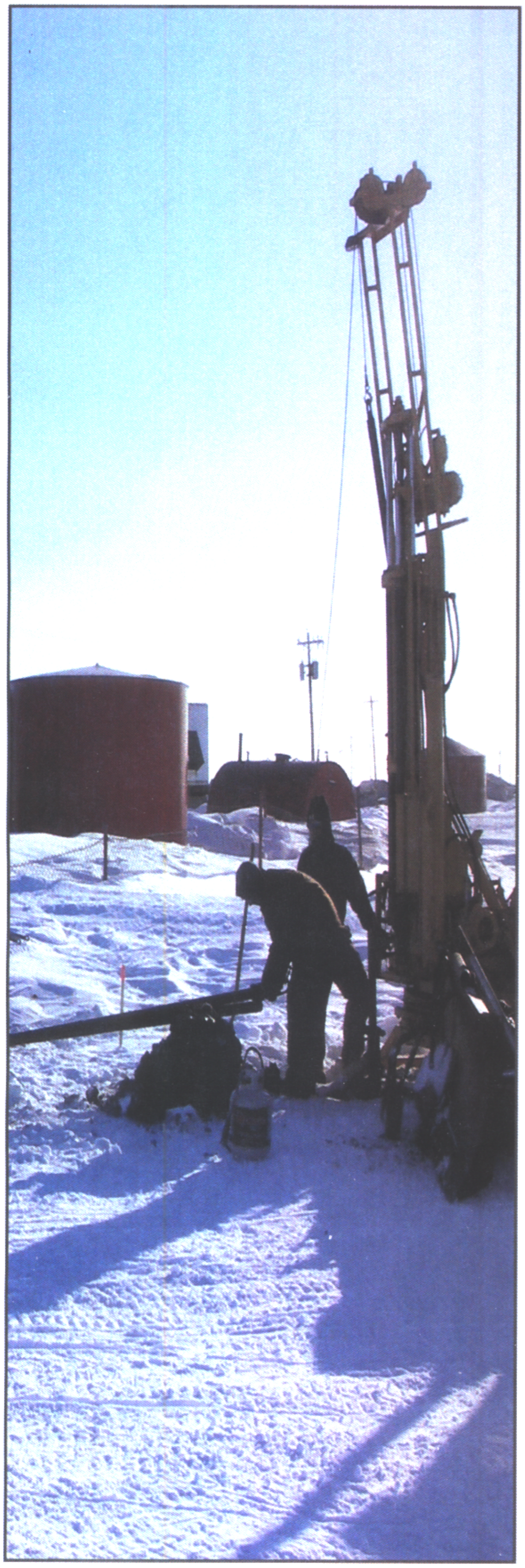

A report prepared for

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Geotechnical Investigation and Engineering Recommendations

Kipnuk Bulk Fuel and Powerplant Facility Kipnuk, Alaska

by

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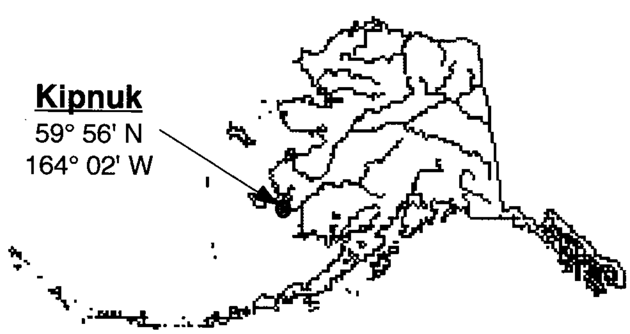
INTRODUCTION

This report presents the results of our soil investigation for the bulk fuel facility and powerplant at Kipnuk, Alaska. The Alaska Energy Authority (AEA), through their prime planning and design consultant, LCMF Inc., is developing program and engineering plans for new fuel storage and primary power generation capacity at Kipnuk. Three major elements for this project include (1) a new bulk fuel tank facility with integral secondary containment, (2) a new diesel piston powerplant either integral with the bulk fuel facility or as a separate structure adjacent to the existing powerplant, and (3) fuel pipeline(s) connecting the fuel tanks to the powerplant. The proposed Kipnuk Light Plant bulk fuel facilities will be located northwest of the existing power plant (Plate 1).



Duane Miller Associates LLC (DMA) conducted a geotechnical site investigation in 1998 advancing three shallow borings near the proposed bulk fuel facility. Based on our 1998 work, preliminary foundation options for the fuel tank and powerplant upgrades were submitted to LCMF. Subsequent to our 1998 site work, a geotechnical investigation was conducted in 2007 including deeper borings and advanced geotechnical laboratory testing to determine soil strength properties. Based on the 2007 findings, the recommended foundation for both the fuel tanks and the powerplant is a driven displacement pipe pile system. Based on preliminary site plans developed by LCMF, the bulk fuel facility containment will be nominally 75 by 150-feet. The largest tanks are vertical with a maximum diameter of 34.5 feet and a maximum height of 18 feet. Estimated initial total fuel volume is approximately 445,000 gallons but expansion capacity is being included at this time.

The existing fuel tanks are founded on fill pads composed of the local silt surrounded by earthen or wooden dikes. The tanks are supported on timber sleepers and show signs of differential settlement. A fuel tank facility for

Kugkaktlik Limited was constructed northwest of the village in 1997 with a soil cement fill pad using the local silt.

This soil investigation was performed in accordance with our proposal to LCMF dated March 7, 2007. The object of the soil investigation was to determine the geotechnical and thermal soil conditions at the primary site and to develop conclusions and recommendations regarding foundation design. During the investigation, we consulted with Mr. Wiley Wilhelm, P.E., Mr. Glen Oen, P.E. and Mr. Joe Daniels of LCMF, Inc. Refinements to the scope of services and project mobilization sharing were coordinated with Mr. David Lockard at AEA and representatives of CVRF in Anchorage. We were assisted in Kipnuk by Mr. Carl Amik and Mr. Paul J. Paul, equipment operators.

The site and boring plan is presented on Plate 1. The inferred logs of the borings are presented on Plates 2 through 4. The soils have been classified in accordance with the Unified Soil Classification System (USCS) described on Plate 5. A summary of sample testing results is presented on Plate 6. Particle size analyses are summarized on Plates 7 and 8. Laboratory secondary strength test results are presented on Plates 9 through 12 for direct shear analysis and Plates 13 and 14 for triaxial analysis. Plate 15 presents summary ground temperature measurements. The results of the moisture content, organic content, sampling blow counts, pore water salinity content and percentage finer than the No. 200 U.S. sieve size are provided on the logs of the borings and in the text in the following section, where appropriate.

INVESTIGATION

Existing data

The investigation started with the collection and compilation of existing data from previous explorations in Kipnuk, including our 1998 site investigation. These data were reviewed in conjunction with historic and recent site aerial photography and review of Google Earth® maps (February and March, 2007) of the Kipnuk area. Summarized findings of prior geotechnical site work are presented below:

1974, Kipnuk High School, Harding Lawson Associates: An investigation for the "old" Kipnuk High School (currently the Kugkaktlik Limited offices) revealed soils consisting of organic silt and silt with some unfrozen brine. The school is founded on timber adfreeze piles with a passive refrigeration system installed in the annulus. Subsequent differential movement of the foundation resulted in an additional investigation by A.W. Murfitt, P.E., in 1983. Discontinuous permafrost was found within the area with the frozen ground having temperatures between 30°F and 32°F. Some visible ice was evident in the permafrost and measured pore water salinity ranged from 2 to 7 ppt (parts per thousand).

1988, K-12 School, Shannon and Wilson, Inc.: A subsurface investigation for the Chief Paul Memorial K-12 School consisted of 4 borings drilled across the site. The borings encountered surficial peat underlain by organic silt and silt with fine sand. All the borings encountered frozen soil with ground temperatures measured from 29°F to 30°F. Unbonded soil was encountered and pore water salinity contents ranged from 1 to 8 ppt. The school is founded on passive refrigeration piling and some differential movement has been observed since construction.

1996, Village Corp. Fuel Facility, DMA: A geotechnical investigation was conducted for the Kugkaktlik Limited tank farm northwest of the village. Site work consisted of shallow test pits advanced into a fill pad constructed of the local silt placed in 1995. The investigation found silt containing organic material in the fill overlying a peat and silt subgrade. Frozen ground was encountered below the active layer. Pore water salinity contents measured between 4 and 26 ppt. The tanks were founded on a fill pad of the local silt treated with Portland cement and underlain by a layer of board insulation and deeper, thermal syphons to maintain the permafrost.

1997, Airport Project, DMA: A geotechnical investigation for the new airport southeast of the village consisted of fourteen borings drilled to investigate the alignment and a possible material site on a point bar upriver of the village. The borings revealed peat underlain by organic silt and silt. Some fine sand was encountered in the silt at deeper depths. Discontinuous permafrost was encountered over the areas explored with unfrozen ground encountered in lower areas of drained lakes and near bodies of water. Pore water salinity contents measured between 1 and 16 ppt.

1997, Water and Sewer Project, DOWL Engineers: A geotechnical investigation was conducted for the U.S. Public Health Service for construction of a water reservoir and sewage lagoon. Two borrow source locations were investigated: northeast of the existing airport and southwest of the village. Twenty-five borings were drilled to a depth of 20.5 feet each. The borings revealed peat underlain by organic silt and silt. Some fine sand was encountered in the silt at deeper depths. Discontinuous permafrost was encountered with some visible ice in the frozen silt. Thaw consolidation tests of the frozen soil showed deformations from 2% to 55%.

1998, Bulk Fuel and Powerplant Upgrade, DMA: Five borings were advanced at two sites (Light Plant site (3 borings) and School District site (2 borings)) in order to develop preliminary foundation recommendations for a new bulk fuel and powerplant facility. All borings were advanced to approximately 30-feet below grade. The borings at the Light Plant site encountered organic silt to 4 to 8 feet below grade then wet, medium stiff inorganic silt below the organic silt. None of the three borings advanced at the Light Plant site encountered frozen soil conditions, except for seasonal surface frost. Pore water salinities ranged from 3 to 19 ppt.

2007 Site Investigation

Two borings to 65 to 70-feet deep were initially planned for the new bulk fuel facility. The purpose of the deeper borings within the area investigated by DMA in 1998 was to confirm subsurface conditions for a pile foundation option. To contain mobilization costs, a skid mounted drill rig was used for the site investigation work using the village corporation dozer to move the rig within the village. If the village excavator was capable, four shallow test pits were also planned for the area surrounding the proposed bulk fuel facility footprint to determine the depth of organic material. Prior to field work, boring locations were determined relative to existing landmarks (tanks, building, etc.) based on preliminary facility layout plans prepared by LCMF (Sheet C-1, Conceptual Design Report, February 06, 2007).

Subsurface Exploration

The field work included a reconnaissance of the project site and exploration of subsurface conditions conducted March 23 through March 27, 2007 by DMA. Exploration included drilling and logging the originally planned two exploration borings. A third boring was completed to confirm the subsurface thermal state. The three test borings were drilled to depths ranging from 48 to 67.5 feet below grade and soil samples were collected at nominal five-foot intervals, unless soil conditions prohibited obtaining representative samples.

Hughes Drilling Service, Inc. of Soldotna, Alaska using a high-torque skid-mounted CME 45 drill rig performed exploration drilling. Borings were advanced with a hollow-stem auger. Soil samples were collected using split barrel and Shelby Tube sampling methods. The drill rig was moved using the village corporation John Deere 450G dozer, operated by local residents. The village equipment was in generally poor operating condition and required continual maintenance by the drilling contractor and village personnel during field work.

DMA field geologist and engineer were Nathan Luzny and Jeff Kenzie, respectively. Patrick Smith and Jason Bussdieker of Hughes Drilling Service were drill rig operators.

The borings were logged and sampled as they were drilled. The sampling was performed using a variety of sampling methods, depending on soil thermal state and anticipated laboratory testing requirements:

- 1.4-inch ID/2.0-inch OD split barrel sampler without brass liners advanced 18-inches with a 140-lb autohammer freefalling 30-inches used for disturbed sampling in frozen soils
- 2.0-inch ID/2.5-inch OD split barrel sampler with brass liners advanced 18-inches with a 140-lb autohammer freefalling 30-inches used for disturbed sampling in unfrozen (thawed) soils
- 3.0-inch OD thin wall Shelby Tube sampler advanced approximately 24-inches with the drill rig hydraulics used for undisturbed sampling in unfrozen (thawed) soils.

The field blow counts to advance the split barrel sampler were recorded at time of sample collection and are noted on our boring logs as drive blow per each 6-inch interval. Samples from the borings were visually classified in the field at time of drilling and representative portions from unlined sampler sealed in double polyethylene bags to maintain soil moisture. Brass liner and Shelby Tube samples were visually classified from exposed end material at time of drilling and sealed with plastic caps and tape to maintain soil moisture.

In-situ soil strengths were measured on recovered brass liner and Shelby Tube samples were with hand-held Pocket Penetrometers (PP) and Torvane (TV) tools. Field measurements from these hand-held tools are presented on the boring logs and summary strength data are discussed in the following section. PP and TV soil strength measurements were also obtained on select laboratory samples.

Neither of the split barrel sampling methods used for this project are the Standard Penetration Test (SPT) method (1.4-inch ID sampler used with a 140-lb cathead drive hammer free falling 30-inches). Thus, field recorded blow counts required numeric adjustment to derive the SPT "N" values. Details of this adjustment are summarized in the following section.

The locations of the borings were established by measuring from existing structures (tanks, buildings, etc.) identified on the conceptual design drawings provided by LCMF. The proposed layout of the fuel system upgrade and the boring locations conducted by DMA for this effort and our 1998 site work at this site are shown on Plate 1. Horizontal control (latitude/longitude) to WGS 84 datum using hand held Garmin eTrex GPS instruments and are noted on the boring logs. Vertical elevations determined with a Garmin eTrex were recorded but should be interpreted with caution owing to inherent GPS equipment accuracy.

Boring TH-1 was completed with both a glue-jointed, sealed 1-inch nominal diameter Schedule 40 PVC pipe for temperature measurement to approximately 65-ft below grade and a glue-jointed, slotted PVC pipe for groundwater depth measurement. TH-2 was completed with a glue-jointed, sealed 1-inch nominal diameter Schedule 40 PVC pipe to approximately 50-ft below grade for ground temperature measurement. All three borings were backfilled with cuttings to grade and marked with survey flagging.

Laboratory Testing

In the laboratory the samples were re-examined to confirm the field classification and to select samples for classification and secondary shear strength testing. The laboratory classification testing included:

- Moisture content (MC)
- Organic content (OLI)
- Moisture/density determination (Dd)
- Particle size analysis (SA, P200)
- Pore water salinity content (Salinity)

Laboratory secondary strength testing conducted on select samples included:

- Direct Shear (DS)
- Unconsolidated, Undrained Triaxial Testing (TXUU)

Two samples (TH-1@17.0-ft and TH-2@ 39.0-ft) were archived in the event additional laboratory testing is necessary. All geotechnical laboratory testing was performed in our Anchorage laboratory. Laboratory test results and data interpretation are presented in the following section.

SITE AND SUBSURFACE CONDITIONS

Geologic Setting

The community of Kipnuk is located in the Yukon-Kuskokwim Lowlands, on the Kuguklik River about three miles from the coast of the Bering Sea. The village is about 100 miles southwest of Bethel. The U.S. Geologic Survey maps the area as undifferentiated surficial deposits consisting of marine, river and deltaic sediments.

The Kuguklik River is a meandering stream that extends about 30 miles east into the flat tundra and lakes complex. Large flows occur in the river due to tide variation and the village is located on an actively eroding bend. Previous studies by DMA show rates of erosion into the bank at the main area of the village of about 7 feet per year in the 30 years up to 1984.

The area around Kipnuk is flat and poorly drained with numerous small to large lakes and small drainages that flow into the Kuguklik River. The elevation of the village is about 10 feet above mean sea level. The tide range in the river is about six feet. The village area can flood if storm surges occur with high tide cycles. Area soils consist of peat over organic silt and silt with fine sand. Permafrost conditions vary widely in the community. The permafrost soils in Kipnuk are relatively warm and contain dissolved salts.

Kipnuk has a subarctic marine climate typified by cool summers and moderately cold winters. Temperatures range between -6°F and 57°F. The village averages 43 inches of snowfall per year and a total of 22 inches of precipitation annually. Snowfall generally begins in early October and ends in late April to mid May but is heavily influenced by winter pack ice and strong winds along the Bering Sea.

Winter conditions appear to be getting warmer. In 2004 we updated our 1999 climate records for 15 different weather stations in Alaska, including Bethel. At all stations we found a significant warming after 1977 and for Bethel the average temperature after 1977 is 2.3° F higher than for the 30 years before 1978. The following table compares climatic data for the Kipnuk area from the *Environmental Atlas of Alaska*, by Hartman and Johnson, 1978, with current design values that we recommend for the village based on our 2004 climate update:

	H&J, 1978	DM&A, 2004
Average Air Temperature	30.0°F	32.3°F
Average Freezing Index	2500°F days	2360° F-days
Design Freezing Index	3700°F days	3420° F-days
Average Thawing Index	2000°F days	2400° F-days
Design Thawing Index	3200°F days	3400° F-days

Based on our revised climatic data, permafrost in some sections of Kipnuk should be considered to be in a degrading thermal state, particularly in lower lying micro-relief areas.

Subsurface Conditions

The proposed location for the new bulk fuel facility is north of the existing powerplant and bulk fuel facility. The area was covered with 1 to 2 feet of hard packed snow during our investigation in March 2007. Three borings (TH-1 through TH-3) were drilled at the location, roughly along the long axis centerline of the proposed facility footprint, Plate 1.

The March 2007 borings at the facility footprint encountered frozen peat and organic silt to depths of 3 to 8 feet and then a non-plastic, wet to saturated medium stiff to stiff mineral silt to approximately 38 to 41-feet below grade. In all three borings, a saturated medium dense to dense fine sand was encountered beneath the silt layer to the depths explored.

Laboratory testing on soil samples from the fine sand layer below approximately 40-feet indicate the material would technically meet the Unified Soil Classification System (USCS) determination for a silty fine sand (SM), a fine sand with 12-% or greater fines content. However, the tested fine sand soil was generally near a borderline classification of a silty fine sand (SM) and a poorly graded fine sand (SP); fine sand with 5 to 12-fines content. Based on engineering behavior of the tested fine sand soil samples, the borderline classification (SM-SP) is assigned to this fine sand material, unless otherwise noted on the logs of the borings.

