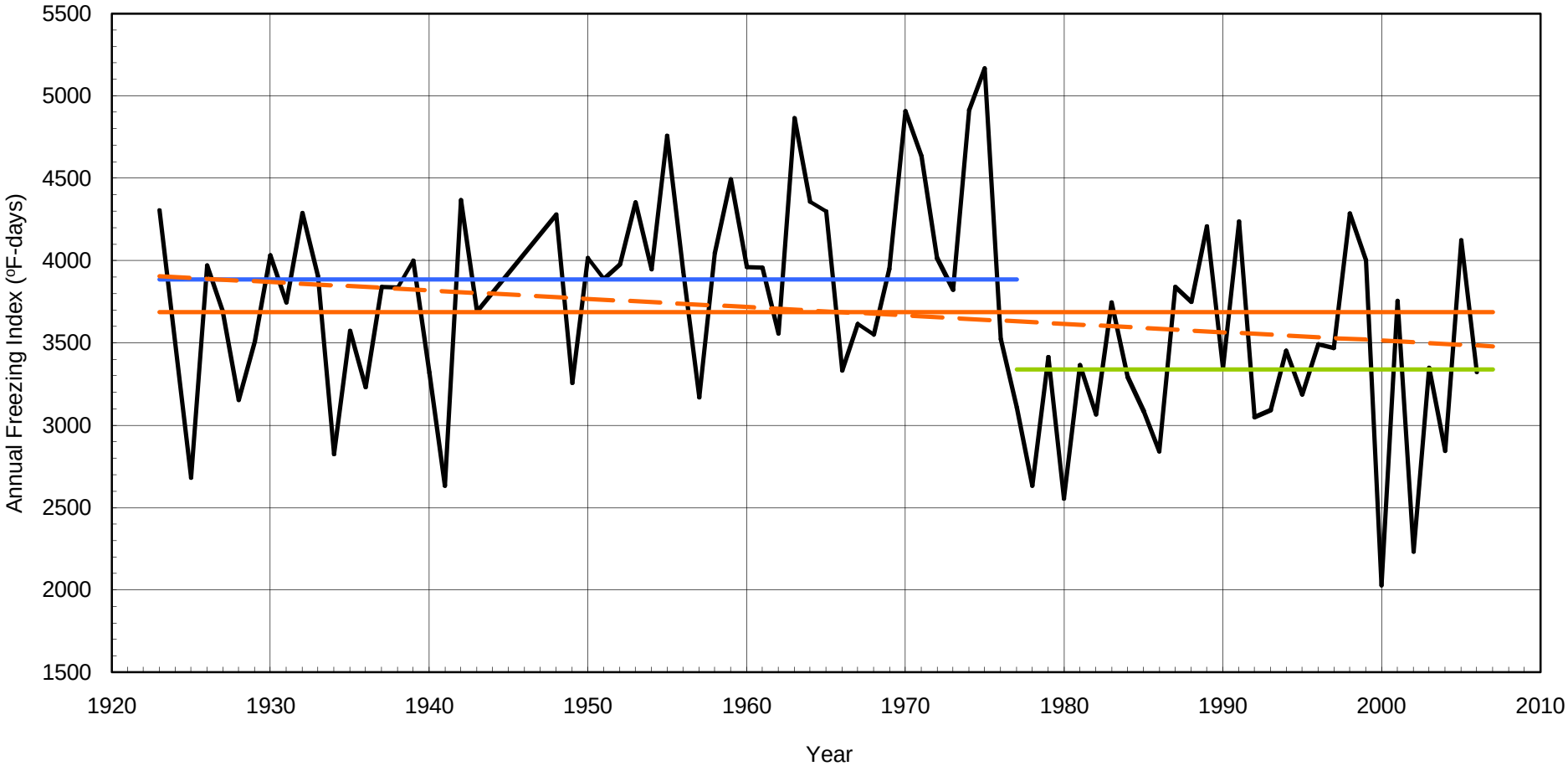
Figure 2



Legend:

- Air Freezing Index (AFI), 1923-2007
- 1923-2007 Mean = 3687, SD = 624
- 1923-1977 Mean = 3885, SD = 569
- 1977-2007 Mean = 3339, SD = 564
- 1923-2007 Linear Regression (Slope = -5.1 °F-days/yr)

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Bethel, Alaska

AIR FREEZING INDEX
BETHEL AIRPORT (PABE)
OBSERVING STATION
1923-2007

2008 31-1-02073-001

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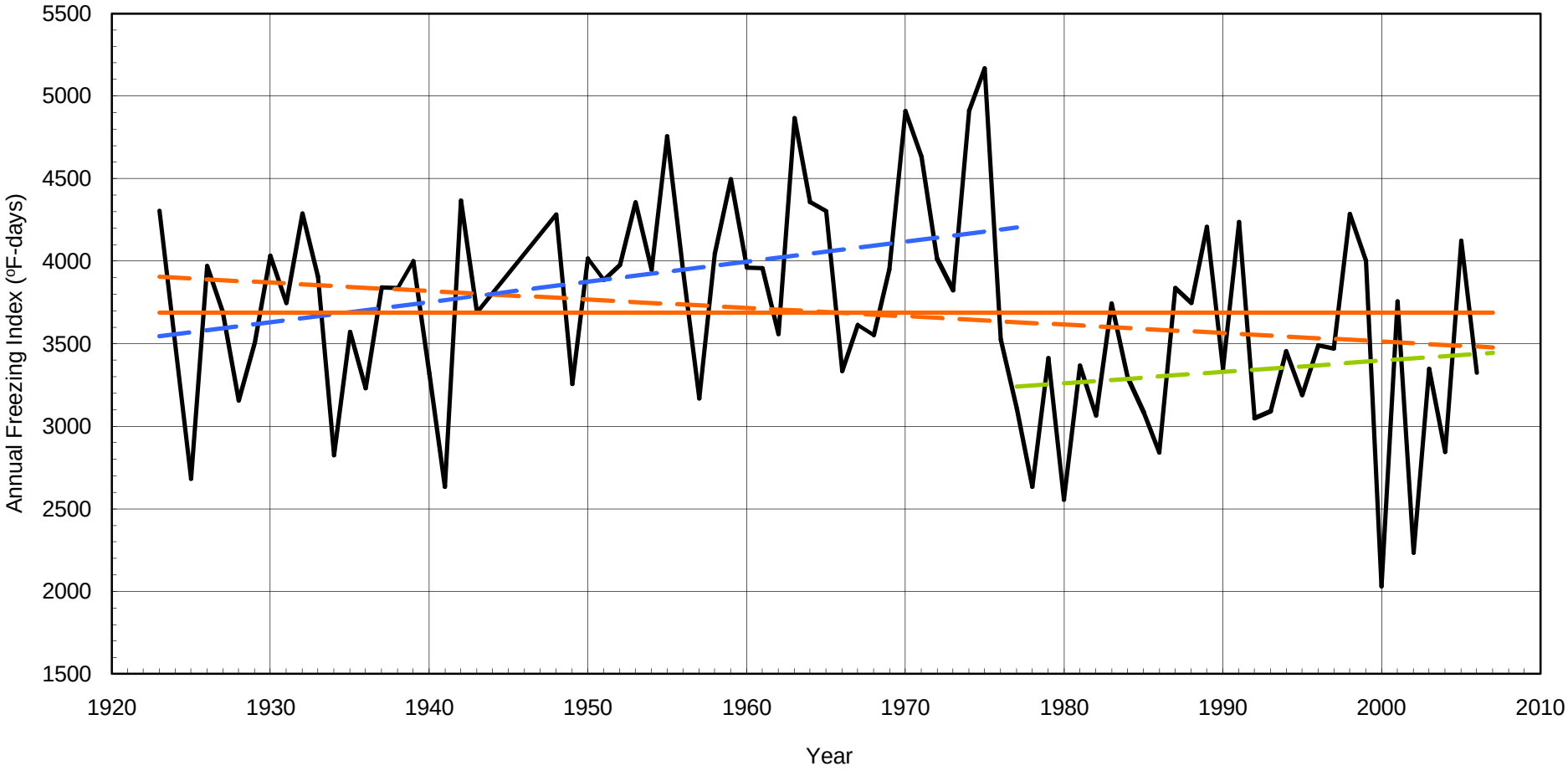
Figure 3



Legend:

- Air Freezing Index (AFI), 1923-2007
- 1923-2007 Linear Regression (Slope = -5.1 °F-days/yr)
- 1923-1977 Linear Regression (Slope = 12.2 °F-days/yr)
- 1977-2007 Linear Regression (Slope = 6.9 °F-days/yr)
- 1923-2007 Mean = 3687, SD = 624

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AIR FREEZING INDEX
BETHEL AIRPORT (PABE)
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1923-2007

