GEOTECHNICAL REPORT
EMBANKMENT STABILITY STUDY
WESTCHESTER LAGOON
ANCHORAGE, ALASKA

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering studies to address State of Alaska concerns for the stability of the Westchester Lagoon embankment slope that will be adjacent to a proposed new fish passage channel intended as an improvement for the Chester Creek drainage in Anchorage, Alaska. The purpose of this geotechnical study was to perform a seepage analysis and a more in depth stability analyses in relation to the proposed improvements. The planned work will improve aquatic habitat and provide a fish passage at the mouth of Chester Creek. Many changes need to be constructed within the local infrastructure in order to do this. Among these changes, a new channel will be constructed to connect the spillway at Westchester Lagoon with an existing intertidal lagoon and then out to Cook Inlet tide water. As part of this study, geotechnical data from 3 of 15 soil borings that were advanced within the proposed area of development were used to construct models for the seepage and stability analyses. The borings that were used for the modeling were Borings B-1, B-2, B-6 and B-10. Presented in this report are descriptions of the subsurface conditions, study methodology, and conclusions and recommendations from our studies.

Authorization to proceed with this work was received from Mr. Dan Billman, of HDR Alaska on February 3, 2002. Our work was conducted in general accordance with our October 5, 2001 proposal.

2.0 SITE AND PROJECT DESCRIPTION

The project area is in the western end of the Westchester Lagoon park area, near the tidal waters of Cook Inlet in Anchorage, Alaska. The improvements will change the existing configuration of the mouth of Chester Creek. Chester Creek lies between the westernmost portion of Westchester Lagoon and extends to the wetlands and mudflats of Cook Inlet. A vicinity map is included as Figure 1.

A site plan is included in Figure 2. The wetland area is continuously marshy and wet. The area is relatively clear of trees except generally near the toe of the railway embankment. Ground cover consisted of a thin organic mat or typical silty clay mud and thick, grass stalks. Several small water channels (approximately 4 feet wide and 2 feet deep) are present in this area, and were likely drainage paths for Chester Creek waters before the lagoon was constructed. Waters present in these channels likely come from seepage through both the railroad and Westchester Lagoon embankments.

Our stability and seepage analyses were based on models of an assumed critical section from the proposed designs for the improvements in the area where the Chester Creek channel will be immediately adjacent to the Westchester Lagoon embankment. The critical section was taken as a slight variation of Profile B-B' in Appendix A, and shown in Figures 2 and 3.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site are depicted in detail in the boring logs and the profile sections presented in our previous report, dated March 2002. Boring logs for Borings B-1, B-2, B-6 and B-10 are included in Appendix A.

The initial soil encountered in the three borings drilled through the existing embankment was silty, gravelly sand (Fill) that was approximately 10 feet deep. Below the fill, the soil is generally composed of an average of 30 feet of clay becoming silty with depth. This clay unit overlies interbedded, coarser-grained silty sands and gravelly sands. As described in our earlier report, the clay is thicker in the northern and northeast reaches of the project area closer to Westchester Lagoon. In this area, the average bottom extent of the clay layer is nearly 30 feet below mean sea level (MSL) or approximately 40 to 50 feet thick. In areas closer to Cook Inlet and further south, the clay layer thins somewhat to around 25 feet in thickness, terminating around 15 feet below MSL. In our borings, the clay was mostly stiff but locally medium stiff to very stiff with blow counts ranging from 4 to 30 blows per foot and averaging around 10 blows per foot. Laboratory analyses indicated that the clay has an average unconfined compressive strength of around 3 tons per square foot (tsf) and average torsional undrained shear strengths of around 1.0 tsf, respectively. This suggests that much of it may be very stiff. Findings from unconfined compressive strength tests generally agree with the Torvane (TV) and Pocket Penetrometer (PP) readings taken in the field. Remolded TV values show that the clay has a relatively low sensitivity, that is, it does not significantly loose its strength when disturbed. According to Atterberg limits tests, the clay is primarily low plasticity material (average plasticity index of around 15) except for Sample S6 from Boring B-6, which was classified as high plasticity and had a plasticity index of 27.

Beneath the clay, our borings found complexly interbedded granular soils. These soils ranged from silty, gravelly sand to gravelly, silty sand. In general, this material was dense with average blow counts of around 30 blows per foot and higher. A layer of sand to slightly silty sand was encountered by some of our deeper borings. This unit seemed to be the primary water-bearing unit in the area. When penetrated, the water level would rapidly rise in the borings to near the surface and in the case of Boring B-10, this unit contained artesian pressures. Because of the relatively high water pressures and cohesionless nature of this unit, it was also prone to severe borehole heaving. In the deepest borings, a layer of very dense, silty sand to gravelly, silty sand was encountered beneath the water bearing sand layer. This material contained enough silt that it would not heave and also did not appear to be saturated.

Existing railroad embankment and pathway dike material encountered by our borings was somewhat variable. The railroad embankment material was relatively loose (as low as 3 blows per foot) or medium dense beneath around 5 feet bgs. In general, the material was classified as slightly silty, gravelly sand with an average frost susceptibility of F2. In both borings advanced through the embankment, the fill layer was approximately 25 feet thick with a relatively sharp boundary between it and the underlying clays. Material in the lagoon dike and under the existing footpath was similar to the material found in the railroad embankment. In Borings B-2 and B-6, the fill was approximately 4 to 5 feet thick. It is our opinion that the 16 feet of fill encountered in Boring B-1 is backfill material used in the foundation around the existing spillway outlet structure shown in Figure 2.

Ground water was typically encountered in our borings that penetrated the water bearing sand layer beneath the clays. As previously mentioned, once the sand layer was penetrated, the water level in the boring rose to nearly the ground surface. The water levels were checked in the observation wells installed in Borings B-4 and B-6 on May 25, 2001, about 8 to 9 days after they were installed. The static water level in Boring B-4 and B-6 were approximately 4.0 and 32.0 bgs, respectively, at this time. It is our opinion that the water level recorded in Boring B-4 is not accurate. When the well was installed, the water level was near the ground surface. Heaving sands made installation of the well difficult and the slotted section of the pipe may not have been positioned across the water-bearing unit.

4.0 ENGINEERING STUDIES

Slope stability and seepage analyses were performed for the Westchester Lagoon embankment. The slope stability analysis will be used to evaluate the potential of the failure of the Westchester Lagoon embankment. The seepage analysis included the evaluation of the hydraulic head distribution beneath the embankment and adjacent to the proposed salmon-passage channel, and the potential for piping at the location of the proposed channel.

4.1 Seepage Analysis

A two-dimensional, steady state, finite element, numerical model (Fastseep 2D) was used to evaluate the hydraulic head distribution between the Westchester Lagoon and the proposed salmon-passage channel. The model dimensions were based on profile B-B' from our previous report, which also provided the largest likely dam face for a worst-case evaluation.

Geologic profiles, water level measurements, and boring logs were used to develop a conceptual model and to determine the hydrogeologic model input. The geologic profiles indicate layers of unconsolidated soils alternating between stiff clay, sandy silt, and silty, gravelly sand and/or slightly-silty sand, overlain by fill deposits placed for the railroad and the Westchester Lagoon embankments. The conceptual model was developed based on the sequence of soils in the vicinity of the Westchester Lagoon embankment as depicted by profile B-B'. This includes a slightly silty to silty sand layer between about elevation –25 feet and –70 feet (which may extend lower), a silt-clay layer from about elevation 10 feet to –25 feet, and fill between about elevation 0 feet and 18 feet. Water levels were measured at about ground surface (elevation 12 feet) in borings on the tail water side of the Westchester Lagoon embankment, and as high as elevation 16 feet within the Westchester Lagoon (based on maximum water level in reservoir).

