A report prepared for

HDR Engineering
2525 C Street
Anchorage, AK 99503

GEOTECHNICAL EXPLORATION
Atka Water Project
Atka, Alaska

by

Duane L. Miller, P.E.
Civil Engineer 3696-E

Laura J. Silliphant
Geologist

DM&A Job No. 4086.055

Duane Miller & Associates
5821 Arctic Boulevard
Anchorage, Alaska 99518
(907) 644-3200
FAX 644-0507

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INTRODUCTION

The city of Atka has received funding for water and wastewater system improvements. HDR, Inc. is implementing the design, and Duane Miller & Associates, DM&A, is providing the geotechnical support to HDR. The geotechnical exploration was performed in accordance with our agreement with HDR, Inc. dated June 30, 2005.

This report presents the results of the soil investigation for the proposed water supply impoundment, the foundation conditions at the water treatment plant (WTP), and two new water storage tanks in Atka, Alaska. The scope of our services was to review existing geotechnical information from the area; explore the soil, bedrock, and groundwater conditions at the proposed sites; and assess the borrow source locations. After gathering and reviewing the data, we developed conclusions and recommendations regarding geotechnical aspects of site preparation and foundation types.

The impoundment site is located approximately 0.6 miles west of the existing WTP in Old Atka Village. The impoundment structure was constructed in 1978 and is comprised of a timber dam with a 4-inch polyvinal chloride (PVC)

pipe and two short sections of intake screens. A recent inspection of the impoundment site found the timber dam is rotting and the slopes above the excavated area are eroding. HDR, Inc. recommends that the timber dam be replaced with a concrete weir founded on bedrock. The approximate dimensions of the new structure will be 60 feet long by 7 feet tall.

The existing WTP is approximately 40 feet long by 24 feet wide. The initial proposal was to extend the WTP along the south side of the existing structure. DM&A conducted their field investigation in alignment with the initial proposal. With further consideration of the water treatment system, the City of Atka and HDR advised to not extend the WTP. Their recommendations are to demolish the existing wood structure, repair the existing concrete slab and footing, install new anchor bolts in the existing footings and slab, and construct a new wood frame building. This report will include the field observations and recommendations for the initially proposed extension.

The proposed water storage tank sites are to be located near the existing 30,000-gallon wood stave storage tanks. The Old Atka Village storage tank is located approximately 175 feet northwest of the WTP. The New Atka Subdivision water storage tank is located approximately 450 feet southwest of the New Atka Subdivision. HDR will prepare design drawings and calculations for tank replacement. The city of Atka will ultimately decide whether to repair the existing wood stave water tanks or replace them.

Three borrow sources are currently being used by the city of Atka: the scoria borrow pit across from the airport, the rock quarry northwest of the fish plant, and the sand dunes across the street from the old fuel tank farm.

INVESTIGATION

Existing Data

The investigation began with a review of the existing data and field observations from previous work in the village. A summary of each reference follows:

- HDR, Inc. investigated and documented the city of Atka's impoundment site, water treatment plant, Old Atka Village water storage tank, and the New Atka Subdivision water storage tank for a preliminary design for the city's water and wastewater system improvements (memorandum dated January 24, 2005). Cores taken from the timber impoundment area showed rotting wood. Hill slopes are eroding into impoundment area. Deterioration was found in the foundation and footings of water treatment plant. Both 30,000-gallon wood stave storage tanks look stable with minor leakage noted. Valves in the water storage tanks freeze up when drained due to demand. HDR's recommendations are to replace the timber dam at the impoundment area with a concrete weir and to fix and reuse the existing foundation at the water treatment plant and extend the facility to south. The city of Atka will need to decide whether to fix valves and perform minor repairs to wood stave tanks or to replace them with new water storage tanks.

- Duane Miller & Associates conducted a geotechnical investigation for water and sewer improvements in Atka (report dated October 13, 2000). The analysis was based on evaluation of existing geotechnical data. Based on the review, DM&A would expect to find a surface layer of compressible peat and organic silts deposited over volcanic ash mixed with old topsoil horizons. These soils have a high moisture content, which would preclude their use as a compacted material. Granular fill material is available from old quarries and beach deposits.

- In 1984, James Hein published the paper: "Reconnaissance geology of southern Atka Island, Aleutian Islands, Alaska" in the USGS bulletin 1609. The paper reviews the geologic history and composition of the volcanic flows and the plutons emplaced on Atka Island. The volcanic rocks are basalt, basaltic andesite, and andesite; and the plutonic rocks are comprised of gabbro, diorite, norite, tonalite, and quartz diorite. The volcanic flows are Tertiary and Quaternary in age, and the pluton emplacement occurred during the Miocene Epoch.

- R&M Consultants, Inc. (report dated June 13, 1980) performed a subsurface soils investigation for development of 18 housing units. The investigation was in an area approximately 1 mile north of the original town site. Five trenches were dug with the city of Atka backhoe to depths of 6 to 9 feet to evaluate site soils and to provide data for foundation recommendations for

the housing. Soils in the housing area generally consisted of soft and wet organic soil with wet silt, sandy silt and organic silt. Cobbles were encountered at a depth of 7 feet in one hole. Groundwater was encountered at in four of the five trenches: at 6-inches (one trench) and 4-feet (3 trenches). The fifth hole was dug on an old beach line and found clean sand to a depth of 6 feet.

- Shannon & Wilson prepared a geotechnical report to provide recommendations for construction of a dock at Nazan Bay. The report is dated October 15, 1993. Six borings were drilled at the proposed dock site to depths of 10 to 17 feet below mudline in water depths of 15 to 25 feet. All of the holes encountered loose to dense sand and silty sand with some fine gravel and then encountered refusal, which was interpreted to be shallow bedrock.

- In a November 1995 report Shannon & Wilson presented results of a subsurface investigation for the upgrade of the unpaved access road and construction of a new bridge between the Atka airport and the proposed new dock in Nazan Bay. They dug forty-one (41) test pits in February 1995 using the local backhoe, collected samples and monitored groundwater conditions. This investigation also evaluated potential borrow sources for subbase or base course materials for the road. The report notes that the military had mined gravel from Dock Creek shoreline and Korovin Bay beaches. Degradation values on samples of possible borrow material were generally low except for gravel from the beach. Natural soil conditions adjacent to and under the old roadway embankment consisted of high moisture content surface organic material over silt and silty sand. Both wind deposited sand and volcanic ash are commonly interlayered with the organic material.

- Engineering recommendations for the Atka Dock Creek Bridge and Road Rehabilitation were presented in a November 1995 report by Shannon & Wilson, Inc. This document is a companion report to the field investigation report noted above and contains all the site specific design recommendations for the dock and 5.6 km of roadway. The report notes the softness of local rock and suggests several soil stabilization methods for local materials.

Subsurface Exploration

We explored the proposed sites for the water supply impoundment, the extension to the water treatment plant (WTP), and the Old Atka Village water storage tank by digging and sampling a total of three test pits to depths ranging from 5.5 to 8.5 feet. In addition, surficial samples were obtained and a Minnesota swamp probe was used to determine the depth of the surface cover along the length of the dam in the impoundment site and around the existing New Atka Subdivision water storage tank. The exploration was performed from August 31,

2005 through September 2, 2005, under the direction of Ms. Laura Silliphant, DM&A Geologist.

The test holes were excavated using a Caterpillar 420D Backhoe operated by Arnold Golodoff, a city of Atka employee. The first test hole, dug on August 31, 2005, was located 38 feet south of the southeast corner of the WTP and was 7.2 feet deep. On September 1, 2005, the remaining two test pits were completed. The second test hole was excavated 22 feet west of the Old Atka Village water storage tank to a depth of 5.5 feet. The third test hole was dug 100 feet northwest of the northwest fence corner and 118 feet from the northeast survey stake of the impoundment area. The total depth of the third test hole was 8.5 feet. Ms. Silliphant observed and logged the soil conditions encountered in each hole and collected representative soil samples.

The test hole locations were located using a cloth tape to measure distances from identifiable landmarks and survey markers and then quantified by use of a handheld GPS system. The coordinates are shown on the boring logs. The datum is NAD-83. The test hole locations are shown on Plate 1.

On September 2, 2005, Ms. Silliphant visited and collected samples from three different borrow sites: the scoria borrow pit across from the airport, the rock quarry northwest of the fish plant, and the sand dunes across the street from the old fuel farm. Soil samples from the borrow sites and the test holes were double sealed in plastic bags, labeled, and then inventoried in preparation for shipment to our Anchorage laboratory.

Laboratory Testing

In our laboratory, the samples were reexamined to verify field classification and to select samples appropriate for testing. The laboratory testing included moisture content, organic content, gradation analysis, dry density, plasticity, and salinity, as appropriate. The logs of the test holes are presented on Plates 2 through 4. The soils have been classified in accordance with the Unified Soils Classification System described on Plate 5. The results of the laboratory tests are shown graphically on the logs of the test holes and are tabulated on the Summary of Samples, Plate 6. Details of the classification tests are shown on the Particle Size Data Sheet, Plate 7. The plasticity chart is found on Plate 8.