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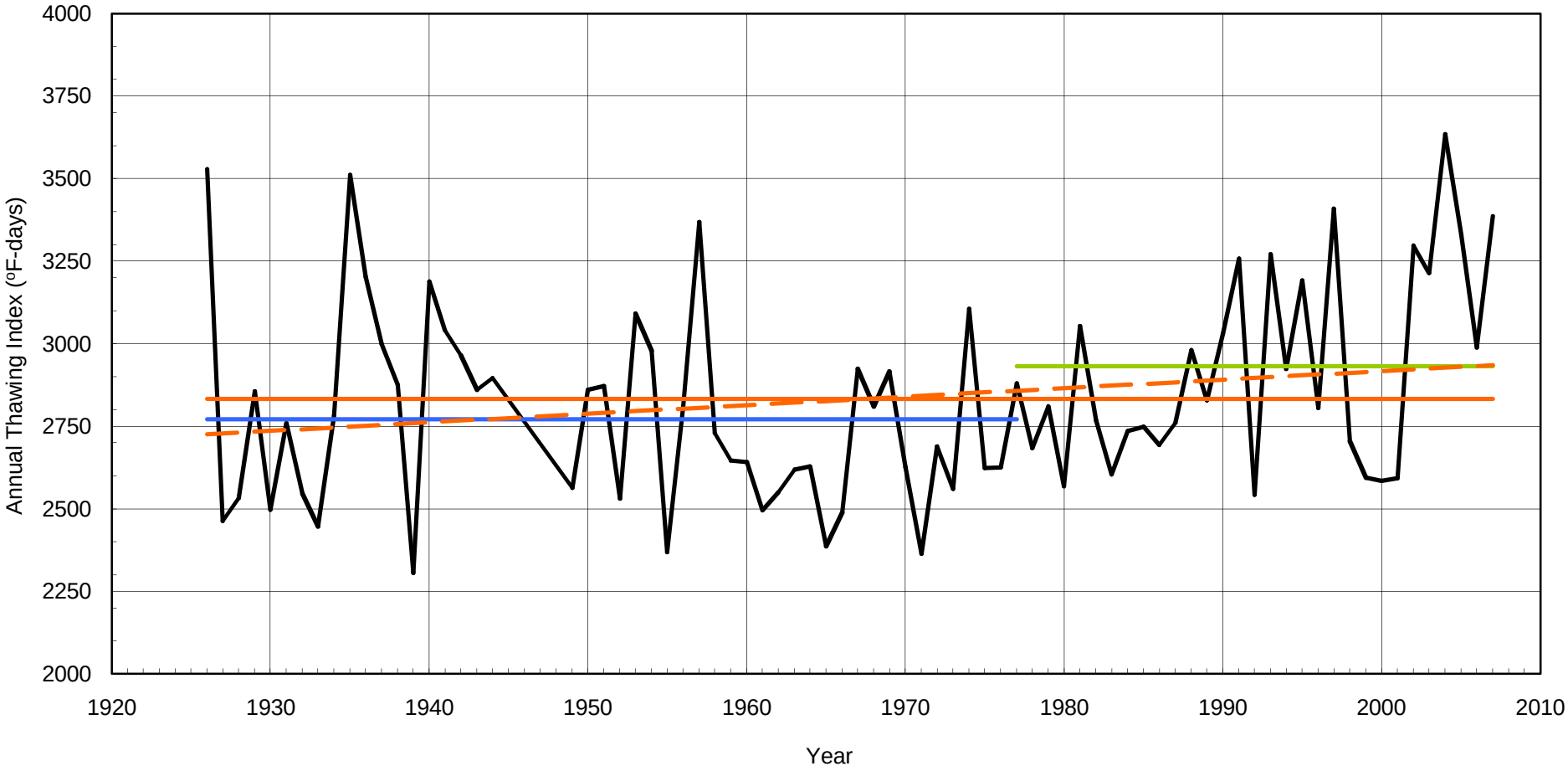
Figure 4



Legend:

- Air Thawing Index (ATI), 1923-2007
- 1923-2007 Mean = 2833, SD = 304
- 1923-1977 Mean = 2771, SD = 290
- 1977-2007 Mean = 2931, SD = 300
- 1923-2007 Linear Regression (Slope = 2.6 °F-days/yr)

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AIR THAWING INDEX
BETHEL AIRPORT (PABE)
OBSERVING STATION
1923-2007

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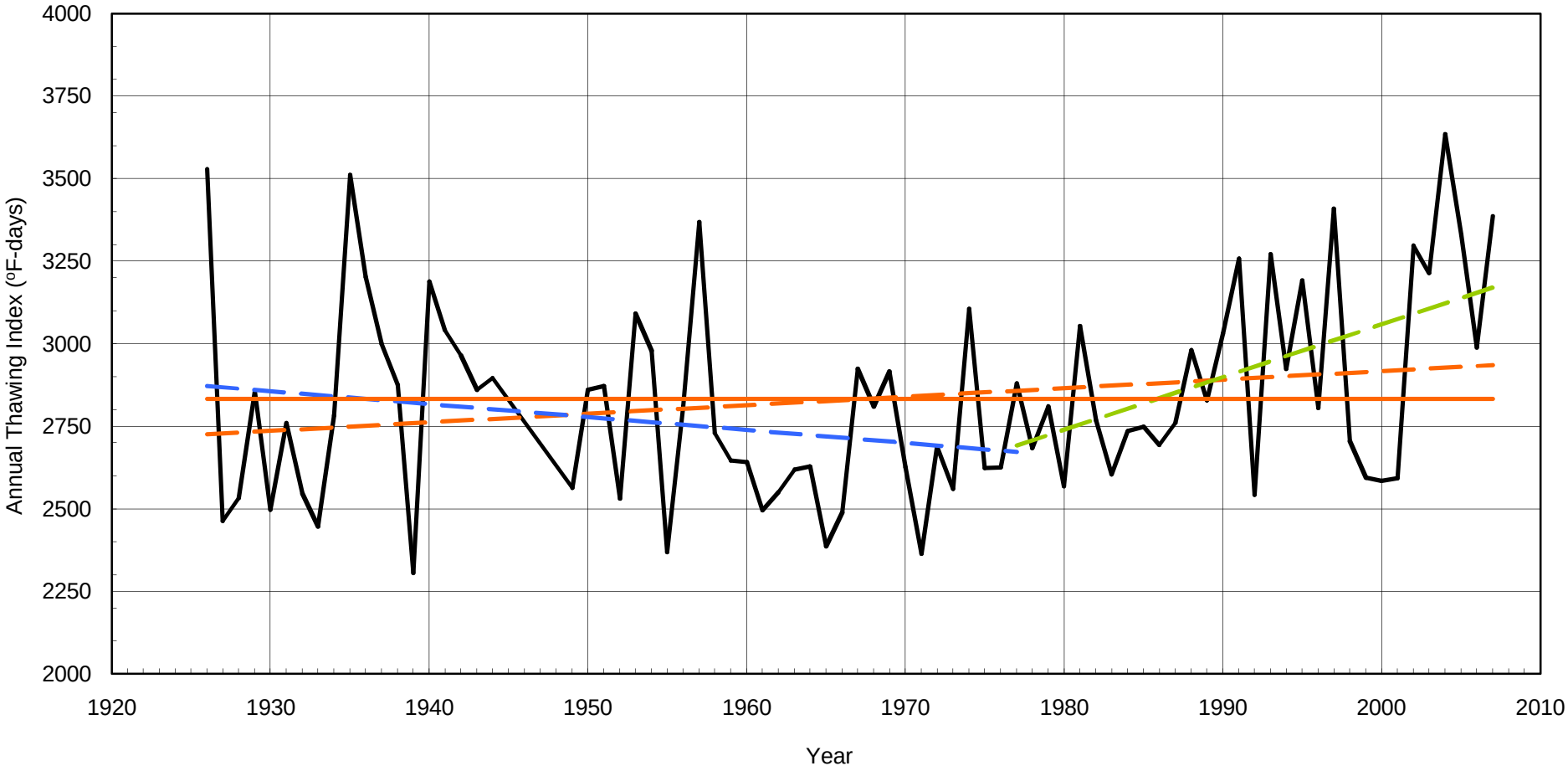
Figure 5



Legend:

- Air Thawing Index (ATI), 1923-2007
- 1923-2007 Linear Regression (Slope = 2.6 °F-days/yr)
- 1923-1977 Linear Regression (Slope = -3.9 °F-days/yr)
- 1977-2007 Linear Regression (Slope = 15.9 °F-days/yr)
- 1923-2007 Mean = 2833, SD = 304

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AIR THAWING INDEX
BETHEL AIRPORT (PABE)
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1923-2007