The numerical model consisted of a three-layer geologic sequence based on the conceptual model; fill deposits overlying the silt-clay layer, above silty sand. Values for hydraulic conductivity were estimated using grain-size distribution curves, the Hazen equation, and published literature values (Freeze and Cherry, 1979). The hydraulic conductivity for the fill deposit layer was varied between 1×10^{-3} and 7×10^{-2} centimeters per second (cm/sec). The hydraulic conductivity for the silt-clay layer was varied between 1×10^{-7} and 1×10^{-3} cm/sec during successive modeling trials in order to evaluate the heterogeneities in the silty clay. The hydraulic conductivity for the silty sand layer was varied between 5×10^{-4} and 5×10^{-3} cm/sec.

For model boundary conditions, we assumed that the Westchester Lagoon was at maximum stage (8 feet of head, elevation 16 feet) and that ground water levels on the tail water side of the embankment were at ground surface (elevation 12 feet). Constant head boundaries were used to simulate: 1) the lagoon stage on the headwater side of the embankment; and, 2) the ground water level and proposed channel water level on the tail water side of the embankment. Exit face boundaries were also used along the tail water side of the embankment above the proposed channel level to evaluate seepage and exit gradients (Figure 3).

Several modeling simulations were performed while varying values of hydraulic conductivity for each layer. We used a large range of hydraulic conductivity for the silt-clay in order to evaluate the impact of potentially higher permeable silt zones within the formation. Additionally, the proposed channel excavation is within the silt-clay layer. The calculated steady-state hydraulic head distribution is provided in Figure 4 (minimum hydraulic conductivity for the silt-clay) and Figure 5 (maximum hydraulic conductivity for the silt-clay). The differences between the two simulations do not appear to be significant relative to exit face gradients at the proposed channel.

4.2 Slope Stability

The slope stability analyses were conducted to estimate whether the Westchester Lagoon embankment would be at risk with the additional depth of downstream face when the cut for the fish passage channel is constructed. Both static stability analyses and deformation analyses under seismic loading using the Makdisi and Seed (1977) procedure were performed for a section perpendicular to the embankment near Section B-B'.

For the static condition, limit equilibrium analyses were performed using the modified Bishop method for circular failure surfaces. Static soil properties were estimated based on the N-values from nearby borings and from the results of torvane and pocket penetrometer tests for undrained shear strength. The water table elevation used in the analyses was based upon monitoring well readings and the results of the seepage analysis described in Section 4.1. A factor of safety of 1.6 was calculated for the critical circle using these procedures.

For seismic stability, we approached the analyses from the perspective of deformation (lateral displacement) of the embankment fill, rather than performing pseudo-static analyses. We used the Makdisi and Seed (1977) procedure to estimate deformations, which included the following steps:

(1) Characterize the soil in terms of residual shear strength and liquefaction susceptibility for a specified level of ground motion.
(2) Evaluate the stability of the slope for static conditions to find the critical circle.
(3) Determine the yield acceleration (k_y), the applied horizontal acceleration that causes the factor-of-safety of the critical circle to be reduced to 1.0.
(4) Estimate k_{max}, the maximum average acceleration for the potential slide mass

(5) Compare k_{max} to k_y to estimate the range of expected displacement. The assumption is that any earthquake causing an average acceleration in the potential slide mass greater than k_y would result in deformation of the embankment.

Since a design level of ground shaking for embankments has not been defined, we performed our initial deformation and liquefaction analyses for ground motions with a return period of approximately 500 years. The 500-year ground motions were estimated from probabilistic seismic hazard analyses (PSHA) performed by the U.S. Geological Survey (USGS, 1996) and available from their website. The rock level peak ground acceleration (pga) of 0.38g from the USGS PSHA was modified by an amplification factor of 10 percent to develop an estimate of pga at the soil surface. The amplification factor of 10 percent is based upon the pga's recommended in the 1997 Uniform Building Code (UBC) for S_B (rock) sites vs. S_D (soil) sites.

Liquefaction analyses were carried out for Borings B-1, B-2, and B-6, using the procedures outlined in NCEER (1997). Only the upper fill materials were potentially liquefiable due to the 500-year ground motions. The residual strength (S_r) of the liquefied soils was initially estimated using the lower quartile of the correlation between N-value and S_r by Seed and Harder (1990). However, since the fill materials are located close to the ground surface and the confining stress is small, the Seed and Harder (1990) correlation gave residual strengths that were greater than the static shear strength. Thus the static shear strengths of the fill materials were used in our seismic analyses. No reduction of undrained shear strength of the cohesive materials was assumed since the soils are medium stiff to stiff.

Profile B-B" in Appendix A shows the soil profile geometry used to determine the yield acceleration. We estimated that the yield acceleration is approximately 0.36g. The next step was to estimate the maximum average acceleration (k_{max}) for the critical slide mass. Using the procedures from Makdisi and Seed (1977), k_{max} is approximately 0.39g, and the estimated deformation is about 3 inches for the 500-year ground motion.

Additional analyses were performed using the Makdisi and Seed (1977) procedure to determine the recurrence interval earthquake needed to cause a deformation that would be necessary for significant risk to the embankment (1 to 3 feet). Based upon our analyses, the 2500-year ground motion is likely to cause between 1 to 3 feet of displacement of the embankment.

5.0 CONCLUSIONS

The potential for piping was evaluated by measuring the steepest exit face gradient at the location of the proposed channel. According to Freeze and Cherry (Groundwater, 1979, p. 482), when assuming a representative soil density of 2 grams per cubic centimeter (g/cm^3), piping will likely occur when the hydraulic gradient is greater than 1 (feet per foot; unitless). In general, the US Army Corp of Engineers (EM 1110-2-1913, p. B-12) recommends maintaining an exit face gradient less than 0.5. Using the model results (Figures 4 and 5), the steepest exit face gradient evaluated at the proposed channel varied between 0.08 and 0.1. We assumed that the water level on the tail water side of the embankment was at ground surface.

Additional model simulations were performed to evaluate exit face gradients (and piping potential) at the proposed channel as the water level on the tail water side of the embankment was lowered. The model indicates if water levels in the channel are lowered 4 feet below ground surface (8 feet lower than the lagoon stage), the exit face gradients increase to about 0.6 at the top of the proposed channel cut (Figure 6).

Seepage (from the lagoon to the proposed channel) was evaluated using a variation of Darcy's Law described by the US Army Corp of Engineers (EM 1110-2-1913, p. B-11). Seepage was calculated independently for the fill material and for the silt-clay. We assumed that the proposed channel would cut into the fill embankment material, as indicated in Profile B-B'. The seepage from the lagoon, through the fill, to the proposed channel was estimated between about 780 and 2,100 gallons per day (gpd) per lineal foot. Hydraulic conductivity was varied between 1×10^{-7} and 1×10^{-3} cm/sec for the silt-clay to account for potential heterogeneities. The potential seepage for the silt-clay ranges widely and depends on the relative amount of silt to stiff clay present in the unit, which controls the hydraulic conductivity. The estimated seepage rate for the silt-clay ranges from about 4 to 370 gpd per lineal foot.

In general, the potential for piping of soil at the proposed channel is low when water levels in the channel are assumed to be about 4 feet lower than the Westchester Lagoon stage (about equal to surrounding ground surface). If the proposed channel does cut through embankment fill material, the potential for piping is greater in the fill than in the silt-clay, and increases along the exposed face of the fill. The potential for piping would increase (in both the fill and the silt-clay) if the water level in the proposed channel was lowered while the lagoon remained at maximum stage because of the increased hydraulic gradient. If water levels in the channel are lowered 4 feet below ground surface (8 feet lower than the lagoon stage), the exit face gradients increase to about 0.6 in the fill. This indicates an exit face gradient exceeding Corp of Engineer recommendations (0.5), although piping theoretically occurs when the exit face gradient is 1. The exit face gradient would continue to increase in both the fill and the silt-clay as the channel water level was lowered, while holding the lagoon at maximum stage.

Our stability calculations indicate that the proposed embankment has a risk of major lateral displacement (1 to 3 feet) in a seismic event having a recurrence interval of 2,475 years. In a 500 year event the risk of displacement is on the order of 3 inches.