SITE and SUBSURFACE CONDITIONS

Physiographic Setting

The city of Atka is located on Atka Island in the Andreanof Island group, 90 miles east of Adak in the Aleutian Islands. Atka Island is geographically and geologically composed of two parts, which are joined by a 3-mile long isthmus separating Korovin and Nazan Bays. The physical terrain of Atka Island is rough and mountainous on the northern part of the island and hilly to steep mountainous on the connecting isthmus and southern part of the island. Both the city of Atka and the areas of interest are located on the southeastern shore of the isthmus. The vegetation covering the hilly to steep mountainous terrain is comprised of grasses, fireweed, and a variety of other herbaceous plants. Grasses cover the sand dunes that run along the isthmus coastlines. Surface drainages are generally short and straight.

Volcanic ash layers cover most of the bedrock on Atka Island. The bedrock on the northern part of the island consists of an uppermost Tertiary and Quaternary volcanic terrane known as the Korovin volcanic complex. The bedrock on the southern part of the island is mostly comprised of Tertiary igneous and volcaniclastic rocks. The isthmus, on which the city of Atka is built, is comprised of volcanic rocks of the Korovin volcanic complex overlying late Tertiary rocks.

The volcanic rocks are basalt, basaltic andesite, and andesite; and the plutonic rocks include gabbro, diorite, norite, tonalite, and quartz diorite. Three main volcanic episodes occurred at Atka Island: a submarine eruption during late Eocene and early Oligocene; a basalt and basaltic andesite eruption during the middle and late Miocene; and the build up of stratovolcano deposits (mostly on the northern part of the island) during the Quaternary. Plutonic emplacement occurred in the southern part of the island during the early, middle, and late Miocene.

Pleistocene glaciation blanketed this part of the Aleutian chain and glacial remnants remain on the higher peaks.

The climate at Atka is characterized as maritime with mild winters and cool summers. The wind is calm only 2.5 percent of the time. Temperature data published by the Alaska Climate Research Center (ACRC) is available for Adak Island, which is 90 miles west of Atka. On Adak, January is the coldest month with a mean temperature of 31.8°F. August is the warmest month with a mean temperature of 50.9°F. The mean annual precipitation on Adak Island is 54.12 inches and the mean annual snowfall is 98.8 inches. ACRC's mean temperature and precipitation values are based on data collected from 1971 to 2000.

Site Conditions

The impoundment site is located approximately 0.6 miles west of the existing WTP at an elevation of approximately 213 feet above Mean Lower Low Water (MLLW). A gravel road and ATV trail lead directly to the north side of the impoundment area. The ATV trail stops approximately 100 feet from the dam. A chain link fence surrounds the impoundment. Hillside erosion is occurring within the fenced area. Vegetated slopes ranging from 15-28 percent still exist (have not sloughed into the impoundment). Within the fenced area, the soil on the north side of the dam was saturated.

The WTP site is in the Old Atka Village at an elevation of approximately 73 feet above MLLW. The building is approximately 40 feet long by 24 feet wide. The concrete foundation, along the south wall of the equipment room, has a small fracture that runs along the length of the wall. Along the south side of the building the footing is exposed. The lot, where the original extension was planned, slopes up to the west. The surface is soft, saturated, and covered with grass.

The water storage tanks sites are near the existing 30,000-gallon wood stave storage tanks. The Old Atka Village water storage site is located 175 feet northwest of the WTP at an elevation of approximately 114 feet above MLLW. A gravel road and driveway to the Old Atka Village water storage tank allows for access to the site. There are two buildings near the site. Building materials and debris are on the ground near the water storage tank. The site is well drained.

The New Atka Subdivision water storage tank is located approximately 450 feet southwest of the New Atka Subdivision at an elevation of approximately 152

feet above MLLW. A walking trail leads up to the New Atka Subdivision water storage tank. The walking trail is grassy and steep so it may not provide suitable site access for construction equipment without upgrading the trail access.

Soil Conditions

The soil conditions for the water improvement sites were explored by digging test holes in areas that were accessible by road and by probing the depth to hard ground in areas that were less accessible. In all test holes, a thin organic mat caps a series of horizontal ash layers that sit upon volcanic bedrock (see Plates 2 through 4).

The soil profile at the impoundment site, test hole AIMP-3, consisted of 6 inches of peat overlying 1.5 feet of organic silt. Red, plastic sandy silt was encountered below the organic silt and continued down to 6.4 feet. From 6.4 feet to 8.5 feet, gray and brown clay interspersed between cobble-sized pieces of bedrock and hard clumps of clay was encountered (see Photo Plate 10). 8.5 feet was the maximum depth that the backhoe could excavate. Water trickled into the test pit from a small seep perched above the clay and bedrock at 6.5 feet.

In addition to the test hole at the impoundment site, Ms. Silliphant probed along the west face of the dam to verify the soil and bedrock profile presented on the as-built drawings. The reference point from which all distances were measured was the north corner of the spillway (see Plate 9 for the soil profile). On the north end of the dam, the soil thickness (ground surface to top of hard surface) was 5.5 feet where the ground has not sloughed. On the south end of the dam, the soil thickness was 3.2 feet (ground surface to top of hard surface). Closer to the spillway, the depth to hard surface was approximately 1.0 to 1.5 feet on the north side and 0.5 to 2.0 feet on the south side. The ashy, organic silt in the sloughed areas on the south side of the dam was also probed and sampled. The soil thickness in the sloughed area was 2.7 feet, and the approximate thickness of the overburden that sloughed was 2.0 feet. The combined sum is 4.7 feet, which is in general agreement with the overburden thickness on the as-built drawings.

At the WTP, test hole AWTP-1 encountered a 1-foot layer of peat over 3 feet of organic silt. Three feet of highly plastic sandy silt was found below the organic silt, and the top of bedrock was reached at 7 feet. While digging the test hole, water continuously seeped from the peat and organic silt layers. The sides of hole also sloughed into the test hole (see Photo Plate 11). Below the southeast corner of the WTP, the footing and the underlying sand fill were exposed in a 1.5 foot backhoe dug test pit. Black, fine to medium grained sand was present beneath the footing (see Photo Plate 12). Water, possibly from a leaking water pipe, immediately began to fill in the hole.

Up the hill from the WTP, test hole OAWT-2 was dug 175 feet northwest of the existing Old Atka Village water tank. The soil conditions were drier than at the WTP. At test hole OAWT-2, 0.5 feet of peat overlies 2.25 feet of organic silt and then 2 feet of highly plastic, sandy silt. Bedrock was encountered at 5 feet (see Photo Plate 13).

The New Atka Subdivision water tank site did not have road access so the soil exploration was limited to probing and soil sampling in the drainage around the water tank. In the drainage, hard ground was encountered at 1.1 feet below the ground surface. The depth to the bottom of the drainage was 2.25 feet. Thus, the total estimated thickness of soil below the water tank is 3.3 to 3.4 feet. Geotextile fabric was also exposed in the drainage. The fabric was placed upon ashy organic silt and sandy silt layers. Organic silt was sampled at 1 foot below the ground surface and sandy silt was sampled at 1.6 feet below the ground surface (see Photo Plate 14).

Samples from each borrow site were collected and shipped to Anchorage, where sieve analyses were completed on the soil samples. The scoria borrow pit is comprised of basalt and weathered basalt. The material is traditionally mined from the wall by scraping it with a toothed bucket. The weathered basalt breaks down into a gravelly material of varying consistency, GP-GM. The sand dunes along Nazan Bay are comprised of black, poorly graded, fine to medium-grained sand, SP. A borrow sample was collected from the sand dune across the street from the fish plant. The rock quarry borrow site is comprised of large boulders of andesite that have been mined out of the rock wall. The rock face appears to be ripable along preexisting joint planes (see Photo Plate 15).

DISCUSSION and CONCLUSIONS

The soils at Atka are relatively uniform. A thin organic mat caps a series of horizontal ash layers that sit upon bedrock. The organic mat ranges in thickness from 6-inches to 1-foot and is comprised of peat. The upper 2 to 2.5 feet of the ash is organic silt, and the lower ash layers are sandy silt. The base of the sandy silt transitions into a weathered bedrock layer comprised of clay and rock fragments.

The geotechnical properties of the soils were noted in the field and verified in the laboratory. The peat and organic silt are both compressible materials. The sandy silt is highly plastic. Four plasticity index (P.I.) tests, a minimum of one sample per test hole, were run on samples from the sandy silt. The P.I. values of the sandy silts ranged from the mid-twenties to the upper-fifties, which are considered reasonable for volcanic fine-grained soils. The liquid limit for the sandy silt samples ranged between 101% and 202%, and the natural moisture content of the samples ranged between 102% and 220% (see Plate 8). In three out of the four samples, the natural moisture content exceeded the liquid limit. In the fourth sample, the natural moisture content closely approached the liquid limit.

Water was encountered in two of the three test pits and while digging near the southeast footing of the WTP. At test hole AWTP-1, water from the peat and organic silt layers (the upper 4 feet) constantly dripped into the open hole. The sides of the test pit sloughed into the open hole as well. The location of the existing WTP is lower on the hillside and not well drained. The water encountered closer to the building, however, may be due to a broken water pipe. The groundwater encountered in the impoundment test hole, AIMP-3, was much deeper (6.5 feet below the ground surface) and slowly trickled out of a small seep perched on the weathered bedrock layer. No groundwater was encountered in the Old Atka Village water tank test hole, OAWT-2.