In all three March 2007 borings, the fine sand was saturated and exhibited considerable heave during drilling. Heave is a condition where the saturated fine sand loses shear strength due to drilling disturbance and is subjected to hydraulic pressure differential between the interior and exterior of the hollow stem auger. This condition resulted in 5 to 20-feet of fine sand rising inside the hollow stem auger relative to the drill bit elevation. The fine sand heave inside the hollow stem auger required the driller to use a variety of methods to work the sampler to the desire sample interval to collect a disturbed but representative *in-situ* soil sample of the material ahead of the drill bit. At several sample intervals, a reduced sample volume was recovered or a sample of *in-situ* material was not possible. These constraints are noted on the logs of the borings.

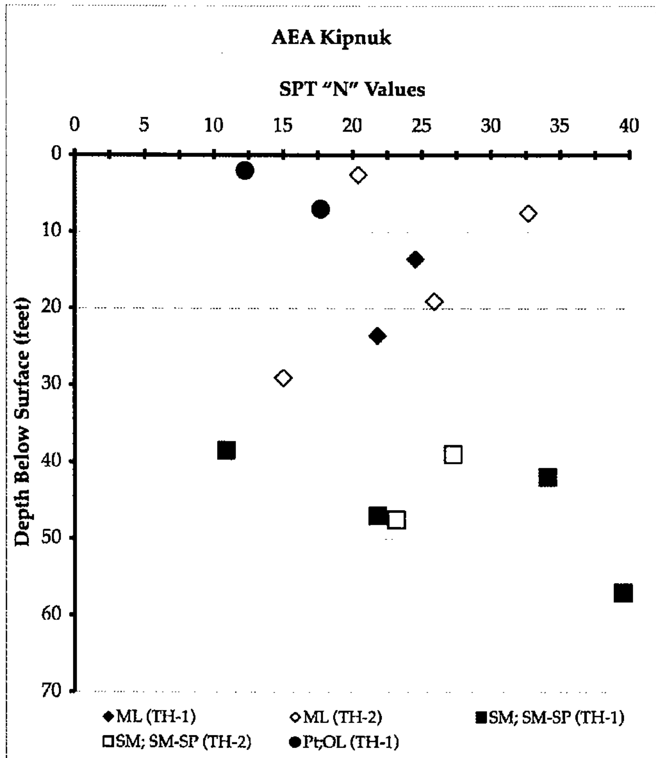
In boring TH-1, a silty fine sand layer is inferred from 47 to 52-feet below grade, with gradational contacts with the fine sand inferred on both the upper and lower boundaries of this silty fine sand layer. In boring TH-2 between the overlying silt and the underlying fine sand, thin stringers of organic silt were noted between 36 and 41-feet below grade. No organic material was encountered in the underlying fine sand layer to termination depth.

Groundwater was observed at approximately 20 feet below grade in all three March 2007 borings but will be expected to rise to near the existing ground level. For geotechnical design purposes we have assumed groundwater will be near the ground surface and that the site would be subject to occasional flooding. Flood level, however, will remain below the base of the bulk fuel containment finish floor elevation.

Our three March 1998 borings advanced in this area (B-3, B-4 and B-5) encountered similar materials as our March 2007 borings (3 to 8 feet of surface peat and organic silt with inorganic, non-plastic wet, medium stiff silt to termination depths, approximately 30-feet below grade. None of our March 1998 borings encountered the fine sand found at approximately 40-feet below grade in our March 2007 site explorations.

Surface frost was encountered to approximately 7 to 8.5 below grade in the three borings advanced in March 2007 but was thinner (to 3.5 to 4.5 below grade) in our March 1998 borings. At 7 to 8.5 feet below grade, the surface frost is considered to be deeper than normal seasonal frost penetration (3 to 4-feet) and most likely represents a deeper frost penetration from recent cooler periods. No permafrost was found below the surface frost in any of our March 2007 and March 1998 borings advanced at this location.

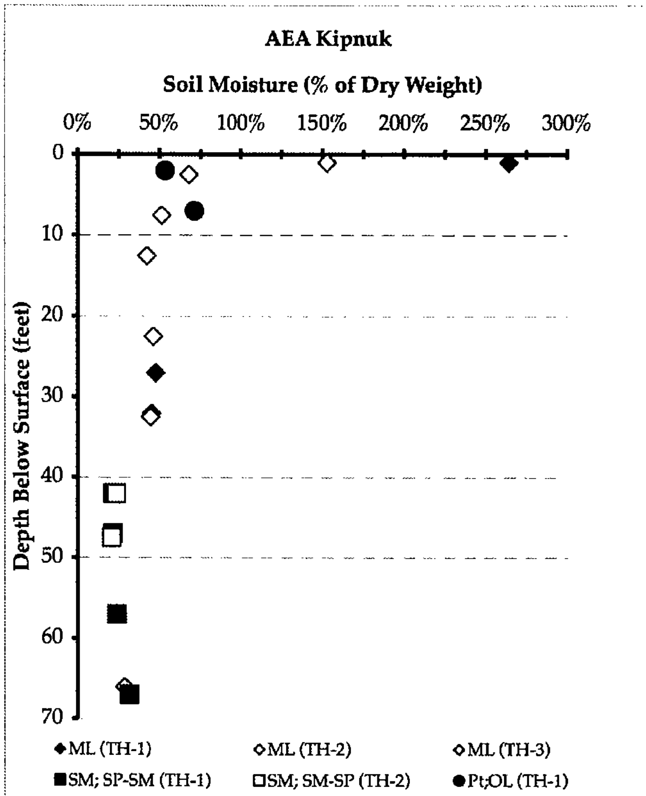
As discussed in the previous section, field blow counts required adjustment for both hammer energy (autohammer versus rope/cathead) and sampler size (2.0-in ID versus 1.4-inch ID) to derive SPT "N" values. Field blow counts to drive the sampler each 6-inch interval is recorded on the boring logs. The field recorded blow counts required to drive the sampler the final 12-inch interval have been adjusted to represented SPT "N" values, summarized below.



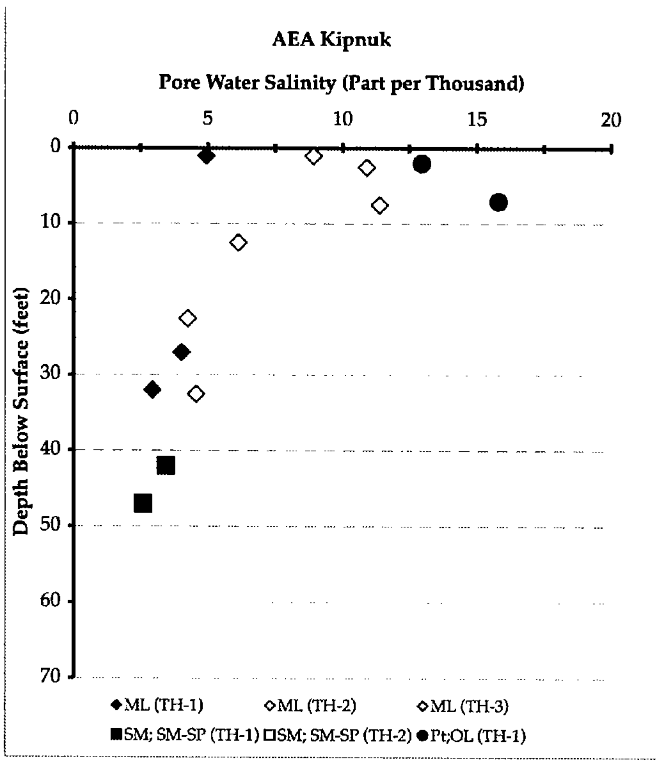
SPT "N" values are a commonly used index of soil density/consistency and are used for geotechnical engineering analyses. As noted on the graph to the right, slightly larger soil density/consistency is evident at approximately 40-feet below grade, the silt/fine sand contact. SPT "N" values in the unfrozen silt layer between 8 to 41 feet below grade generally ranged from 15 to 34 and averaged about 23 blows per foot. SPT N-values in the unfrozen fine sand layer below the silt generally ranged from 11 to 39 and averaged about 26 blows per foot. At these "N" values, the silt is considered medium stiff to stiff and the fine sand layer is considered medium dense to dense.

Soil moistures as a percent of dry weight are summarized in the graph to the left. Average soil moisture for the inorganic silt and underlying fine sand were 62-% and 23-%, respectively. At these soil moisture contents, the inorganic silt is considered wet to saturated and the underlying fine sand as saturated. Field observation of recoverd soil samples and groundwater levels measured at time of drilling appear to support the laboratory soil moisture values.

Pore water salinities were measured in nearly all recovered samples and reflect a trend of decreasing pore water salinity with depth in borings TH-1 and TH-2. In permafrost areas pore water salinity depresses the freezing point in the soil and can significantly impact the soil thermal state and foundation engineering design for permafrost areas, particularly in 'warm' permafrost areas.



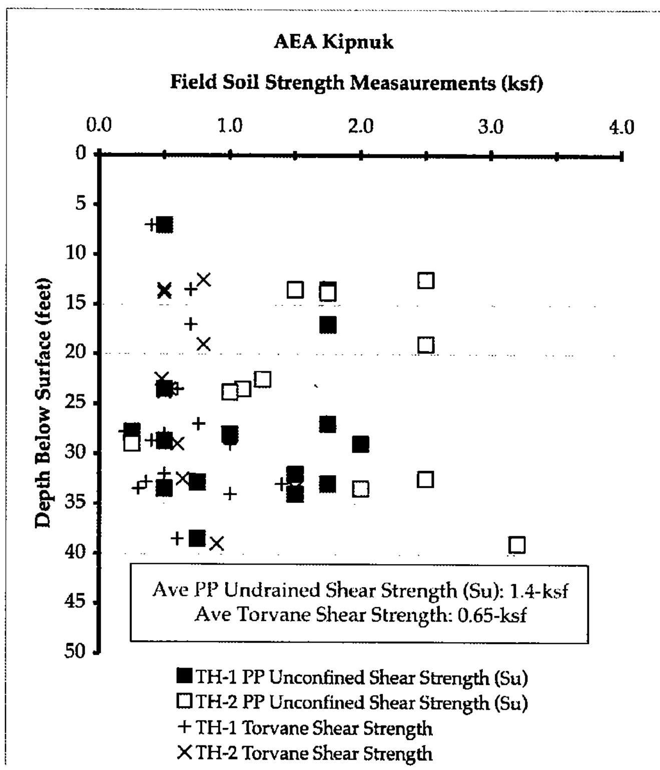
Since the key foundation bearing soils at this site (inorganic silt and underlying fine sand) are unfrozen, freezing point depressions are not a significant foundation design consideration. However, pore water salinity may present a longer-term corrosion concern.



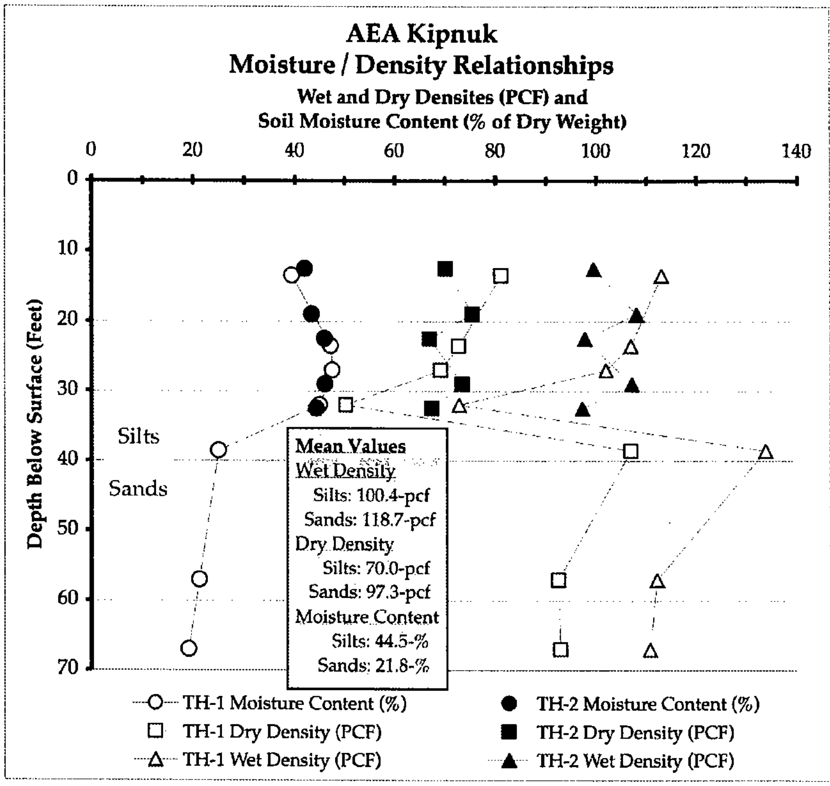
As noted on the graph above, average pore water salinities below the surface frost layer are generally below 5 parts per thousand (ppt) and appear to be significantly greater in the uppermost 5 to 8-feet. This may be a result of periodic saltwater intrusion during wind blown high tide flooding or possibly are resultant of salt concentration due to seasonal frost.

Field measurements of soil shear strength were collected at time of drilling and during geotechnical laboratory testing using a hand-held Pocket Penetrometer and Torvane instruments. Pocket Penetrometer testing provides an indication of the unconfined, undrained shear strength of soil and Torvane testing provides a direct measurement of undrained shear strength of the soil sample. Soil strengths based on hand-held instruments may be subject to a wide range of variation and should be carefully considered when used for engineering design.

However, trends in undrained soil strength can be inferred from theses data. In the graph provided to the left, both Pocket Penetrometer (PP) and Torvane (TV) shear strength data are summarized. Torvane data indicate average shear strength on the order of 0.65-kips per square foot (ksf) and Pocket Penetrometer average shear strengths on the order of 1.4-ksf. Note that only the inorganic silt samples were measured for field shear strength values, the fine sand samples were retained for secondary direct shear testing.



Moisture/density testing was conducted on 12 samples recovered from borings TH-1 and TH-2 to determine wet and dry density of the inorganic silt and underlying fine sand. As summarized on the graph to the right, average dry density of the inorganic silt is approximately 70-pounds per cubic foot (pcf) increasing to approximately 97-pcf in the fine sands. Respective wet densities are approximately 100 and 120-pcf.



Four samples from either brass liner split barrel sampler or an auger grab sample had Direct Shear secondary soil strength analyses. These samples are primarily from the saturated sandy material:

- TH-1 @ 38.5-ft (brass liner)
- TH-1 @ 57-ft (brass liner)
- TH-1 @ 67-ft (grab sample, remolded analysis)
- TH-2 @ 29-ft, (brass liner)

The Direct Shear test provides an estimate of the internal angle of friction (ø) and cohesion (c) assuming a direct shear plane through the soil sample under varying loading states. Based on testing results, internal friction angles (ø) were found to be on the order of 30 to 35 degrees and a remolded sample ø of approximately 26-degrees, Plates 9 through 12. The fine sand soil is considered non-cohesive.

Two Unconsolidated, Undrained Triaxial (TXUU) analyses were conducted on the siltier materials recovered from either a brass liner split barrel sample or thin wall Shelby Tube sample for shear strength analysis:

- TH-1 @ 27-ft (brass liner)
- TH-2 @ 32.5-ft (Shelby Tube)

The TXUU test provides an estimate of the internal undrained shear strength (Su) of the samples under varying loading states. Undrained shear strengths (Su) on the order of 600 to 750 pounds per square foot (psf) were determined, Plates 13 and 14.

Subsurface temperatures were collected at TH-1 and TH-2 in June, 2007 by Roy Paul, Jr. of United Utilities, Inc. At each boring, temperatures were recorded at selected depths using a Beaded Stream™ thermistor system and were plotted for comparison, Plate 15. For each location, a thermistor string was placed in a pre-installed 1-inch diameter PVC pipe and left untouched for more than one hour to allow for equilibration with ambient air temperatures. At shallow depths, ambient soil temperature is influenced by air temperature. During data collection, air temperature at the project site was approximately 67°F. As depth below ground surface approaches 0.0 feet, subsurface temperature approaches air temperature. Observed temperatures in each boring vary inversely with depth below ground surface to a depth of approximately 16 feet. Below this depth, ambient subsurface temperatures remain above 32°F and fluctuate with depth on a magnitude of 0.1°F. At both boring locations, soil is unfrozen to depth below approximately 20 feet. This confirms that the proposed stratum of sand is unfrozen, and will provide suitable support for a displacement pile foundation system.

DISCUSSION AND CONCLUSIONS

Geotechnical recommendations for the three key components of this project (bulk fuel tank, powerplant and pipeline) are addressed below. Construction sequencing and methodology recommendations follow our geotechnical recommendations. Finally, construction observation recommendations are provided.

For the bulk fuel facility, conceptual-level structural engineering calculations have determined short-term pile loads in the range of 65 to 70-kips. An estimated maximum load of slightly greater than 66-kips was provided for design purposes. Sustained loads are slightly lower than short-term loads, thus short-term loading conditions were used for foundation design. The powerplant axial loads per pile are lower, in the range of 50 to 55-kips for both short term and sustained loading conditions. Based on observed site conditions and discussions with the civil and structural engineering team, we recommend the bulk fuel facility and powerplant be constructed as a pile supported structure.

An engineered fill section was considered in our 1998 conceptual report for this area. The 1998 conceptual report suggested an inorganic silt core for the fill section placed as a surcharge over the organic soils to accelerate consolidation. After consolidation, the inorganic silt would be regraded and capped with an armor material. In our 1998 recommendations, a surcharge equal to the weight of a full 14-foot high tank was suggested, requiring an additional 8 feet of fill placed above the grade of the fill pad where the tanks will be supported.

Based on geotechnical properties encountered below the depth of our 1998 borings and AEA's preliminary project scheduling, a pile-supported structure is recommended for the bulk fuel facility and the powerplant. A pile-supported structure will eliminate fill placement and an imported armor cap as well as eliminate the surface organic consolidation lag period prior to installation of top-side elements.