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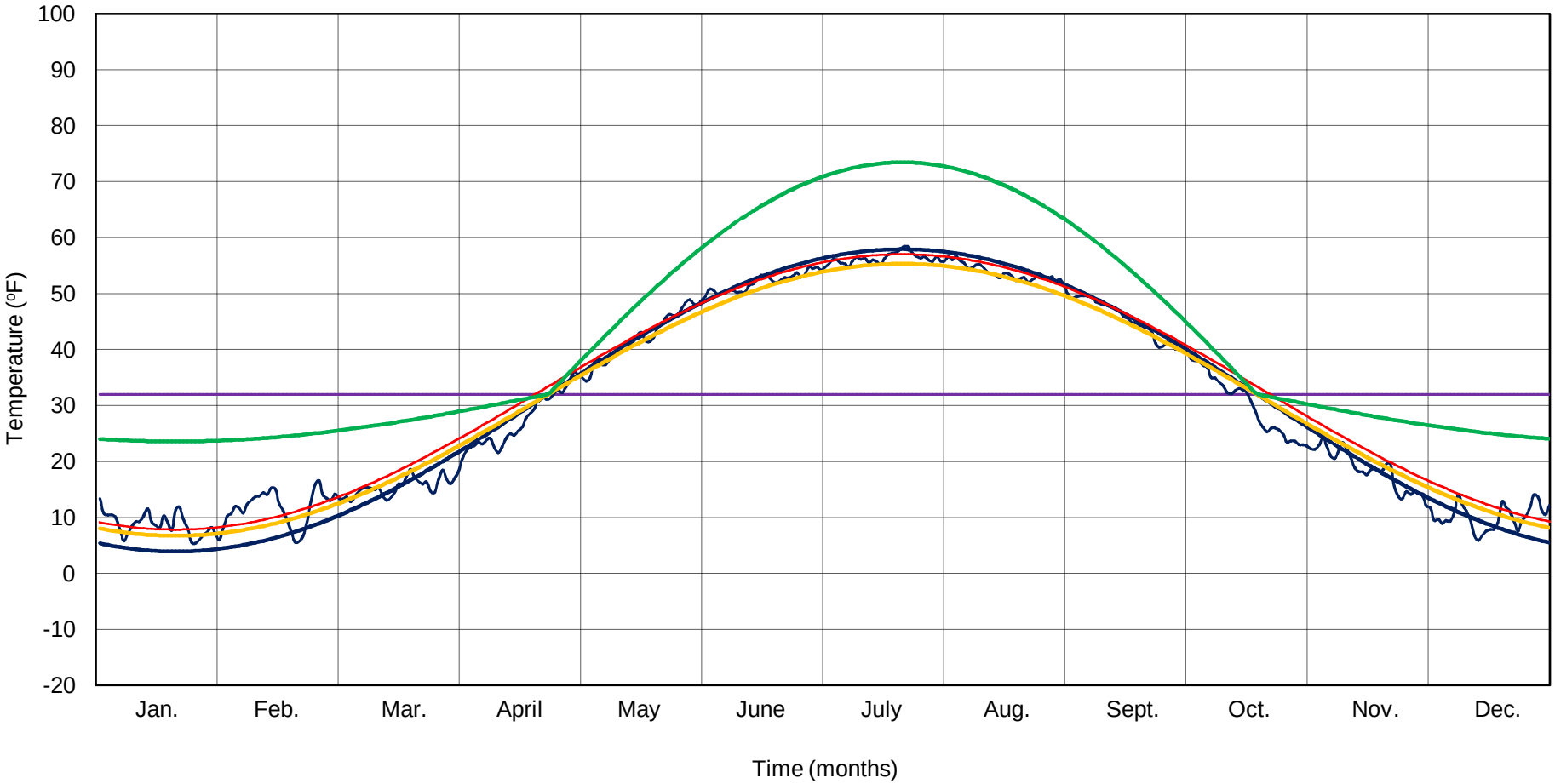
Figure 6



Legend:

- 32 Degrees
- Bethel Mean Air Temperature 1977 - 2007
- Simulated Mean Air Temperature
- Simulated Condenser Temperature
- Simulated Interior Ground Surface Temperature (nt = 0.9, nf = 0.9)
- Simulated Exterior Ground Surface Temperature (nt = 1.6, nf = 0.3)

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SIMULATED CONDENSER AND GROUND SURFACE TEMPERATURES

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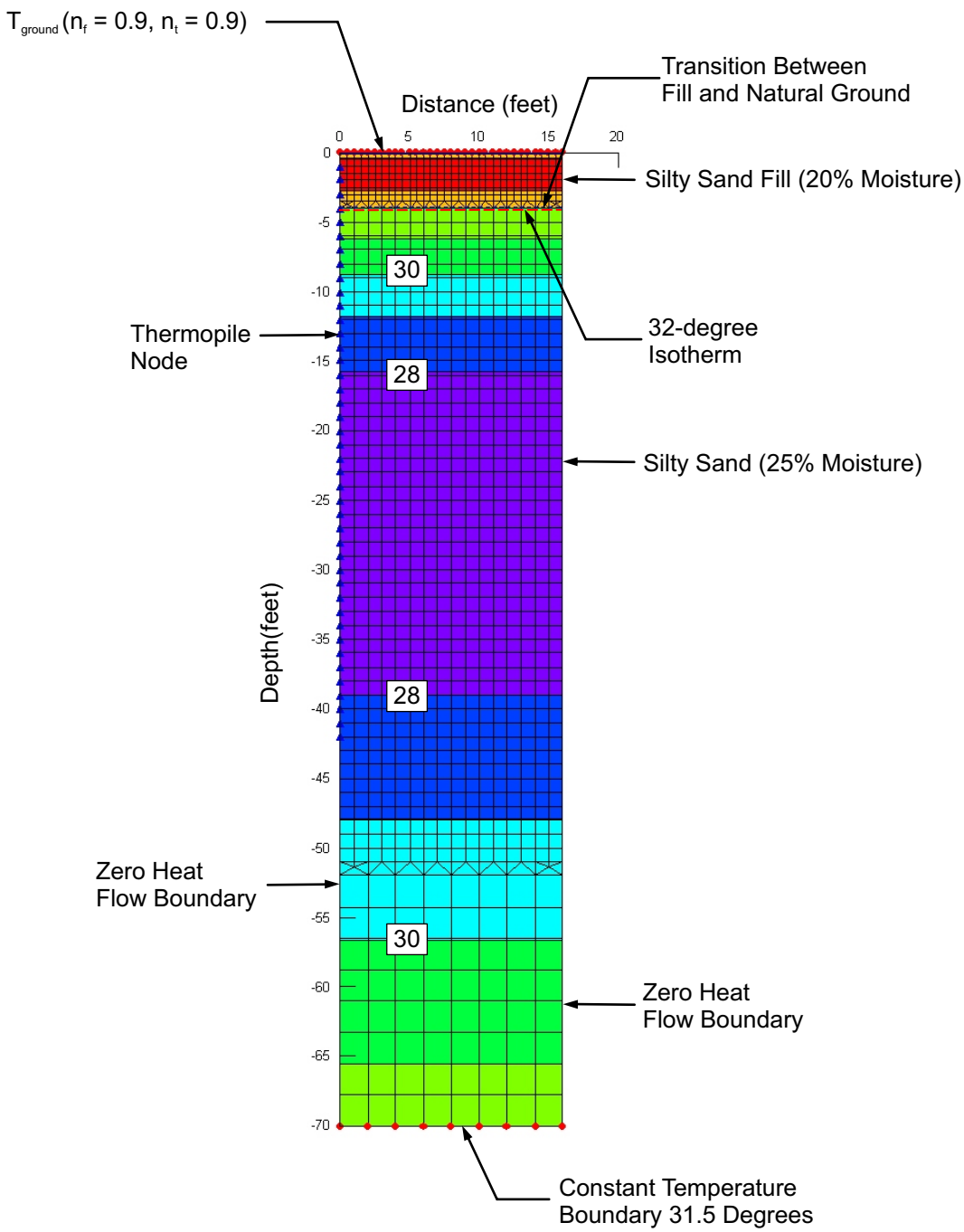
Figure 7



Not to Scale

Notes:

Thermopile Group Conductance = 19.6 Btu/Day-°F-Ft

Isotherms Drawn at 1 Degree Intervals

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INTERIOR PILE SIMULATION
END OF THAWING SEASON

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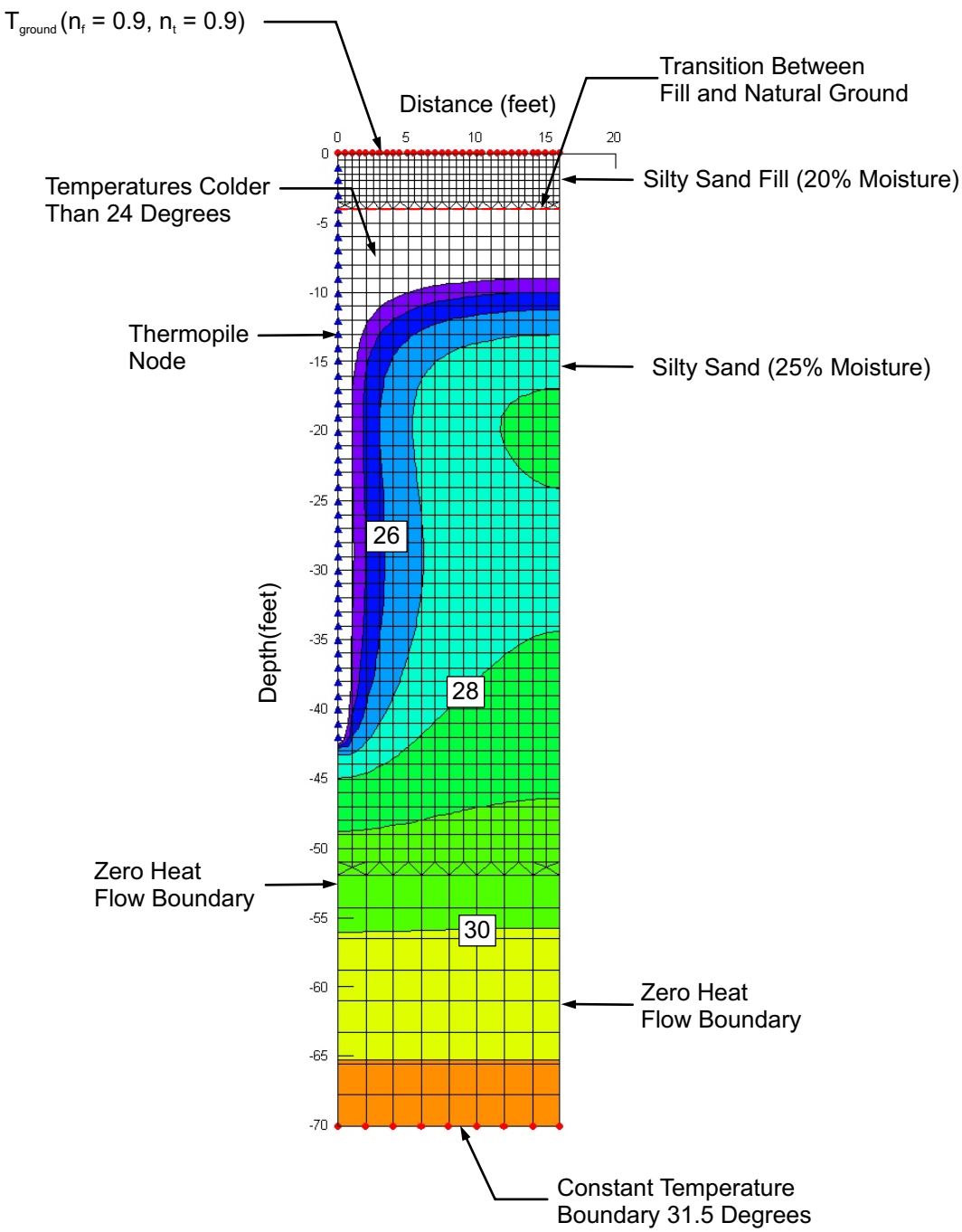
Figure 8



Not to Scale

Notes:
Thermopile Group Conductance = 19.6 Btu/Day-°F-Ft
Isotherms Drawn at 1 Degree Intervals

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INTERIOR PILE SIMULATION
MIDDLE OF THAWING SEASON

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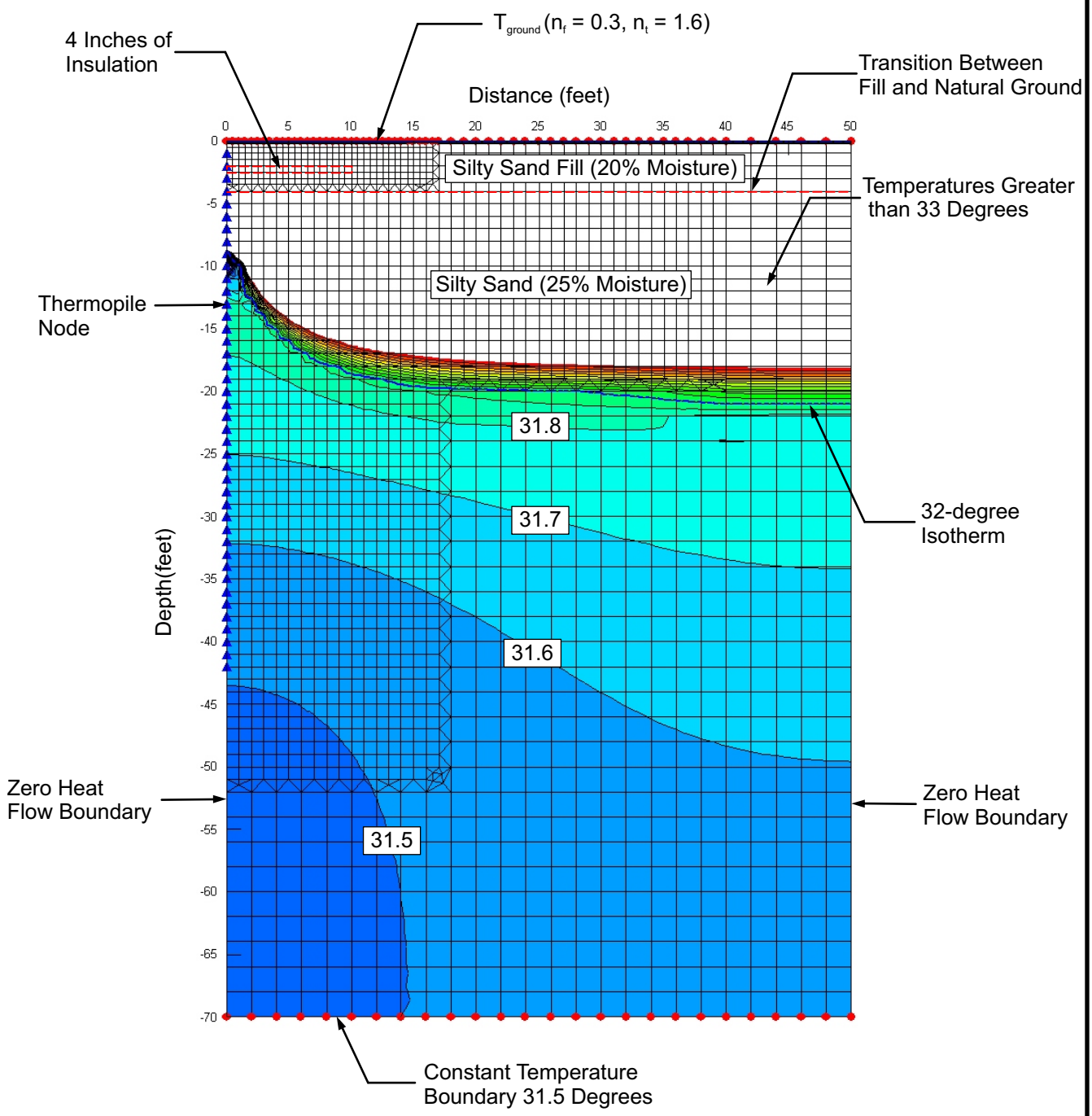
Figure 9



Not to Scale

Notes:

Thermopile Conductance = 19.6 Btu/Day-°F-Ft
Isotherms Drawn at 0.1 Degree Intervals