The sieve analyses indicate that the material from both the sand dune borrow site and the scoria borrow pit will be suitable for embankment fill with some restrictions. The dune sand will provide adequate bearing for the proposed structures; however, if the fine sand is saturated, it may not adequately compact. The fine sand will also be expected to drift if it is exposed at the surface

and not stabilized with vegetation. The scoria borrow material is less uniform in gradation and may require mechanical processing if a uniform gradation is required. The scoria borrow material will also have a greater fines content if crushed, which could increase the soil moisture sensitivity during compaction as well as frost susceptibility. The rock from the rock quarry is probably suitable for riprap at the impoundment site. The rock may require on-site processing to meet final riprap specifications. The rock face also appears to be ripable along preexisting joint planes (see Photo Plate 15). Blasting will most likely be required to get a smaller size class.

If any borrow site is being considered for concrete aggregate, we strongly recommend a design mix be conducted to assure concrete strength and cure rates can be achieved.

RECOMMENDATIONS

Geotechnical engineering recommendations are provided for the following project elements:

- Water Impoundment Site
- Water Treatment Plant Addition
- Water Storage Tanks (Old Atka Village and New Atka Subdivision sites)

Water Impoundment Site

Key improvements at the Water Impoundment Site will include:

- Removal of the existing timber dam and replacement with a new reinforced concrete structure.
- A spillway will be constructed near the center of the new dam structure with riprap planned on both up and down gradient faces of the spillway.
- A new water intake system and a new water supply pipe will be installed near the base of the new dam structure.
- Improved site access is planned for the new dam structure to facilitate site work, construction and maintenance.

New Dam Structure

Before constructing the new dam at the existing impoundment site, an upstream diversion system will be needed to maintain water supply to the city during construction. After removal of the existing timber structure, all surficial material within the footprint of the new dam structure will have to be removed to expose a firm, competent bedrock surface. Safe design of the side slopes along the excavation footprint will have to take into account a wide range of factors such as equipment loads and proximity, time and sequence of work, proximity and height of spoil piles, weather, and workmanship. Most of these variables are most easily monitored or controlled by the contractor. The contractor should be responsible for performing the trenching, drain placement, and backfill operations in a safe manner consistent with state and federal labor and safety codes. Within the impoundment site, vegetated slopes ranging from 15 to 28 percent remained stable. Surface and groundwater is expected to enter the

excavation so water control will be necessary to keep water from ponding during construction.

The new concrete dam structure must be seated on firm, competent bedrock. Based on anticipated site conditions, the surface of the bedrock may require over-excavation to remove loose or weathered materials and hand clearing may be needed in areas. Once firm, competent bedrock is exposed, surface cleaning with compressed air (air lance) or high-pressure water should be conducted to confirm suitable bearing surface. We recommend an experienced geologist or geotechnical engineer confirm suitable dam bearing surface prior to constructing the dam structure

Once suitable bedrock is confirmed, grouted vertical reinforcing bars are recommended to key the dam structure to the bedrock. Civil and structural engineering design will determine the reinforcing bar size and corrosion requirements. We expect that a 2-foot minimum embedment into competent bedrock would be sufficient.

Reinforcing bars should be installed in pre-drilled holes allowing a nominal 1-inch radial annulus around each bar. The boreholes should be plumb and must be free of deleterious matter prior to installing the reinforcing bars.

Grout should consist of Portland Type III cement at nominal cement to water ratio of between 0.40 and 0.45. All grout must be pre-mixed with potable water at 45°F to 55°F at time of placement and maintained at this temperature range during curing. Grout should not be placed in boreholes with visible ice or ground temperature below 45°F. Grout should be tremie placed in a manner that prevents air pockets or bridging to occur.

We anticipate water will be present in the boreholes but that a sufficiently dense grout will displace this water if tremie placement methods are used. Grout plasticizers and accelerants may be considered and we would recommend confirming a grout design mix prior to construction. Grout must be allowed to cure for at least 72 hours and until compressive strength tests have confirmed grout strength has developed. On-site oversight by an experienced material technician is recommended during grout mixing and placement.

After reinforcing bars have been installed and grouted, the concrete dam can be completed in accordance with engineering specifications. After the concrete structure has been placed and cured, we recommend an Alaska Department of Transportation and Public Facilities (ADOT/PF) Class I Riprap be placed along the spillway and intake structure system on both upstream and downstream faces of the concrete dam. The riprap will be used to develop lateral resistance and to reduce overturning moment. The riprap should extend out from the dam structure in each direction at a 1.5H : 1V slope. A 12-inch wide bench at the top. Riprap must be placed on sound bedrock. Outside the riprap prism, non-organic fill such as dune sand may be placed at a 2H : 1V slope. Non-organic fill is considered non-structural and as such will not sustain engineering loads. The non-organic fill should be placed in nominal 12-inch lifts and mechanically compacted to 90-percent of modified Procter, ASTM D-1557. A thin organic fill section (topsoil) , erosion mat (jute mat or coconut fiber) and seeding is recommended atop the non-riprap fill section.

Site geometry along the downstream side of the dam structure may require a large fill section. The fill section may be terminated against a shallow retaining structure, such as gabions or riprap, to reduce fill volume. If a toe structure is planned, we recommend a design review to determine for conformity with our geotechnical recommendations.

Civil engineering design will not permit water seepage between the concrete dam and the bedrock nor permit water seepage along the water line penetration through the dam. Piping penetrations through the dam structure should be designed as to not permit water seepage under both static and dynamic (earthquake) loading conditions. Piping connections between the dam penetration and the downslope water supply line should be designed to allow for both vertical and horizontal movements resulting from earthquake forces and seasonal temperature flux. Depending on exposed bedrock geology and quality, seepage through the bedrock may occur. Based on existing conditions and reasonably expected bedrock geology, we do not anticipate the need for water seepage control systems under the concrete dam, such as cut off walls or grout curtains. However, if such geologic conditions become evident during site work, we should be immediately notified.

Water Intake Structure and Downstream Water Pipe

A water intake structure will be installed on the upgradient side of the new dam and a new water supply line will be installed through the dam and down gradient of the dam. The upstream intake structure should be founded on a compact riprap section and may warrant a basal concrete thrust block. Downstream of the dam face, the new water pipe will be buried within the riprap section. The water pipe must be capable of resisting an effective overburden pressure in the range of 1,200 pounds per square foot (psf). If a ductile iron or high density polyethylene (HDPE) material is used for the new water pipe section through the dam and the riprap section, then a nominal 6-inch bedding section surrounding the pipe using sand from the sand dune borrow site is recommended. If polyvinyl chloride (PVC) or similar pipe material is planned, a more extensive bedding section may be required. We should be contacted for pipe bedding recommendations if material other than ductile iron of HDPE is planned.

Regardless of water pipe material, the point of exit at the dam face may induce large stress on the pipe if the underlying fills section settles. It is important that the fill material underlying the water pipe, within 4-feet of the dam face, be adequately placed and compacted to reduce post-construction settlement. If Class I rip-rap is used under the water pipe at the dam face, mechanical vibratory compaction is recommended. Typically, riprap does not mechanically compact under ASTM D-1557 specifications due to the large, angular particle size. In lieu of the ASTM D-1557 specification, we recommend a performance-based mechanical compaction effort to reduce post construction settlement. For Class I riprap, 12-inch thick lifts subjected to four (4) passes with vibratory plate compactor with the following recommended performance-based specifications:

Vibrations per minute (VPM, minimum)	5,000
Impulse (centrifugal) force (lbs, minimum)	5,000
Travel rate (feet per min, maximum)	20

Outside the dam fill prism, the water pipe will remain buried to the water treatment plant. We understand two options are under consideration for the water line outside the dam fill prism: (1) connect to the existing PVC water line or (2) complete replacement. If complete replacement is conducted, the new water pipe will most likely be placed in an excavated trench, approximately 3 to 5-feet below existing grade. The water pipe should be designed to withstand a static vertical load in the range of 750 psf.

If the water line is designed to allow for differential settlement on the order of 4 to 6-inches over an 8-foot section and a suitable HDPE material is used for the water pipe, trench excavation can be used as unclassified backfill. However, rock fragments larger than 3-inch diameter or sharp-edged coarse gravel (material greater than 1-inch diameter) should not be placed directly adjacent to the water pipe. Trench backfill should be mechanically compacted to 85-percent of dry density as determined by ASTM D-1557 and mounded above the water line to allow for additional settlement and positive drainage away from the pipeline. Based on reasonably anticipated soil conditions, a 6 to 8-inch mound should be sufficient to accommodate long-term settlement.

If the water line cannot tolerate differential settlements stated above, additional trench preparation including overexcavation of soft or yielding soils, granular backfill and pipe bedding may be required to control settlements. Granular fill may require check plugs along the water line to control groundwater piping of finer grained soils.

Impoundment Site Access Improvements

An improved access road to the impoundment site will be required for construction access and post-construction maintenance access. Based on site geometry, the preferred alignment appears to be a 16-foot wide road from below the elevation of the impoundment site and then access the site along the same contour elevation as the impoundment site. One turnout will be planned along the access. Site work for access improvements will be kept below the impoundment reservoir and can be coupled with dam fill prism excavation work.