6.0 RECOMMENDATIONS

The stability of the embankment in relation to a seismic event was analyzed using the most stringent conditions indicated by our borings, i.e. the ground water level at or above elevation 10 feet. Based on these studies, the risk of displacement is about 3 inches for a 500 year event. The risk of displacement of an amount that can be expected to cause failure (1 to 3 feet) can be related to an over 2,000 year event. In our opinion, the current design geometry for the channel will not pose a danger to the stability of the Westchester Lagoon embankment.

Seepage studies indicate that water level differences between the tail water and reservoir of greater than 8 feet with the reservoir at maximum stage will cause an exit face gradient in seepage water level that approaches, or slightly exceeds, the Corps of Engineers limit of 0.5. The risk of exceeding this limit is greater in the embankment fill. This risk can be reduced either by making sure that the channel is a sufficient distance from the embankment that the channel crest will not intercept the embankment fill, or by decreasing the permeability of the embankment material. A relatively easy means by which this can be accomplished is with a cutoff wall, constructed to a depth that is below the channel bottom, along the portion of the embankment adjacent to the channel.

7.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. It is assumed that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If, during construction, subsurface conditions different from those encountered in these and prior explorations are observed or appear to be present, Shannon & Wilson should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to review those portions of the plans and specifications pertaining to earthwork in the proximity of the lagoon embankment to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachments in Appendix B "Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of the reports.

Sincerely,

SHANNON & WILSON, INC.

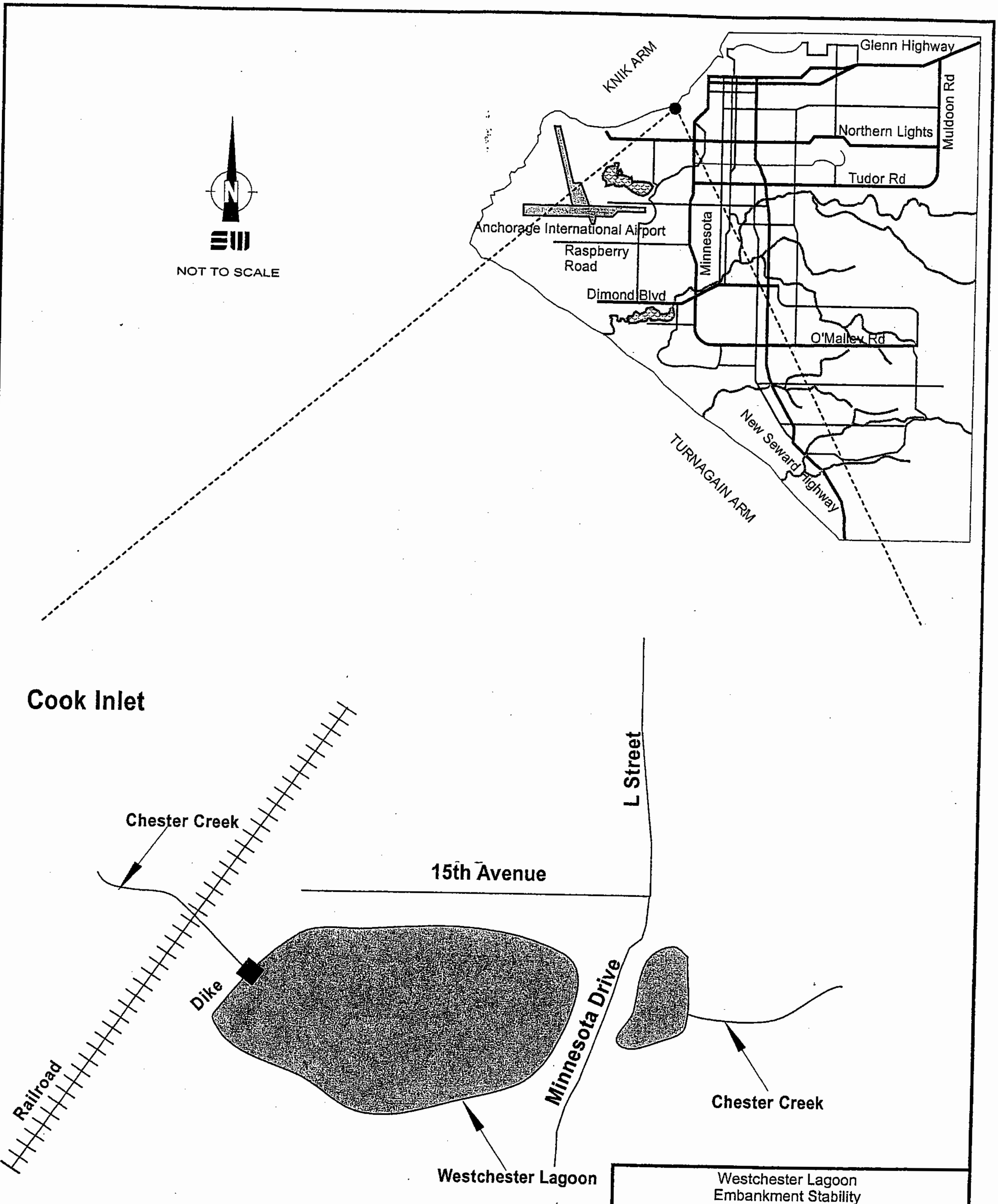
Prepared by:

Grover L. Johnson, P.E.
Principal Engineer

STATE OF ALASKA
7/12/02
49th
GROVER L. JOHNSON
CE-8833
REGISTERED PROFESSIONAL ENGINEER

Reviewed by:

William S. Burgess, P.E.
Associate



Westchester Lagoon
Embankment Stability
Anchorage, Alaska

VICINITY MAP

July 2002 | 32-1-01406-02

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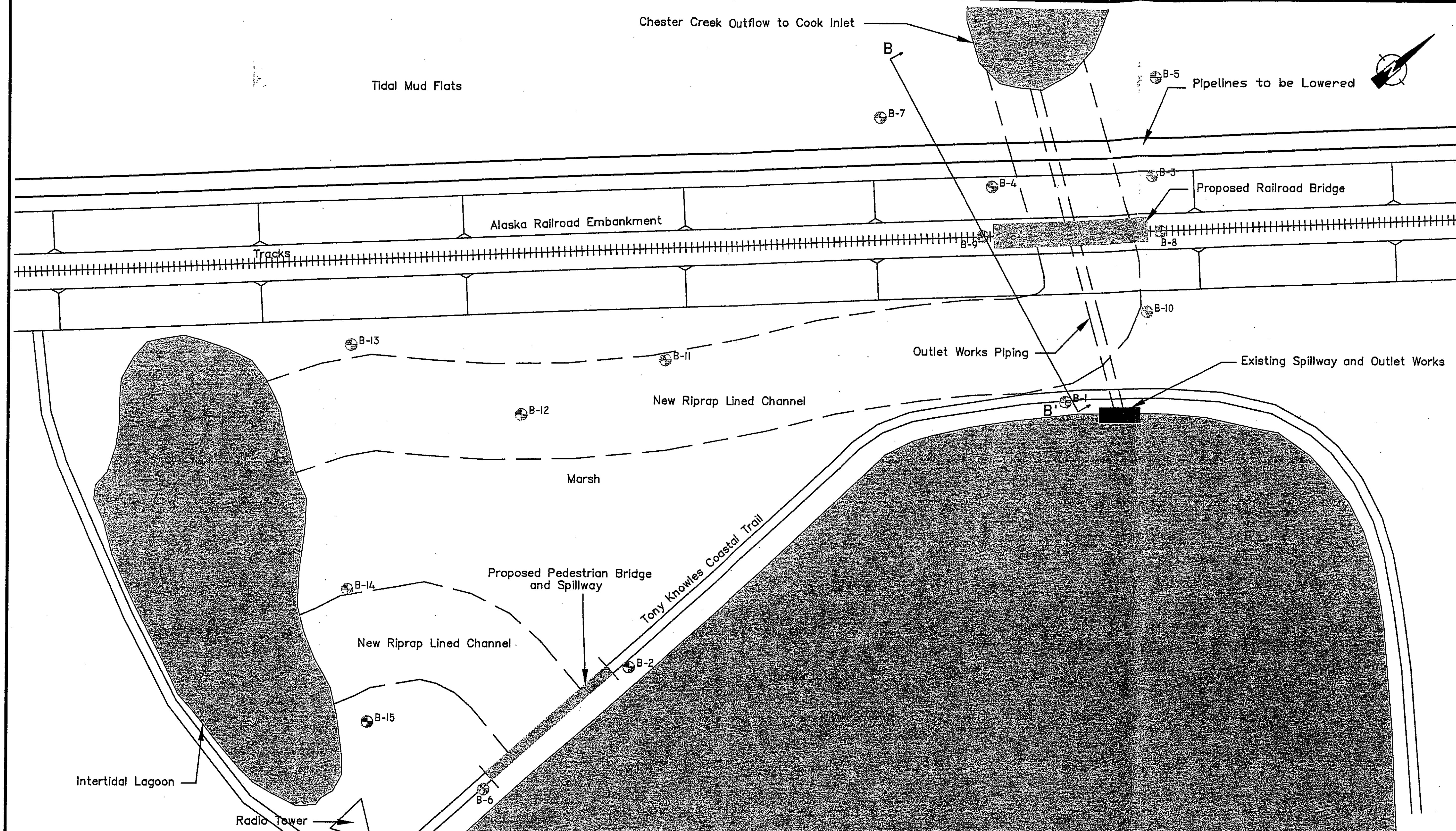
Fig. 1