Bulk Fuel Facility

Based on pile capacity methods developed by the US Naval Command Facilities (NAVFAC DM-7) and the US Army Pile Design Manual (EM-1110-2-2906), a 14-inch diameter displacement-type (closed end) pipe-pile driven to at least 55 feet below existing grade is expected to develop an allowable per-pile capacity of 66-kips with a factor of safety of at least 3 for the bulk fuel facility. A 1/3 increase in this allowable capacity is permitted for short-term transient loading conditions. Displacement piles are recommended. For this site, displacement piles are expected to develop both adhesion (skin friction) and end bearing. Displacement pile should penetrate at least ten (10) feet into the saturated fine sand, and a 15 foot penetration into the fine sand is preferred. No special bond break is needed in the active layer zone if the piles are installed to at least 55 feet below existing grade. If the piles are driven in the winter when the surface is frozen, a pilot hole of the pile diameter may be necessary through the frozen material.

Tension capacity will be significantly less than axial compression capacity. Assuming a maximum uplift load of no greater than 10-kips under full buoyant conditions for the 14-inch pile, resistance to uplift will have a factor of safety of at least 3 at a 55 foot embedment.

Powerplant

Conceptual site development plans include a powerplant adjacent to the existing generation facility. If so, the new powerplant will be a separate structure from the bulk fuel facility and would require a fuel pipeline system connecting the two facilities. Preliminary engineering layout indicates a nominal 36-ft by 48-ft above grade structure for the powerplant. The powerplant is expected to house four piston generators, day fuel storage tanks, and electric switch gear. If a pile supported structure is used, preliminary per pile loads in the range of 55-kips are expected.

Site specific explorations were not conducted within the proposed powerplant footprint adjacent to the existing generation facility under this scope of work. However, boring B-3 from our 1998 site work appears to be relatively near the existing generation facility. Boring B-3 encountered subsurface materials similar to those encountered during this exploration effort, a medium stiff to stiff inorganic silt to approximately 30-feet below existing grade. Surface peat and organic silt thickness at boring B-3 was similar to that encountered at the bulk fuel facility, approximately 7 to 8 feet thick with seasonal frost extending to 4 feet below grade (in 1998).

Assuming a saturated, unfrozen fine sand similar to that encountered at approximately 40-feet below grade at the bulk fuel facility site is present at the proposed powerplant site, a pile supported structure for the new powerplant is recommended.

A 12-inch closed end (displacement) pipe pile driven to at least 55 feet below existing grade is expected to develop a 55-kips capacity with a factor of safety of at least 3, including allowing 1/3-inch for short-term transient loading conditions. As with the bulk fuel facility pile foundation recommendations, the powerplant piling should penetrate at least ten (10) feet into the saturated fine sand and fifteen (15) foot penetration into the fine sand material is preferred. Likewise, a bond break is not considered necessary to reduce seasonal frost forces if a 55-ft embedment is attained. Pre-drilling may be necessary to facility pile driving if seasonal frost is present at time of pile installation.

Settlements are expected to be similar to those estimated for the bulk fuel facility. Lateral capacity and pile deflection can be refined once lateral loadings and above grade riser elevations are provided for the powerplant. The axial capacity discussed above will require refinement by DMA as the powerplant layout and loading configuration is refined.

Fuel Pipeline

Assuming the powerplant is placed outside the bulk fuel system, fuel pipeline(s) between the bulk fuel facility and the powerplant will be necessary. Pipelines are expected to impose a low axial load. If pipelines are pile supported, frost uplift forces will control pile embedment. We recommend pipeline piles, if used, be installed to at least 40-feet below grade to resist frost uplift forces depending on lateral loads. Open-end pipe pile or HP pile should are suitable for foundations. We expect the structural analysis will define the required pipeline pile dimensions.

Lateral Capacity

Preliminary calculations indicated a lateral load at the pile cap of approximately 10-kips can developed for the bulk fuel facility and we have assumed a similar lateral loading for the powerplant for preliminary design. We have assumed the pile caps will extend approximately five (5) feet above grade.

The lateral loads on the piles will be resisted by the mineral soils below the surface organic layer. The lateral loads will be resisted by passive soil pressures developed against the 14-inch pipe shaft by the mineral soils. The surface organic soils will provide minimal lateral resistance, except when these soils are frozen.

For design purposes, for the 14-inch diameter pipe we have assumed the following conditions:

- Moment of inertia of approximately 373 in^4
- Pipe is free to rotate at the pile cap
- Pipe is laterally loaded to 10 kips [assumed design lateral load per pile]
- Lateral load is at pile cap, approximately 5-feet above grade
- Organic silts are not removed

Based on these assumptions, we estimate the deflection at the ground surface will be approximately 1-inch. The point of fixity can be assumed to average approximately 7 feet below grade. Lateral load and deflections were determined following methods developed by both Matlock & Reese and NAVFAC DM-7.

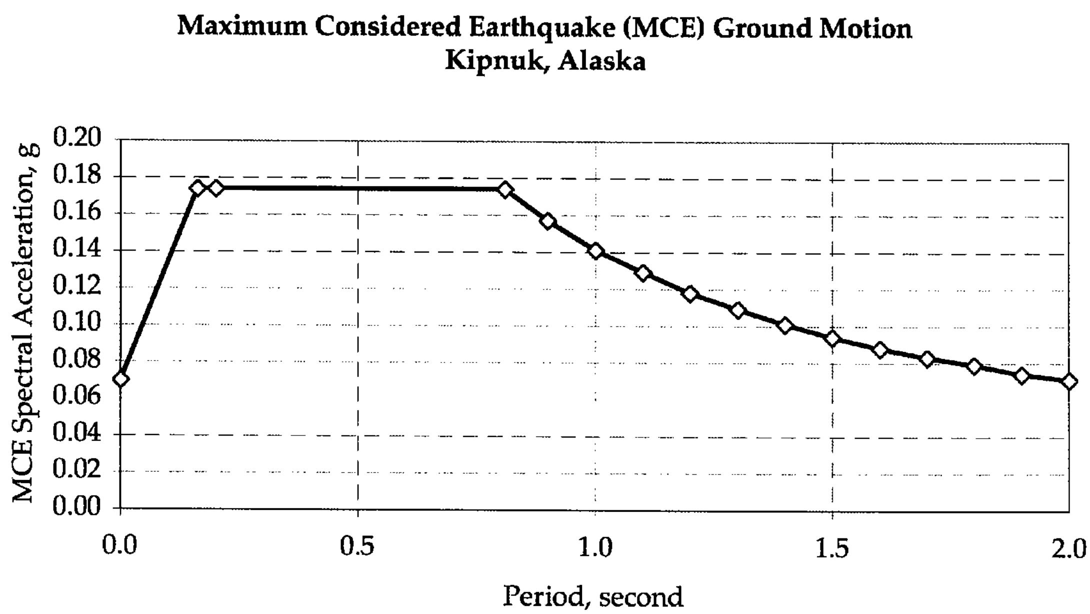
The strength and deformation of the unfrozen ground will control the lateral capacity and structural deflections and the uppermost 5 to 8 feet of the site soils will be soft to very soft when unfrozen. Pile deflections at ground surface will be reduced if the pile is fixed against rotation compared to a pile that is free to rotate. Methods such as above grade diagonal bracing can be considered to reduce pile rotation. If diagonal bracing is considered, it should be kept above ground level so that frost heave forces are not developed along non-vertical members.

Settlement

Pile settlement is generally developed by several elements including the elastic compression of the pile due to the imposed load and settlement of the soil under the pile tip due to the imposed load at the pile cap. An additional component may include settlement due to loads imposed along the pile shaft (negative skin friction). Negative skin friction may result from a variety of influences including thaw consolidation of frozen soils, surcharge consolidation resulting from loads imposed on the ground surrounding the pile, and other factors. We understand site development will not result in consolidation (surcharge) settlement but that some contribution of negative skin friction as the uppermost non-seasonally frozen material thaws is expected. However, the largest component of pile settlement will be due to loading at the pile cap. We estimate total settlement to be less than 1.5-inch for the bulk fuel facility with less than 0.75-inch differential over a 20-year design life, if piles are installed in accordance with our recommendations.

Geohazards

The area is considered a relatively low seismic risk hazard area. Maximum considered earthquake (MCE) ground motions are summarized below based on International Building Code (IBC 2000) and US Geological Survey (USGS) databases. Spectral parameters for Site Class "D" relatively firm to stiff soils with blow counts (N values) between 15 and 50 are provided below:



Mapped Spectral Accelerations (0.2 sec short period (Ss) and 1 sec period (S1))

Period (sec)	MCE Sa (%g)	
0.2	10.9	MCE Value of Ss, **Site Class B**
1.0	5.9	MCE Value of S1, **Site Class B**

Spectral Parameters **for Site Class D**

Period (sec)	MCE Sa (%g)		
0.2	17.4	$S_{MS} = FaSs$	Fa = 1.6
1.0	14.1	$S_{M1} = FvS1$	Fv = 2.4

Design Spectral Response Acceleration Parameters **for Site Class D**

Period (sec)	MCE Sa (%g)	
0.2	11.6	$S_{DS} = 0.667\ S_{MS}$
1.0	9.4	$S_{D1} = 0.667\ S_{M1}$

The general design spectral response acceleration (Sa) will require determination of the fundamental period of the specific structure.

Bulk Fuel and Powerplant Pile Foundation Considerations

Pore water salinity is present in the near surface soil at elevated concentrations. If corrosion is considered a concern, the initial five (5) feet of the pile below ground level may be provided with corrosion protection without significantly reducing axial capacity.

Pile should be installed vertical and no closer than eight (8) pile diameters (approximately 9 feet, center to center) to avoid the potential for group effect. If piles are closer than four (4) pile diameters (center to center), group interaction may control axial capacity. We should be contacted to verify our capacity analysis if pile-to-pile spacing of less than six (6) pile diameters is being considered.

The potential for pile heave (float) due to driving adjacent pile is considered remote. However, if pile float should occur, a staggered pile installation pattern should be implemented. Any pile that should float can be re-struck to depth and re-seated to develop axial capacity.

Pile wall thickness will be a function of the drive hammer used for installation, lateral stiffness requirements and other factors. We have assumed a Standard wall thickness of at least 0.375-inch will be used but a higher-grade steel or thicker-wall section may be required pending actual drive hammer used and the lateral capacity requirements. We recommend conducting a WAVE equation analysis as part of the pile section analysis once structural and constructability reviews are completed. We can conduct this analysis as part of our design review process.

Installation of the driven piles should require a crane, leads and a pile hammer with adequate energy. We recommend installation equipment be suitable for installing each pile without the need for splicing. End plates will be required for all displacement piles. End plates may be flat plates or conical points. Generally, conical points have sixty degree configurations and are available with an inside flange. Conical points generally cost more than flat plates. End plates should be thick enough to resist all driving stresses. Plates must be flush with the pile outside diameter and not extend outside the pipe pile for welding. A flush fit end cap is needed to not impact the adhesion capacity.

Based on recommendations developed by the US Army Corps of Engineers (Pile Driving Manual, TI-818-03), single acting diesel pile hammers in the rated range of 60,000 to 75,000 ft-lb of energy should be suitable for installing the 14-inch diameter close end pipe pile to the recommended embedment depths. Project specifications should note both a minimum and maximum pile hammer energy that will facilitate acceptable installation while not damaging the pile. While WAVE equation analysis will refine drive energy requirements, single acting diesel pile hammers in the range of 60,000 to 75,000 ft-lbs should be considered for installation of the bulk fuel facility and powerplant pile.

We recommend a pile load test be conducted on the initial piles installed at the bulk fuel facility. ASTM D1143-81 (1994) E1 "Standard Test Method for Piles Under Static Axial Compressive Load", while withdrawn in 2006, should be used as an informational procedure for an axial pile loading analysis to confirm capacity. Alternatively a pile driving analyzer (PDA) system can be used to provide real-time pile installation analysis and capacity confirmation. We can provide details for both axial load testing and PDA analyses as part of our design review effort.

Constructability Considerations

Several key pile installation constructability issues should be addressed, as the design and bid documents are refined. Assuming a driven pile foundation system is used, tracked heavy equipment construction activity should be prohibited on a barren site surface until at least 18-inches of frost penetration has occurred. Unfrozen near surface soils are generally moisture sensitive organic silts that will readily loose strength rut under repeated light trafficking loads.

Significant damage to the organic mat may impact pile axial and lateral capacity. Assuming a pile foundation is used, the contractor should be required to submit a detailed pile driving plan for approval by the owner or engineer prior to allowing installation of any pile or construction trafficking on the site.

Two pile hammer systems are generally used for driven pile installation; a fixed lead and a free lead system. A fixed lead system is preferred but a free lead system can be used, provided the selected contractor can demonstrate a reliable performance record with free lead systems. The pile and leads should be carefully plumbed before the start of driving. Pulling the pile into position should not be allowed after driving has started. The head of the pile should fit square in the hammer but the steel driving cap should not restrain the pile from rotating. Pile hammer cushion and drive cap configuration should be detailed by the contractor as part of his submittal prior to driving any pile.

Piles should be transported, stored and handled by the contractor at the site in a manner that will not result in pile shaft bowing (sweep) or piles becoming out of round. All piles should be visually inspected prior to installation for conformance with the contract specifications. At completion, each pile's centerline should be within 3 inches of the design horizontal location and the plumbness should be within 1-inch per 10 feet of vertical. Before driving, each pile should be marked with a horizontal line at every 12 inches from the tip of the pile. At every five feet the line should be labeled with the distance from the tip.

The piles should be driven in a continuous manner. A driving record should be kept of the installation of each pile. The record should include the time driving started and ended, any times when driving stopped, any preboring or spudding, hammer energy, blows per foot for each foot driven and blows for the last increment if less than one foot, location and plumbness for the installed pile.

Design Review and Construction Observation

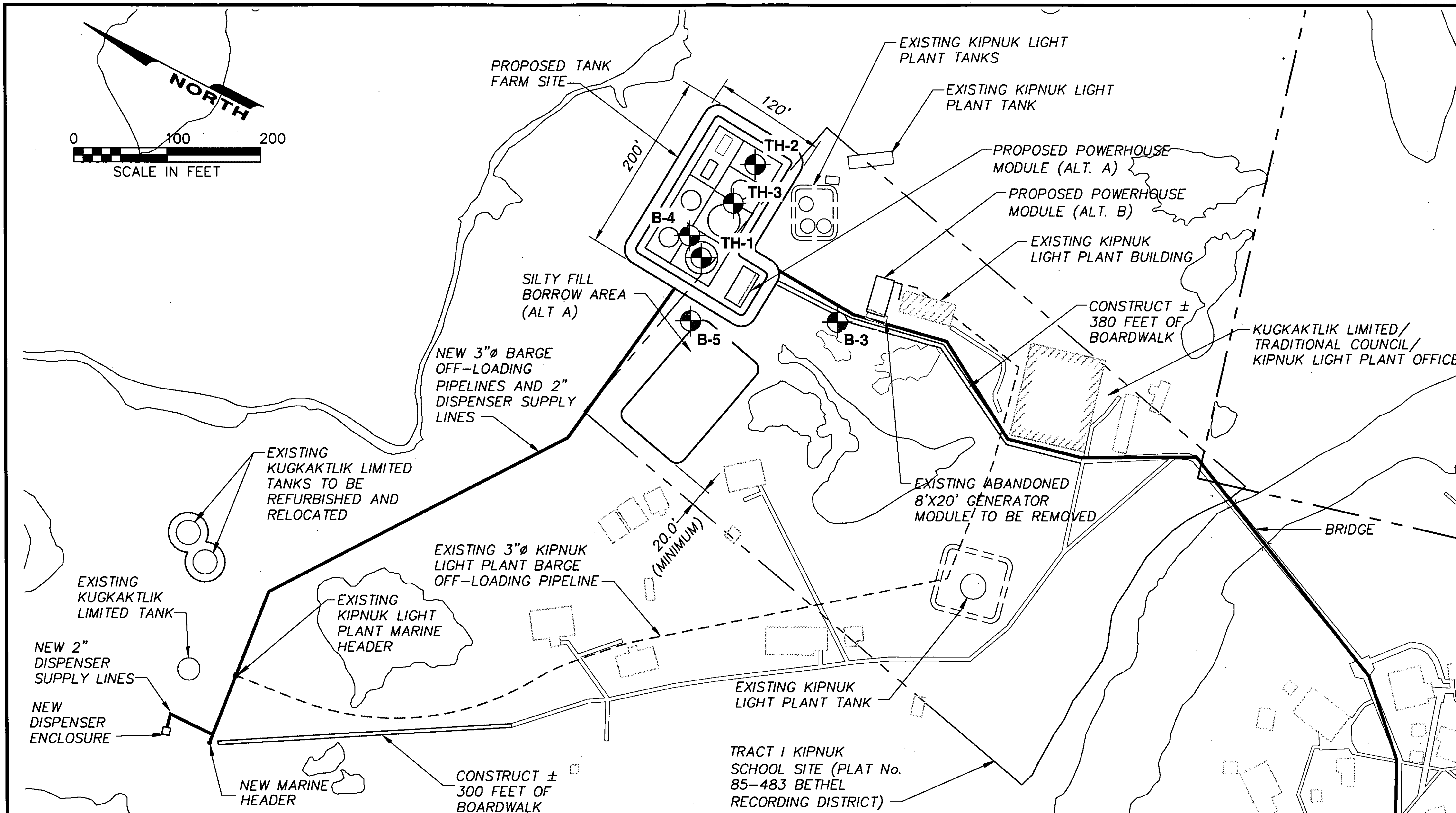
The plans and specifications should be reviewed by us as they develop to assure conformance with our design intent. Pile installation should be under the full-time observation of an experienced engineer or inspector. Inspection will permit the detection of unanticipated conditions and allow verification that the work is done in accordance with the intent of the recommendations in this report.

CLOSURE

The conclusions and recommendations developed for this report are based, in part, on the subsurface soils encountered and recovered at this site. As with any geotechnical investigation, site conditions may vary from those encountered during the site investigation. If differing subsurface conditions are encountered than those presented in this report, we should be contacted immediately to verify or modify our geotechnical recommendations.

Pile driving may encounter subsurface conditions different than those expected as the basis for our recommendations. We recommend the Owner retain competent, trained personnel experienced in remote site pile installation for on-site construction observation during pile installation. Construction observation should include daily reports that are forwarded to us for review.