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EXTERIOR PILE SIMULATION
END OF THAWING SEASON

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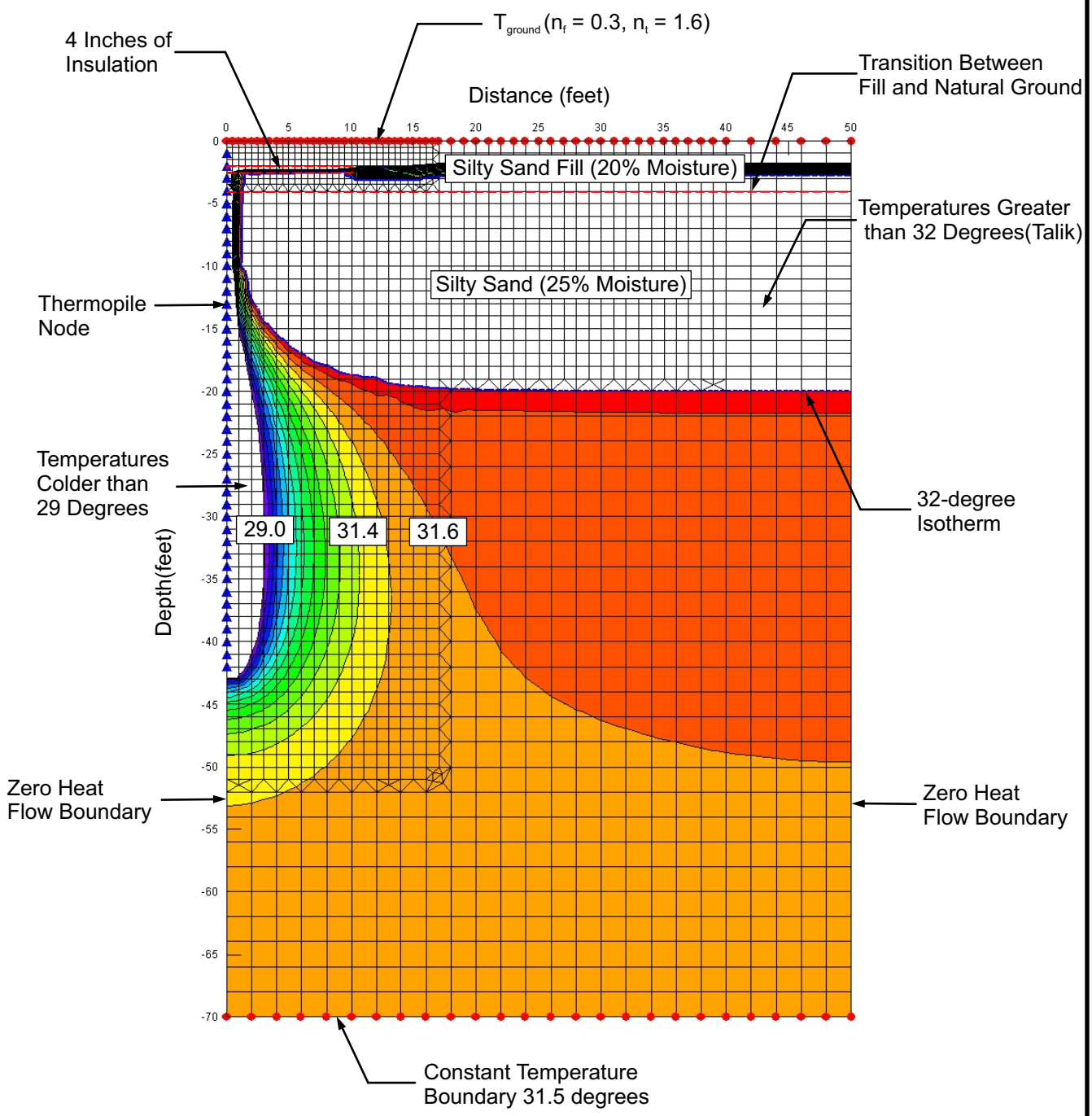
Figure 10



Not to Scale

Notes:

Thermopile Conductance = 19.6 Btu/Day-°F-Ft
Isotherms Drawn at 0.2 Degree Intervals

Geotechnical Design Study
Tundra Women's Coalition
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EXTERIOR PILE SIMULATION
MIDDLE OF FREEZING SEASON

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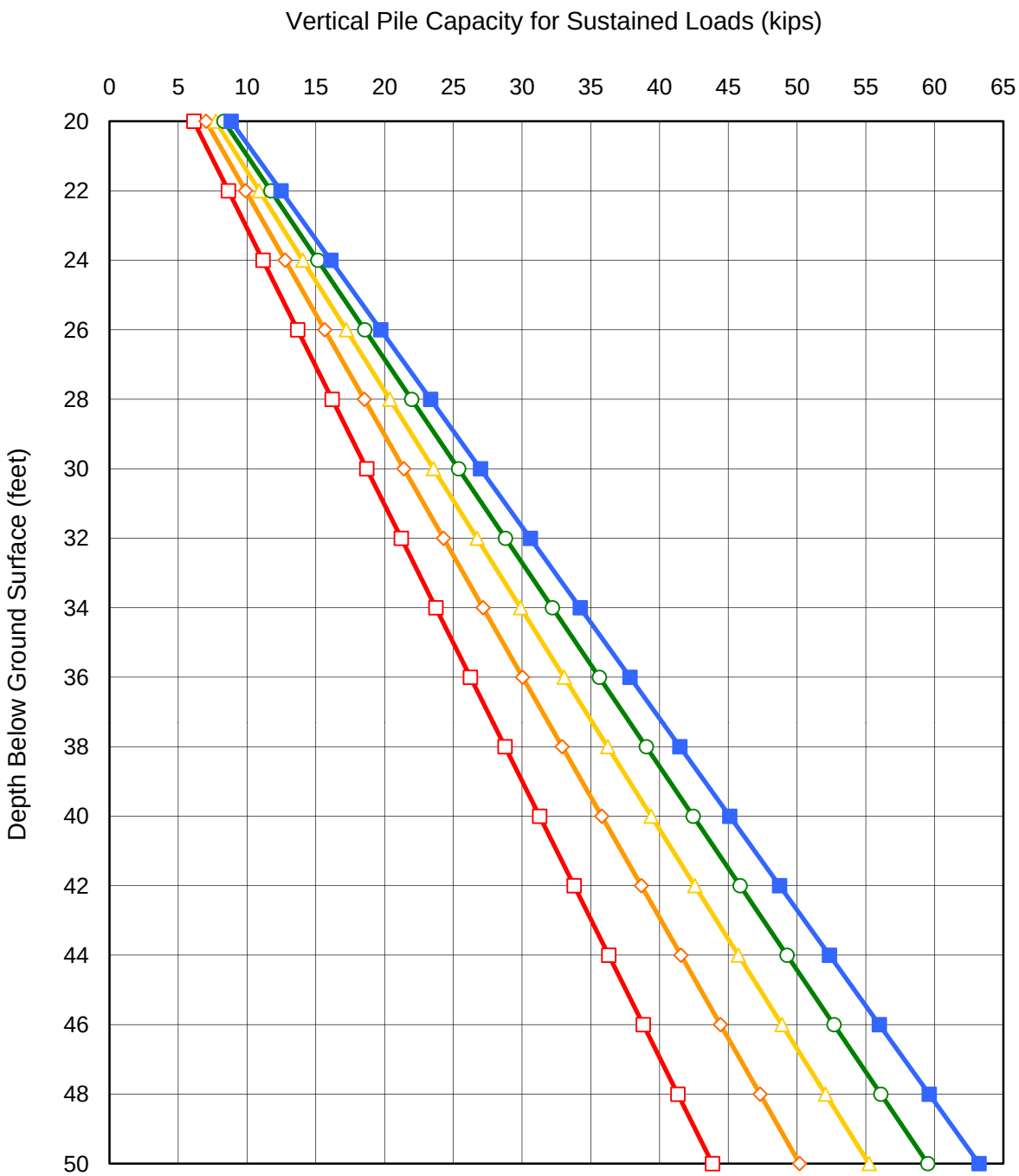
Figure 11



INTERIOR PILE:
Unbonded Length: 16 ft
Design Temp.: 31.0 F

- 1.0 in / 20 yr
- 1.5 in / 20 yr
- 2.0 in / 20 yr
- 2.5 in / 20 yr
- 3.0 in / 20 yr