LEGEND

B-1 Approximate Location of Boring B-1 Advanced by Shannon & Wilson, Inc.

B—B' Approximate Location of Profile B-B', Exhibited on Figure 9 from July 2001 Geotechnical Report Included in Appendix A with Boring Logs

0 40 80 160
APPROXIMATE SCALE IN FEET

WESTCHESTER LAGOON
EMBANKMENT STABILITY
Anchorage, Alaska

SITE PLAN

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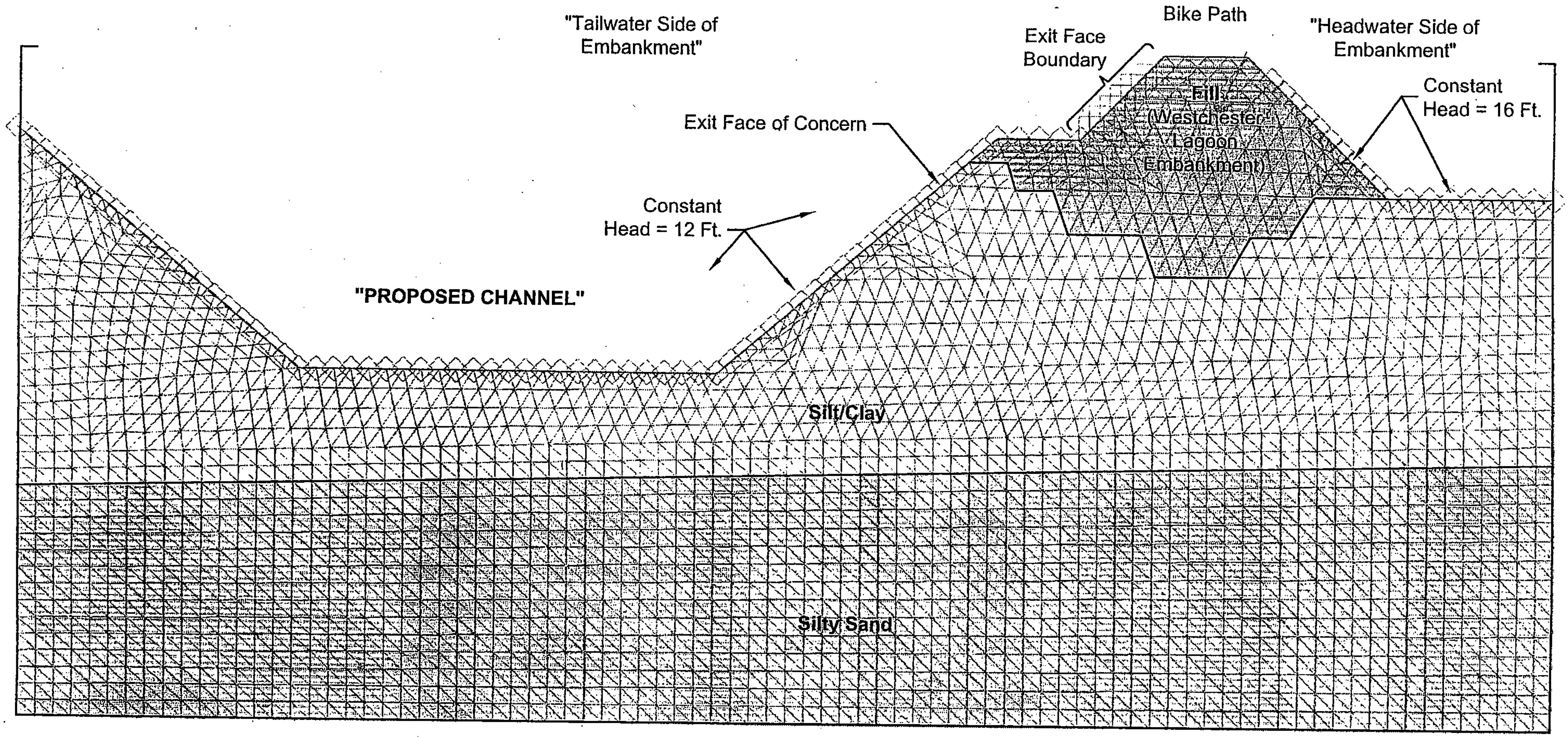
FIG. 2



Not to Scale

NOTE

Critical section based on Profile B–B' (see Appendix A)

Westchester Lagoon
Embankment Stability
Anchorage, Alaska

MODEL BOUNDARIES

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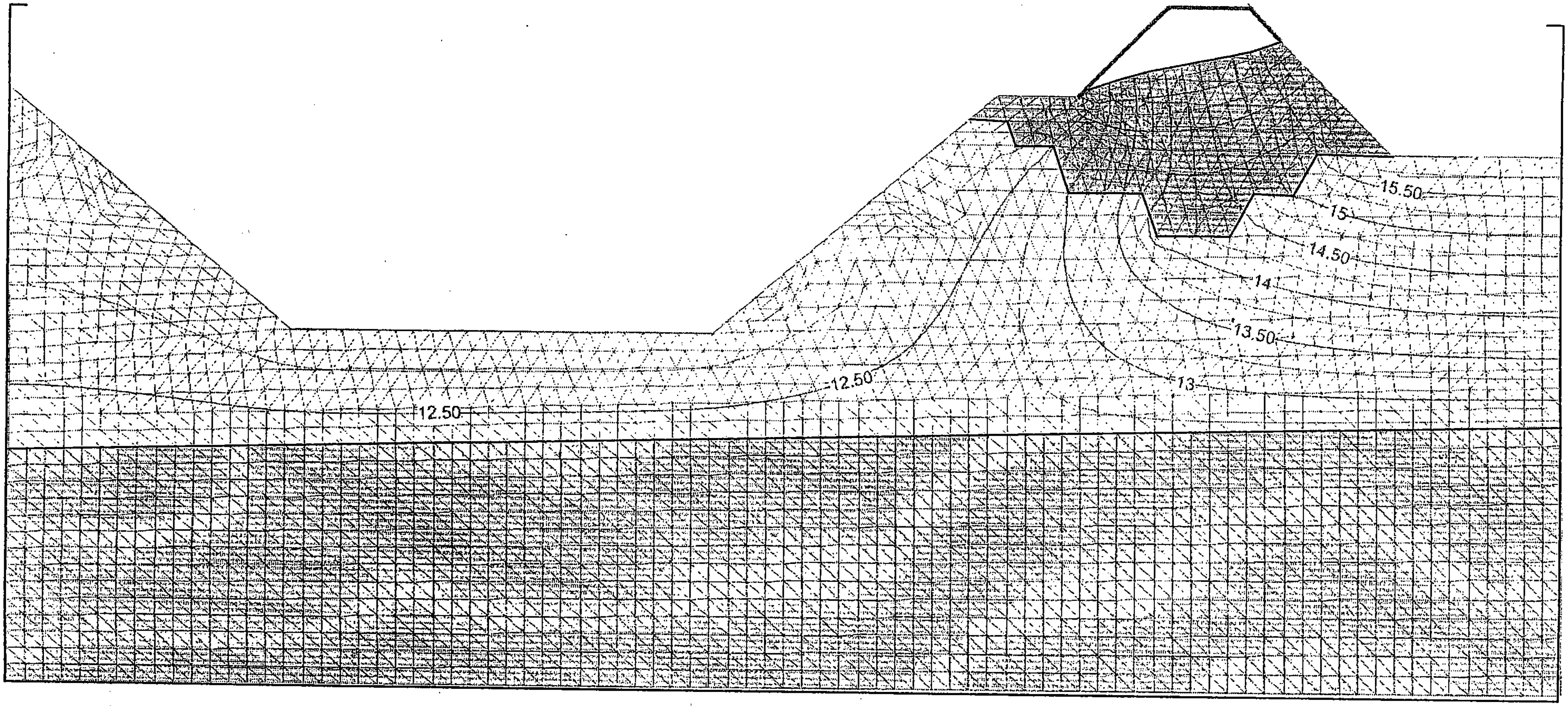
FIG. 3

01406_2fig3.dwg 1:1 7/02 Shannon & Wilson, Inc.



NOT TO SCALE

KEY

— 15 — Equipotential contour corresponding to 15 feet of head.
Based on boundary conditions shown in Figure 3

Westchester Lagoon
Embankment Stability
Anchorage, Alaska

MODEL RESULTS
LOW HYDRAULIC CONDUCTIVITY

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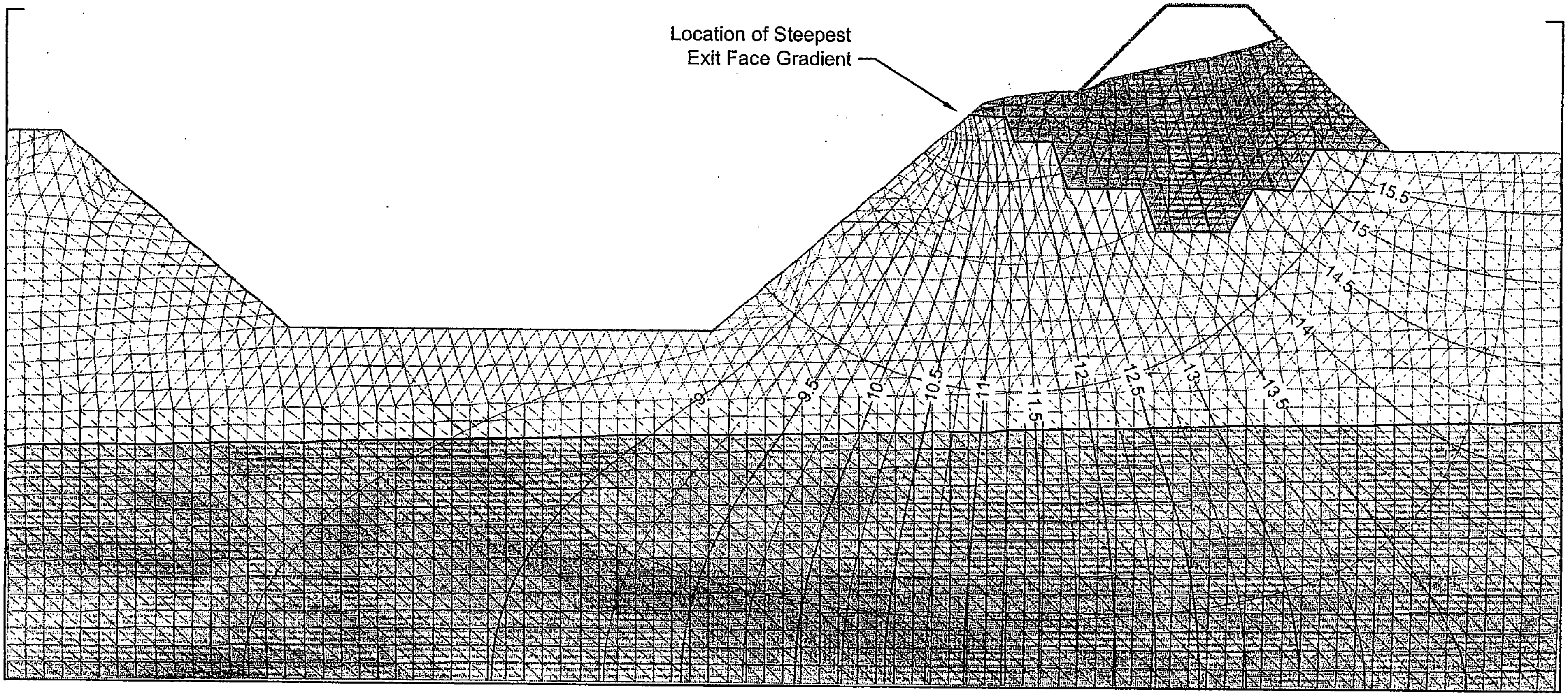
FIG. 4



Not to Scale

KEY

— 15 — Equipotential contour corresponding to 15 feet of head. Based on boundary conditions shown in Figure 3

Westchester Lagoon
Embankment Stability
Anchorage, Alaska

MODEL RESULTS
HIGH HYDRAULIC CONDUCTIVITY

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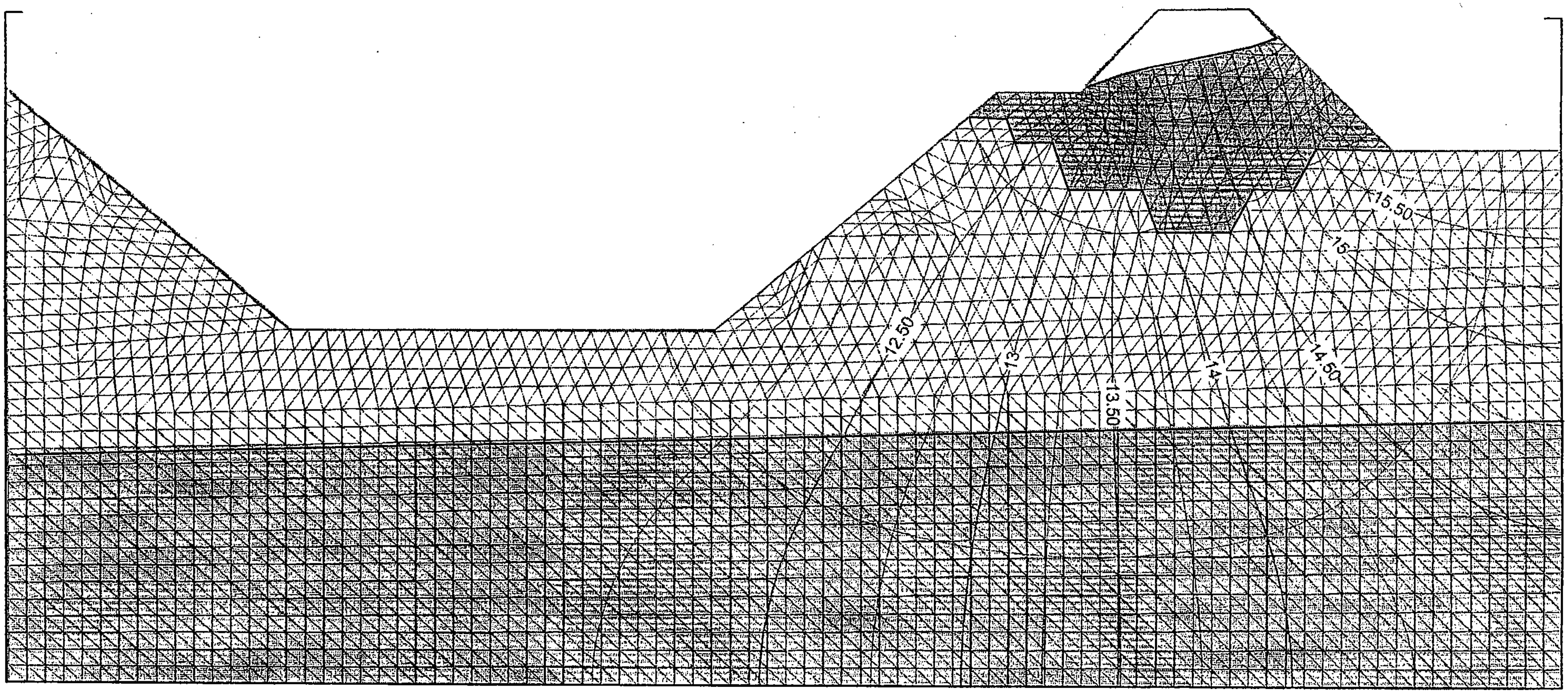
FIG. 5



Not to Scale

KEY

— 15 — Equipotential contour corresponding to 15 feet of head. Based on boundary conditions shown in Figure 3

Westchester Lagoon
Embankment Stability
Anchorage, Alaska

MODEL RESULTS
LOW CHANNEL WATER LEVEL

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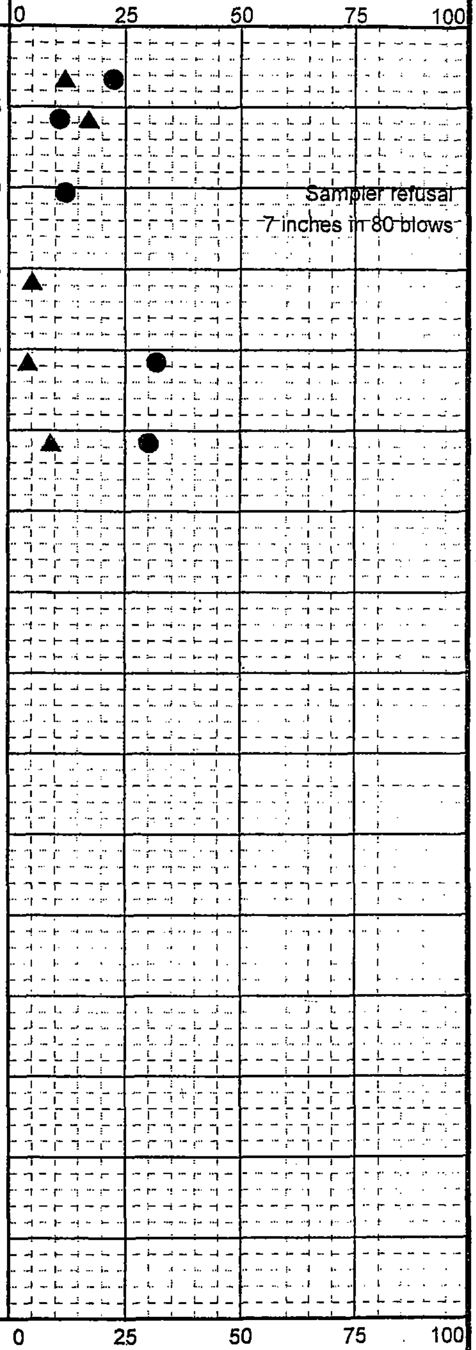
FIG. 6

APPENDIX A

BORING LOGS
and
Profile B-B'

MATERIAL DESCRIPTION 18 Ft. Relative to MSL	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Medium dense, brown, slightly silty, sandy GRAVEL; dry to moist (FILL)	2.0		S-1		5
Medium dense, dark brown, silty, gravelly SAND; moist to wet (FILL) S2: Gravel = 27%, Sand = 55%, Silt = 18%; F2	16.0		S-2 S-3		10 15
Medium stiff to stiff, gray CLAY; moist Transitioning to sandy CLAY at 25.5 feet bgs	26.5		S-4 S-5 S-6	Groundwater Not Encountered During Drilling on May 15, 2001	20 25
Bottom of Boring Boring Completed May 15, 2001					30 35 40 45 50 55 60 65 70 75

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



● % Water Content

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Split Spoon Sample
- 2" O.D. Split Spoon Sample
- Shelby Tube
- Ground Water Level ATD
- Perched Water Level

NOTES

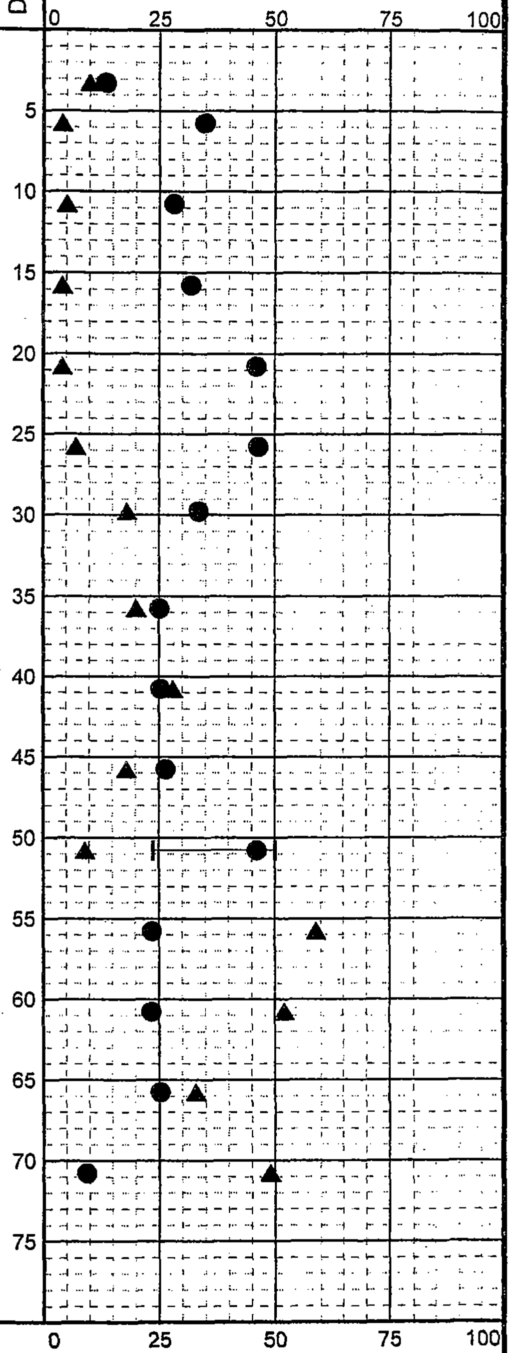
1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Pocket pen values are represented by PP. Torsional force vane values are represented by TV. Percent passing the number 200 sieve is represented by P200.

Chester Creek Improvements Anchorage, Alaska	
LOG OF BORING B- 1	
July 2001	32-1-01406
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**Fig. A-1**

S&WGEO1 LOG 32-1-0-1.GPJ S&W GEO1.GDT 7/10/02

MATERIAL DESCRIPTION 17 Ft. Relative to MSL	Depth, Ft.	Samples	Ground Water
Medium dense, brown, slightly silty, gravelly SAND; moist (FILL)	2.0	S-1	5/15/2001 ∇
Medium dense, dark brown, silty, gravelly SAND; moist (FILL) S1: Gravel = 29%, Sand = 51%, Silt = 21%; F3	4.5	S-2	
Medium stiff, gray CLAY; moist Contains decayed organic material between 19 and 32 feet bgs Strong sulfur odor in these samples	32.0	S-3, S-4, S-5, S-6, S-7, S-8	
Stiff, gray CLAY to slightly sandy CLAY; moist	53.0	S-9, S-10, S-11, S-12	
Dense to very dense, gray, silty SAND; moist to wet	67.0	S-13, S-14, S-15	
Dense, gray, silty, gravelly SAND; wet	71.5	S-16	
Bottom of Boring Boring Completed May 15, 2001			

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



● % Water Content

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

LEGEND

- • Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Split Spoon Sample
- 2" O.D. Split Spoon Sample
- Shelby Tube
- ∇ Ground Water Level ATD
- ▼ Perched Water Level

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Chester Creek Improvements
Anchorage, Alaska

LOG OF BORING B- 2

July 2001 | 32-1-01406

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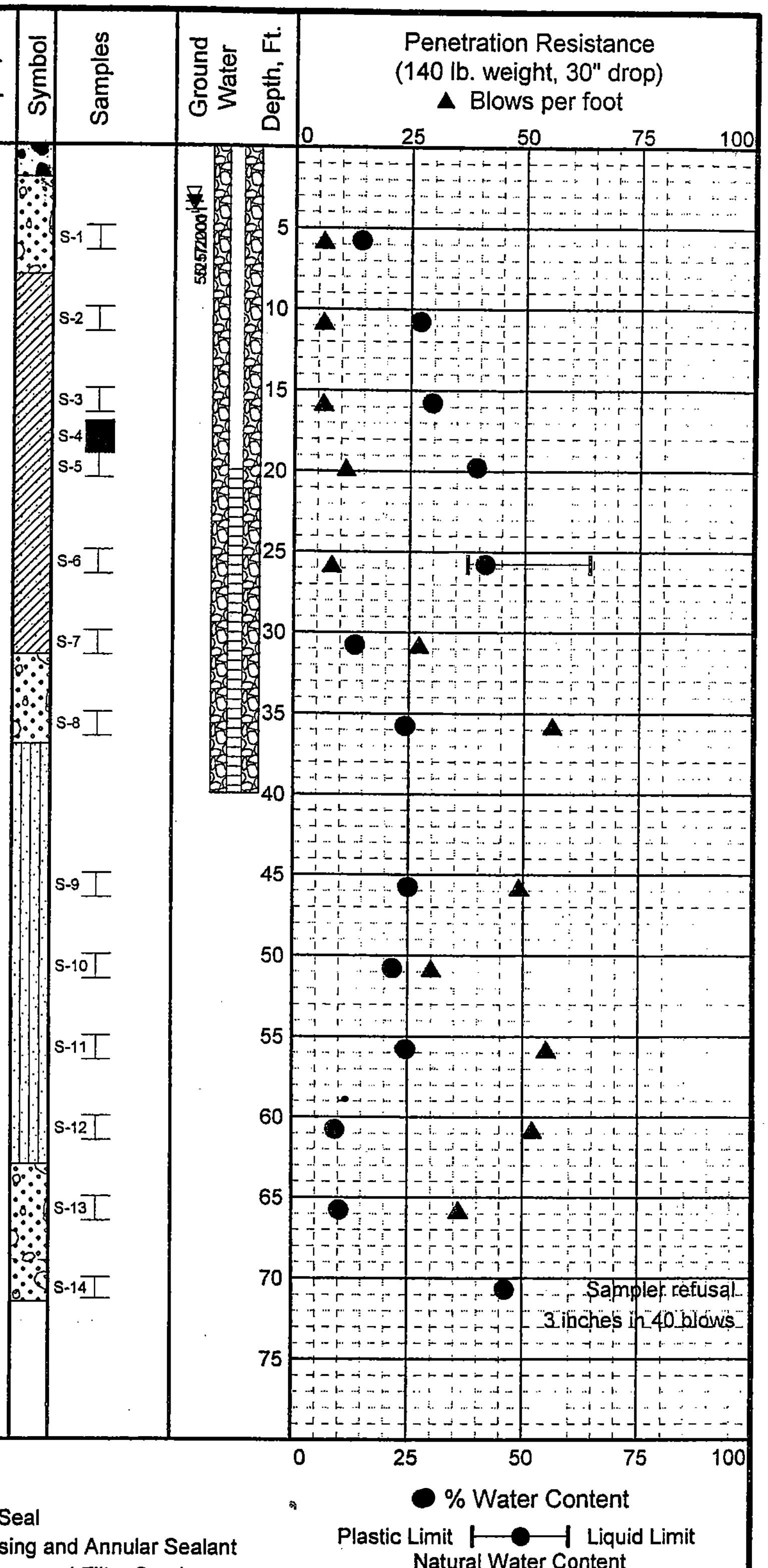
Fig. A-2

S&WGEO1 LOG 32-1-0-1.GPJ S&W GEO1.GDT 7/10/02

MATERIAL DESCRIPTION 19 Ft. Relative to MSL	Depth, Ft.
Medium dense, slightly silty, sandy GRAVEL; moist (FILL)	2.0
Medium dense, silty, gravelly SAND; moist (FILL) S1: Gravel = 29%, Sand = 51%, Silt = 19%; F3	8.0
Stiff, gray, slightly sandy CLAY; moist Contains decayed organic silt and sulfur odor below 17 feet bgs	31.5
Dense, gray, silty, gravelly SAND; wet	37.0
Hard, gray, sandy SILT; wet	63.0
Dense to very dense, gray, silty, gravelly SAND; wet	71.5
Bottom of Boring Boring Completed May 17, 2001	



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Split Spoon Sample
- 2" O.D. Split Spoon Sample
- Shelby Tube
- Surface Seal
- Solid Casing and Annular Sealant
- Well Screen and Filter Sand
- Cuttings Backfill
- Ground Water Level ATD
- Perched Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Pocket pen values are represented by PP. Torsional force vane values are represented by TV. Percent passing the number 200 sieve is represented by P200.

Chester Creek Improvements
Anchorage, Alaska

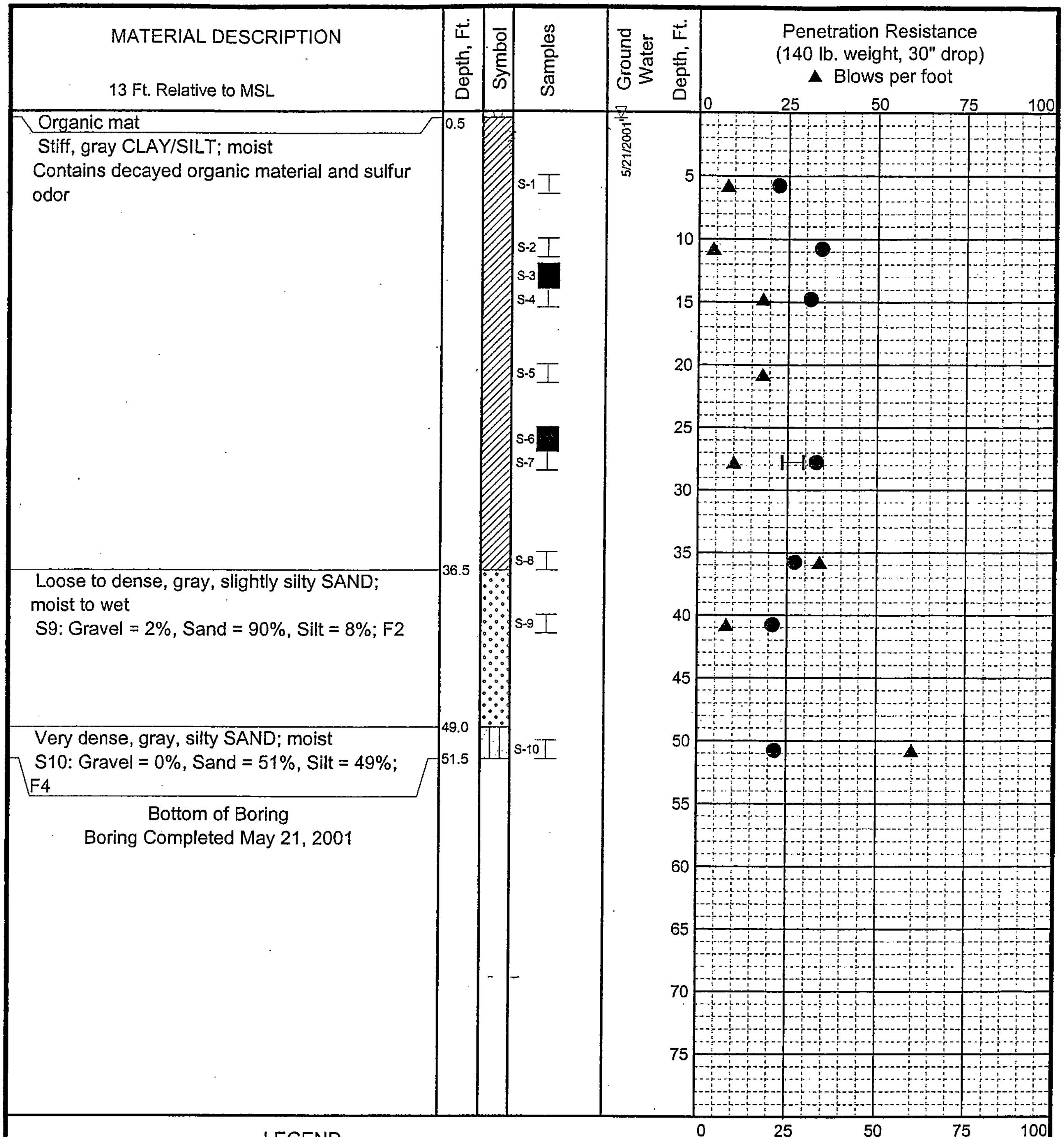
LOG OF BORING B- 6

July 2001 32-1-01406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. A-6

S&WGEO1 LOG 32-1-0-1.GPJ S&W GEO1.GDT 7/10/02



LEGEND

- * Sample Not Recovered
- ⌶ 2" O.D. Split Spoon Sample
- Ⅲ 3" O.D. Split Spoon Sample
- ⌶ 2" O.D. Split Spoon Sample
- ■ Shelby Tube
- ▽ Ground Water Level ATD
- ▼ Perched Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Pocket pen values are represented by PP. Torsional force vane values are represented by TV. Percent passing the number 200 sieve is represented by P200.

Chester Creek Improvements
Anchorage, Alaska

LOG OF BORING B- 10

July 2001 | 32-1-01406

SHANNON & WILSON, INC.
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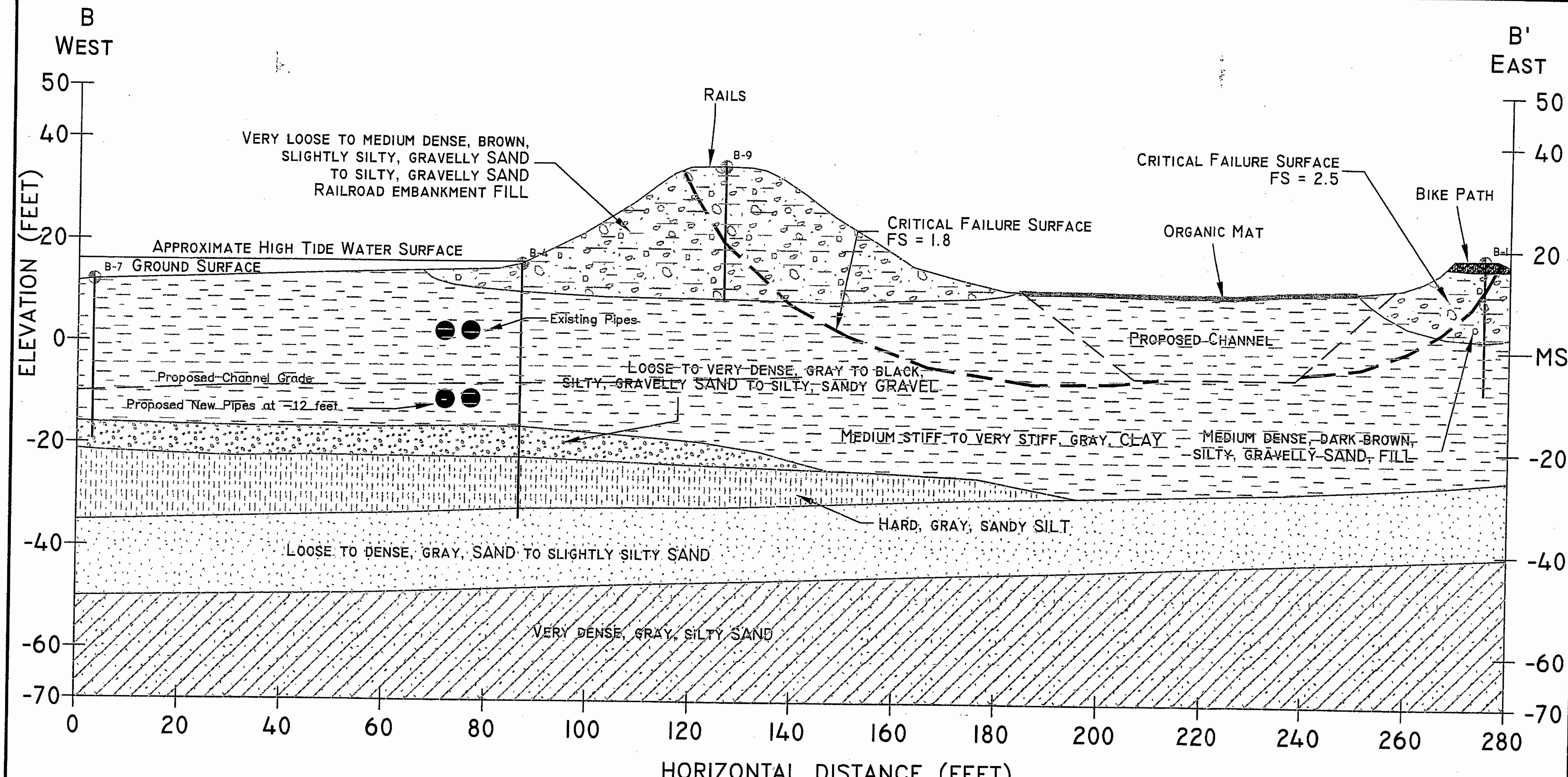
Fig. A-10

S&WGEO1 LOG 32-1-0-1.GPJ S&W GEO1.GDT 7/12/02



LEGEND

B-1 Approximate Location of Boring B-1 Advanced by Shannon & Wilson, Inc. May 2001

NOTES

- Boring locations and elevations based on site survey performed by Alaskan Consulting Surveyors in May 2001.
- Noted elevations are in feet above mean sea level MSL.
- Lithologic boundaries shown above are interpolated from boring data and should be considered approximate. Actual soil boundary locations may vary especially as the distance from adjacent borings increases
- Critical failure surfaces shown above are for both dynamic and static loading conditions. The noted factors of safety (FS) are for the most critical loading condition, in this case, dynamic loading.

Chester Creek Improvements
Anchorage, Alaska

PROFILE B-B'

July 2001 32-1-01406

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 9

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to 32-1-01406
Dated: July 2002
To: HDR Alaska
Re: Westchester Lagoon Embankment Stability

Important Information About Your Geotechnical/Environmental Report

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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