We appreciate the opportunity to assist you with this project. If additional assistance is needed, feel free to contact us at your convenience.



Base from dwg Site Layout with Alt Sites dated 2/6/07 © LCMF LLC. Borehole locations are approximate.

2007 DMA Test Hole

1998 DMA Test Hole

Duane Miller Associates LLC
Job No.: 4095.111 / 131
Date: June 2007

SITE MAP
AEA Bulk Fuel and Powerplant Facility
Kipnuk, Alaska

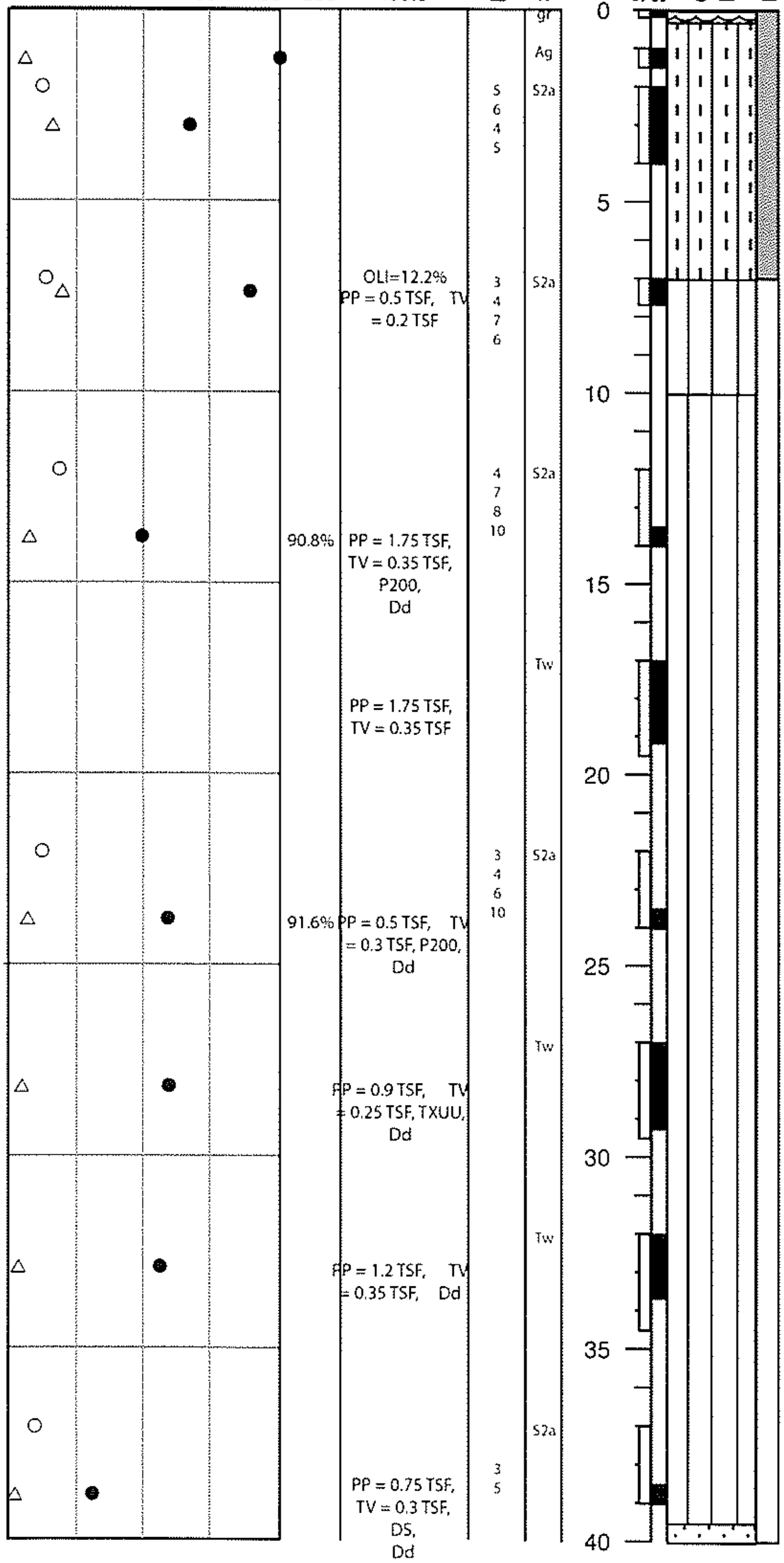
Plate
1

DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-01

Date Drilled: March 23, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.8" W 164°02'14.4" (WGS 84)
Elevation: 16 ft. (GPS, approximate)



Depth (ft)	P200	Other Tests	Blow Counts	Sampler Type	Description
0				gr	**PEAT (Pt)** Organic mat
				Ag	**ORGANIC SILT (OL)** (Nbn) Dark brown, with organics and interbeds of fiberous Peat (Pt) (70%), weakly bonded
			5 6 4 5	S2a	
~7		OLI=12.2%, PP = 0.5 TSF, TV = 0.2 TSF	3 4 7 6	S2a	**SILT (ML)** (Nbn) Gray, medium stiff, moist, with interbeds of Peat (Pt) (30%)
10					**SILT (ML)** Dark gray, stiff to very stiff, moist, nonplastic, micaceous
~12	90.8%	PP = 1.75 TSF, TV = 0.35 TSF, P200, Dd	4 7 8 10	S2a	
~17		PP = 1.75 TSF, TV = 0.35 TSF		Tw	Small amount of water on the top end of the Shelby sample
~22	91.6%	PP = 0.5 TSF, TV = 0.3 TSF, P200, Dd	3 4 6 10	S2a	
~27		PP = 0.9 TSF, TV = 0.25 TSF, TXUU, Dd		Tw	
~32		PP = 1.2 TSF, TV = 0.35 TSF, Dd		Tw	
~37		PP = 0.75 TSF, TV = 0.3 TSF, DS, Dd	3 5	S2a	
40					**SAND (SP-SM)** Gray, medium dense, moist, micaceous, fine-grained, trace silt

Duane Miller Associates LLC
Job No.: 4095.111/131
Date: June 2007

LOG OF TEST BORING TH-01
AEA Bulk Fuel/ Powerplant Facility
Kipnuk, Alaska

DRAFT

Plate
2.1

DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-01

Date Drilled: March 23, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.8" W 164°02'14.4" (WGS 84)
Elevation: 16 ft. (GPS, approximate)

Moisture Content % (●), PL & LL (|—|), Salinity (△) and Sampling Blows/ft (○)
0 20 40 60 >80

Depth (Feet): 40, 45, 50, 55, 60, 65, 70, 75, 80

P200	Other Tests	Blow Counts	Sampler Type
8.4%	SA	8 12 13	S2a
30.2%	SA	16	S2a
12.4%	SA, DS, Dd	7 9 14 15	S2a
12.7%	SA, DS (remolded), Dd (remolded)		Ag

Description

SAND (SP-SM) Gray, medium dense to dense, saturated, fine-grained, trace silt

Encountered 7 feet of fine sand heave at 40 feet

Grades to:

SILTY SAND (SM) Gray,medium dense to dense, saturated, non plastic, micaceous

Fine sand heave, no sample recovered at 52 feet

Grades to:

SAND (SP-SM) Gray, medium dense to dense, saturated, micaceous, fine-grained, trace silt

Fine sand heave, no sample recovered at 62 feet

Encountered 24 ft of heave at 67 feet

Test Boring completed at 67.5 feet

Installed 1" slotted PVC to 65 feet, hand slotted for piezometer

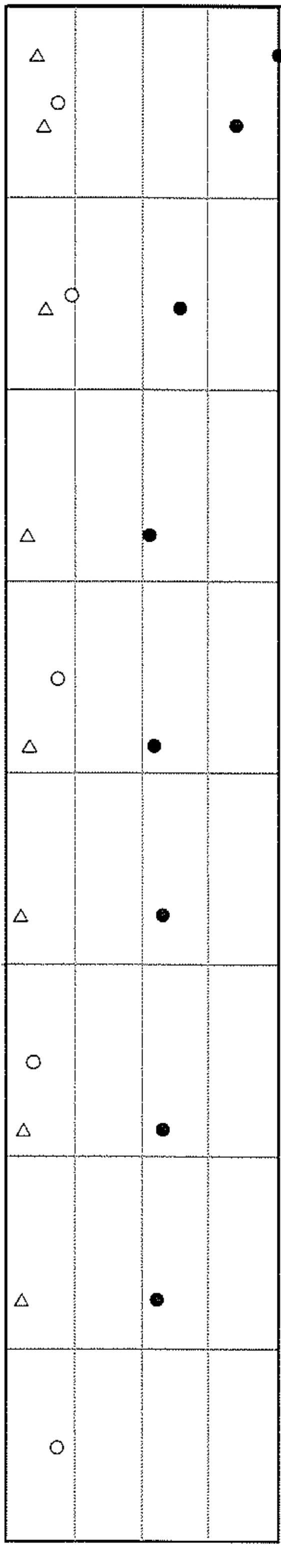
DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-02

Date Drilled: March 22, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.9" W 164°02'11.9" (WGS 84)
Elevation: 10 ft. (GPS, approximate)

Moisture Content % (●), PL & LL (⊢⊣), Salinity (△) and Sampling Blows/ft (○)
0 20 40 60 >80



P200	Other Tests	Blow Counts	Sampler Type
			Ag
		6 7 8	S2a
	OLI=3.8%	5 8 11 13	S2a
			Tw
	PP = 2.5 TSF, TV = 0.4 TSF, Dd		
88.7%	PP = 2.5 TSF, TV = 0.4 TSF, Dd	5 7 8 11	S2a
			Tw
	PP = 1.25 TSF, TV = 0.24 TSF, Dd		
85.9%	PP = 0.25 TSF, TV = 0.3 TSF, DS Dd	3 3 5 6	S2a
			Tw
	PP = 2.5 TSF, TV = 0.32 TSF		
	PP = 3.2 TSF, TV = 0.45 TSF	3 6 9 11	S2a

Depth (Feet): 0, 5, 10, 15, 20, 25, 30, 35, 40
Sampling Interval Samples | Graphic Log | Frozen

Description

PEAT (Pt) Organic mat

ORGANIC SILT (OL) (Nbn) Dark brown, micaceous, poorly bonded, no visible ice

SILT (ML) (Vx) Gray, with thin organic layers, poorly bonded, with 25 - 30% visible ice, weakly bonded

SILT (ML) (Vx) Gray and brown, with thin organic layers, micaceous, poorly bonded, with 5 - 10% visible ice

SILT (ML) Gray to dark gray, medium stiff to stiff, moist, micaceous, non plastic

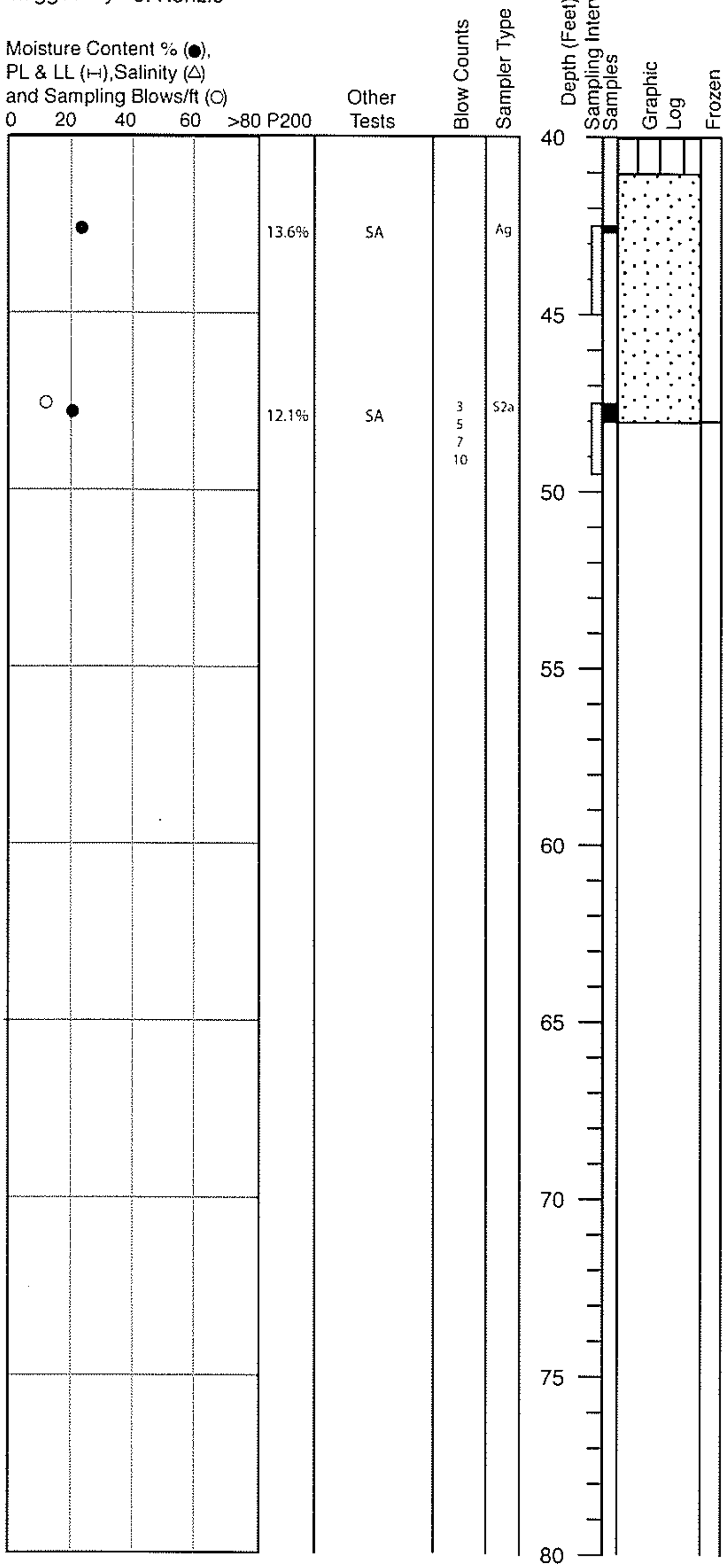
SILT (ML) Dark gray with thin discontinuous layers of organic silt, moist-wet, medium stiff to stiff

DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-02

Date Drilled: March 22, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.9" W 164°02'11.9" (WGS 84)
Elevation: 10 ft. (GPS, approximate)



Description

SILT (ML) Dark gray with thin discontinuous layers of organic silt, moist-wet, medium stiff to stiff

SAND (SP-SM) Dark gray, saturated, medium dense, micaceous, fine grained, trace silt. Greater silt content at 41 to 43 feet, bordering on Silty Sand classification

Encountered 14 feet of heave at 42.5 feet, no sample recovered from within the Shelby.

Encountered 20 feet of heave at 47 feet.

Test boring completed at 48 feet.

Installed 1" slotted PVC to 49 feet, hand slotted for piezometer

DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-03

Date Drilled: March 27, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.9" W 164°02'11.9" (WGS 84)
Elevation: -

Moisture Content % (●), PL & LL (⊢⊣), Salinity (△) and Sampling Blows/ft (○): 0, 20, 40, 60, >80 | P200 | Other Tests | Blow Counts | Sampler Type | Depth (Feet) | Sampling Interval Samples | Graphic Log | Frozen | Description

Depth (Feet)	Description
0	**PEAT (Pt)** Organic mat
2	**SILT (ML)** Dark gray, micaceous, no organics
20	Groundwater encountered at 20 feet
39	**SAND (SP-SM)** Gray, saturated, micaceous, fine-grained, trace silt

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

Duane Miller Associates LLC
Job No.: 4095.111/131
Date: June 2007

LOG OF TEST BORING TH-03
AEA Bulk Fuel/ Powerplant Facility
Kipnuk, Alaska

DRAFT

Plate 4.1

DUANE MILLER ASSOCIATES LLC

Project: Kipnuk Bulk Fuel Pad / Powerplant Facility
DMA Job No.: 4095.111/131
Logged By: J. Kenzie

Log of HOLE: TH-03

Date Drilled: March 27, 2007
Contractor.: Hughes Drilling Service
Equipment: CME 45, skid mounted, 140# Autohammer
GPS Coord.: N 59°56'26.9" W 164°02'11.9" (WGS 84)
Elevation: -

Moisture Content % (●), PL & LL (⊢⊣), Salinity (△) and Sampling Blows/ft (○)
0 20 40 60 >80

Depth (Feet)	P200	Other Tests	Blow Counts	Sampler Type	Description
40					**SAND (SP-SM)** Gray, saturated, micaceous, fine-grained, trace silt
65	42.6%	SA	26 44 38 30	Ag	**SILTY SAND (SP-SM)** Gray, dense, saturated

Depth scale: 40, 45, 50, 55, 60, 65, 70, 75, 80

Column headings: Depth (Feet), Sampling Interval, Samples, Graphic Log, Frozen

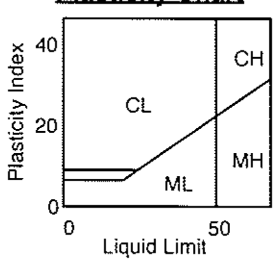
Test Boring completed at 67 feet

Boring advanced without sampling to confirm thermal state of soils

Note: Sample 65 feet collected from auger bit and may not be representative of *in-situ* soil conditions.