Geotechnical Design Study
Tundra Women's Coalition
Bethel, Alaska

ALLOWABLE AXIAL CAPACITY
SLURRY-BACKFILLED 12-INCH
INTERIOR PIPE PILE

2008 31-1-02073-001

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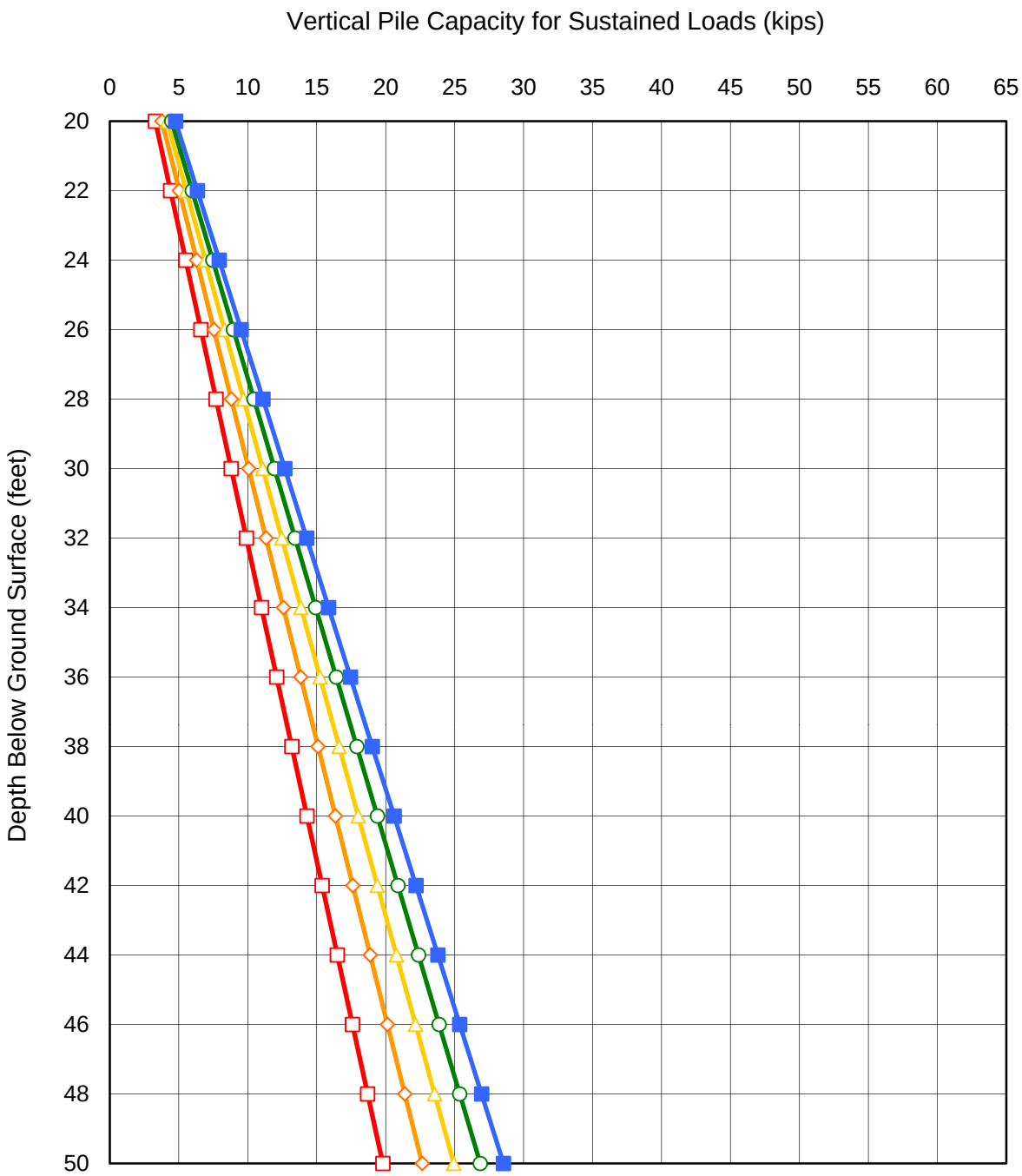
Figure 12



EXTERIOR PILE:
Unbonded Length: 16 ft
Design Temp.: 31.5 F

- 1.0 in / 20 yr
- 1.5 in / 20 yr
- 2.0 in / 20 yr
- 2.5 in / 20 yr
- 3.0 in / 20 yr

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Tundra Women's Coalition
Bethel, Alaska

ALLOWABLE AXIAL CAPACITY
SLURRY-BACKFILLED 12-INCH
EXTERIOR PIPE PILE

2008 31-1-02073-001

SHANNON & WILSON, INC.
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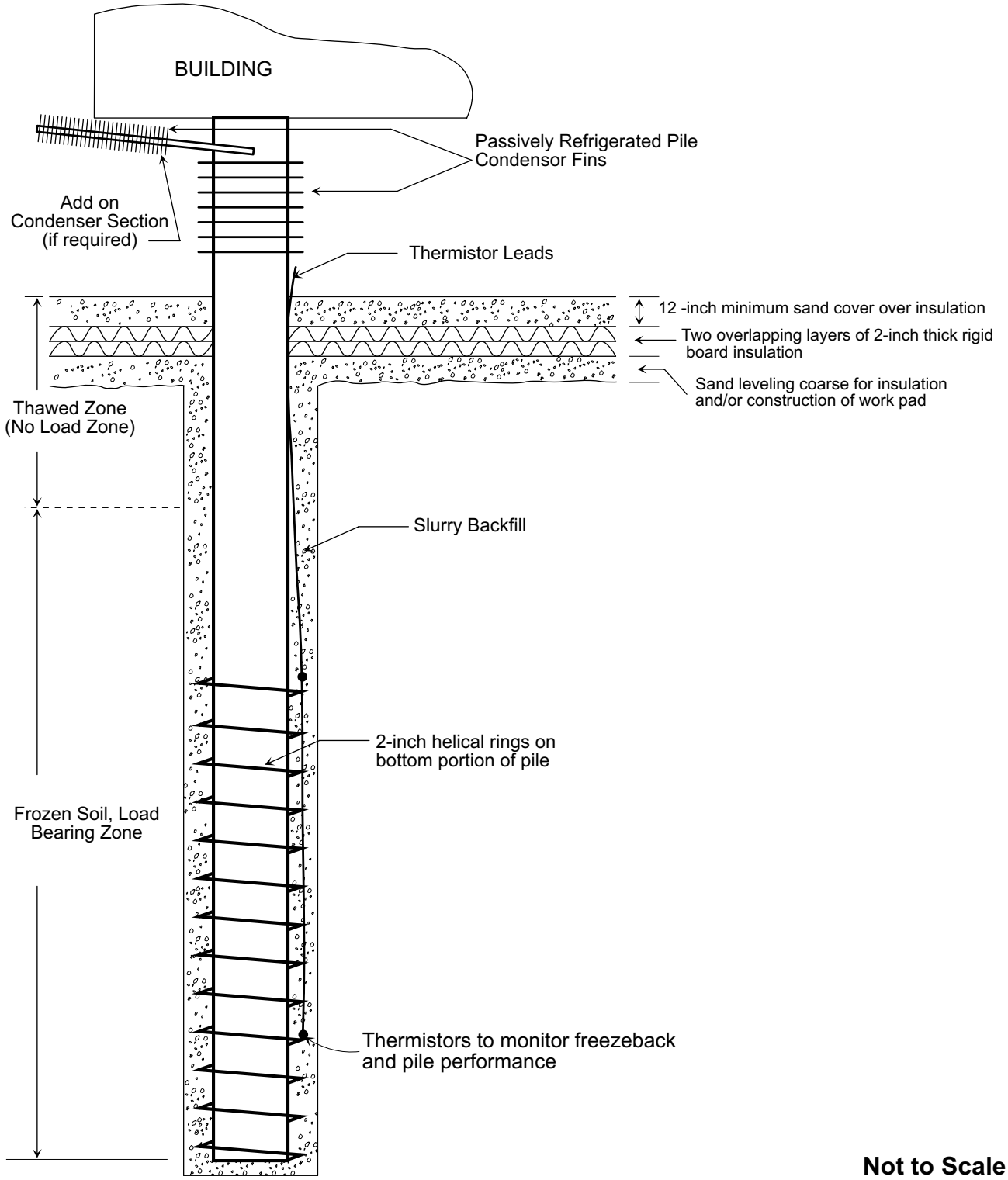
Figure 13



Not to Scale

NOTES:
SEE REPORT FOR DETAILS

Geotechnical Design Study
Tundra Women's Coalition
Bethel, Alaska

PASSIVELY REFRIGERATED SLURRY BACKFILLED PILE FOUNDATION RECOMMENDATIONS

2008 31-1-02073-001

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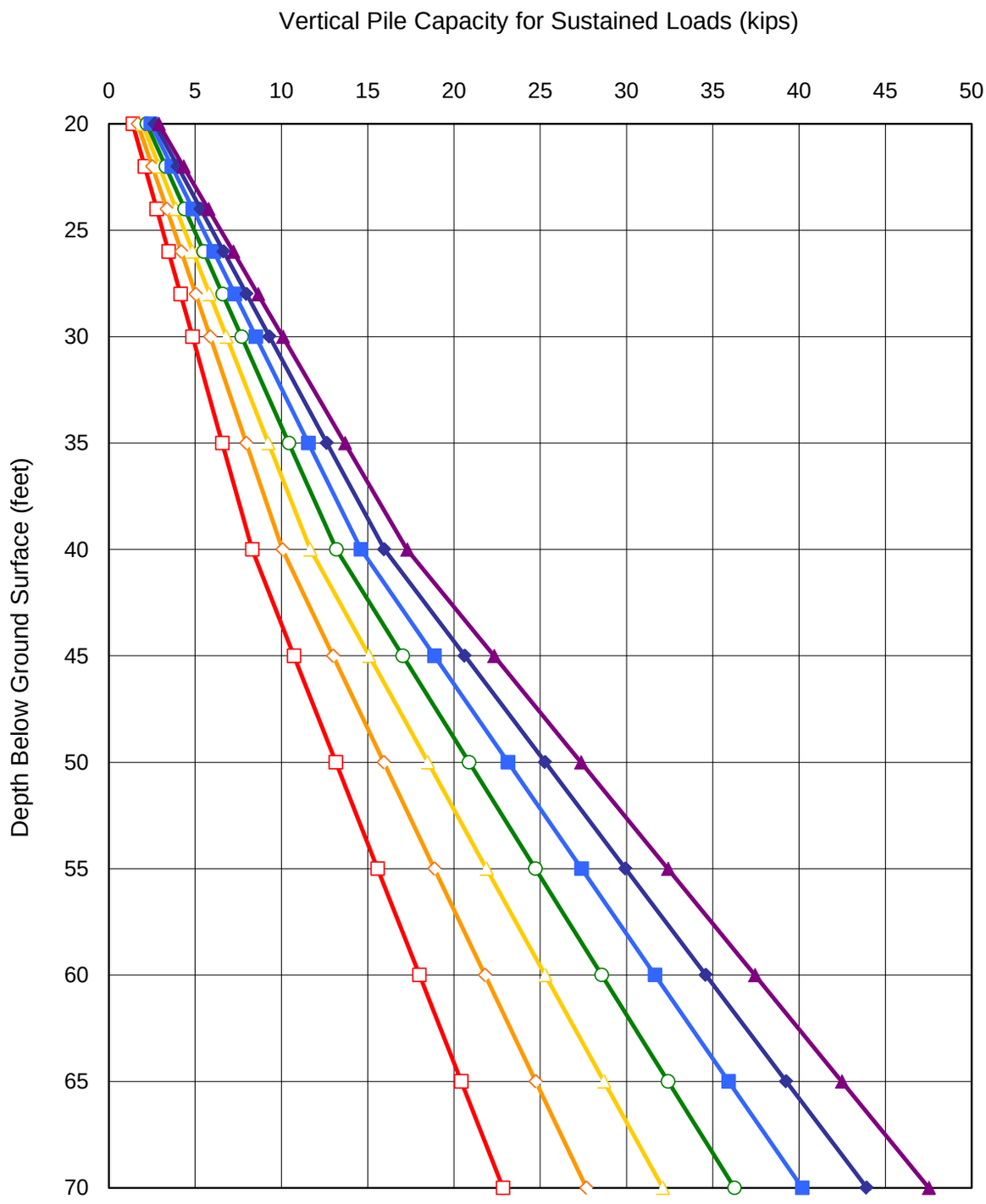
Figure 14