Access roads, pads and turnouts can be designed for several levels of durability, depending on capital costs and site needs. For this project, a minimal prism is preferred with the understanding that longer-term maintenance will be incurred. We recommend the following geotechnical elements:

- Hand clear the alignment of large obstructions (greater than 8-inches). Maintain organic mat in an undisturbed state.
- Place a geotextile fabric (Propex 2002 or 4510 (or similar) in accordance with manufacturer's recommendations
- Place 18-inches of well-graded granular fill, such as the material from the scoria borrow site, compact to 90-percent of modified Proctor (ASM D-1557)
- Crown for surface drainage
- Constructed fill prism side slope at 1.5H : 1V
- If necessary for site access, cut backslopes at 3H : 1V, install thin topsoil layer, vegetation mat (jute or coconut fiber) and seed.
- Along upslope sides, provide for drainage swale at toe of fill prism

If areas of higher elevation are encountered along the access prism, isolated removal of the organic mat and underlying organic or mineral soil may be necessary to maintain grade. If so, careful excavation and placement of geotextile and granular fill is advised. If culverts are needed to control site drainage, CMP or HDPE culverts are recommended with geotextile wrap and Type I rip-rap at culvert entrance and exit points. With this design, additional effort should be anticipated to maintain the stability of the fill prism particularly at culverts and locations where the organic mat is removed. Isolated pumping of underlying fine soils may occur during and after construction of the impoundment.

Water Treatment Plant

An extension to the existing WTP was originally proposed. This extension be a slab-on-grade with continuous concrete foundations, similar to the existing WTP building. The excavation footprint under the foundation wall and the slab-on-grade will require excavation to firm mineral soil (sandy silt). All overlying organic material under the proposed building footprint will require removal and offsite disposal. Owing to the high moisture sensitivity of the mineral bearing soils, we strongly recommend careful site work, construction staging, and caution accessing the exposed sandy silts with construction equipment during

wet periods. If the sandy silt bearing soils should become disturbed a severe loss of cohesion and soil strength may occur.

Under the footings, the in-situ mineral sandy silt should be leveled and proof compacted to an unyielding surface prior to foundation construction. If granular fill is required to attain foundation grade, a geotextile similar that discussed in the previous section is recommended between the in-situ soil and granular fill. For this site, granular fill consisting of locally available clean fine sand can be used. This fine sand should be placed in 8-inch thick lifts vibratory compacted to 95-percent of maximum dry density as determined by ASTM D-1557.

The fine sand available from the local borrow sites would be suitable for use as fill at the WTP site. The fill material should be free of organic material, thawed, and contain less than 5-percent (by dry mass) material finer than the U.S Number 200 sieve size. An alternative borrow site (the scoria site) may be substituted for the fine sand material with some restrictions on grain size and frost concerns. If the scoria material is planned for backfill under foundation or the slab-on-grade at the WTP site, we should be contacted in order to provide additional geotechnical recommendations.

The WTP building can be supported on a continuous spread footing foundation placed on a properly prepared base of in-situ mineral sandy silt or compact fine sand. The perimeter footings should be placed at least 24 inches below exterior finish grade, and interior footings, if used, should be at least 12 inches below the finish floor. Continuous strip footings should be at least 12 inches wide and column footings should be at least 24 inches wide. Footings can be designed using a bearing pressure of 3,000 pounds per square foot (psf) for dead plus real live loads. The bearing pressure can be increased by 50% for short term transient loads such as wind and seismic.

Uplift resistance can be achieved by the static weight of the soil above the concrete perimeter footings. Assuming an 8-inch thick concrete foundation wall, a nominal 4-inch section can be used to resist uplift loads. Assuming 24-inch embedment, uplift resistance on the order of 250 pound per lineal foot of perimeter foundation wall can be assumed with a factor of safety of 1.5.

If constructed on a properly prepared base, perimeter foundation wall total settlement will be less than 1-inch over a 20-year design life. Most of the settlement will occur as the loads are applied so post-construction differential settlements are expected to be less than 0.5-inch over this period.

The footings will develop resistance to lateral loads by passive pressure against the face of the footing and by friction on the base of the footing. Passive pressure can be calculated as an equivalent fluid pressure of 350xH psf, where H is the depth below adjacent grade. If the footings are constructed as cast-in-place elements, the friction on the base can be calculated as 0.30 times the vertical dead load on the footing. To restrict potential movements, these lateral resistances already include a factor of safety of two.

Slab-on-grade should be placed on a compact, fine sand fill as discussed above. The fine sand will have greater capillary forces relative to a well-graded material. We recommend a capillary break be used under the slab. Typical capillary breaks such as 4 –inches of crushed, washed aggregate (pea gravel or similar) or synthetic liners may be suitable. If isolated interior column foundations are planned, we recommend a minimum 4-inch thick fine sand fill section between the top of the column foundation and the base of the slab be used. If a slab-on-grade floor in constructed atop a properly prepared base of fine sand as discussed above, a bearing capacity of 500 psf can be used, with a factor of safety of 1.5.

The foundations and slab-on-grade should be maintained above freezing throughout the structure's life. Site grading should direct drainage away from the structure. If roof drainage will impact the foundations, we recommend a foundation drain system of 4-inch perforated drain pipe be installed. The foundation drain should include a geotextile wrap to reduce siltation and be directed away from the structure to a dry well or to daylight down slope of the structure. Any exterior underground or at-grade appurtenances accessing the structure should be designed to permit frost or settlement related movement without damaging the structure. This would include items such as drain lines, fuel tanks, stairways, and garage or door accessways.

Old Atka Village Water Tank Site

A new potable water tank is planned for the Old Atka Village site. This tank will be on the order of 60,000-gallons, steel frame and unheated. The water tank will be seated directly on fill material. It will not be supported above grade.

The tank should be supported on the sandy silt material underlying the surface organics and compressible organic silt. All soft and compressible materials should be removed from the water tank area, and the base of the excavation should extend to a horizontal distance of at least five feet beyond this area. Locally available fine sand is recommended for fill material.

The exposed sandy silt soil should be proof rolled to identify soft or yielding zones. Soft or yielding zones will require overexcavation, backfill and compaction with fine sand. A geotextile, similar to that discussed previously, should be placed atop the prepared sandy silt surface. Fine sand fill can then be placed in 8-inch lifts and compacted to 95-percent of maximum dry density per ASTM D-1557.

If the fine sand is used for fill under the tank, a concrete or all weather wood (AWW) timber ring wall is recommended.

Several options may be feasible for uplift resistance. One option is to bury the ringwall at least 2-feet below finish grade with a nominal 4-inch uplift bearing surface, similar to the WTP building. If so, uplift resistance on the order of 200 pound per lineal foot of ringwall can be assumed. A second option is to install anchor rods grouted at least 2-feet into competent bedrock, similar to those discussed in the Impoundment section. If so, approximately 1,500 pounds of uplift resistance per anchor can be assumed. A third option is to use A.B. Chance (or similar) soil anchors under the timber ringwall to increase uplift resistance. The soil anchors can be installed in either the in-situ soil or the fill section but should extend at least 3-feet below the base of the ringwall and be tensioned to reduce free play. If soil anchors are being considered, we should be contacted to refine geotechnical engineering recommendations.

If constructed above a compacted fill section as discussed above, bearing pressures of 1,500 psf can be assumed with a 50 % increase for short-term transient loads. Post construction settlements under the tank would be expected

to be less than 1 inch. Greater settlements should be anticipated along the tank wall if a ringwall system is not used.

Since the tank will not be heated, any external connections should be designed to permit differential movement without damage to the tank shell. Because the fine sand fill can be wind transported under certain conditions, we recommend any exposed fine sand fill sections be vegetated or mechanically stabilized as soon as possible.

HDR is also considering installing a 6-inch HDPE fused waterline down a short, steep (25-30%) embankment between the Old Atka water tank and the WTP. The trench for the pipe will be 4 feet deep. Due to the wet, maritime climate, potential slope failure of the backfill material needs to be considered on the steep slope. After the trench is excavated, it should be backfilled with granular fill consisting of locally available clean fine sand. At least 6 inches of sand should be placed in the bottom of the trench and vibratory compacted to 95% of maximum dry density as determined by ASTM D-1557. Then the 6-inch HDPE pipe should be installed and covered with sand in thin lifts and vibratory compacted to 95% of maximum dry density as determined by ASTM D-1557. The top 12 inches of the trench should be filled with the gravel material from the local scoria borrow site to add friction and stability. This upper fill material should be placed in a mound, covered with a thin layer of topsoil and vegetated to facilitate drainage. If additional stability of the fill on the slope is required, another option would be to place cement treated plugs every 10 to 20 feet within the sand material that overlies the pipe. Placing cement treated plugs at the crest would provide additional support to the waterline.

New Atka Subdivision Water Tank

The new water tank for the New Atka Subdivision will be nearly identical to the Old Atka water tank. However this site is underlain by a relatively thin layer of soil over bedrock. All soil should be removed to bedrock and imported fine sand fill as discussed in the prior tank section should be used. A concrete or AWW ringwall is recommended that can be anchor grouted to the bedrock. Anchor rods should be grouted at least 2-feet into competent rock.

Bearing capacities, settlement, appurtenance and erosion protection are similar to those discussed under the previous water tank discussion.

At all sites, corrosion issues must be addressed for the tank base in direct contact with fill material and subgrade anchor systems. Likewise, we recommend any fill proposed for use under the tanks be tested for salinity and corrosion potential prior to use.

Summary, Review, and Construction Monitoring

The observed subsurface materials should be adequate as foundation materials for the proposed improvements. We recommend that caution be exercised when exposing sandy silt soils since they are considered moisture sensitive and would not be expected to regain cohesion or strength if disturbed during wet periods. Local borrow sites are acceptable. Each borrow site (fine sand and the scoria site) has limitations with respect to soil moisture sensitivity, frost susceptibility, and erosion behavior.

Several elements of the geotechnical recommendations are predicated on perceived geologic conditions; for example the quality of the bedrock under the new dam structure and suitability of silty soils for foundation bearing during wet conditions. We recommend on-site observation by a trained, experienced geologist or geotechnical engineer during construction to assure conformance with our recommendations. Materials testing of anchor rod grouts, fill compaction and other general construction concerns should be documented with daily reports submitted to us for review. We recommended that we review project plans and specifications at the 35-percent, 65-percent and 95-percent milestones to confirm alignment with our geotechnical design assumptions.

ILLUSTRATIONS

Plate 1	Test Hole Locations
Plate 2 through 4	Logs of Test Holes
Plates 5	Soil Classification Chart
Plate 6	Summary of Samples
Plate 7	Particle Site Data
Plate 8	Plasticity Chart
Plate 9	Dam Cross Section
Plates 10 through 15	Site Photographs

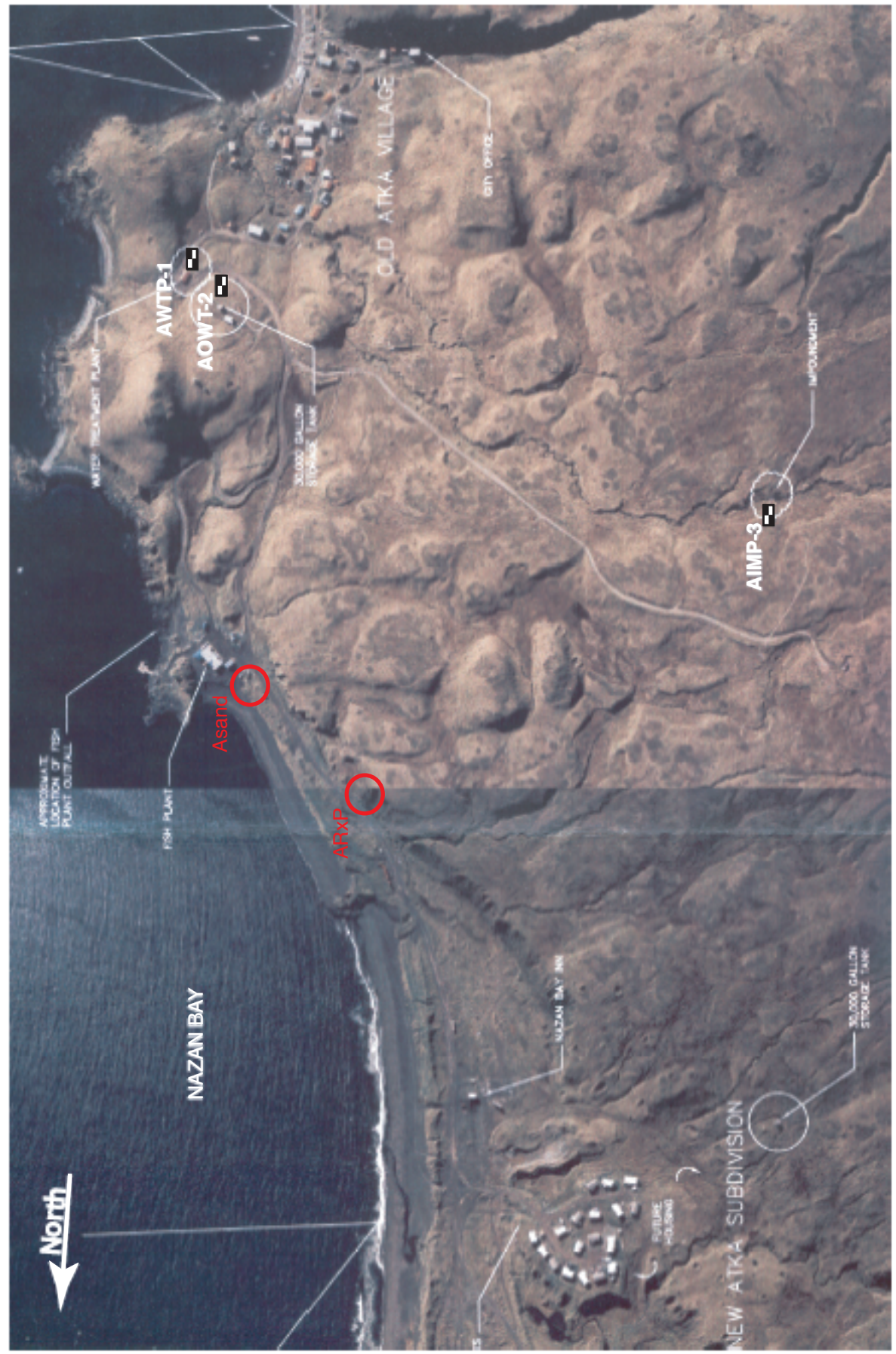


Figure adapted from HDR annotated air photos.

AWTP-1 Approximate Test Hole Location

Asand Approximate Borrow Site Location

200 ft 400 ft
Aproximate Scale

Note. Scoria borrow site is NE of airstrip. Airstrip is due north of New Atka Subdivision and is not visible on this air photo.

Duane Miller & Associates
Job No.: 4086.055
Date: March 2006

TEST HOLE LOCATIONS
Atka Water Project
Atka, Alaska

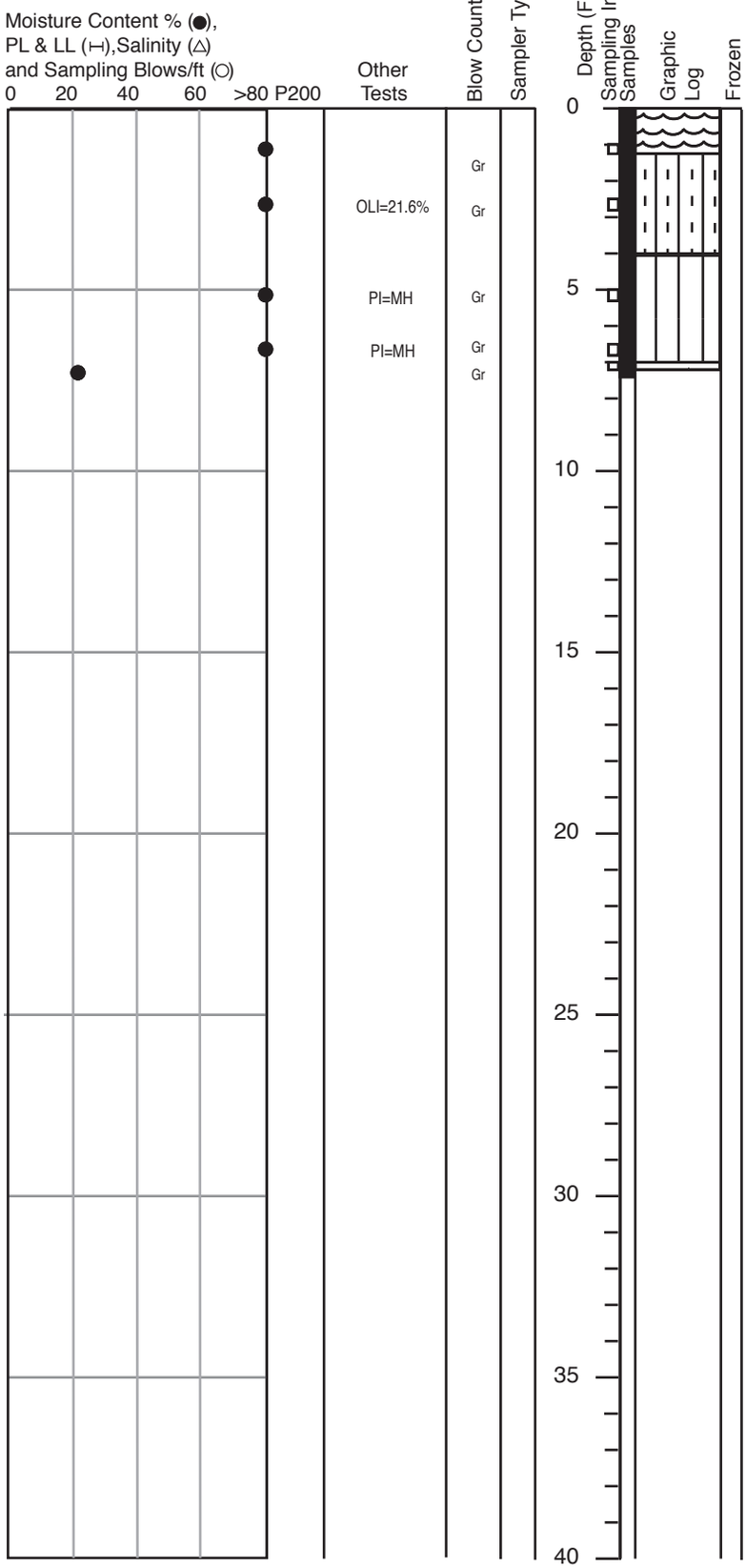
Plate
1

DUANE MILLER & ASSOCIATES

Project: Atka Water Project
DM&A Job No.: 4086.055
Logged By: L. Silliphant

Log of HOLE: AWTP-1

Date Drilled: 9/1/05
Contractor.: City of Atka
Equipment: Catepillar 420D
GPS Coord.: N 52°12'00.7", W 174°11'48.2" NAD 83
Elevation: 67'



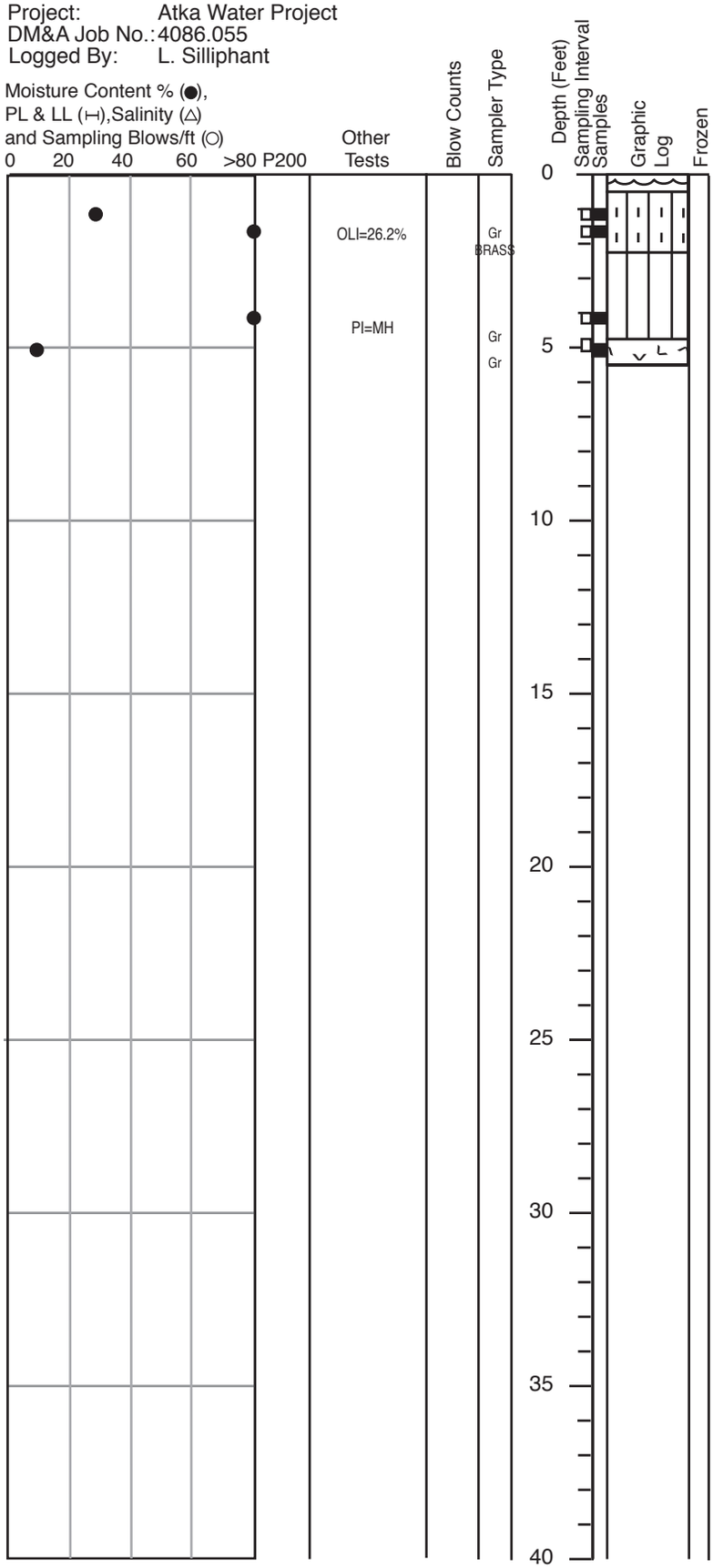
Description
Peat (Pt) Brown, fibrous, soft, wet,
Organic Silt (OL) Brown to red-brown, fibrous, w/ trace fine sand, soft, wet, (ash), water slowly dripping into test pit, 2.5" tan silty ash layer at 2.8'
Sandy Silt (MH) Light brown, w/ some fibrous organics (roots), wet, plastic (ash)
Red, medium stiff, moist, plastic below 6'
Bedrock (Rock) Green, cobble size pieces of volcanic rock to 6", w/ brown and gray clay interspersed between pieces of bedrock, moist
Bottom of test pit at 7.2'. Note: sides of test pit slumping into hole due to high moisture content in the soil.

DUANE MILLER & ASSOCIATES

Project: Atka Water Project
DM&A Job No.: 4086.055
Logged By: L. Silliphant

Log of HOLE: OAWT-2

Date Drilled: 9/1/05
Contractor.: City of Atka
Equipment: Catepillar 420D
GPS Coord.: N 52°12'02.2", W 174°11'50.6" NAD 83
Elevation: 109'



Description

Peat (Pt) Red-brown, fibrous, soft

Organic Silt (OL) Black, w/ some fibrous organics (roots), w/ trace fine to medium sand, soft, dry (ash)

Sandy Silt (MH) Red, medium stiff, moist, plastic, hard to get out of bucket, (ash)

Bedrock (Rock) Black and green cobble size pieces of bedrock to 4" (andesite), w/ tan clay interspersed between pieces of bedrock.

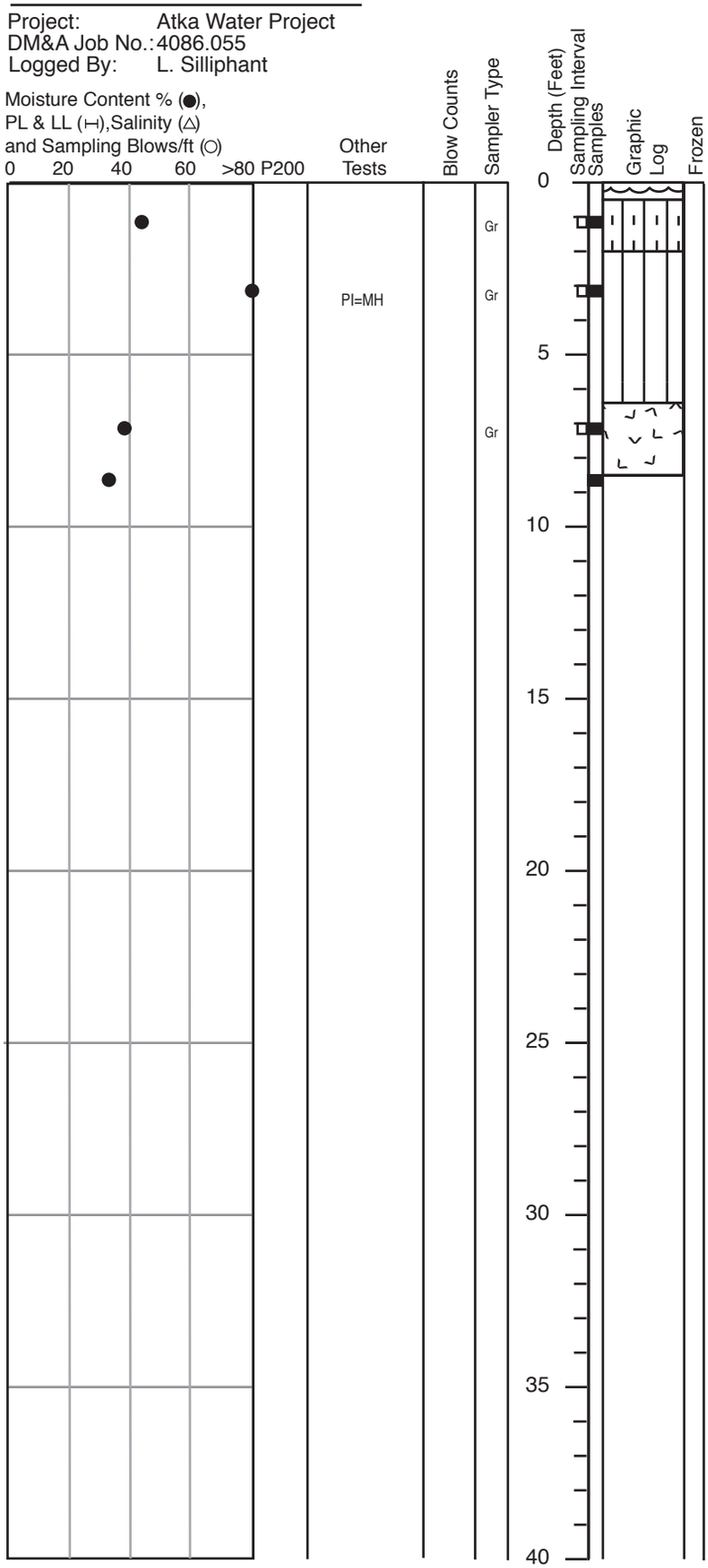
Bottom of test pit at 5.5'.

DUANE MILLER & ASSOCIATES

Project: Atka Water Project
DM&A Job No.: 4086.055
Logged By: L. Silliphant

Log of HOLE: AIMP-3

Date Drilled: 9/1/05
Contractor.: City of Atka
Equipment: Catepillar 420D
GPS Coord.: N 52°12'09.9", W 174°12'26.3" NAD 83
Elevation: 260'



Description

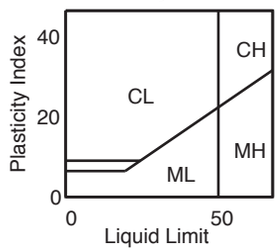
Peat (Pt) Brown, fibrous, soft

Organic Silt (OL) Black, soft, moist, w/ tan, silty ash layer at 1.2'

Sandy Silt (MH) Red, w/ trace fibrous organics, moist, plastic (ash)

Weathered Bedrock (CL+Rock) Brown and gray, cobble size pieces of bedrock to 6" (porphrytic basalt and andesite), brown and gray clumps of hard clay (weathered bedrock), w/ gray and brown clay interspersed between hard clay balls and pieces of bedrock, moist

Bottom of test pit at 8.5'. Note: water seeping into test pit from a small seep perched above the clay and bedrock interval.

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** **Plasticity Chart** 	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

UNIFIED SOIL CLASSIFICATION SYSTEM

KEY TO TEST DATA

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

Strength Data
XXX(YYY), where
XXX = $(\sigma_1-\sigma_3)/2$
YYY = σ_3

KEY TO SAMPLE TYPE

gr = Grab sample
Ag = Auger grab
Ab = Auger bulk
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
P = Pushed Sampler
Tw = Shelby tube
Ss = 1.4" ID split barrel w/ 140 lb. manual hammer
Cc = 3.25" continuous core barrel

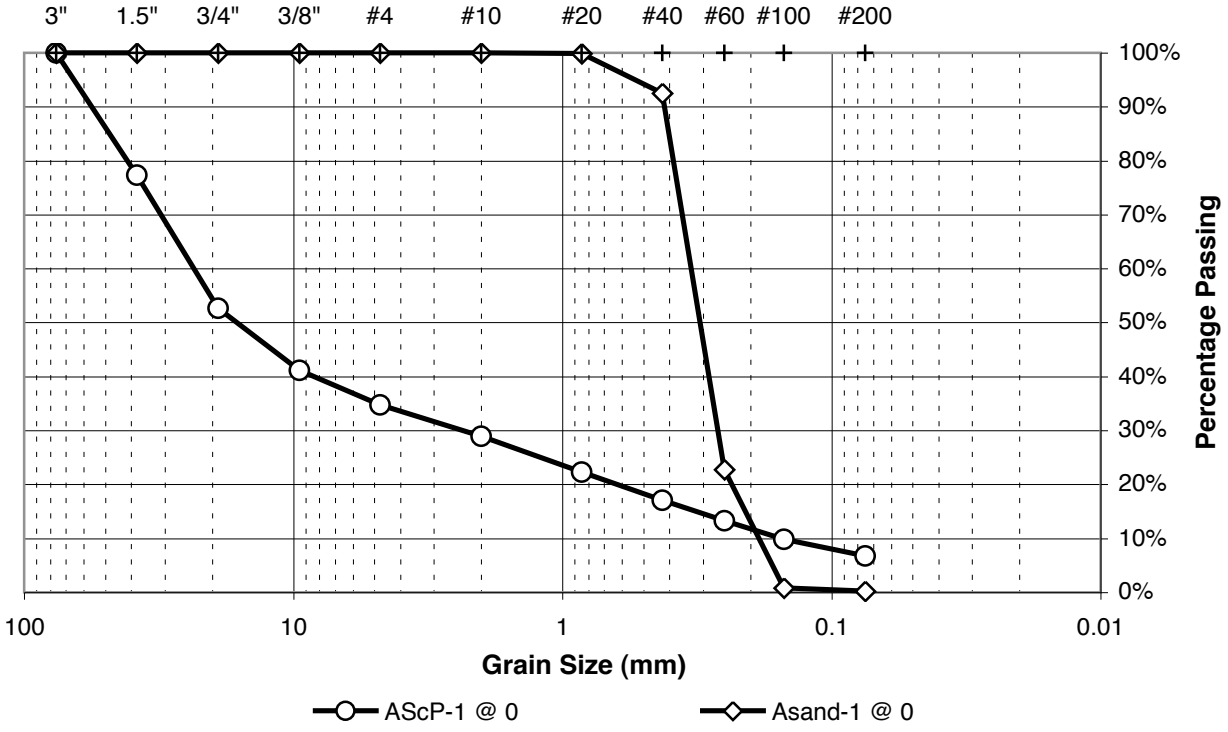
GROUP	ICE VISIBILITY	DESCRIPTION		SYMBOL	
N	Segregated ice not visible by eye	Poorly bonded or friable		Nf	
		Well bonded	No excess ice	Nb	Nbn
			Excess microscopic ice		Nbe
V	Segregated ice is visible by eye and is one inch or less in thickness	Individual ice crystals or inclusions		Vx	
		Ice coatings 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	Soil Type (USCS)	Thermal State	Sampler Type	Moisture Content	Dry Density	Organic Loss	Salinity	Passing #200	Other Tests
AWTP-1	1.0 ft.	Pt	Unfrozen	Gr	164.7%					
AWTP-1	2.5 ft.	OL	Unfrozen	Gr	208.3%		22%			
AWTP-1	5.0 ft.	MH	Unfrozen	Gr	102.0%					PI
AWTP-1	6.5 ft.	MH	Unfrozen	Gr	124.1%					PI
AWTP-1	7.2 ft.	ROCK	Unfrozen	Gr	22.0%					
OAWT-2	1.0 ft.	ML	Unfrozen	Gr	28.9%					
OAWT-2	1.5 ft.	OL	Unfrozen	Brass	154.2%	26 pcf	26%			
OAWT-2	4.0 ft.	MH	Unfrozen	Gr	220.5%					PI
OAWT-2	4.9 ft.	ROCK	Unfrozen	Gr	9.8%					
AIMP-3	1.0 ft.	OL	Unfrozen	Gr	44.1%					
AIMP-3	3.0 ft.	MH	Unfrozen	Gr	140.4%					PI
AIMP-3	7.0 ft.	CL + Rock	Unfrozen	Gr	38.5%					
AIMP-3	8.5 ft.	CL + Rock	Unfrozen	Gr	33.4%					
AWTP-SE	1.5 ft.	SP	Unfrozen	Gr	69.3%			0 ppt		
NAWT-4	1.6 ft.	ML	Unfrozen	Gr	137.9%					
NAWT-4	1.0 ft.	OL	Unfrozen	Gr	154.2%					
DAM-5	1.5 ft.	OL	Unfrozen	Gr	159.1%					
AScP-1	0.0 ft.	GW-GM	Unfrozen	Gr	8.5%				6.8%	
AScP-2	0.0 ft.	GW-GM	Unfrozen	Gr						
Asand-1	0.0 ft.	SP	Unfrozen	Gr	6.0%			0 ppt	0.3%	
ARxP-1	0.0 ft.	Rock	Unfrozen	Gr						

SUMMARY OF SAMPLES
Atka Water Project
Atka, Alaska

Boring =>	AScP-1	Asand-1
Depth =>	0.0 ft.	0.0 ft.
3" =>	100%	100%
1.5" =>	77%	100%
3/4" =>	53%	100%
3/8" =>	41%	100%
#4 =>	35%	100%
#10 =>	29%	100%
#20 =>	22%	100%
#40 =>	17%	93%
#60 =>	13%	23%
#100 =>	10%	1%
#200 =>	6.8%	0.3%
Analysis of Data		
D10 size =>	0.153 mm	0.186 mm
D30 size =>	2.329 mm	0.264 mm
D50 size =>	16.197 mm	0.308 mm
D60 size =>	23.371 mm	0.332 mm
Coeff. of Uniformity, Cu =	152.27	1.79
Coeff. of Curvature, Cc =	1.51	1.13
Gravel (+#4) percentage =	65.3%	0.0%
AASHTO Gravel (+#10) =	71.0%	0.0%
Sand percentage =	27.9%	99.7%
Fines percentage =	6.8%	0.3%
Unified Soil Class Symbol =	**GW-GM**	**SP**



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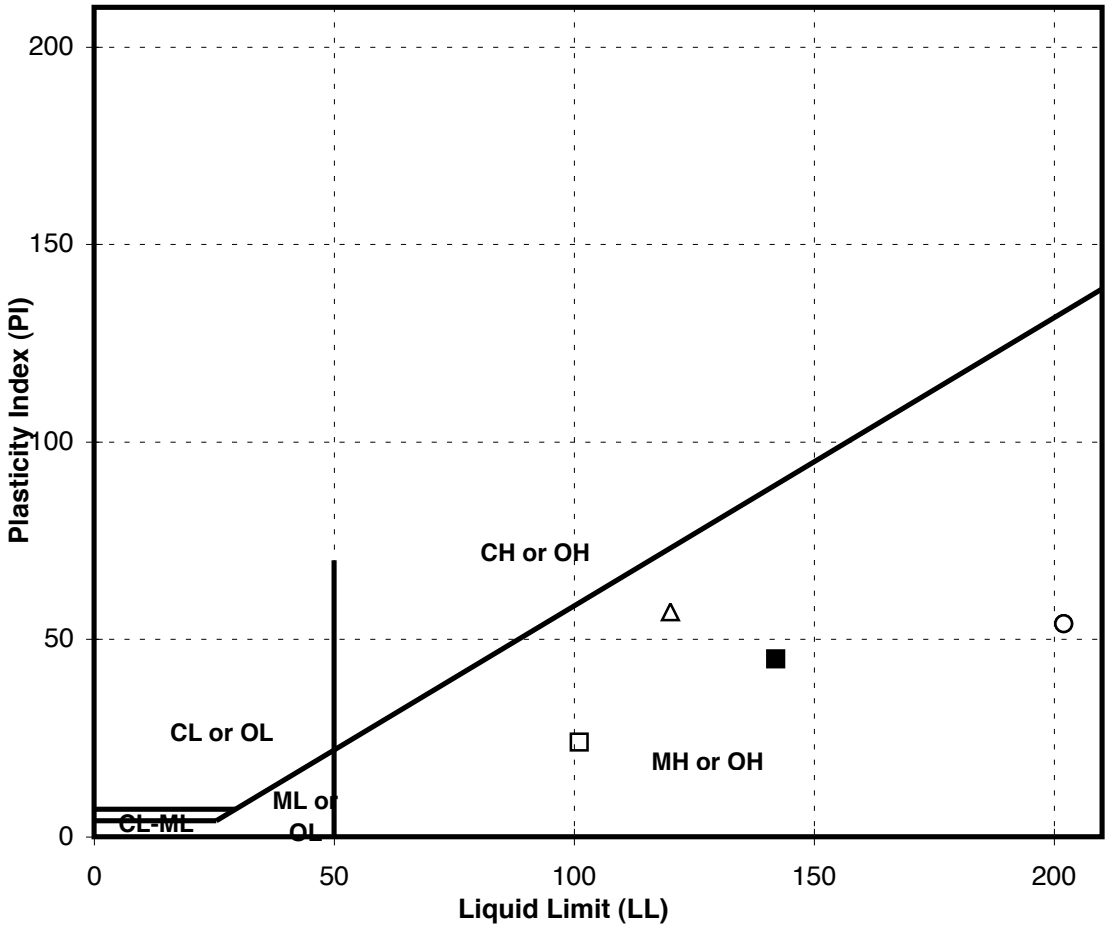
March 2006

PARTICLE SIZE DATA
Atka Water Project
Atka, Alaska



	Sample Location	Plastic Limit	Liquid Limit	Plasticity Index	Natural Moisture Content	USCS
□	AWTP @ 5.0 ft	77	101	24	102.0%	MH
△	AWTP @ 6.5 ft	63	120	57	124.1%	MH
○	OAWT @ 4.0 ft	148	202	54	220.5%	MH
■	AIMP @ 3.5 ft	97	142	45	140.0%	MH

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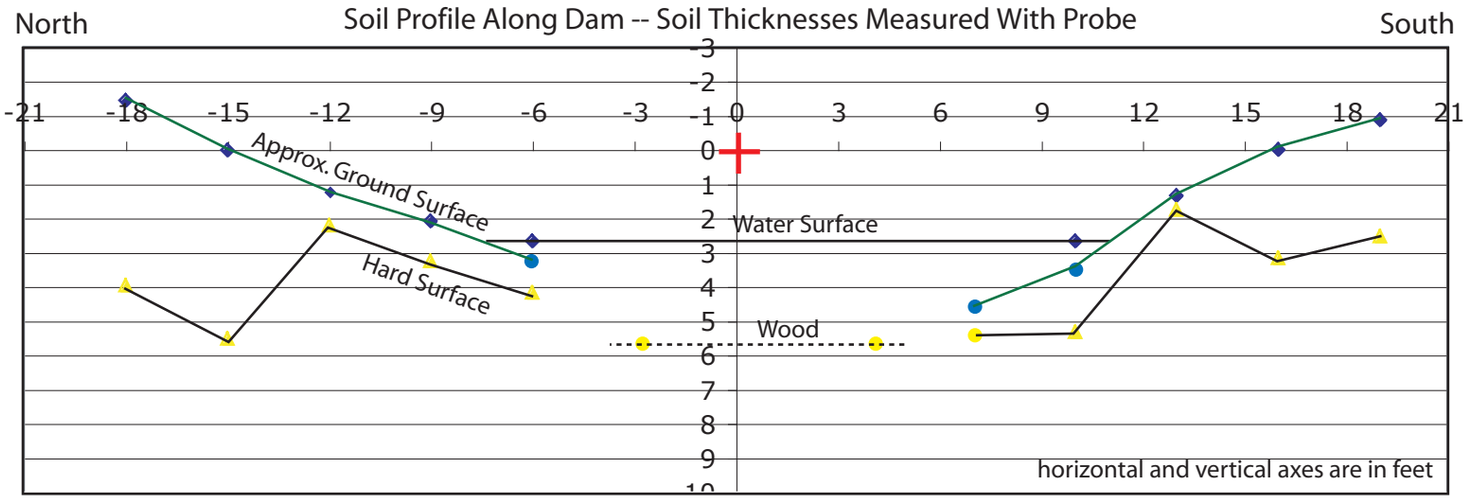
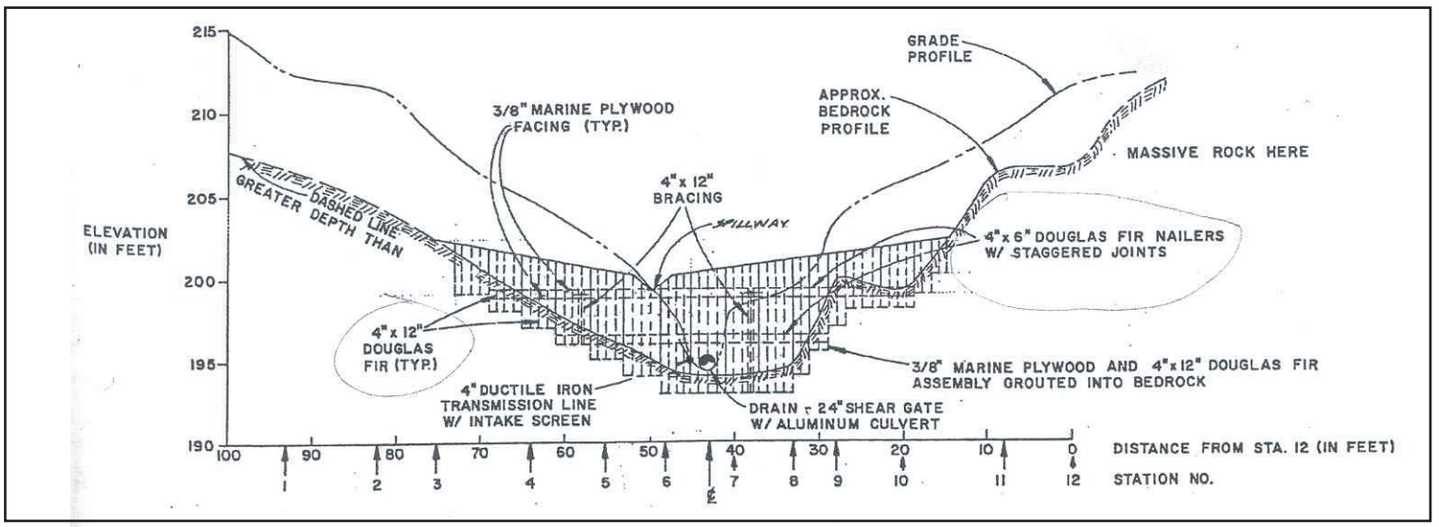
March 2006

PLASTICITY CHART
Atka Water Project
Atka, Alaska



Dam Cross Section -- March, 1978 As Built



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Date: March 2006

DAM CROSS SECTION
Atka Water Project
Atka, Alaska

Plate
9

PHOTO 1. Water Impoundment Area, Test Hole AIMP-3.

PHOTO 2. Water Impoundment Area, Test Hole AIMP-3. Clay and rock material dug out from depths of 6.5 feet to 8.5 feet.

PHOTO 1. Water Treatment Plant, Test Hole AWTP-1.

PHOTO 2. Water Treatment Plant, Test Hole AWT-1. WTP is blue building on right. Old Atka Village water tank in background.

PHOTO 1. Southeast footing of WTP. Sand fill is from sand dune borrow site along Nazan Bay (see photo 2). Water encountered immediately.

PHOTO 2. Atka sand dune borrow site. Leonty for scale.

PHOTO 1. Old Atka Village Water Tank, Test Hole OAWT-2.

PHOTO 2. Old Atka Village Water Tank, Test Hole OAWT-2.

PHOTO 1. New Atka Subdivision Water Tank Site.

PHOTO 2. New Atka Subdivision Water Tank Site in background.

PHOTO 1. Atka Scoria Pit Borrow Site.

PHOTO 2. Atka Rock Quarry Borrow Site.