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS 50% or more larger than #200 sieve, 0.075 mm	**GRAVELS** More than half of the coarse fraction is larger than #4 sieve size, > 4.75 mm.	Clean gravels with little or no fines	GW	Well graded gravels, sandy gravel
			GP	Poorly graded gravels, sandy gravel
		Gravels with more than 12% fines	GM	Silty gravels, silt sand gravel mixtures
			GC	Clayey gravels, clay sand gravel mixtures
	SANDS More than half of the coarse fraction is smaller than #4 sieve size, < 4.75 mm.	Clean sands with little or no fines	SW	Well graded sand, gravelly sand
			SP	Poorly graded sands, gravelly sand
		Sands with more than 12% fines	SM	Silty sand, silt gravel sand mixtures
			SC	Clayey sand, clay gravel sand mixtures
FINE GRAINED SOILS > 50% finer than #200 sieve	**SILTS and CLAYS**	Liquid limit less than 50	ML	Inorganic silt and very fine sand, rock flour
			CL	Inorganic clay, gravelly and sandy clay, silty clay
			OL	Organic silts and clay of low plasticity
		Liquid limit greater than 50	MH	Inorganic silt
			CH	Inorganic clay, fat clay
			OH	Organic silt and clay of high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soil

Plasticity Chart



UNIFIED SOIL CLASSIFICATION SYSTEM

KEY TO TEST DATA

Con = Consolidation
Dd = Dry Density (pcf)
D1557 = modified Proctor
MA = Sieve and Hydrometer Analysis
LL = Liquid Limit
NP = non Plastic
OLI = Organic Loss
PI = Plastic Index
PL= Plastic Limit
PP = Pocket Penetrometer
RD = Relative Density
SA = Sieve Analysis
SpG = Specific Gravity
TS = Thaw Consolidation
TV = Torvane
TXCD = Consolidated Drained Triaxial
TXCU = Consolidated Undrained Triaxial
TXUU = Unconsolidated Undrained Triaxial

Strength Data
XXX (YYY), where:
$XXX = (\sigma_1 - \sigma_3)/2$
$YYY = \sigma_3$

KEY TO SAMPLE TYPE

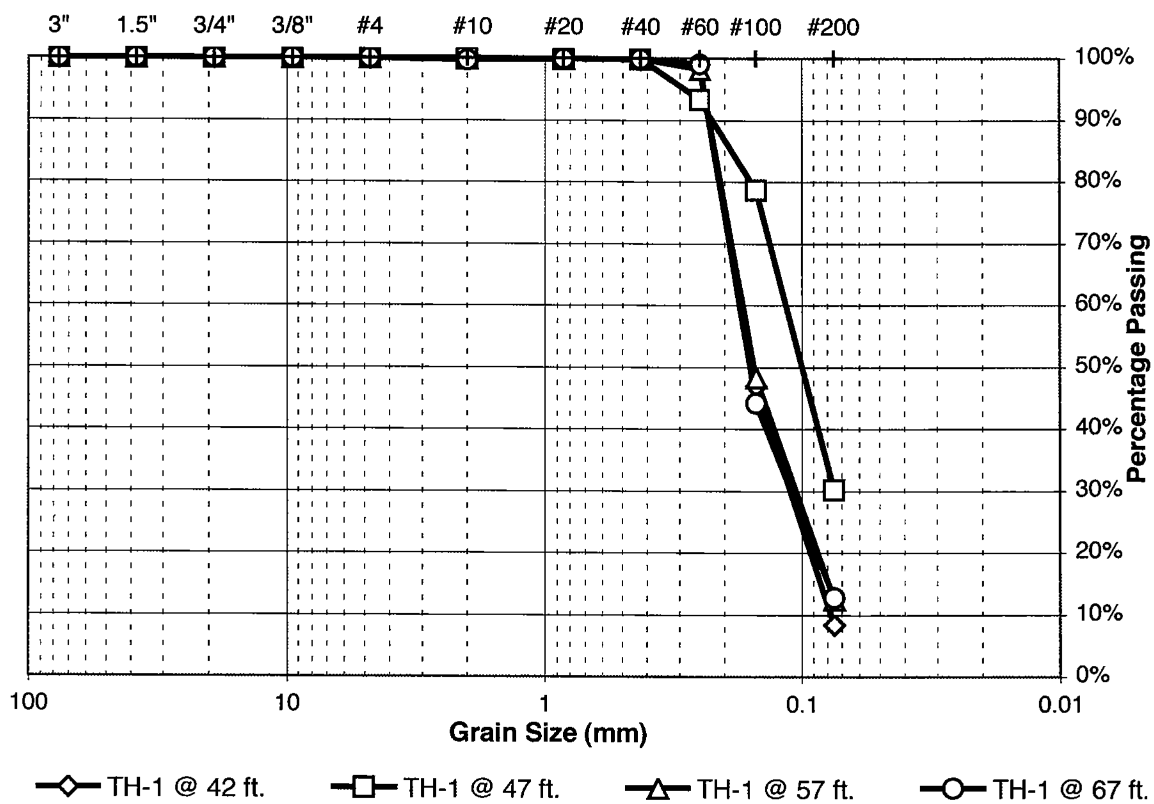
Gr = Grab sample
Ag = Auger grab
Ab = Auger bulk
Ac = Air chip
Sh = 2.5" ID split barrel w/ 340 lb. manual hammer
Sh* = 2.5" ID split barrel w/ 140 lb. manual hammer
Sha = 2.5" ID split barrel w/ 340 lb. automatic hammer
S2* = 2.0" ID split barrel w/ 140 lb. manual hammer
S2a = 2.0" ID split barrel w/ 140 lb. automatic hammer
Tw = Shelby tube
Ss = 1.4" ID split barrel w/ 140 lb. manual hammer (Standard Penetration Test Method)
Ssa = 1.4" ID split barrel w/ 140 lb. automatic hammer
Cc = 3.25" continuous core barrel

GROUP	ICE VISIBILITY	DESCRIPTION		SYMBOL	
N	Segregated ice not visible by eye	Poorly bonded or friable		Nf	
		Well bonded	No excess ice	Nb	Nbn
			Excess microscopic ice		Nbe
V	Segregated ice is visible by eye and is one inch or less in thickness	Individual ice crystals or inclusions		Vx	
		Ice coatings on particles		Vc	
		Random or irregularly oriented ice		Vr	
		Stratified or distinctly oriented ice		Vs	
		Uniformly distributed ice		Vu	
ICE	Ice greater than one inch in thickness	Ice with soil inclusions		ICE + soil type	
		Ice without soil inclusions		ICE	

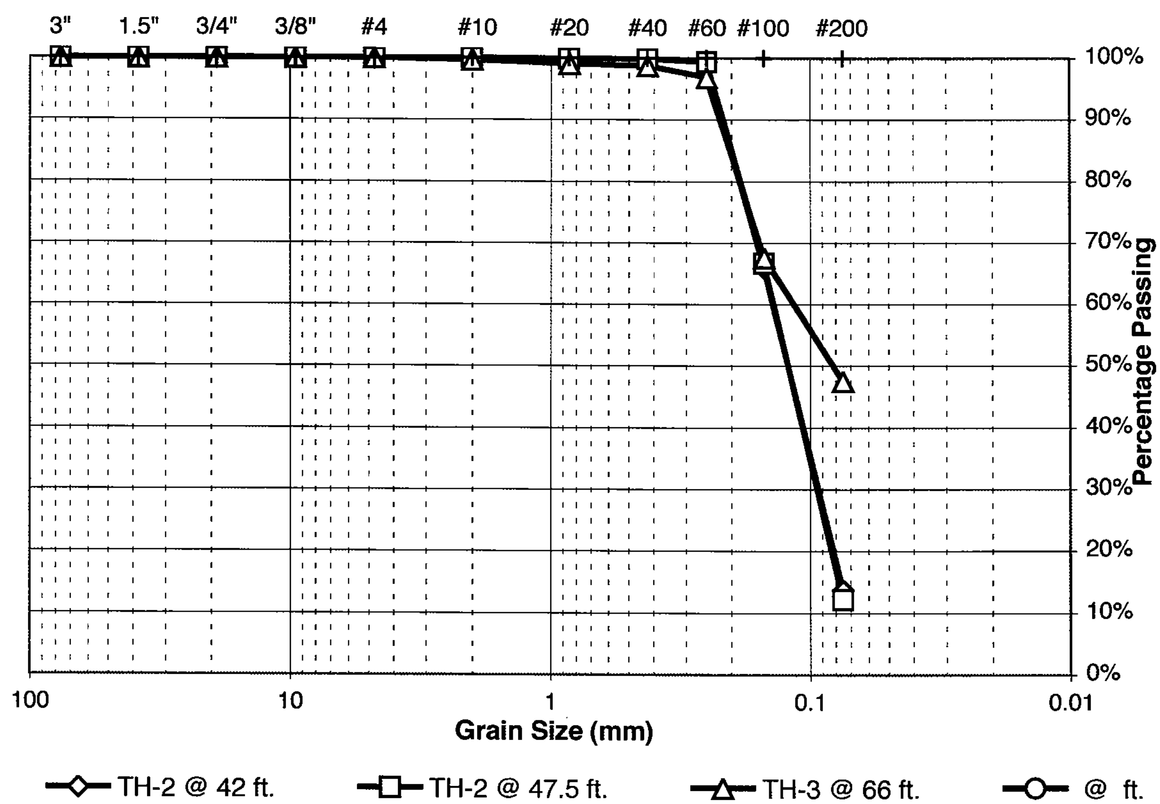
ICE CLASSIFICATION SYSTEM

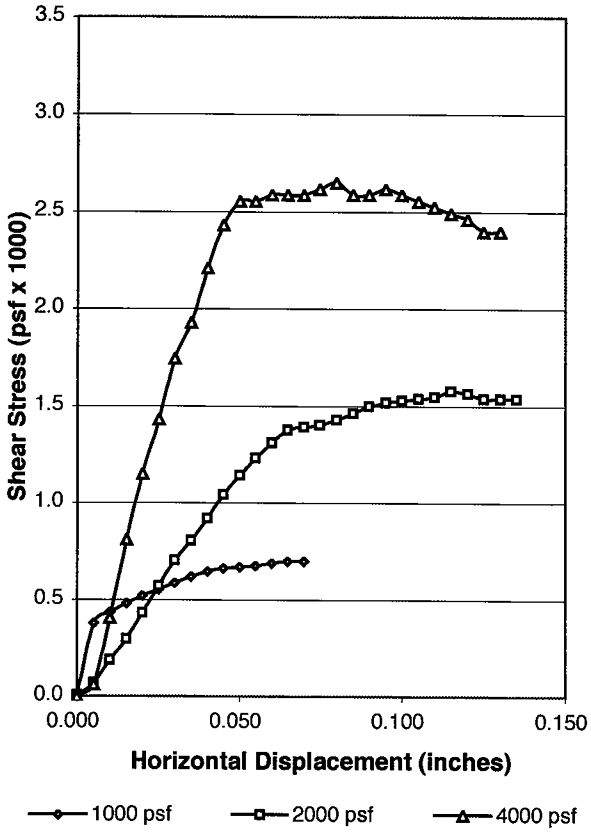
Test Hole	Sample Depth (ft)	Soil Type (USCS)	Sampler Type	Thermal State	SPT "N" Value	Moisture Content (%)	Salinity (ppt)	OLI (%)	Gravel (%)	Sand (%)	Passing # 200 (%)
AEA-TH-1	1.0	OL	Ag	F		264.0	4.9				
AEA-TH-1	2.0	OL	S2a	F	12	53.4	13.0				
AEA-TH-1	7.0	ML	S2a	UF	18	71.1	15.8	12.2			
AEA-TH-1	13.5	ML	S2a	UF	24	39.4	6.2		0	9	90.8
AEA-TH-1	17.0	ML	Tw (archive)	UF							
AEA-TH-1	23.5	ML	S2a	UF	22	47.1	5.7		0	8	91.6
AEA-TH-1	27.0	ML	Tw	UF		47.4	4.0				
AEA-TH-1	32.0	ML	Tw	UF		45.0	2.9				
AEA-TH-1	38.5	ML	S2a	UF	11	25.0	1.9				
AEA-TH-1	42.0	SP	S2a	UF	34	21.5	3.4		0	92	8.4
AEA-TH-1	47.0	SP	S2a	UF	22	21.3	2.6		0	70	30.2
AEA-TH-1	57.0	SP	S2a	UF	39	24.1	2.4		0	88	12.4
AEA-TH-1	67.0	SP	Ag	UF		31.5	2.9		0	87	12.7
AEA-TH-2	1.0	OL	Ag	F		152.7	8.9				
AEA-TH-2	2.5	ML	S2a	F	20	67.6	10.9				
AEA-TH-2	7.5	ML	S2a	F	33	51.0	11.4	3.8			
AEA-TH-2	12.5	ML	Tw	UF		41.9	6.1				
AEA-TH-2	19.0	ML	S2a	UF	26	43.3	6.7		0	11	88.7
AEA-TH-2	22.5	ML	Tw	UF		45.9	4.2				
AEA-TH-2	29.0	ML	S2a	UF	15	46.0	5.1		0	14	85.9
AEA-TH-2	32.5	ML	Tw	UF		44.3	4.5				
AEA-TH-2	39.0	ML	S2a (archive)	UF	27						
AEA-TH-2	42.0	SP	Ag	UF		23.4			0	86	13.7
AEA-TH-2	47.5	SP	S2a	UF	23	20.5			0	88	12.1
AEA-TH-3	66.0	SP	Ag	UF		28.4	0.3		0	57	42.6

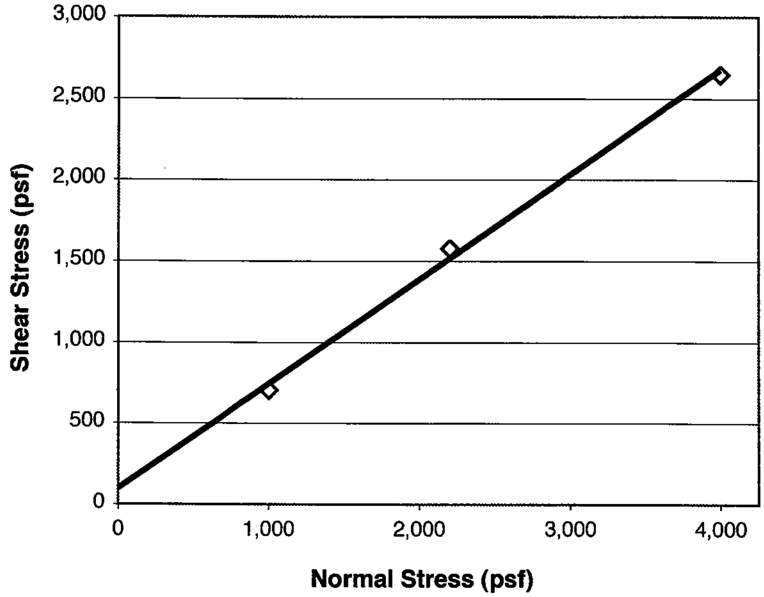
Boring =>	TH-1	TH-1	TH-1	TH-1
Depth =>	42.0 ft.	47.0 ft.	57.0 ft.	67.0 ft.
3" =>	100%	100%	100%	100%
1 1/2" =>	100%	100%	100%	100%
3/4" =>	100%	100%	100%	100%
3/8" =>	100%	100%	100%	100%
#4 =>	100%	100%	100%	100%
#10 =>	100%	100%	100%	100%
#20 =>	100%	100%	100%	100%
#40 =>	100%	100%	100%	100%
#60 =>	99%	93%	98%	99%
#100 =>	47%	79%	48%	44%
#200 =>	8.4%	30.2%	12.4%	12.7%
Analysis of Data				
D10 size =>	0.077 mm	N/A	N/A	N/A
D30 size =>	0.111 mm	N/A	0.105 mm	0.110 mm
D50 size =>	0.154 mm	0.100 mm	0.153 mm	0.158 mm
D60 size =>	0.170 mm	0.115 mm	0.169 mm	0.174 mm
Coeff. of Uniformity, Cu =	2.21	N/A	N/A	N/A
Coeff. of Curvature, Cc =	0.93	N/A	N/A	N/A
Gravel (+#4) percentage =	0%	0%	0%	0%
Sand percentage =	91.6%	69.8%	87.6%	87.3%
Fines percentage =	8.4%	30.2%	12.4%	12.7%
Unified Soil Class Symbol =	**SP-SM**	**SM**	**SM**	**SM**



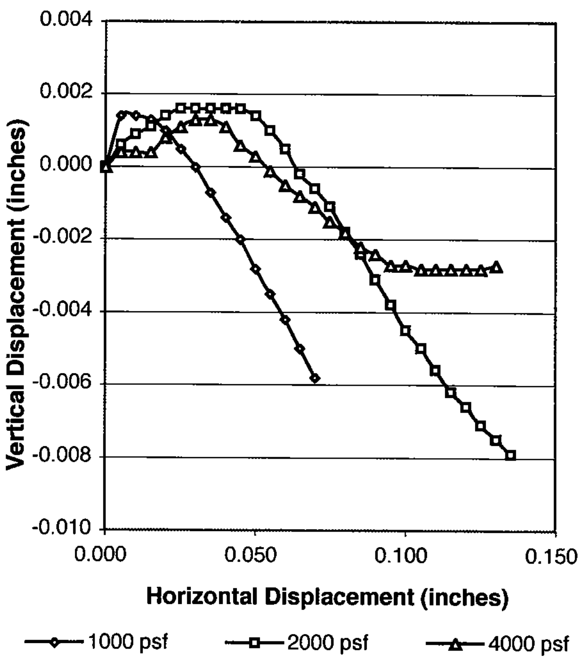
Boring =>	TH-2	TH-2	TH-3	
Depth =>	42.0 ft.	47.5 ft.	66.0 ft.	
3" =>	100%	100%	100%	
1 1/2" =>	100%	100%	100%	
3/4" =>	100%	100%	100%	
3/8" =>	100%	100%	100%	
#4 =>	100%	100%	100%	
#10 =>	100%	100%	100%	
#20 =>	100%	100%	99%	
#40 =>	100%	100%	99%	
#60 =>	100%	99%	97%	
#100 =>	66%	67%	68%	
#200 =>	13.7%	12%	48%	
Analysis of Data				
D10 size =>	N/A	N/A	N/A	
D30 size =>	0.093 mm	0.094 mm	N/A	
D50 size =>	0.121 mm	0.121 mm	0.082 mm	
D60 size =>	0.139 mm	0.138 mm	0.115 mm	
Coeff. of Uniformity, Cu =	N/A	N/A	N/A	
Coeff. of Curvature, Cc =	N/A	N/A	N/A	
Gravel (+#4) percentage =	0%	0%	0%	
Sand percentage =	86.3%	87.9%	52.4%	
Fines percentage =	13.7%	12.1%	47.6%	
Unified Soil Class Symbol =	**SM**	**SM**	**SM**	