- 6-inch pipe
- 8-inch pipe
- 10-inch pipe
- 12-inch pipe
- 14-inch pipe
- 16-inch pipe
- 18-inch pipe

Design Criteria:
Settlement: 2 in / 20 yr
FS: 2.0
Unbonded Length: 16 ft
Design Temp.: 31.3 - 31.5 F

Geotechnical Design Study
Tundra Women's Coalition
Bethel, Alaska

DRIVEN PIPE PILE CAPACITY

2009

31-1-02073-001

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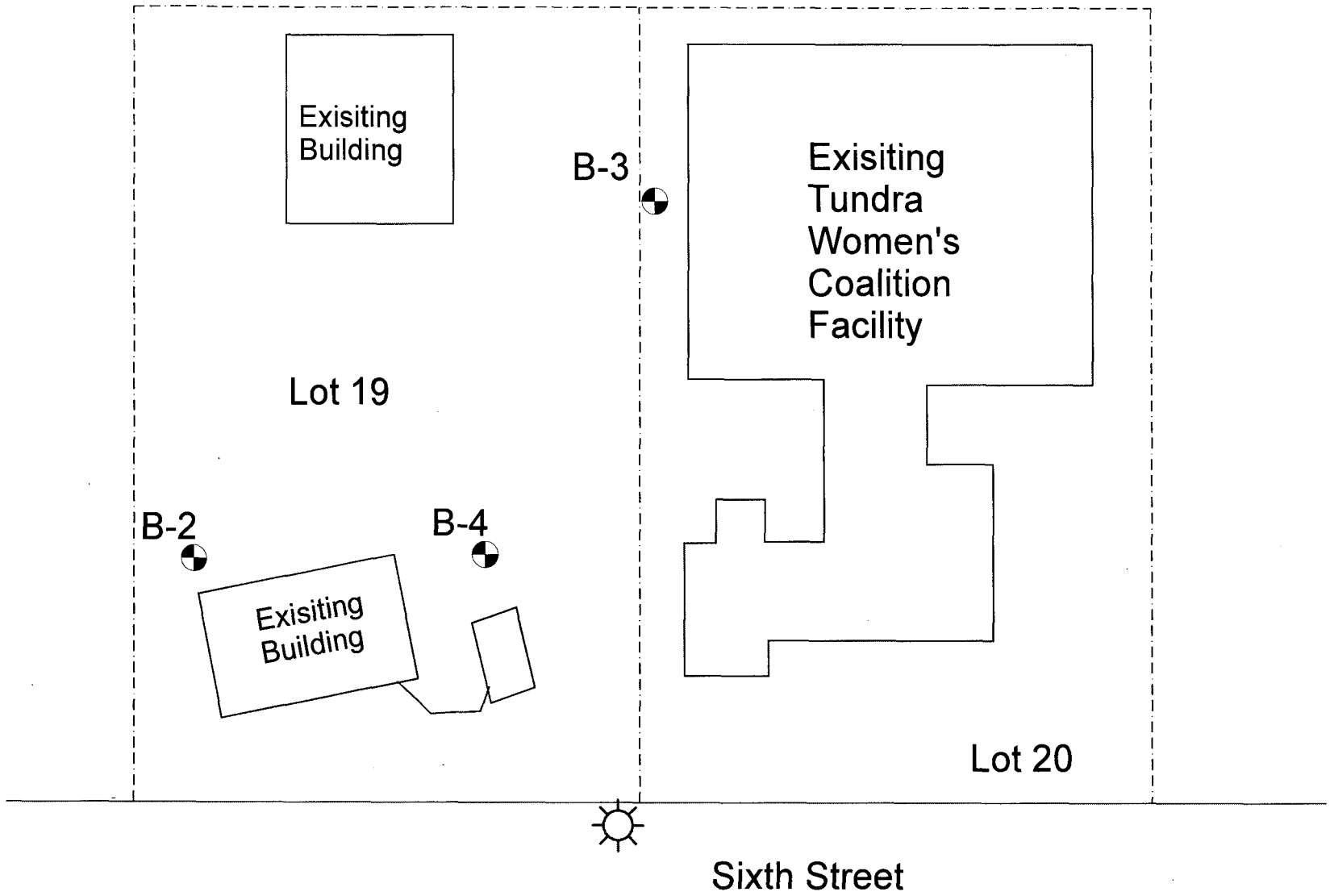
Figure 15

APPENDIX A

SOIL LOGS FROM PREVIOUS STUDIES



LEGEND

B-2 Approximate Location of Boring B-2 advanced by Shannon& Wilson, May 2007

NOTE: Boring B-1 eliminated from field program because of access restrictions

0 30 60
APPROXIMATE SCALE IN FEET

N

Tundra Women's Coalition Addition
Bethel, Alaska

SITE PLAN

June 2007 | 32-1-01899

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

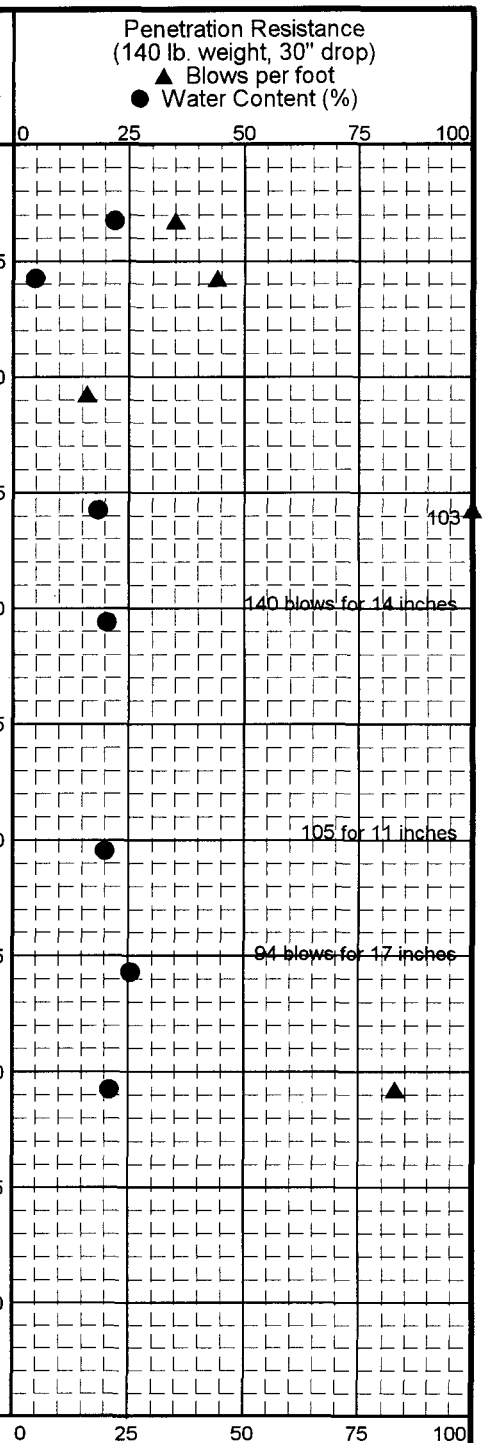
Fig. 2

MATERIAL DESCRIPTION Approx. Elevation: 2 ft below 6th Street	Depth, Ft.	Samples
Dark brown, silty, fine SAND; moist [FILL] S1: 0% Gravel, 51% Sand, 49% Fines (F4)	2.5	S1
Frozen to medium dense, light brown to gray, silty, fine SAND; [Nbn] -thawed from ground surface to 2.8 ft -moist below 7.0 ft -dark brown peat seam at 2.5 ft		S2 S3 *
Frozen, gray, silty, fine SAND; Vc to Vs S4: 18.0% Fines (F2)	12.0	S4 S5
Frozen, gray, silty, fine SAND; Nbe S7: 25.0% Fines (F4)	30.0	S6 S7
Bottom of Boring Boring Completed May 10, 2007	41.5	S8

Symbol

Ground Water Depth, Ft.