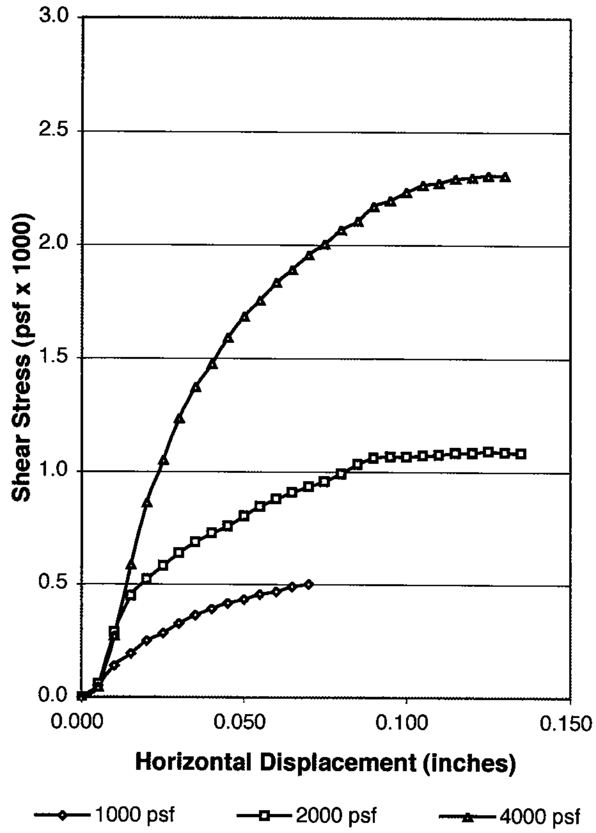


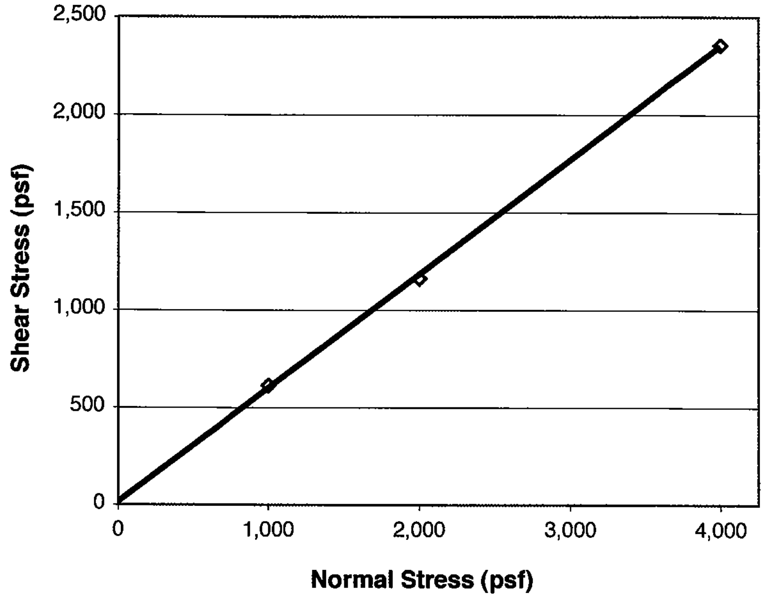
Boring	TH-1	Moisture	25.0%
Depth	38.5 ft	Dry Density	107 pcf
Phi	32.8°	Assumed SpG	2.70
Cohesion	100 psf		
Classification	Silty Sand		



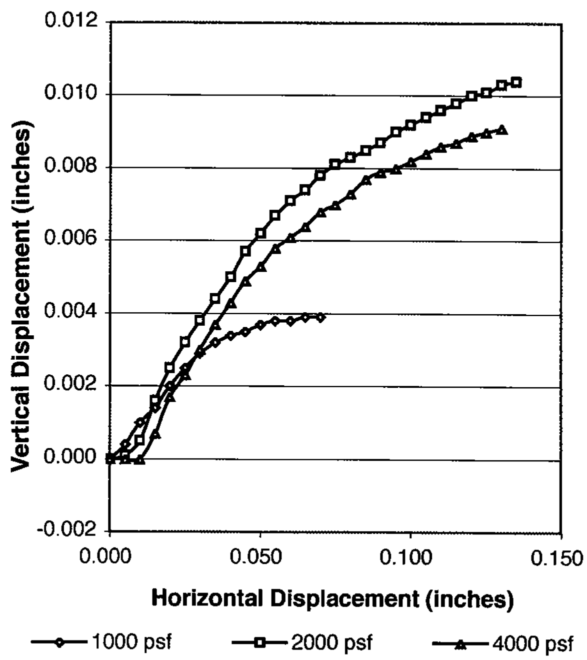
	Normal Stress	1000 psf	2000 psf	4000 psf
Initial	Moisture	20.4%	19.2%	20.0%
	Height	1.000 in	1.000 in	1.000 in
	Wet Density	131 pcf	129 pcf	128 pcf
	Dry Density	108 pcf	108 pcf	107 pcf
	Void Ratio	0.554	0.564	0.574
Before Shear	Post-Consol Void Ratio	0.524	0.533	0.509
Final	Moisture	22.0%	21.2%	20.6%
	Height	0.987 in	0.985 in	0.958 in
	Wet Density	133 pcf	131 pcf	134 pcf
	Dry Density	110 pcf	109 pcf	112 pcf
	Void Ratio	0.533	0.541	0.509

Max Shear	700 psf	1580 psf	2650 psf
Time to Failure	0:28:04	0:28:08	0:20:48





Boring	TH-1	Moisture	21.3%
Depth	57.0 ft	Dry Density	92 pcf
Phi	30.3°	Assumed SpG	2.70
Cohesion	20 psf		
Classification	Fine Sand		



	Normal Stress	1000 psf	2000 psf	4000 psf
Initial	Moisture	19.2%	19.7%	20.6%
	Height	1.000 in	1.000 in	1.000 in
	Wet Density	111 pcf	108 pcf	110 pcf
	Dry Density	90 pcf	90 pcf	91 pcf
	Void Ratio	0.805	0.863	0.853
Before Shear	Post-Consol Void Ratio	0.748	0.781	0.741
Final	Moisture	-138.8%	24.3%	24.3%
	Height	0.957 in	0.940 in	0.923 in
	Wet Density	115 pcf	113 pcf	117 pcf
	Dry Density	98 pcf	96 pcf	99 pcf
	Void Ratio	0.728	0.751	0.710

Max Shear	610 psf	1160 psf	2360 psf
Time to Failure	0:53:03	0:41:37	0:34:26

Duane Miller Associates LLC
Job No.: 4095.111/131
June 2007

DIRECT SHEAR TEST DATA
AEA Bulk Fuel and Powerplant Facility
Kipnuk, Alaska

Plate
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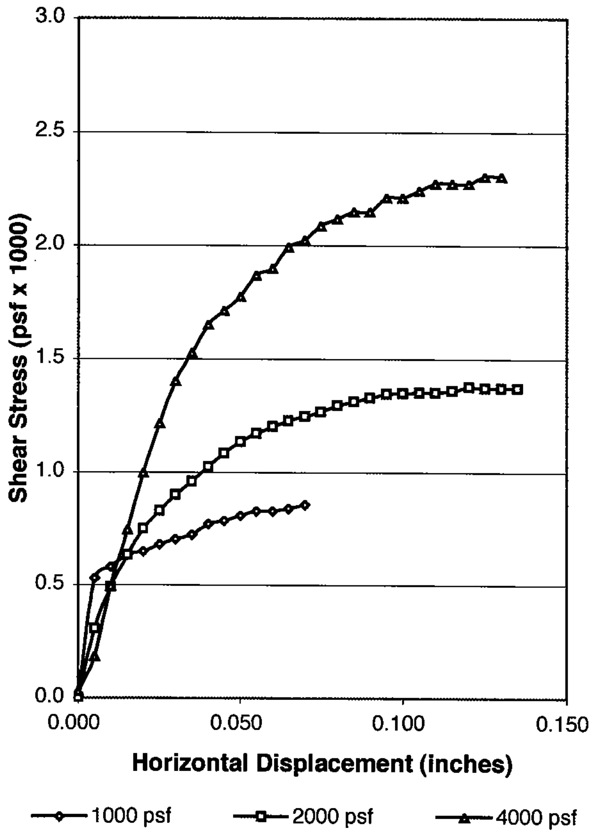
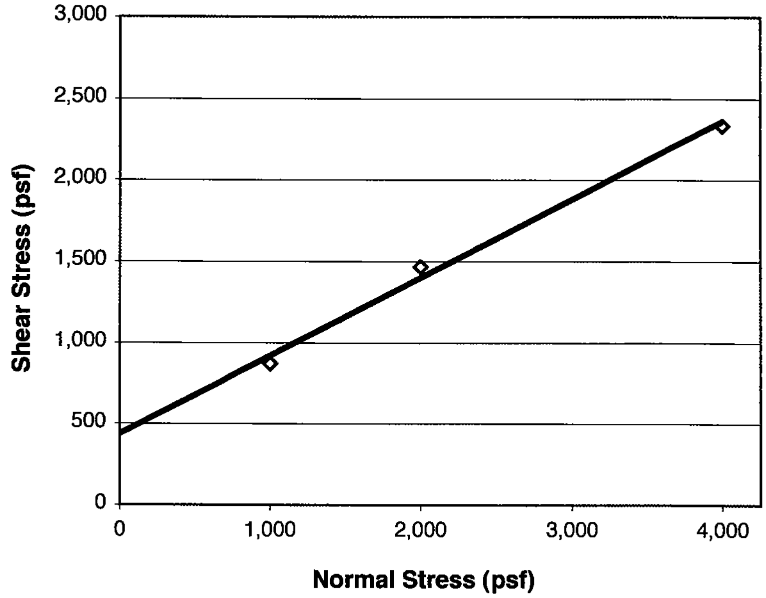
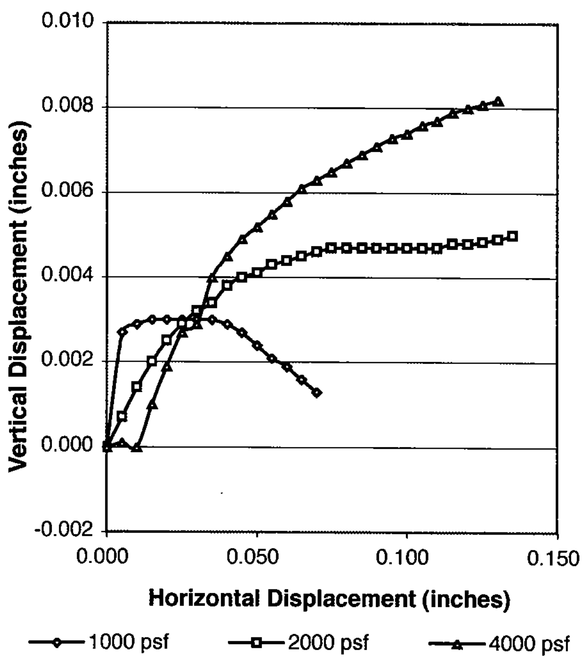




Boring	TH-1	Moisture	19.2%
Depth	67.0 ft	Dry Density	93 pcf
Phi	25.7°	Assumed SpG	2.70
Cohesion	440 psf		
Classification	Fine Sand		



	Normal Stress	1000 psf	2000 psf	4000 psf
Initial	Moisture	19.0%	19.0%	19.2%
	Height	1.000 in	1.000 in	1.000 in
	Wet Density	111 pcf	110 pcf	111 pcf
	Dry Density	93 pcf	93 pcf	93 pcf
	Void Ratio	0.802	0.820	0.808
Before Shear	Post-Consol Void Ratio	0.757	0.748	0.717
Final	Moisture	22.8%	23.7%	24.2%
	Height	0.972 in	0.951 in	0.937 in
	Wet Density	114 pcf	115 pcf	117 pcf
	Dry Density	96 pcf	97 pcf	100 pcf
	Void Ratio	0.752	0.730	0.694

Max Shear	870 psf	1470 psf	2340 psf
Time to Failure	0:28:53	0:38:10	0:40:00

Duane Miller Associates LLC
Job No.: 4095.111/131
June 2007

DIRECT SHEAR TEST DATA
AEA Bulk Fuel and Powerplant Facility
Kipnuk, Alaska

Plate
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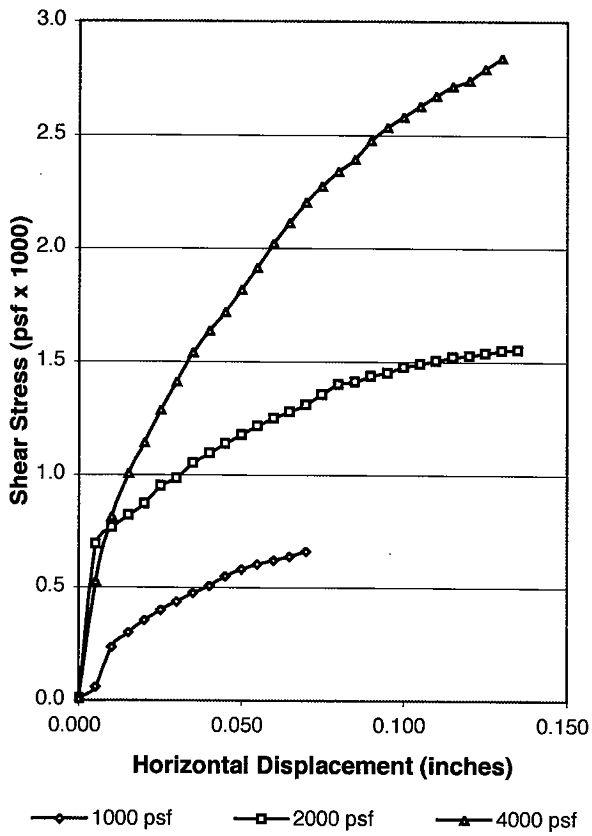
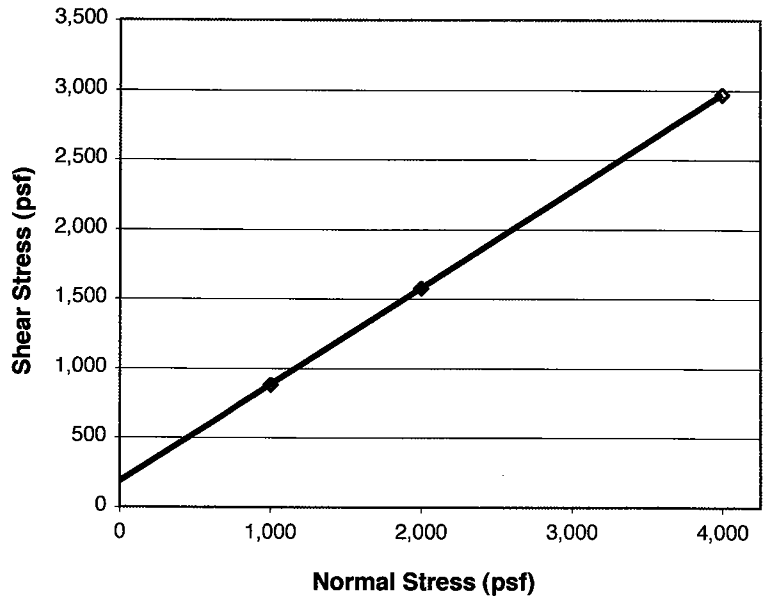
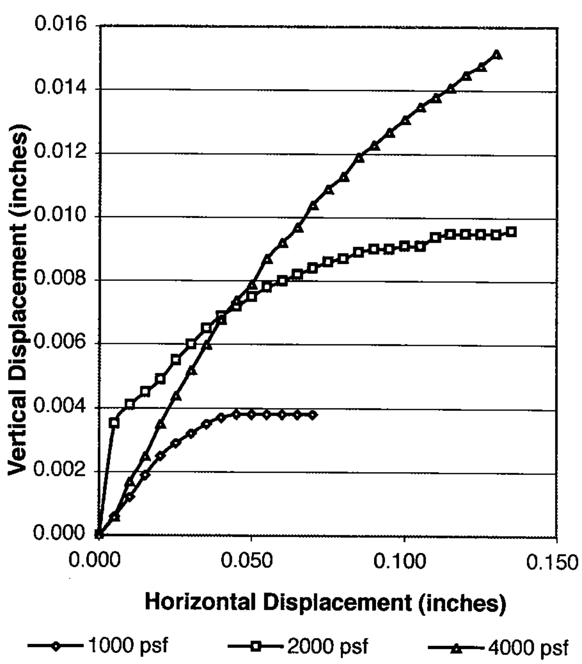




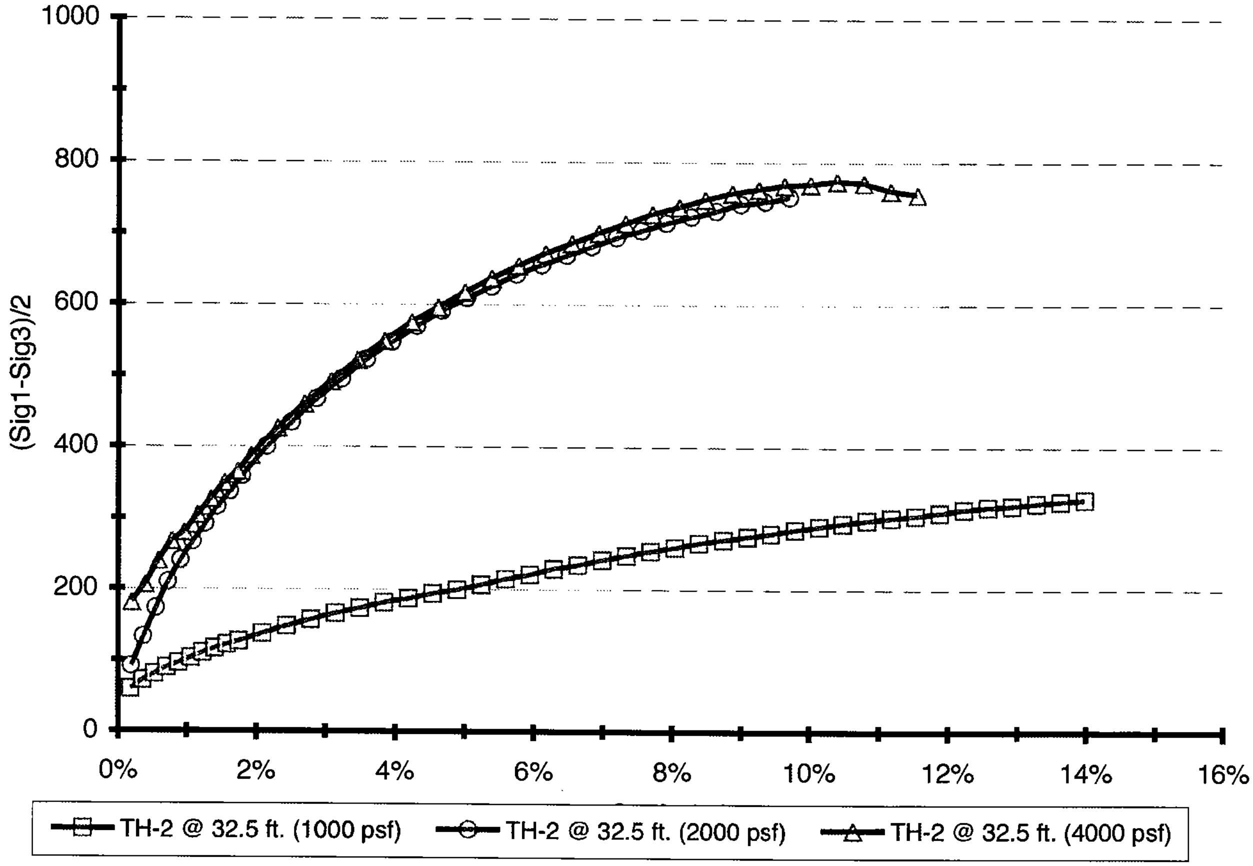
Boring	TH-2	Moisture	46.0%
Depth	29.0 ft	Dry Density	73 pcf
Phi	34.8°	Assumed SpG	2.7
Cohesion	190 psf		
Classification	Silty Fine Sand		

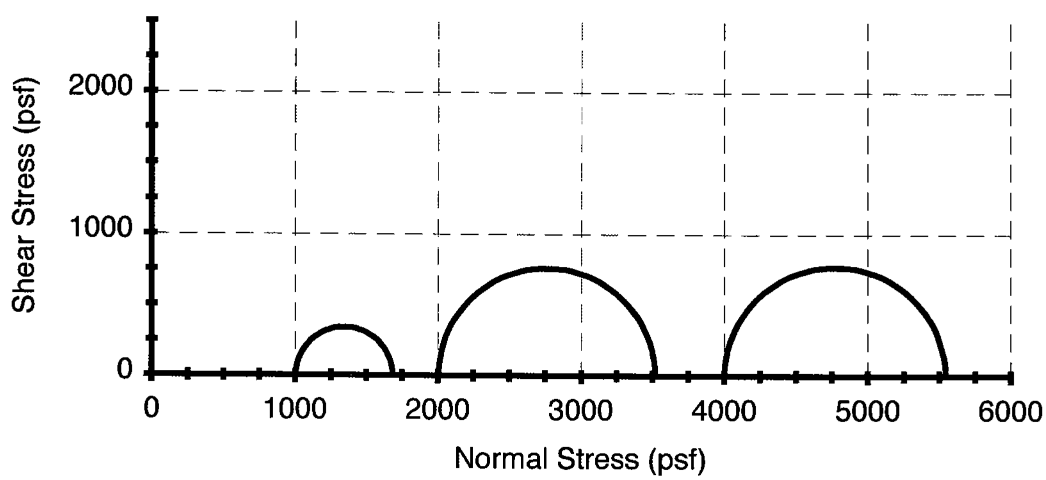


Initial	Normal Stress	1000 psf	2000 psf	4000 psf
	Moisture	46.6%	45.0%	45.8%
	Height	1.000 in	1.000 in	1.000 in
	Wet Density	106 pcf	107 pcf	106 pcf
	Dry Density	74 pcf	74 pcf	73 pcf
	Void Ratio	1.333	1.291	1.312
Before Shear	Post-Consol Void Ratio	1.210	1.090	1.032
Final	Moisture	45.1%	42.2%	40.5%
	Height	0.935 in	0.898 in	0.859 in
	Wet Density	112 pcf	117 pcf	121 pcf
	Dry Density	77 pcf	82 pcf	85 pcf
	Void Ratio	1.180	1.058	0.986

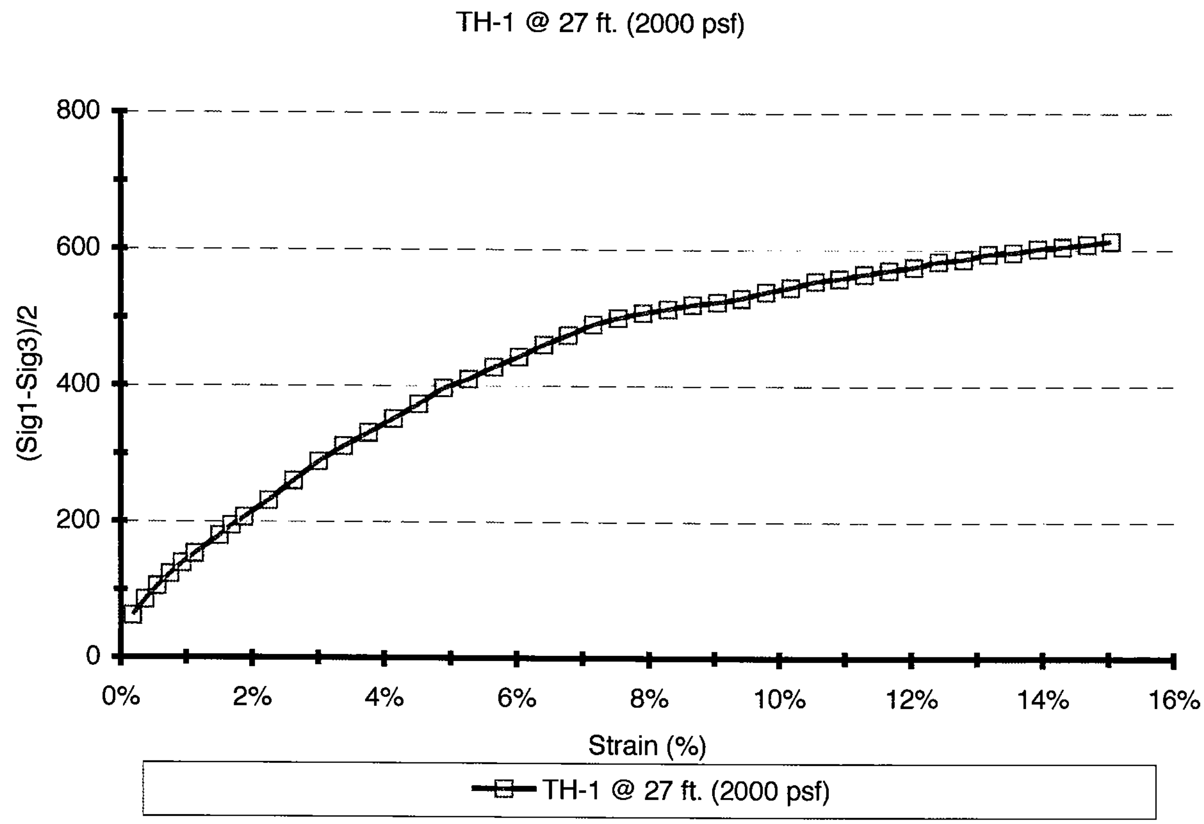
Max Shear	890 psf	1580 psf	2970 psf
Time to Failure	0:46:15	0:47:22	0:34:50

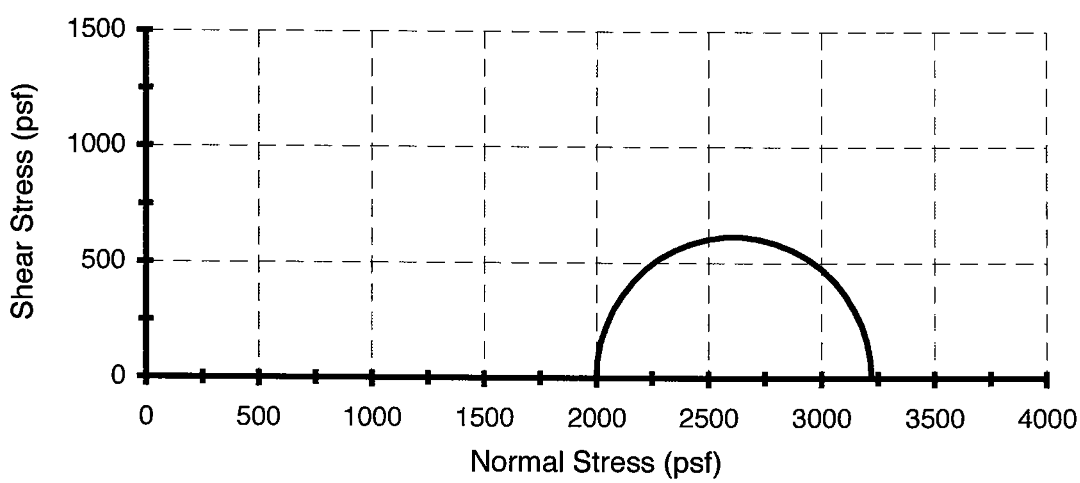
Boring	Sample Depth	Moisture Content	Dry Density	Saturation (Assumed SpG =2.70)	σ_3	$(\sigma_1 - \sigma_3)/2$	USCS
TH-2	32.5 ft	42.9%	73 pcf	90%	1000 psf	340 psf	ML
TH-2	32.5 ft	44.1%	74 pcf	97%	2000 psf	760 psf	ML
TH-2	32.5 ft	43.1%	77 pcf	102%	4000 psf	770 psf	ML



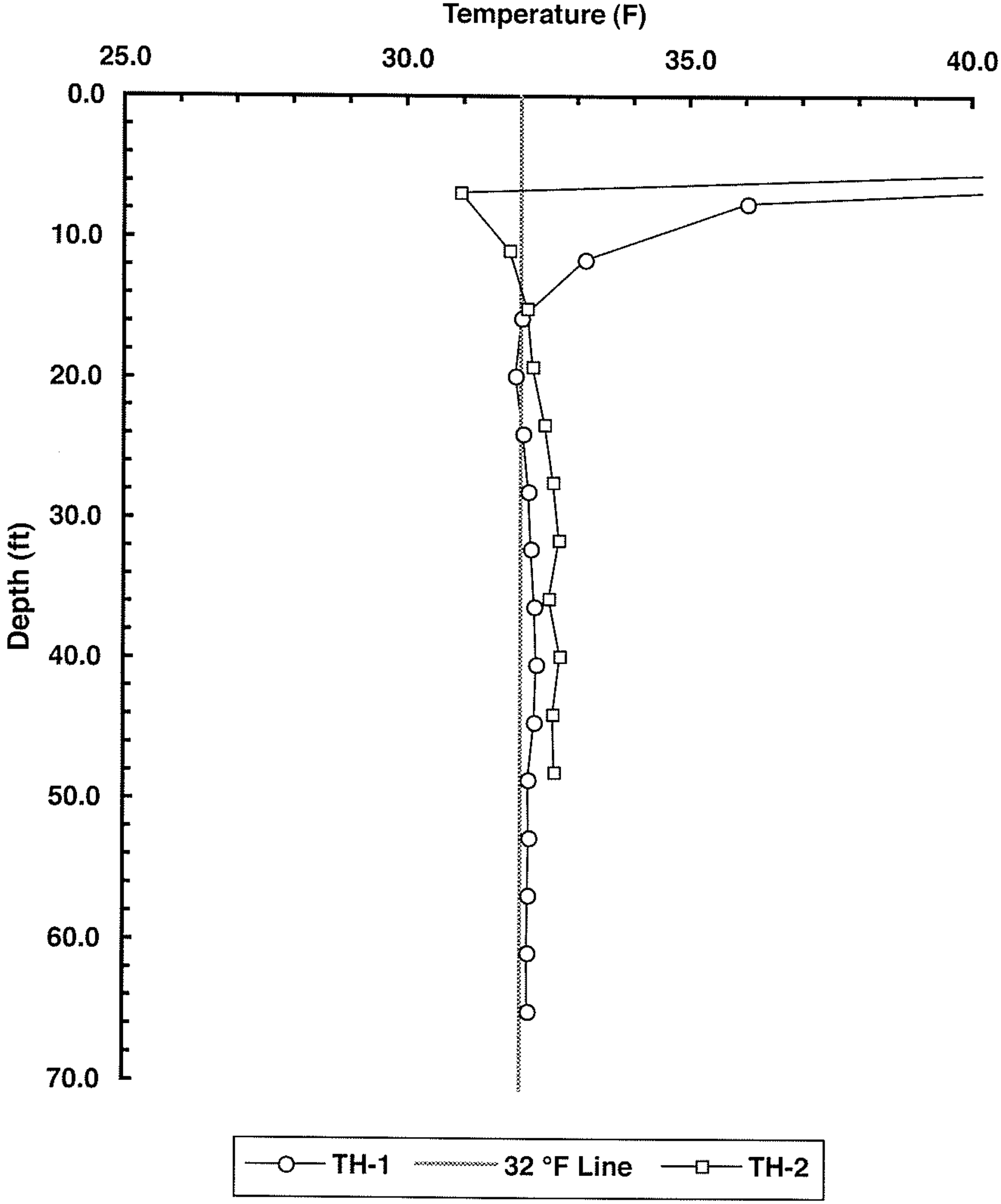


Boring	Sample Depth	Moisture Content	Dry Density	Saturation (Assumed SpG =2.61)	σ_3	$(\sigma_1 - \sigma_3)/2$	USCS
TH-1	27.0 ft	47.5%	73 pcf	101%	2000 psf	610 psf	ML





Measured Temperatures
AEA Fuel Tank Farm Site
June 1, 2007



Appendix A

1998 DMA Site Investigation Borings
Kipnuk Light Plant

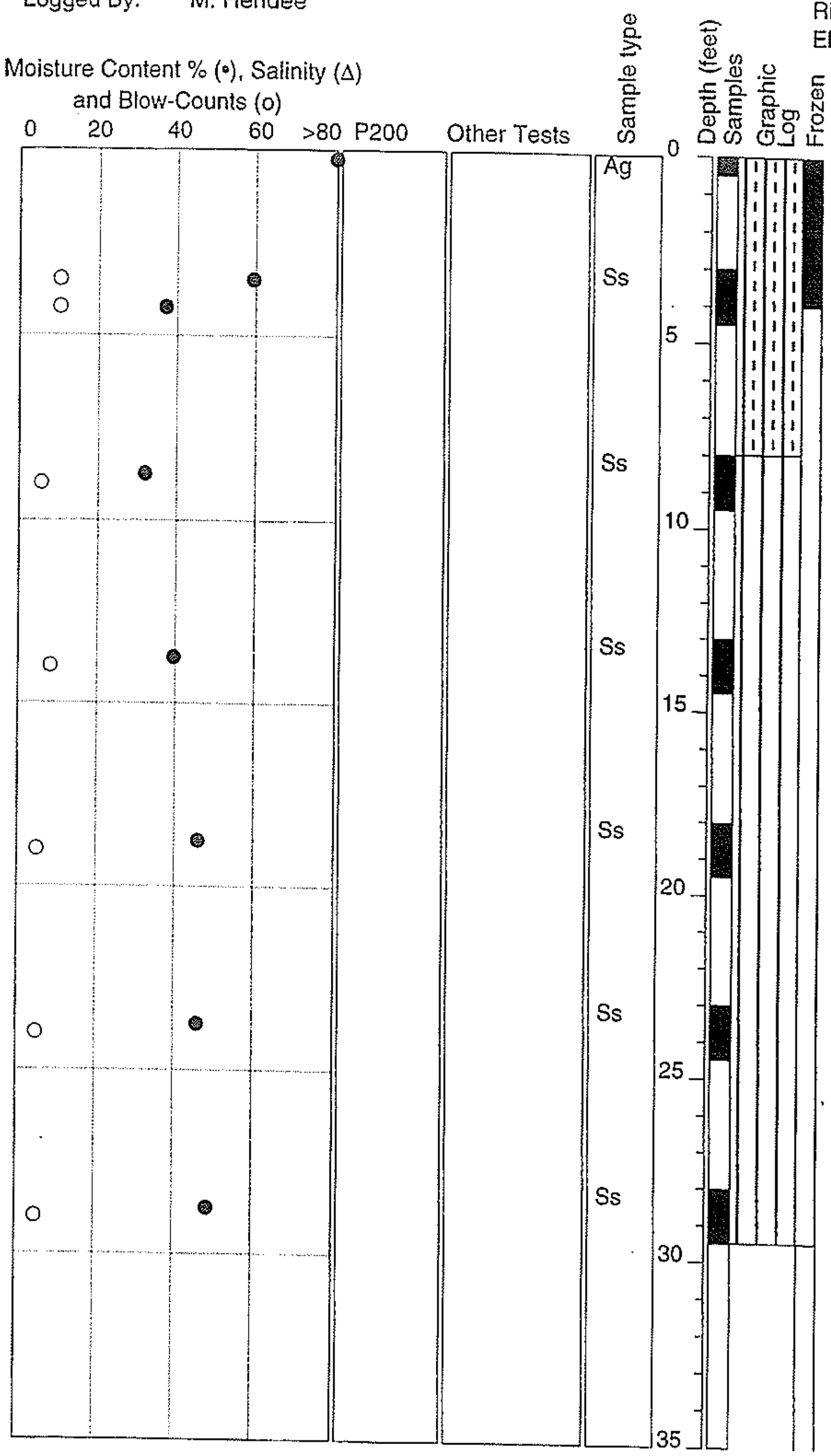
Boring B-3
Boring B-4
Boring B-5

DUANE MILLER & ASSOCIATES

Project: Kipnuk Fuel Tanks
DM&A Job No. :4095.44
Logged By: M. Hendee

Log of HOLE : B-3

Date Drilled: March 10, 1998
Contractor: Denali Drilling
Rig Type: CME-45 W/ 6" O.D. Hollow Stem
Elevation: 89.5 ft*



Light Plant Tank Farm

Description

ORGANIC SILT: (OL) (Vx) Gray-brown, seasonally frozen to 4' w/ trace visible ice, wet & medium stiff below below 4'

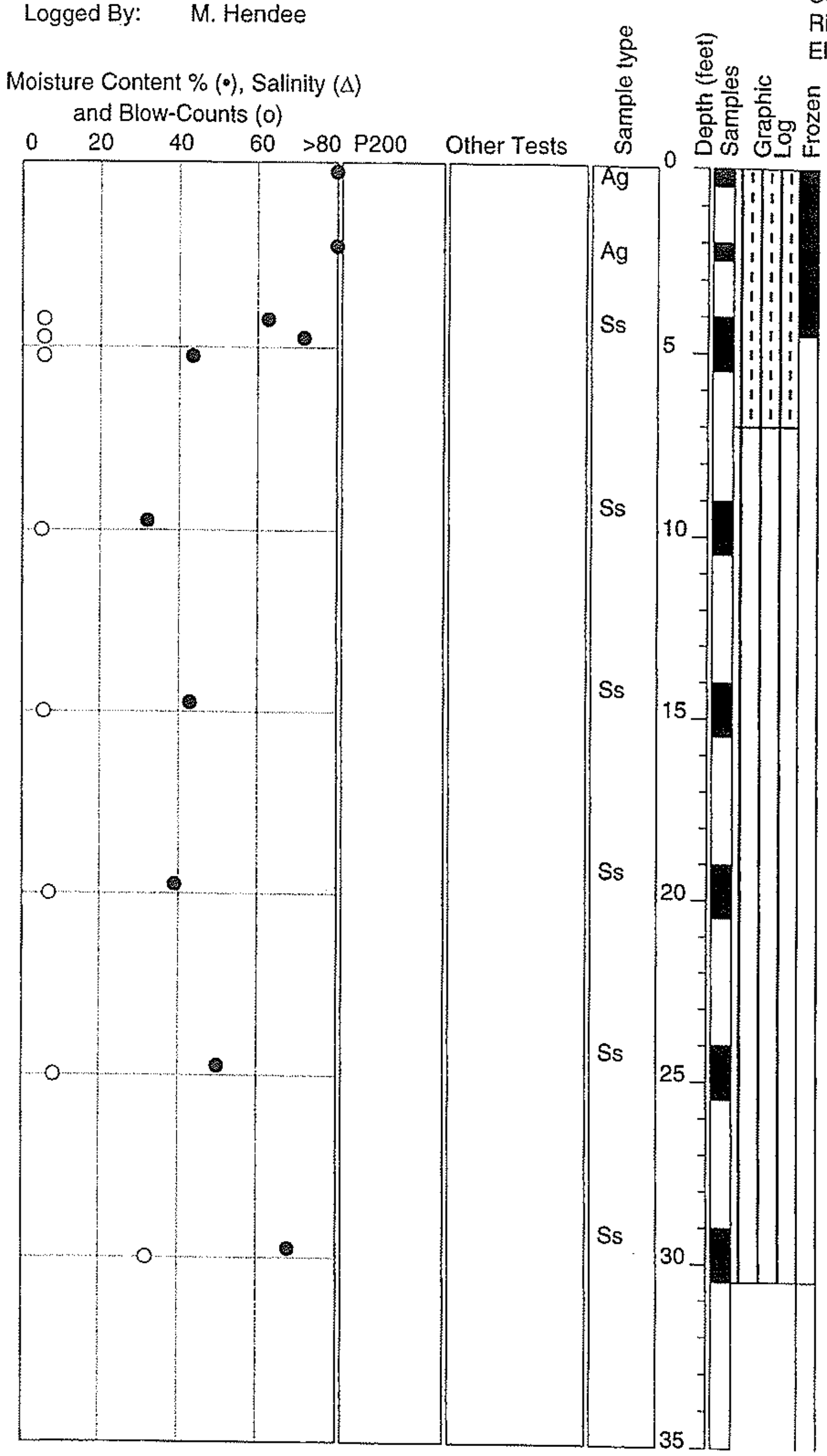
SILT: (ML) Dark gray w/ trace sand, wet, medium stiff

Closed, 3/4" I.D. PVC installed to 29.5'

*Elevations were measured with a hand level. An arbitrary datum of 100.0 feet was established at the top of the southwest corner of the burned trailer.

DUANE MILLER & ASSOCIATES
Project: **Kipnuk Fuel Tanks**
DM&A Job No. :4095.44
Logged By: M. Hendee

Log of HOLE : B-4
Date Drilled: March 10, 1998
Contractor: Denali Drilling
Rig Type: CME-45 W/ 6" O.D. Hollow Stem
Elevation: 89.5 ft*



Light Plant Tank Farm

Description

ORGANIC SILT: (OL) (Nb) Gray-brown w/ layers of fibrous Peat (Pt), seasonally frozen to 4.5' w/ no visible ice, wet & medium stiff below below 4.5'

SILT: (ML) Dark gray w/ trace sand, wet, medium stiff

Stiff below 28'

*Elevations were measured with a hand level. An arbitrary datum of 100.0 feet was established at the top of the southwest corner of the burned trailer.

Duane Miller & Associates
Arctic & Geotechnical Engineering
Job No.: 4095.44
Date : May 1998

LOG of BORING B-4
Fuel System Upgrade
Kipnuk, Alaska

Plate
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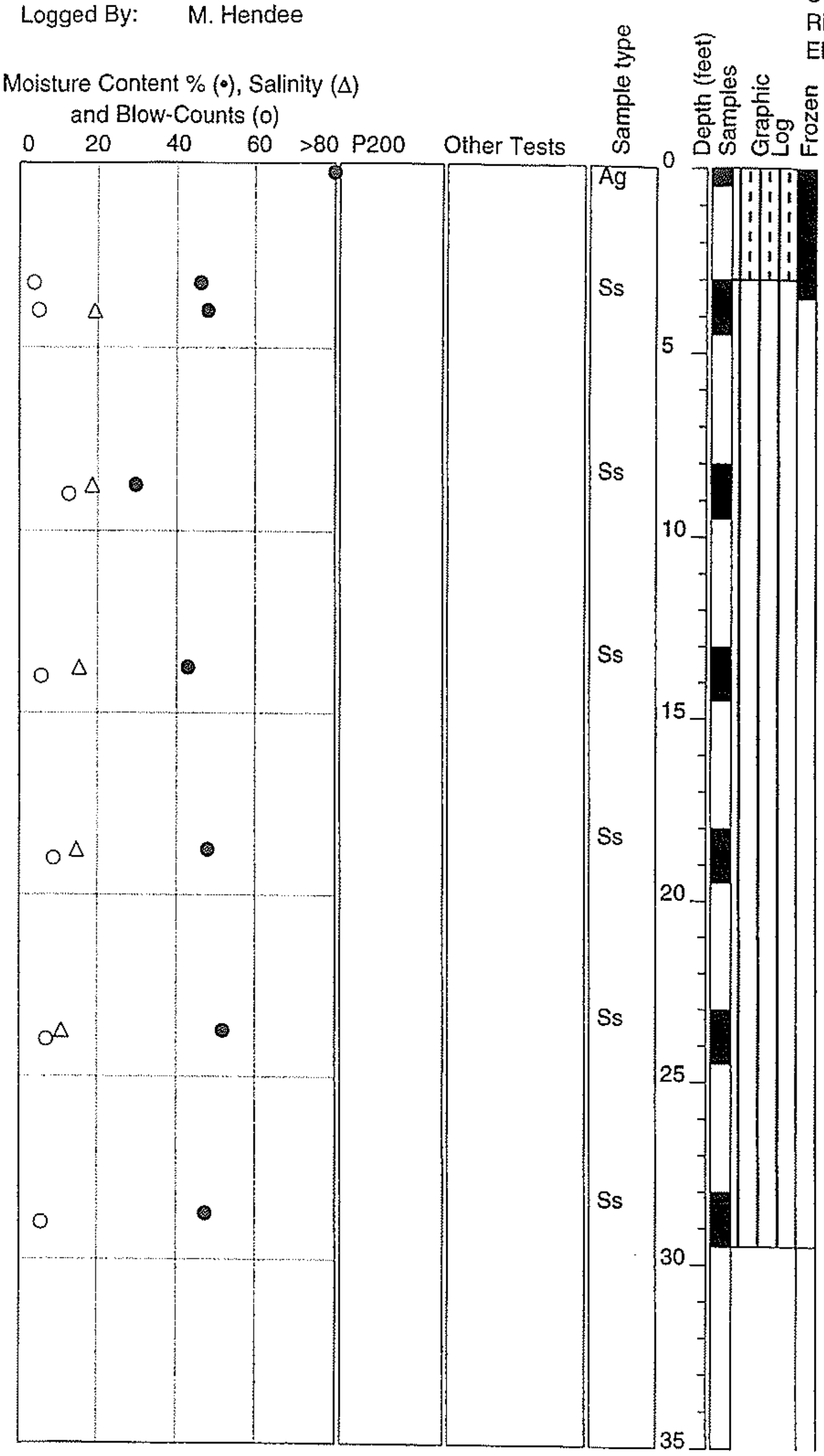
DUANE MILLER & ASSOCIATES
Project: Kipnuk Fuel Tanks
DM&A Job No. :4095.44
Logged By: M. Hendee

Log of HOLE : B-5

Date Drilled: March 10, 1998
Contractor: Denali Drilling
Rig Type: CME-45 W/ 6" O.D. Hollow Stem
Elevation: 88.5 ft*

Light Plant Tank Farm



Description

ORGANIC SILT: (OL) (Vx) Gray-brown, frozen w/ trace visible ice as crystals

SILT: (ML) (Vr) Dark gray w/ trace sand, seasonally frozen to 3.5' w/ visible ice as randomly oriented lenses 1/16" thick, wet & medium stiff below 3.5'

*Elevations were measured with a hand level. An arbitrary datum of 100.0 feet was established at the top of the southwest corner of the burned trailer.

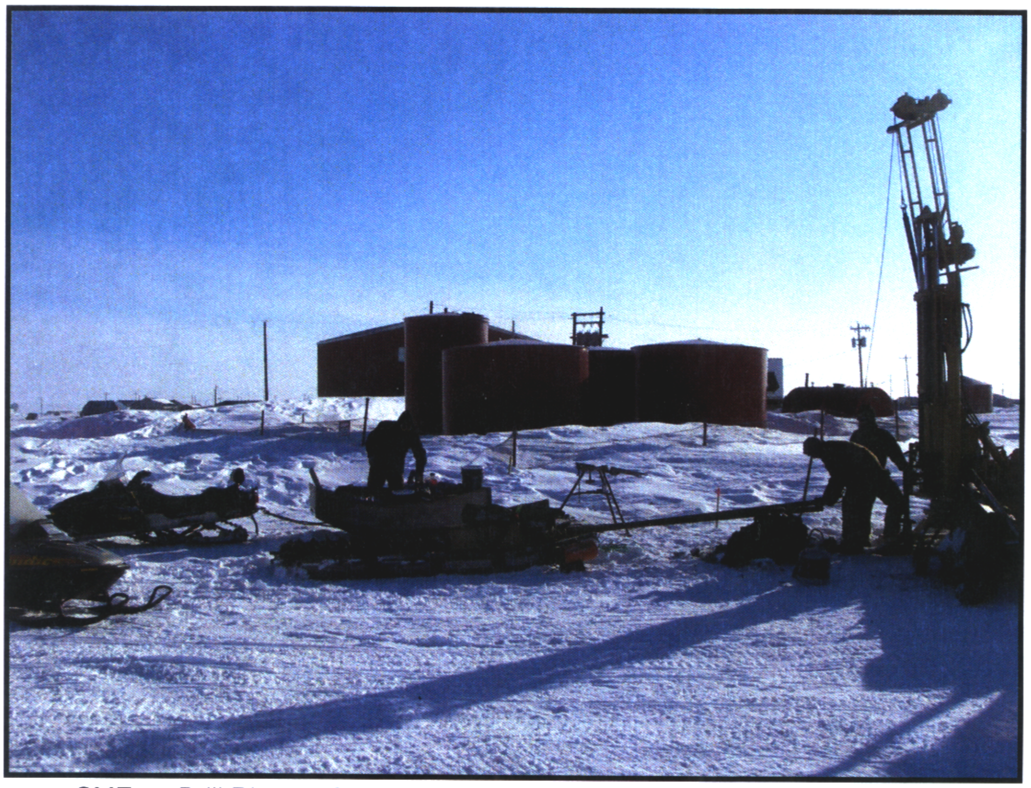
Duane Miller & Associates
Arctic & Geotechnical Engineering
Job No.: 4095.44
Date : May 1998

LOG of BORING B-5
Fuel System Upgrade
Kipnuk, Alaska

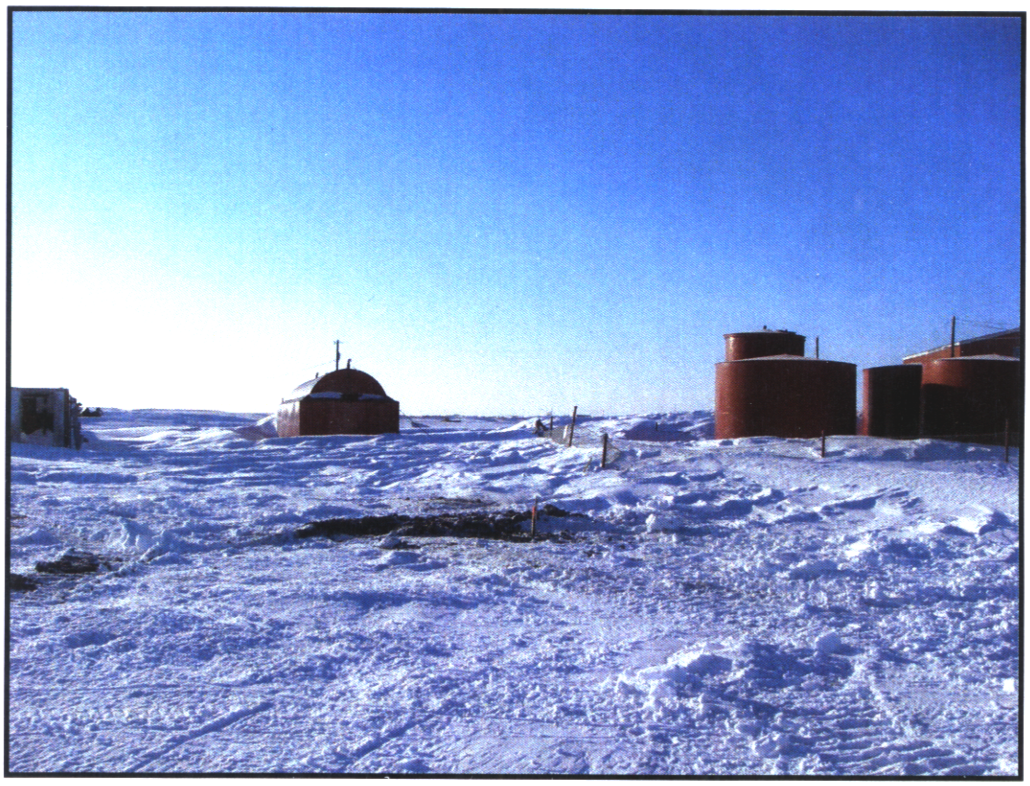
Plate
6

Appendix B

Representative Site Photographs
2007 DMA Site Investigation



CME-45 Drill Rig and Support Equipment, Kipnuk Fuel Tank Site, March 2007

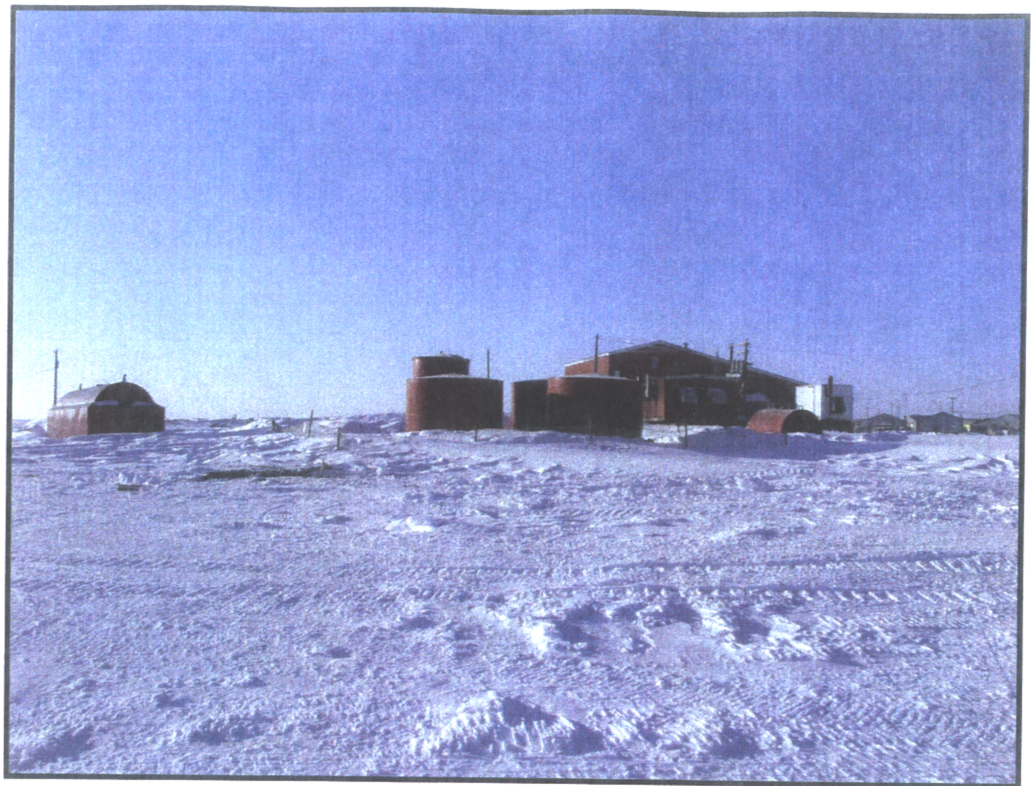


General Site Conditions, Kipnuk Fuel Tank Site, March 2007

Duane Miller Associates LLC
Job No.: 4095.111/131
Date: June 2007

SITE PHOTOGRAPHS
AEA Bulk Fuel and Powerplant Facility
Kipnuk, Alaska

Plate
B-1



Existing Fuel Storage Facility and Corporation Offices, March 2007

CME-45 Drill Rig (Skid Mounted) and Village Corporation JD 450 Dozer, March 2007