Groundwater not encountered during drilling on May 10, 2007



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- B Auger Cuttings
- ▮ Frozen
- ▽ Ground Water Level At Time Of Drilling
- ● Water Content (%)
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Tundra Women's Coalition
Bethel, Alaska

LOG OF BORING B-2

June 2007 | 32-1-01899

SHANNON & WILSON, INC.
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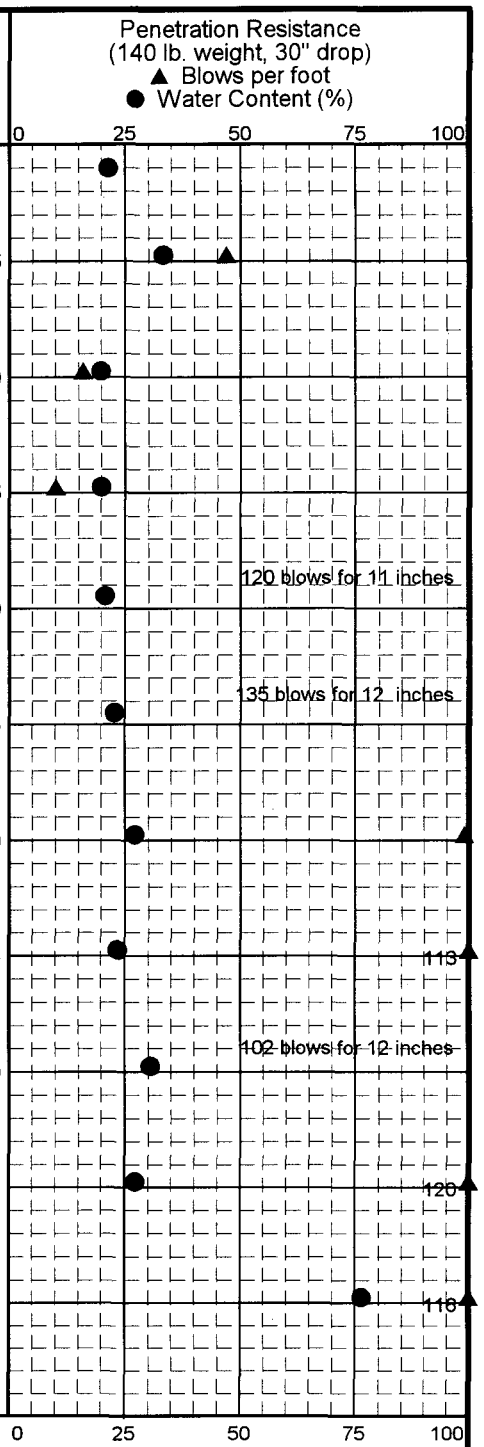
Fig. A-4

GEOTECHNICAL LOG 01899 TWC.GPJ S&W GEO1.GDT 7/2/07

MATERIAL DESCRIPTION Approx. Elevation: 0.5 ft below 6th Street	Depth, Ft.	Symbol	Samples	Ground Water Depth, Ft.
Medium dense to frozen, brown, silty, very fine SAND; moist to frozen [Nbn]; [FILL] -frozen below 3.0ft			S1	
S1: 0% Gravel, 69% Sand, 31% Fines (F4)	4.5		S2	
Frozen, reddish brown PEAT [Nbn] -frozen, silty sand layer, 3 in. thick, at 4.6 ft	7.0			
Medium dense, brown, silty, fine SAND; moist -with peat seams at 9 ft			S3	
S4: 25.8% Fines (F4)			S4	
Frozen, gray to brown, silty, fine SAND; [Nbe to Vc]; with frequent peat seams	16.0		S5	
			S6	
S7: 40.2% Fines (F4)			S7	
			S8	
			S9	
-visible ice cristals scattered below 44 ft			S10	
S11: 22.4% Fines (F4)	50.5		S11	
Bottom of Boring Boring Completed May 12, 2007				

Groundwater not encountered during drilling on May 12, 2007



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- B Auger Cuttings
- ▼ Ground Water Level At Time Of Drilling
- ■ Frozen
- ● Water Content (%)
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
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Tundra Women's Coalition
Bethel, Alaska

LOG OF BORING B-3

June 2007 | 32-1-01899

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. A-5

GEOTECHNICAL LOG 01899 TWC.GPJ S&W_GEO1.GDT 7/2/07

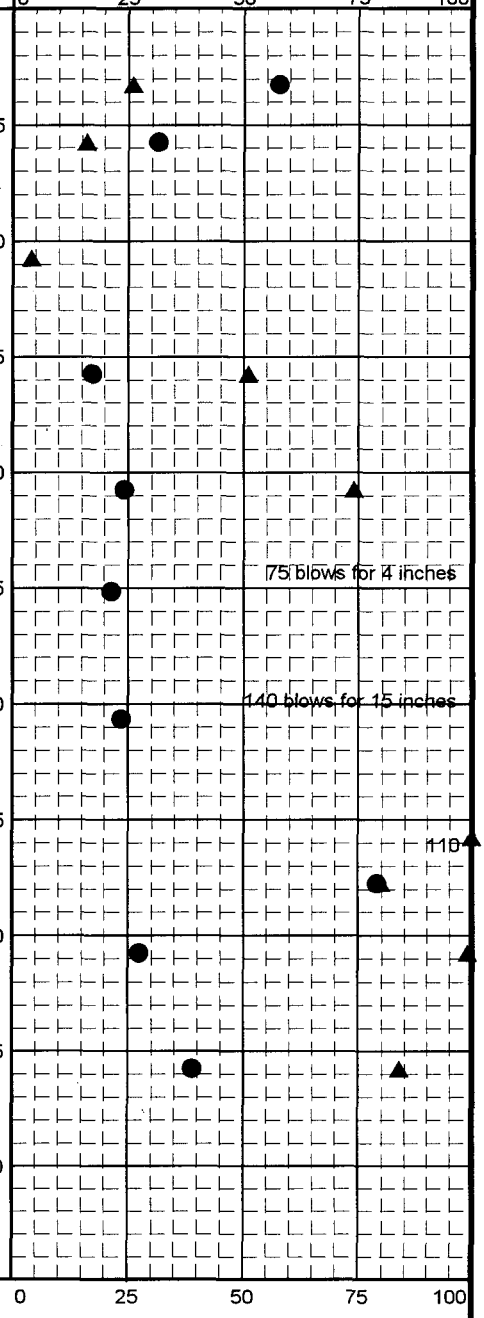
MATERIAL DESCRIPTION Approx. Elevation: 1.5 ft below 6th Street	Depth, Ft.	Samples
Medium dense, brown, silty, fine SAND; moist to wet [FILL]		
Frozen, dark brown PEAT; [Nbn] -frozen ash layer, 2 in thick, at 3.1 ft -frozen, sandy silt layer, 2 in thick, at 3.3 ft	3.0 4.5	S1 S2
Frozen to loose, brown, silty, fine SAND to sandy SILT; moist S2: 0% Gravel, 46% Sand, 53% Fines (F4) -frozen to 5.3 ft [Nbn]		S3 *
Frozen, brown, silty, fine SAND; [Nbe to Vc] S5: 39.3% Fines (F4) -Visible crystals [Vc] below 25ft	15.8	S4 S5 S6 S7
ICE [Vs]; with occasional sand particles	33.5	S8 S9
Frozen, gray to brown, silty, fine SAND; [Vc]; laminated with peat seams S10: 36.7% Fines (F4)	38.0	S10 S11
Bottom of Boring Boring Completed May 11, 2007	46.5	

Symbol

Ground Water Depth, Ft.

Groundwater not encountered during drilling on May 11, 2007

Penetration Resistance
(140 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- B Auger Cuttings
- ▼ Ground Water Level At Time Of Drilling
- ■ Frozen
- ● Water Content (%)
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Tundra Women's Coalition
Bethel, Alaska

LOG OF BORING B-4

June 2007 | 32-1-01899

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. A-6

GEOTECHNICAL LOG 01899 TWC.GPJ S&W GEO1.GDT 7/2/07

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Date: December 2008
To: USKH Inc
Re: Geotechnical Design Study, Tundra Women's Coalition, Bethel, Alaska

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/ environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland