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IMPROVEMENTS TO THE WASTEWATER TREATMENT PLANT
PHASE ONE
ABERDEEN, WASHINGTON

1.0 INTRODUCTION AND SCOPE OF WORK

The draft report presents the preliminary results of our field explorations and geotechnical engineering studies completed for the proposed Phase One Improvements to the Wastewater Treatment Plant (WWTP) located in Aberdeen, Washington. The purpose of these studies was to provide foundation recommendations for the proposed improvements to the existing plant. Our scope of work included subsurface explorations and engineering analyses to develop foundation recommendations and associated construction considerations. The results of our studies are summarized in this draft report.

Our work was performed in general accordance with the scope of the geotechnical work (Task 1 - Predesign) described in the subconsultant professional agreement between Tetra Tech/KCM, Inc. (Tt/KCM) and Shannon & Wilson, Inc., which was authorized on January 11, 2001. It should be noted that our scope of work has been slightly modified because of the elimination of the proposed selector tank and the addition of a UV disinfection tank.

2.0 SITE AND PROJECT DESCRIPTION

Our understanding of the project and the site conditions is based on the documents that were provided by Tt/KCM, which include the Project Plan document dated January 11, 2001, and a previous geotechnical report dated May 17, 1977, prepared by Roger Lowe Associates for the additions to the plant, which were completed in the 1970s. The existing treatment plant is located on the shore of the Chehalis River in Aberdeen, Washington, as shown on Figure 1.

It is our understanding that the original plant was constructed in 1959. The original structures are supported on timber piles down to 60 to 80 feet. About 5 feet of fill was placed on the plant site during the original plant construction. The heavy pile-supported structures constructed in 1959 reportedly settled from 14 to 16 inches at the time of the fill placement. Additionally, several inches of settlement occurred in the 10 to 20 years following the 1959 construction.

Additions to the plant were constructed in the late 1970s, which are supported on 12.75-inch outside-diameter (O.D.), concrete-filled steel pipe piles. It was recommended in the 1977 geotechnical report that these piles be bearing on the very dense layer located about 135 feet below the existing ground surface. We were not able to obtain the records of the construction completed in the 1970s. It is our understanding that no settlements were reported for the 1970s additions. We also understand that about two years ago, however, below ground, concrete-encased electrical conduits settled excessively.

The proposed phase one improvements include a new secondary clarifier, a blower building, and the UV disinfection tank (see Figure 2). The site is relatively flat, ranging from elevations 13 to 17 feet (within the area of the three proposed building). Elevation datum used in this report is referenced to the Mean Lower Low Water (MLLW). Almost one-half of the land required for the proposed secondary clarifier is currently owned by Weyerhaeuser. A drainage ditch, an 18-inch-diameter storm drain pipe, and a 42-inch-diameter storm drain pipe are currently located within the footprint of the proposed secondary clarifier, as shown on Figure 2.

It is our understanding that the UV disinfection tank and the blower building will be located at or near grade. For the secondary clarifier tank, the bottom center will be at about elevation –3 feet (about 18 feet below the ground surface) and the bottom edge will be at about elevation 5 feet (about 10 feet below the ground surface). We understand that the approximate design load for the secondary clarifier tank is 2,000 pounds per square foot (psf). The design loads for the UV disinfection tank and the blower building are still being developed.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

The field explorations consisted of three borings drilled with a Mobile 95 truck-mounted rig at the locations shown on Figure 2. The locations of the borings were surveyed by Berglund, Schmidt, and Associates, Inc. The borings were drilled using mud-rotary techniques by Geo-Tech Explorations, Inc. of Seattle, Washington, under a subcontract to Shannon & Wilson, Inc. Borings CB-1, CB-2, and CB-3 were drilled to depths of about 151, 92, and 138 feet, respectively. The logs of the three borings are presented on Figures A-2 through A-4 (Appendix A).

Soil samples were obtained during drilling using 2-inch outside diameter and 3-inch-diameter Osterberg samplers. To obtain disturbed samples, Standard Penetration Tests (SPTs) were performed as shown on the boring logs. The SPT consisted of driving a 2-inch outside-diameter

split-spoon sampler a total distance of 18 inches into the bottom of the boring with a 140-pound hammer falling 30 inches. The numbers of blows required to advance the sampler for each 6-inch increment were recorded. The blow count recorded for the last 12 inches of penetration is termed Standard Penetration Resistance (N-value). When penetration resistance exceeded 50 to 100 blows for 6 inches or less of penetration, the test was terminated and the number of blows and the corresponding penetration recorded. To obtain undisturbed cohesive soil samples, a 3-inch O.D. Osterberg sampler (a special sampler often used to sample very soft soils) was used.

Soil samples retrieved using split-spoon and Osterberg samplers were returned to our laboratory in Seattle for testing. Laboratory testing included visual classification, natural water content, Atterberg limits, fines content, triaxial undrained shear strength tests, and one consolidation test. The results of the water contents and the Atterberg limits tests are presented on the boring logs (Figures A-2 through A-4). The results of the other laboratory tests are presented in Appendix B.

A monitoring well was installed in boring CB-3 (the clarifier boring) to measure the depth of the groundwater following completion of drilling and to perform slug tests. The purpose of the slug test was to estimate the permeability of a soil layer. The results of the slug test are presented on Appendix B. In addition, a vibrating wire transducer piezometer (VWP) was installed in the monitoring well for five days to monitor the fluctuation of the groundwater table with the variation of the tide level in the river.

4.0 SUBSURFACE CONDITIONS

Based on the results of the three current and the two previous borings, the subsurface conditions at the locations of the three proposed buildings are relatively uniform, as shown on Figure 3. The subsurface conditions consist of about 6 to 10 feet of fill layer overlying 120-foot-thick estuarine deposits which overlie alluvial deposits. (Estuarine deposits result when fresh water of a river comes into contact with sea water.) The fill layer consists of a mixture of sandy gravel, silty sand, and wood debris with a relative density varying from loose to very dense.

In general, the estuarine deposits at the site consist of stratified very soft to stiff (locally very stiff to hard) silts of medium to high plasticity and very loose to medium dense silty fine sand to fine sandy silt, of none to very low plasticity. The estuarine, soft to medium stiff silt layers are very compressible, as indicated by the standard penetration resistances (N-values). Also, the results of the consolidation test indicate that the lower silt layer is normally consolidated with an overconsolidation ration (OCR) of 1.1. OCR is the ration of the maximum past effective

pressure over the existing effective pressure. Our visual classification indicates the presence of abundant organics in the estuarine silt layer, which is reflected in the water contents and the Atterberg limits.

The estuarine silty sand to sandy silt layers vary in relative density from very loose to medium dense, with N-values ranging from 0 to 22 blows/foot. In the previous borings (B-203 and B-204), N-values in these layers were higher than the ones recorded in the current borings. The logs of the previous boring B-203 and B-204 are presented in Appendix C. The difference in N-values between the previous and the current borings can be attributed to the presence of wood fragments and the method of drilling. Based on our understanding of the geology, the current N-values were used in the engineering analysis presented in Section 5.0. The fines contents of the tested samples ranged from 47 to 74 percent. Some samples of these layers include small percentage of clays (estimated to be 5 to 10 percent), as indicated by the very low plasticity. Considering the fines content, plasticity, N-values and groundwater depth, the estuarine silty sandy to sandy silty layers encountered in the current borings are likely susceptible to liquefaction.

The estuarine deposits overlie very dense alluvial deposits consisting of gravelly sand and sandy gravel located at a depth of about 133 feet below the existing ground surface.

The results of the slug test performed on boring CB-3 indicate that the permeability of the top silty sandy layer located approximately from 28 to 33 feet below the existing ground surface is about 7.4×10^{-6} cm/sec, which is consistent with our experience with similar soils. The data obtained from the VWP indicate that the groundwater table at the location of boring CB-3 is about 5 feet (Elevation 10 feet) below the existing ground surface. The data also suggest very weak communication between the groundwater table and the river as indicated in Figure 4. In addition, the 1977 geotechnical study performed for the 1970s additions indicates that the groundwater in the top silt layer (approximately from 10 to 30 feet below the ground surface) was at about El. 8.0 feet, with weak communication with the river elevation.

For the fill layer, it was observed during drilling that groundwater flowed into the drilled hole during the high tide with a relatively high rate. During the low tide, the drilling fluid was seeping out of the drilled hole. Based on these observations, it appears that this fill layer is very pervious with strong communication with the adjacent river.

5.0 ENGINEERING STUDIES AND RECOMMENDATIONS

This section presents the results of the engineering studies and the foundation design recommendations for the proposed three structures. The engineering studies included seismic and settlement analyses for the site and pile foundations for support of the new structures. Also presented in this section are earthwork recommendations for the proposed secondary clarifier and considerations for other construction associated issues.

5.1 Seismic Analyses

The seismic analyses for the subject sites consisted of:

- Liquefaction analysis, which included estimate of soil strength reduction, settlement, and lateral spreading due to liquefaction.
- Estimate of free-field motion.
- Site-specific ground response analysis for the proposed structures.

According to the 1997 building code, it is assumed that the seismic design of the proposed structures will be based on ground motions with a 10 percent probability of being exceeded in 50 years. The main source of seismic hazardous at the site is movement on the Cascadia Subduction Zone. The U.S. Geological Survey (USGS) map indicates that for 10 percent probability (a recurrence interval of 475 years), the site peak ground acceleration (PGA) is 0.3g. Consequently, we recommend that a site PGA of 0.3g be used in the seismic analyses. Based on the subsurface conditions discussed above, the site soil profile is classified as soil type S_F because of potential liquefaction. For soil type S_F, the 1997 UBC requires a site-specific ground response analysis.

To perform a site-specific ground response analysis, ground motion time histories representative of actual earthquake motions should be used. We selected three ground motions, each from a different source with different characteristics, to obtain a reasonable broad envelope of spectra. The three motions were previously used in our seismic retrofit studies completed for Wishkah River/Heron Street Bridge and the Chehalis River Bridge in Aberdeen, Washington. The three ground motions are: (1) Gilroy No. 6 recorded during the 1989 Loma Prieta Earthquake, (2) Cholame recorded during the 1996 Parkfield Earthquake, and (3) the Highway Test Lab record from the 1949 Olympia Earthquake. The three ground motion histories were scaled to PGA of 0.3g.

The soil properties required for the site-specific analysis were estimated based on the SPT values and the laboratory testing results and also on field measurements (shear wave velocities) obtained from the seismic studies for the two bridges mentioned above.

Using the results of the site-specific ground response analysis, we performed a liquefaction analysis considering the worst of the three ground motions presented above. The liquefaction analyses suggest that the estuarine silty sandy to sandy silty layers would liquefy under the design PGA of 0.3g and an earthquake magnitude of 7.5. The results of the liquefaction analyses are shown on Figure 5. Liquefaction would cause ground settlement of about 12 inches and soil strength reductions as discussed in Section 5.3.

Additionally, because of the vicinity of the Chehalis River to the treatment plant site, lateral spreading of more than 6 inches (lateral movements towards the river) might occur due to liquefaction of the top layer (from 28 to 33 feet below existing ground surface). In the analyses, it was assumed that the river bottom is at Elevation –40 feet based on our verbal communication with Burglund, Schmidt, and Associates. It should be noted that the state-of-the-art analysis method of lateral spreading is based on case histories, which are limited in depth of liquefied soils to about 33 feet. As such, lateral spreading at the subject site due to soil liquefaction at 45 feet and below, although above the river bottom, has a low probability of occurrence. It is expected that lateral spreading will have a significant impact on the foundation design.

Regardless of liquefaction occurrence, the subsurface layers would be subjected to lateral deformations during an earthquake shaking, each layer with a different deformation magnitude and, sometimes, direction. The lateral deformations are critical and should be considered in the deep foundation design. The results of the free-field motion analyses are presented on Figure 6. It should be noted these lateral deformations would occur during the shaking prior to liquefaction.

For the seismic design of the structures, two methods are typically used in the design: (1) response spectrum and (2) equivalent base shear. For the response spectrum analysis, the results of the three motions are presented on Figure 7 assuming a structural damping of 5 percent. For the equivalent base shear method, the seismic coefficients required for the analyses are presented in the following table.

Zone Factor (Z)	3	
Soil Type	S_F	
C_a	0.18	.36
C_v	0.45	.94

As described above, the proposed plant site would likely experience liquefaction under the design PGA of 0.3g and an earthquake magnitude of 7.5. Our additional analyses also indicate that the proposed project site would not be subjected to liquefaction and lateral spreading under a magnitude 6.5 earthquake with a PGA of 0.2g.

5.2 Settlement Analysis

As mentioned in the Subsurface Conditions section, the stratified estuarine layers are very compressible. The normally-consolidated silt layers would probably continue settling due to secondary compression (rearrangement of soil structure after loading). In addition, decaying of the abundant organics in the silt layers may cause additional settlement. These two sources, separately or together, might cause the settlement of the concrete-encased electrical cables as mentioned in Section 2.0. Furthermore, settlements would occur due to liquefaction as mentioned in Section 5.1 These settlements would likely cause downdrag on pile foundations supporting the proposed new structures.

5.3 Foundation Design Recommendations

5.3.1 Design Considerations

Because of the settlement concern, particularly in the silt layers, shallow foundations are not recommended to support the proposed new structures. Ground improvements, such as surcharging, are not practically feasible at the site because of the relatively thick silt deposits, which need to be improved, and also the relative long period required to reduce potential settlement. Because of this, we recommend supporting the structure on deep foundations. The deep foundation must be designed as end bearing driven into the underlying very dense alluvial layer because of the high compressibility of the overlying layers. In addition, the on going silt settlement would cause a downdrag force on the proposed piles, resulting in additional vertical loads. We recommend that steel pipe piles ranging between 12- and 16-inch O.D. be considered to support the proposed new structures. We further recommend that the steel pipe piles be driven

closed-end to the underlying very dense alluvial deposits to achieve the required ultimate capacity. The pipe piles should be filled with concrete after driving.

Piles should also be designed to resist the lateral loads. For the proposed secondary clarifier, part of the lateral loads will be resisted by the difference between the passive and active pressures within the buried portion of the structure. The recommended static passive and active pressures in term of equivalent fluid weights in pounds per cubic foot (pcf) are presented in the table below.

Location	Equivalent Fluid Weights (pcf)			
	Static Active	Static Passive	Seismic Active	Seismic Passive
Above GWT	36	260	48	350
Below GWT	18	130	24	175

63

Notes:
1. Static and seismic passive fluid weights include factors of safety of 1.5 and 1.0, respectively.
2. GWT: Groundwater Table.

In addition to the static loads and the downdrag force, the proposed piles will be subjected to additional loads due to earthquake shaking. These loads include lateral loads on the top of the piles; free-field induced loads; lateral loads on the piles due to lateral spreading; and seismic, net lateral earth pressures. The recommended seismic active and passive lateral earth pressures are presented in the table above. Furthermore, seismic shaking would cause a reduction in soil strengths. The table below presents three cases with the corresponding soil strengths under which the piles should be designed.

Case Description	Loading Description	Reduction in Soil Strength
Static	* Static axial + downdrag * Static lateral – static passive pressure + static active pressure	No reduction
During Earthquake Shaking	* Static axial + downdrag + seismic axial * Static lateral + seismic lateral – seismic passive pressure + seismic active pressure + free-field loads	No reduction
Immediately After End of Shaking	* Static axial + down drag + (seismic axial/2) * Static lateral + (seismic lateral/2) + static active pressure + lateral spreading loads	Soil reduction all estuarine layers (see table below).

Note: Lateral earth pressures are applied only for the secondary clarifier case.

The following table summarizes the recommended soil strengths for static and seismic cases.

Depth (ft)	Soil Type	Static		Seismic	
		ϕ'	S_u (psf)	ϕ'	S_u (psf)
0-10	Fill	32	---	32	--
10-28	Cohesive Silt	26	---	--	480
28-33	Silty Sand to Sandy Silt	32	---	9.0 (liquefaction)	---
33-45	Cohesive Silt		600	--	480
45-65	Silty Sand to Sandy Silt	30	--	3 (liquefaction)	--
65-75	Silty Sand to Sandy Silt	32	--	7 (liquefaction)	--
75-110	Cohesive Silt		1800		1450
110-133	Cohesive Silt		2100		1680
133-	Gravelly Sand	42		42	

Note: c'=0.0 for all soils.

It is expected that the seismic cases would be the governing cases for the pile design. It is also expected that the lateral spreading loads on the piles would significantly increase the foundation cost. As an alternative to reduce the foundation cost, soil improvements such as vibro-compaction (stone column) can be used to mitigate the liquefaction and, as a consequence, the lateral spreading potentials. The recommended depth of the stone columns is about 40 feet. The improved areas should extend at least 40 feet horizontally beyond the perimeter of the proposed structure to isolate the proposed structure. This improved area beyond the perimeter of the structure would act like a buffer between the structure area and the rest of the site during seismic loading.

5.3.2 Loading Conditions

Based on the above discussion, it is our opinion that the City of Aberdeen and Tt/KCM evaluate the pros and cons of the seismic design criteria as required by the 1997 UBC and decide if they will proceed with these requirements with or without ground improvements, or use different seismic requirements. In summary, there are three loading conditions for consideration.

Condition 1 – Static Loading

Under static loading conditions, vertical loads of the proposed new structures can be supported by pile foundations. Lateral loads may be resisted by piles and/or the passive pressures generated within the buried portion the structures.

Condition 2- Magnitude 6.5 and PGA = 0.2g Earthquake

Under the earthquake, the potential of liquefaction and lateral spreading is minimal, the foundation design requirements would be the same as those under static loading conditions.

Condition 3 – Magnitude 7.5 and PGA = 0.3g Earthquake (1997 UBC)

Under this earthquake as required by the 1997 UBC, liquefaction and lateral spreading are likely to occur. Vertical loads of the proposed new structures can be supported by pile foundations. However, it will be very difficult to use most foundations to resist lateral spreading. One of the feasible deep foundations would be large-diameter drilled shafts installed many feet into the underling very dense alluvial deposits. Installation of these long and large-diameter drilled shafts will be very difficult and costly. We, therefore, recommend that ground improvements, such as vibro-compactions as described earlier, be considered in the foundation design. After the recommended ground improvement, the foundation design requirements would be similar to those under static loading conditions.

5.3.3 Pile Axial Capacity

As mentioned earlier, steel pipe piles ranging from 12- to 16-inch O.D. are recommended to support the proposed new structures. We recommend that these steel pipe piles be driven closed-end approximately 3 to 5 feet into the underlying very dense alluvial deposits. Based on the subsurface conditions encountered in the borings, relative densities as determined by N-values, and our experience in similar soils and project conditions, we used an in-house computer program to estimate pile axial capacities. The pile capacity estimates include the effect of downdrag (negative skin friction) caused by ground settlements due to the weight of the existing site fill and the seismic loadings. Assuming that the steel pipe piles are driven closed-end and penetrate 3 feet into the underlying very dense alluvial deposits, the estimated downdrag; ultimate positive skin friction; ultimate end bearing; allowable design load; and ultimate uplift resistance for 12-, 14-, and 16-inch O.D. steel pipe piles are presented in the following:

Pile O.D. (inches)	Downdrag (tons)	Ultimate Positive Skin Friction (tons)	Ultimate End Bearing (tons)	Allowable Design Load (FS=2) (tons)	Ultimate Uplift Resistance (tons)
12	42	78	78	57	120
14	50	90	107	74	140
16	56	103	140	94	159

As indicated, a factor of safety (FS=2) of 2 was used to calculate the allowable compressive design load. An appropriate factor of safety should be used to estimate the allowable uplift resistance. The recommended factors of safety are 3 for long-term loads such as wind loadings and 1.5 for short-term loads such as seismic loadings.

We understand that the approximate design load for the secondary clarifier is 2,000 psf (1 tsf). Thus, the 12-, 14-, and 16-inch O.D. steel pipe piles would be able to support about 60, 75, and 95 square feet of the plan area for the secondary clarifier, respectively. The design loads of the blower building and UV disinfection tank were not available when this report was prepared. However, the pile foundation design can be completed once their design loads become available.

5.3.4 Pile Lateral Resistance

The computer program LPILE^{PLUS} (Reese and Wang, 1997) may be used to generate P-Y curves for the lateral resistance analysis of piles. Based on subsurface conditions as interpreted from the explorations accomplished at the site, the recommenced parameters for input into the LPILE^{PLUS} program are given in Table 1.

The recommended efficiency (reduction) factors due to pile group action are listed below. The efficiency factors are based on recommendations presented in a 1998 ENSOFT Seminar (ENSOFT, 1998).

EFFICIENCY FACTORS FOR PILE GROUPS

Pile Spacing	Efficiency Factor
2D	0.6
3D	0.7
4D	0.8
5D	0.9
6D	1.0

5.4 Pile Installation

Fixed-lead pile driving equipment is recommended to drive the steel pipe piles. The use of hanging or swinging leads is not recommended, unless they are constructed so that they can be held in a fixed position during driving operations. An air-, steam-, hydraulic-, or diesel-powered hammer may be used for driving the proposed steel pipe piles. The Contractor should furnish the information required on the Pile and Driving Equipment Data Sheet, shown on Table 2, seven days in advance of the scheduled pile driving. We recommend that the steel pipe piles be driven with an air, steam, or hydraulic hammer that has a rated energy ranging between 30 and 50 foot-kips depending on the pile sizes. For a diesel hammer, a rated energy between 25 and 60 foot-kips and a ram weight of 6.6 kips is recommended. Final driving criteria should be based on the actual hammer-pile-soil system.

Wave Equation Analyses for Pile driving (WEAP) will be performed after receiving the design loads and determining the pile size. Because of the effects of downdrag, the piles will have to be overdriven to a higher ultimate load than that obtained simply by multiplying the allowable pile capacity by the factor-of-safety (FS). The ultimate pile capacity required to establish the driving criteria should be determined using the following equation:

Ultimate Capacity = [(Allowable Capacity + Downdrag Load) x (FS = 2.0)] + Downdrag Load

All pile driving should be monitored by taking a continuous driving record of each pile. For this purpose, the Contractor should be required to mark the pile in 1-foot increments. Additional 1-inch increments between the 1-foot marks may be required. The pile-driving record should be complete. The form should have spaces to record hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record should include tip elevation, driving criteria, and initials of inspectors making final acceptance of the pile. The pile-driving records should be reviewed on a daily basis.

All driven piles should be checked for possible heave prior to cutoff. The heave should be checked by surveying the elevation of each pile butt after a pile is driven within the group. The heave data should be provided to the Owner and Engineer to determine if redrive will be required. In general, if the heave is greater than 0.15- to 0.25-inch, the pile should be redriven to the original cutoff or tip elevation.

It is often difficult to estimate the energy delivered by diesel hammers. The Saximeter, developed by Pile Dynamic, Inc., can be used to record hammer strokes and provide an estimate

of the driving energy of diesel hammers. If the Contractor selects a diesel hammer, we recommend that a Saximeter be used during pile driving.

5.5 Buried Pipes

The proposed improvements include pipe installation between the proposed and existing structures. It is our understanding that the existing pipes are not pile supported but no settlement-related problems were reported, except for the concrete-encased electrical conduits. For pile-supported structures, differential settlements between the pipe ends are not anticipated. Significant differential settlement, however, may occur between pipe ends and the midpoint of a pipe, depending on the pipe length and stiffness. We recommend that the proposed pipes be designed assuming that the ground surface would settle about 2 inches under static condition. Under seismic conditions, the ground surface would settle about 12 inches if liquefaction occurs. The designer should evaluate the risk associated with ground settlements due to seismic loading versus supporting the pipes on deep foundations.

As an alternative, it is also our understanding that a utility tunnel is currently under consideration to house the existing and proposed pipes to avoid excessive settlements. In such a case, we recommend supporting the utility tunnel on deep foundations bearing on the underlying very dense alluvial soils.

5.6 Earthwork Recommendations

As mentioned before, the construction of the clarifier would require about a 10-foot-deep excavation below the existing ground surface at the rim of the clarifier. Because of the presence of soft silt immediately below the bottom of the clarifier, shoring the excavation will require a relatively deep penetration. Due to the relative remoteness of the proposed clarifier from the other existing structures, a more economical alternative is to lay back the excavation at 1.75 Horizontal to 1.0 Vertical (1.75H:1V). That means the excavation would start about 17 feet from the perimeter of the proposed clarifier. For safe working conditions and prevention of ground loss, excavation slopes should be the responsibility of the Contractor since he/she will be at the job site to observe and control the work. In our opinion, the soils can be excavated with conventional earthwork equipment.

As mentioned in the subsurface conditions, the existing fill layer is very pervious and is hydraulically connected to the river. As such, a high rate of groundwater flowing through the fill

into the excavation is expected. The Contractor will be required to provide a dewatering plan for the excavation.

Excavated soils from the secondary clarifier will consist of fill and an estuarine silty soil of high plasticity. These soils are not recommended to be used for backfill behind the clarifier wall. Instead, we recommend the fill material consisting of clean, granular soil, with no more than 5 percent fines content, which should be non-plastic. The fill should be placed in 8-inch-thick loose lifts and compacted to 92 percent of the American Society for Testing and Materials (ASTM) D 1557 maximum dry density. The water content of the fill soils should be within ±2 percent of their optimum moisture content.

6.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based upon site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions at the location of the proposed improvements (i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations). Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. Our conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as interpreted from the explorations.

If, during construction, subsurface conditions different from those encountered in the field explorations are observed or appear to be present during excavations and pile installation, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission 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 concerning the changed conditions or the time lapse.

This report was prepared for the exclusive use of Tt/KCM. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface

conditions such as those interpreted from the exploration logs and presented in the discussions of subsurface conditions included in this report.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely by taking soil samples from test borings. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

The scope of our geotechnical services did not include any environmental assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site, or for evaluation of disposal of contaminated soils or groundwater should any be encountered. Also, the scope of our geotechnical services did not include any corrosion evaluation for the proposed pipes.

Shannon & Wilson, Inc. has prepared a document, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports. This document is included in this report as Appendix D.

SHANNON & WILSON, INC.

Ming-Jiun (Jim) Wu, P.E.
Senior Vice-President

RM:JW/rm

TABLE 1
RECOMMENDED PARAMETERS FOR DEVELOPMENT OF P-Y CURVES USING LPILE^{PLUS}

Reference Boring Number	Boundary Depth (feet)		Soil Type	Effective Unit Weight (pcf)		Cohesion c (psf)		Friction Angle ϕ (degrees)		Modulus of Subgrade Reaction, k (pci)		ϵ_{50}
	Upper	Lower		Static	Liquef.	Static	Liquef.	Static	Liquef.	Static	Liquef.	(%)
CB-1 and CB-3	0.0	10	Sand	115		0	0	32	32	40	40	--
	10	28	Clay	50		600	480	0	0	50	40	2
	28	33	Sand	50		0	0	32	9	30	3	--
	33	45	Clay	50		600	480	0	0	50	40	2
	45	65	Sand	50				30	3	20	2	--
	65	75	Sand	50		0	0	32	7	30	3	--
	75	110	Clay	50		1,800	1,450	0	0	400	320	0.7
	110	133	Clay	50		2,100	1.,680	0	0	700	560	0.5
	133-		Sand	65		0	0	42	42	150	150	--

NOTES:

1. The values above do not include a reduction in efficiency (an efficiency factor) for deep foundation group action.
2. Liquef. = liquefied soil conditions.

TABLE 2
Pile and Driving Equipment Data

Contract No.: _____ Structure Name and/or No.: _____

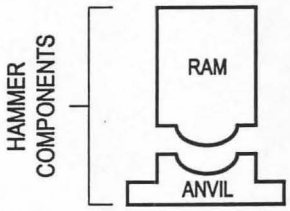
Project: _____ _____

_____ Pile Driving Contractor or Subcontractor: _____

County: _____ _____
(piles driven by)

HAMMER

Manufacturer: _____ Model: _____
Type: _____ Serial No.: _____
Rated Energy: _____ @ _____ Length of Stroke
Explosive Force: _____
(for diesel hammers)

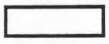


RAM

Ram Weight: _____ Ram Length: _____
(for diesel hammers)
Ram Cross Sectional Area: _____

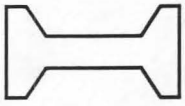
ANVIL

(with diesel hammers) Anvil weight: _____



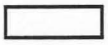
CAPBLOCK

Material: _____ Area: _____
Thickness: _____
Modulus of Elasticity - E: _____ (psi)
Coefficient of Restitution - *e*: _____



PILE CAP

Helmet
Bonnet
Anvil Block
Drivehead
Weight: _____



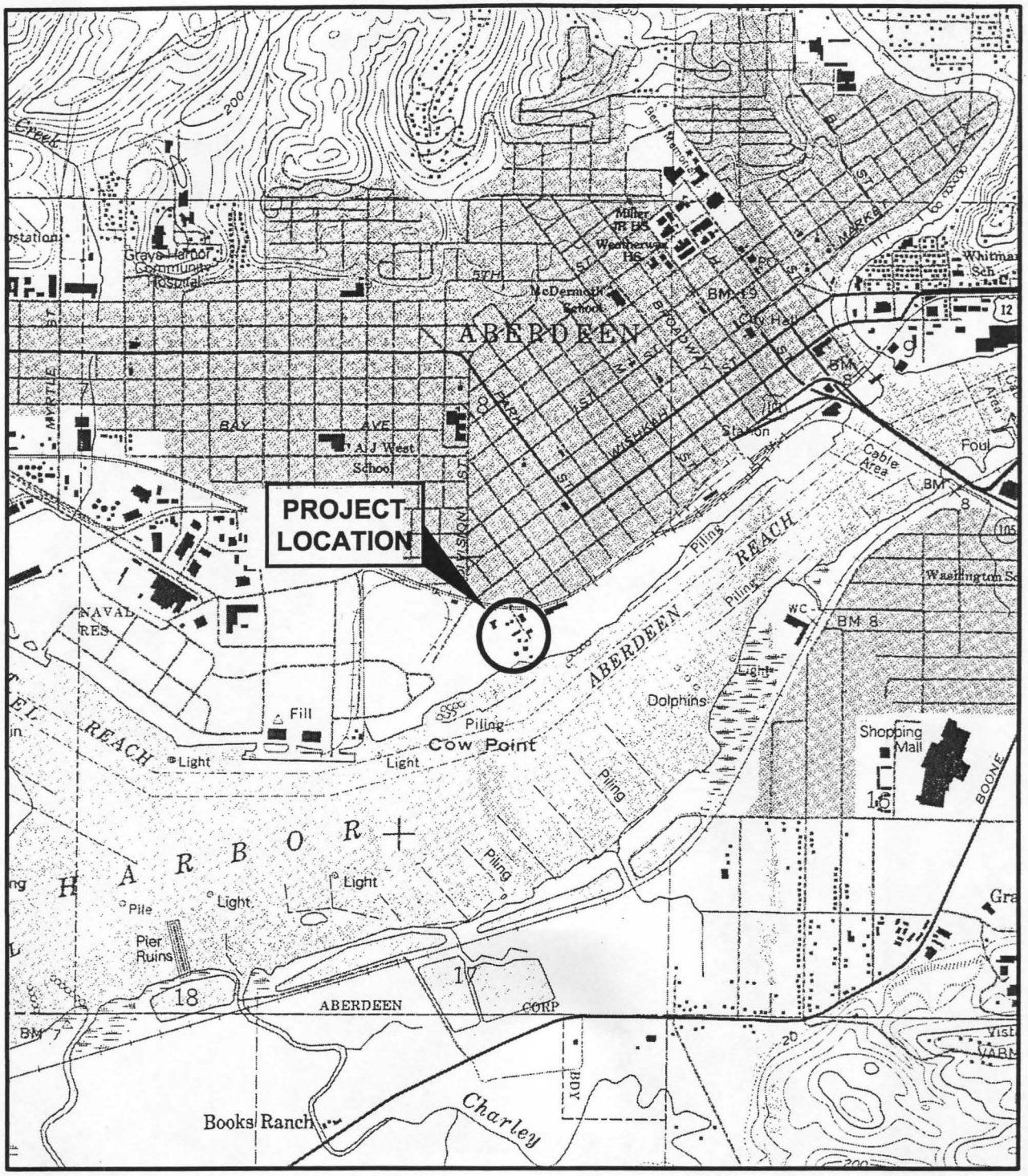
CUSHION

Cushion Material: _____ Area: _____
Thickness: _____
Modulus of Elasticity - E: _____ (psi)
Coefficient of Restitution - *e*: _____

PILE

Type: _____
Pile Size: Length (in leads): _____
Diameter: _____
Wall Thickness: _____ Taper: _____
Material: _____ Weight/Ft.: _____
Design Pile Capacity: _____ (tons)
Description of Splice: _____
_____
Tip Treatment Description: _____
_____

File: I:\Drafting\211\09279-001\21-1-09297-001 Table 2.dwg Date: 03-26-2001 Author: CVD



0 1/2 1

Scale in Miles

NOTE

Map adapted from 1:24,000 USGS topographic map of Aberdeen, WA dated 1957, photorevised 1994.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

VICINITY MAP

March 2001 | 21-1-09279-001

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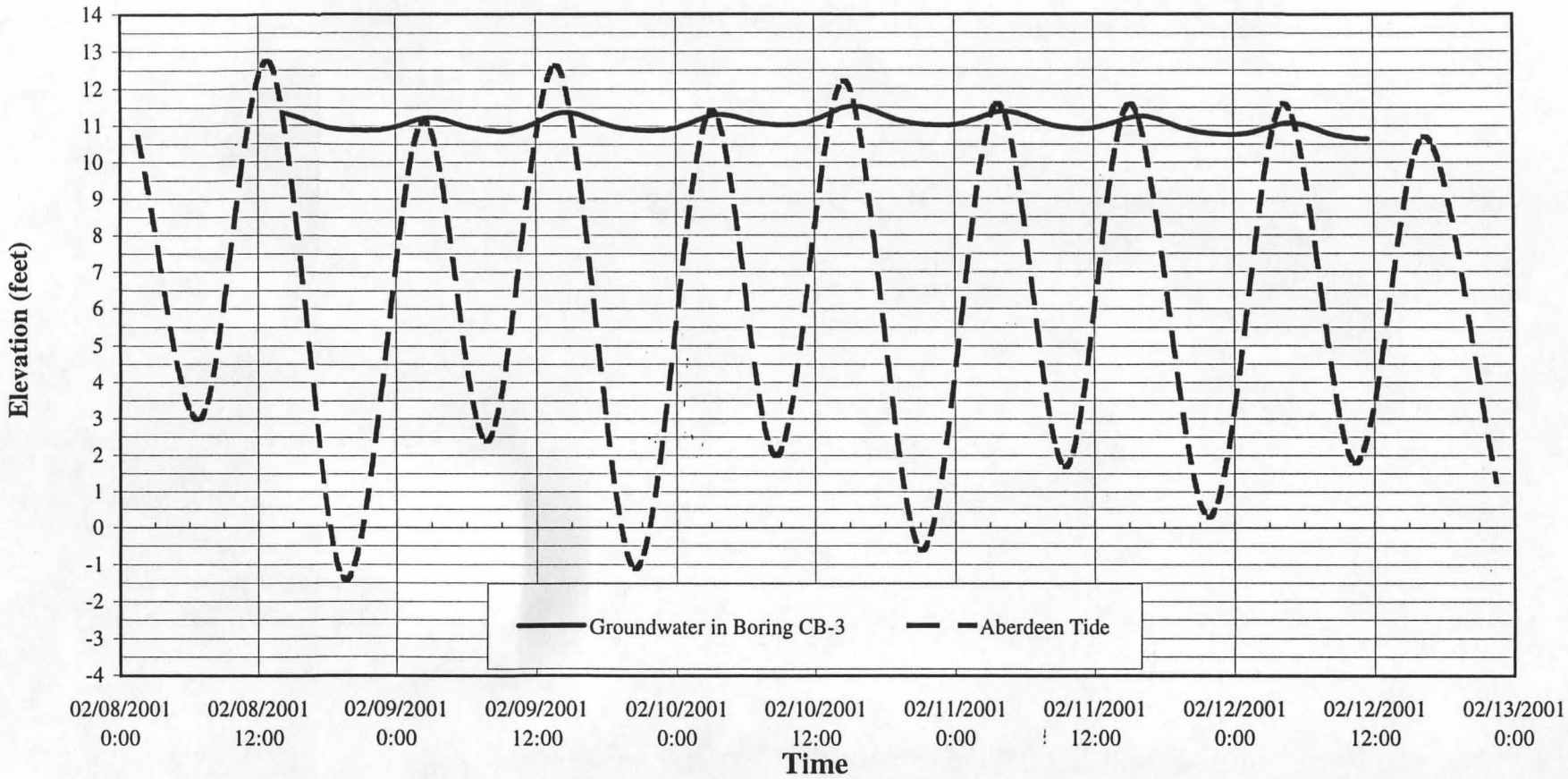
FIG. 1

File: I:\Drafting\211\09279-001\21-1-09297-001 Fig 1.dwg Date: 03-05-2001 Author: CVD



Notes:

1. Elevations are referenced to the Mean Lower Low Water (MLLW) datum.
2. Tide data obtained from Aberdeen Station (www.dairiki.org/tides).

Imrovements to the Wastewater Treatment Plant - Phase One
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TIDE DATA & GROUNDWATER MEASUREMENTS IN BORING CB-3

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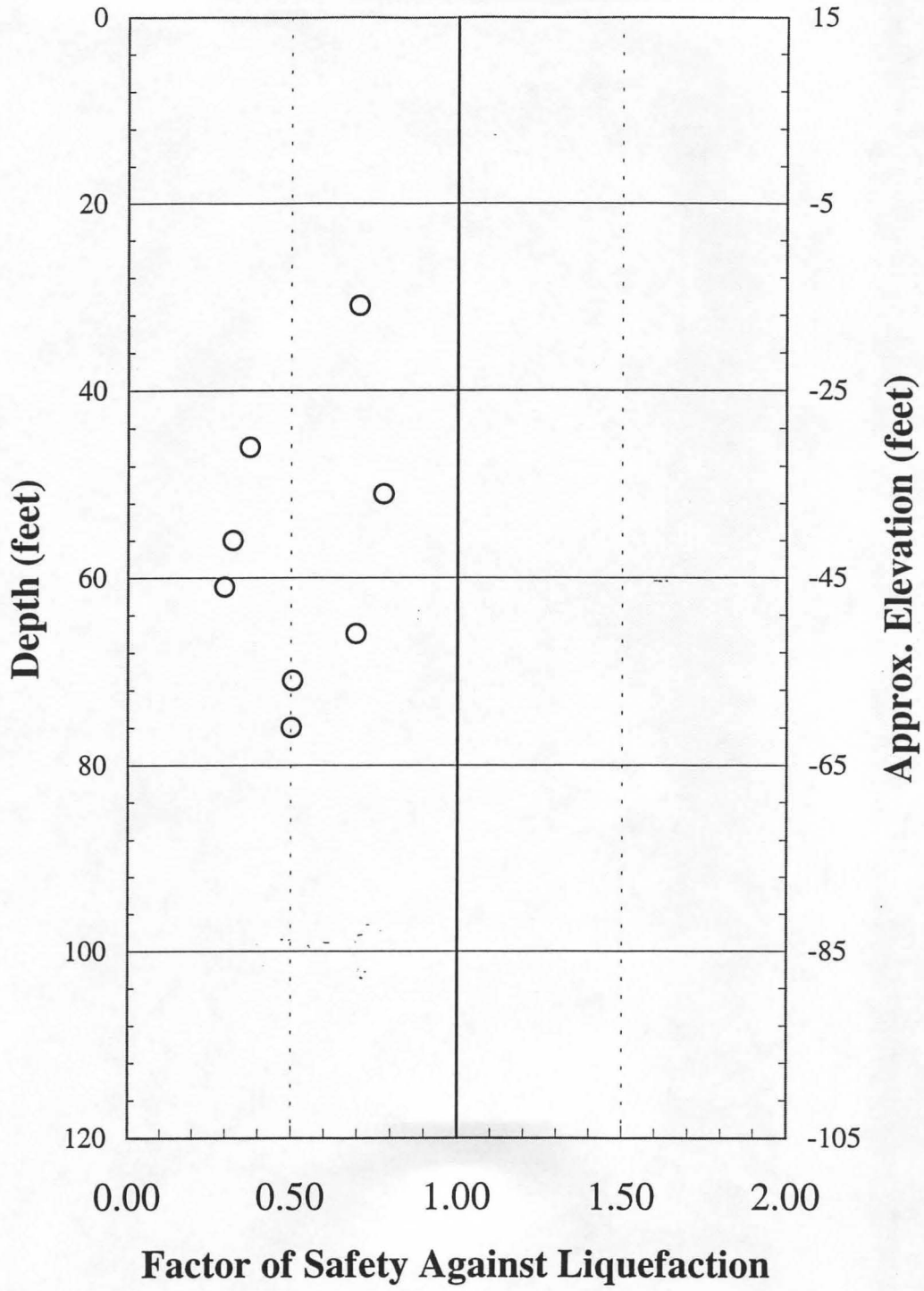
FIG. 4

FIG. 4



Note:
Factors of safety against liquefaction are based on a Magnitude 7.5 and 0.30g site PGA

Improvements to the Wastewater Treatment Plant - Phase One
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LIQUEFACTION ANALYSIS

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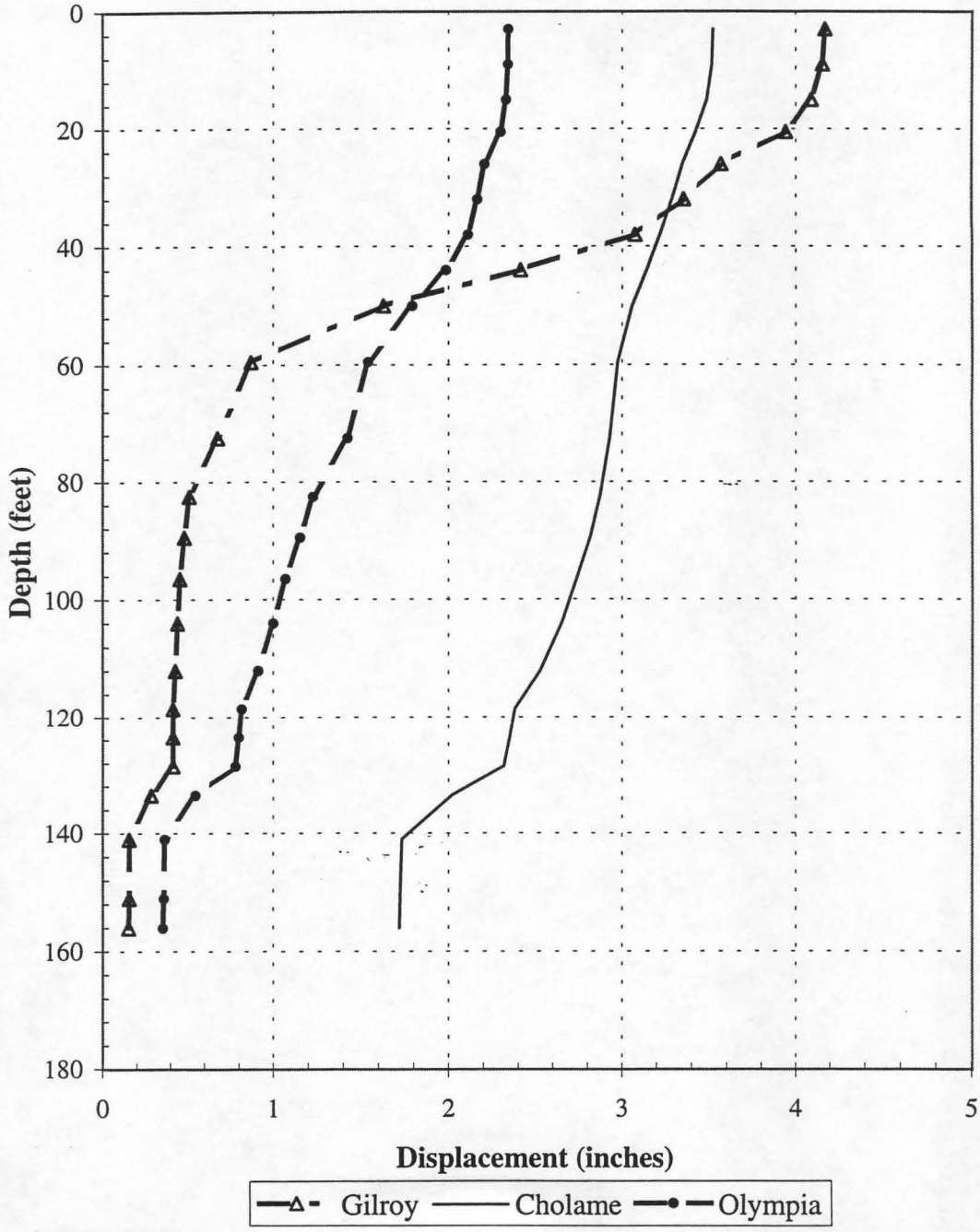
FIG. 5



Improvements to the Wastewater Treatment Plant - Phase One
Aberdeen, Washington

ESTIMATED FREE-FIELD GROUND DISPLACEMENTS

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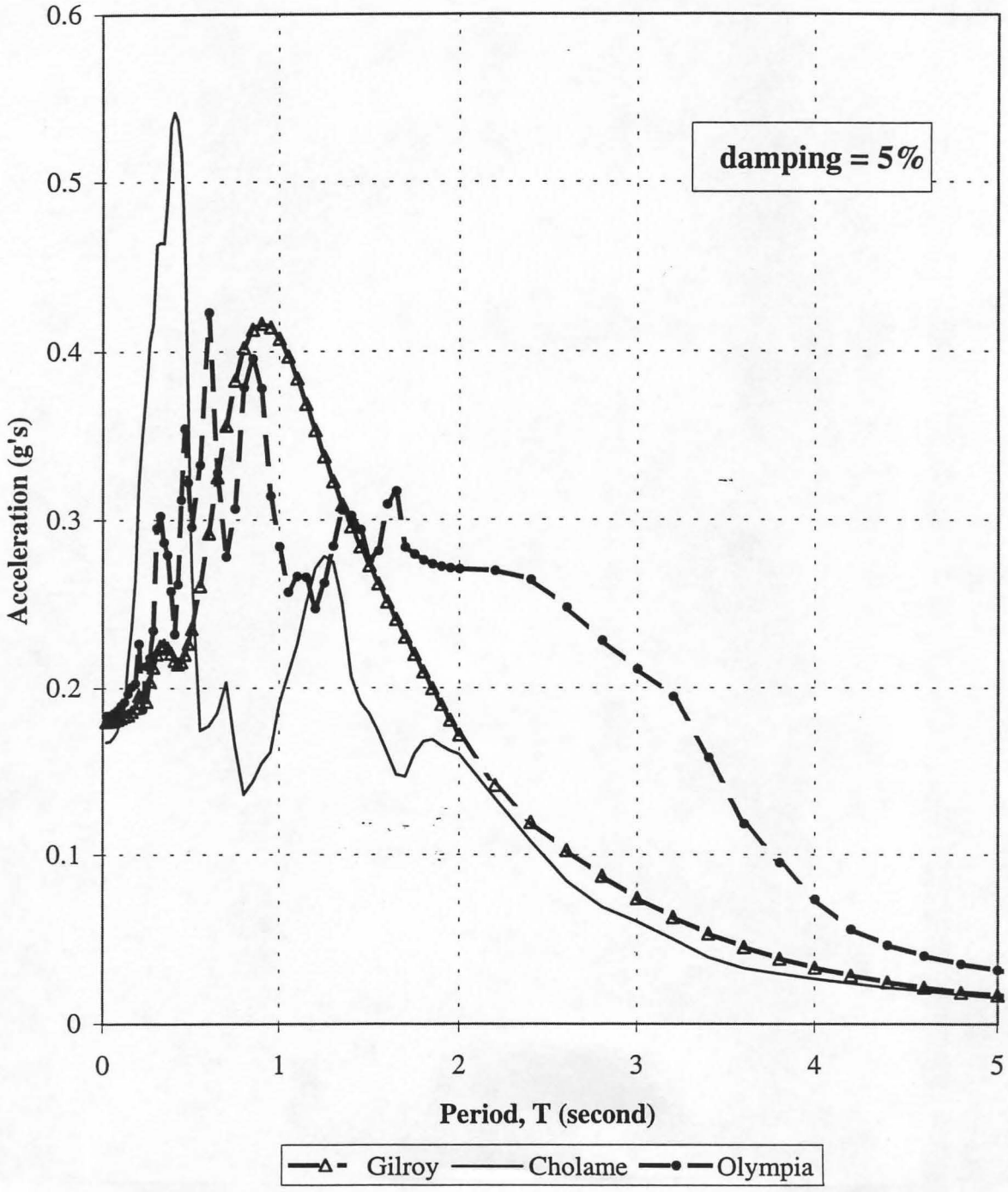
FIG. 6



Note:
The three motions were scaled to 0.3g pga

Improvements to the Wastewater Treatment Plant - Phase One
Aberdeen, Washington

RESPONSE SPECTRA

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FIG. 7

APPENDIX A

CURRENT BORINGS

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major constituents are capitalized (SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not Applicable or Not Available
OD	Outside Diameter
OVA	Organic Vapor Analyzer
PID	Photoionization Detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split Spoon sampler
SPT	Standard Penetration Test
USC	Unified Soil Classification
WLI	Water Level Indicator

GRAIN SIZE DEFINITIONS

DESCRIPTION	SIEVE SIZE
FINES	< #200 (0.8 mm)
SAND*	
• Fine	#200 - #40 (0.4 mm)
• Medium	#40 - #10 (2 mm)
• Coarse	#10 - #4 (5 mm)
GRAVEL*	
• Fine	#4 - $\frac{3}{4}$ inch
• Coarse	$\frac{3}{4}$ - 3 inches
COBBLES	3 - 12 inches
BOULDERS	> 12 inches

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED/COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	<2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Cement/Concrete	Asphalt or PVC Cap
Bentonite Grout	Cobbles
Bentonite Seal	Fill
Slough	Ash
Silica Sand	Bedrock
2" I.D. PVC Screen (0.020-inch Slot)	Gravel

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

SOIL CLASSIFICATION AND LOG KEY

March 2001 21-1-09279-001

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 2

File: I:\Drafting\211\09279-001\21-1-09297-001 Fig A-1.dwg Date: 03-05-2001 Author: CVD

UNIFIED SOIL CLASSIFICATION SYSTEM
(From ASTM D 2488-93 & 2487-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL ②		TYPICAL DESCRIPTION
Coarse-Grained Soils *(more than 50% retained on No. 200 sieve)* *[use Dual Symbols for 5 - 12% Fines (i.e. GP-GM)]* ①	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels ① *(less than 5% fines)*	GW		Well-Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
			GP		Poorly Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
		Gravels with ① Fines *(more than 12% fines)*	GM		Silty Gravels, Gravel-Sand-Silt Mixtures
			GC		Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean sands ① *(less than 5% fines)*	SW		Well-Graded Sands, Gravelly Sands, Little or No Fines
			SP		Poorly Graded Sand, Gravelly Sands, Little or No Fines
		Sands with ① Fines *(more than 12% fines)*	SM		Silty Sands, Sand-Silt Mixtures
			SC		Clayey Sands, Sand-Silt Mixtures
Fine-Grained Soils *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Inorganic Silts of Low to Medium Plasticity, Rock Flour, or Clayey Silts With Slight Plasticity
			CL		Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
		Organic	OL		Organic Silts and Organic Silty Clays of Low Plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	CH		Inorganic Clays of Medium to High Plasticity, Sandy Fat Clay, Gravelly Fat Clay
			MH		Inorganic Silts, Micaceous or Diatomaceous Fine Sands or Silty Soils, Elastic Silt
		Organic	OH		Organic Clays of Medium to High Plasticity, Organic Silts
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor		PT		Peat, Humus, Swamp Soils with High Organic Content (See D 4427-92)

NOTES

1. Dual Symbols (symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols (symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND) indicate that the soil may fall into one of two possible basic groups.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

SOIL CLASSIFICATION AND LOG KEY

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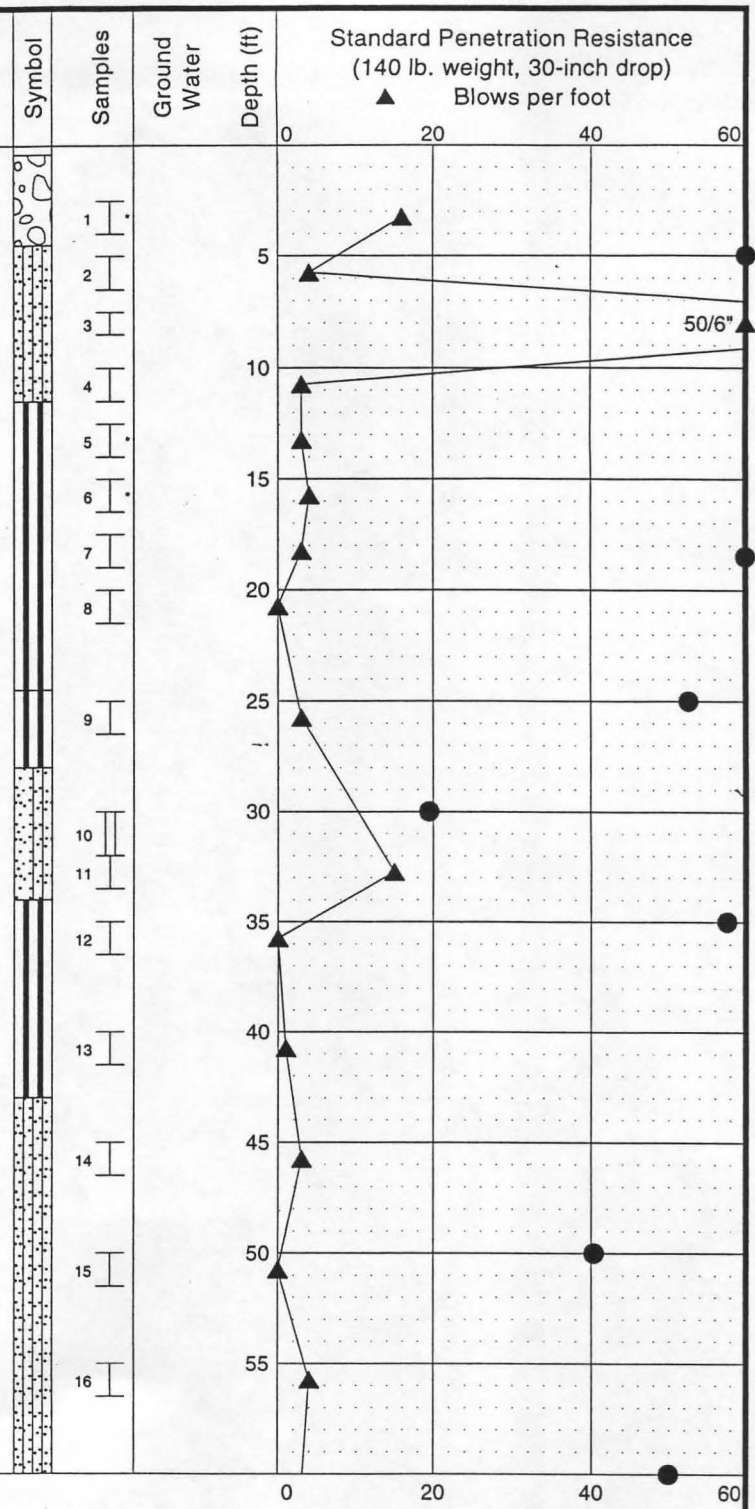
FIG. A-1
Sheet 2 of 2

File: I:\Drafting\211\09279-001\21-1-09297-001 Fig A-1.dwg Date: 03-05-2001 Author: CVD

SOIL DESCRIPTION Surface Elevation: 14.73 Ft.	Depth (ft)
Asphalt.	0.4
Medium dense, dark gray, sandy GRAVEL, trace of silt; wet; (Hf) GP/GP-GM.	4.5
Very loose, dark gray, slightly fine to medium sand, slightly silty to silty fine gravelly coarse SAND; wet; abundant wood debris; (Hf) SM. - split-spoon bouncing on wood from 7.5 to 8.5 feet	11.5
Soft to very soft, gray, slightly fine sandy to fine sandy, slightly clayey SILT; moist; massive, scattered zones of organic silt, abundant organic fragments, abundant fine sand pockets towards bottom; (He) MH.	24.5
Interbedded, soft, gray, slightly clayey SILT, trace of fine sand and very loose, fine (to medium) SAND, trace of silt; moist; bedded, abundant organics in silt, micaceous, scattered shell fragments; (He) MH/SM.	28.0
Medium dense, gray, slightly silty SAND; wet; massive, scattered wood fragments, siltier towards top; (Ha) SP-SM/SP.	34.0
Very soft, green-gray to gray, slightly clayey SILT, trace of fine sand; moist; massive to faintly bedded; scattered to abundant organic fragments (in growth position towards bottom); scattered shell fragments; micaceous; (He) MH. - possible dark brown organic layer from 39 to 40 feet, based on slough in S-13	43.0
Very loose to medium dense, gray, fine sandy SILT and silty fine SAND; moist to wet; massive to bedded, scattered to abundant organics, scattered shell fragments, micaceous; (He) SM/ML. - bioturbated/intermixed with slightly fine sandy silt from 43 to 63 feet	

CONTINUED NEXT PAGE



LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- ∇ Ground Water Level ATD

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-1

February 2001 | 21-1-09279-001

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 1 of 3

DRAFT

MASTER LOG 21-09279.GPJ SHAN_WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: MGI

SOIL DESCRIPTION Surface Elevation: 14.73 Ft.	Depth (ft)	Symbol	Samples	Ground Water	Depth (ft)
- laminated to bedded with abundant silt seams and brown organic-rich seams from 65 to 70.5 feet			17, 18		65
	70.5		19		70
Medium stiff, gray, slightly clayey, fine sandy SILT; moist; massive to bioturbated, abundant shell fragments and organic fragments, micaceous; (He) ML/SM.	73.0				
Interbedded, medium dense, gray, trace of silt to slightly silty fine SAND and very stiff SILT, trace of clay and fine sand; wet; abundant organics, micaceous; (He) SM/SP/ML. - tan, clayey silt seams at 76 and 76.2 feet - scattered dark brown organic-rich seams betwee 80 and 80.5 feet			20, 21		75, 80
	85.5		22		85
Medium dense to soft, gray, slightly silty fine SAND grading to slightly clayey, fine sandy SILT; moist; bedded, abundant silt seams, abundant organics, scattered shell fragments; (He) MH/SM.			23		90
	93.0				
Medium stiff and stiff, gray, slightly clayey SILT, trace of fine sand; moist; massive, faintly bedded, abundant organic fragments, scattered zones of organic silt; (He) MH/OH.			24, 25		95, 100
			26		105
			27		110
			28		115

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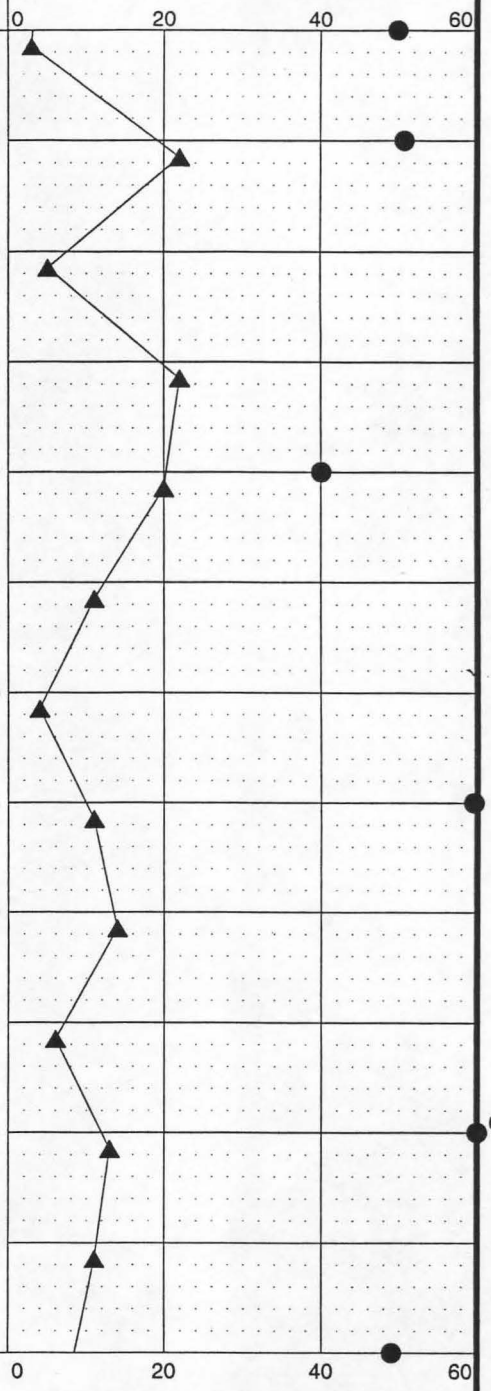

LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- ▽ Ground Water Level ATD

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-1

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FIG. A-2
Sheet 2 of 3

DRAFT

MASTER_LOG 21-09279.GPJ SHAN_WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: MGI

SOIL DESCRIPTION Surface Elevation: 14.73 Ft.	Depth (ft)	Symbol	Samples	Ground Water	Depth (ft)	Standard Penetration Resistance (140 lb. weight, 30-inch drop) ▲ Blows per foot
			29			
Stiff to medium stiff, green-gray, slightly clayey SILT; moist; massive, scattered blocky texture, scattered organics towards bottom; (He) ML/MH. - silty fine sand seam from 130 to 130.2 feet - dark brown mottling at 131.4 feet	124.5		30 31 32		125 130	
Very dense, gray, silty, sandy GRAVEL; wet; massive, angular to rounded clasts, scattered andesite grains, locally clayey (weathered?); (Ha) GM/GW.	132.5		33 34 35		135 140 145	50/6" 50/6" 50/4"
BOTTOM OF BORING COMPLETED 2/6/2001	150.8		36		150 155 160 165 170 175	51/4"

0 20 40 60

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- ▽ Ground Water Level ATD

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-1

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SHANNON & WILSON, INC.
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FIG. A-2
Sheet 3 of 3

DRAFT

MASTER_LOG 21-09279.GPJ SHAN_WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: MGI

SOIL DESCRIPTION Surface Elevation: 16.08 Ft.	Depth (ft)
Crushed Rock (2.0" minus); (Hf) GP.	
Medium dense to very dense, gray-black to gray, silty SAND and slightly silty, gravelly SAND; moist to wet; scattered debris (asphalt, wood, concrete, etc.); (Hf) SM. - concrete between 9.5 and 12 feet, inferred from drill action and limited sample recovery	3.5
Stiff to medium stiff, olive-gray to gray, slightly clayey SILT, trace of fine sand to slightly fine sandy, slightly clayey SILT; moist to wet; massive to faintly bedded, abundant wood pieces, micaceous, scattered fine sandy silt pockets towards bottom; (He) MH. - wood debris from 14 to 19 feet - increasing fine sand from 30.0 to 35.5 feet	12.0
Medium stiff, dark brown, organic SILT, trace of fine sand; moist; massive, abundant wood fragments; (Hp) OL.	35.5 36.5
Medium stiff, gray, slightly clayey SILT, trace of fine sand; moist to wet; abundant organic fragments, micaceous; (He) MH. - brown organic pocket with fine sandy silt at 41.2 feet	43.0
Very loose and medium dense, silty fine SAND and fine sandy SILT; trace of clay, moist to wet; bioturbated, abundant silt seams and pockets, scattered organic fragments, scattered shells, micaceous; (He) SM/ML.	

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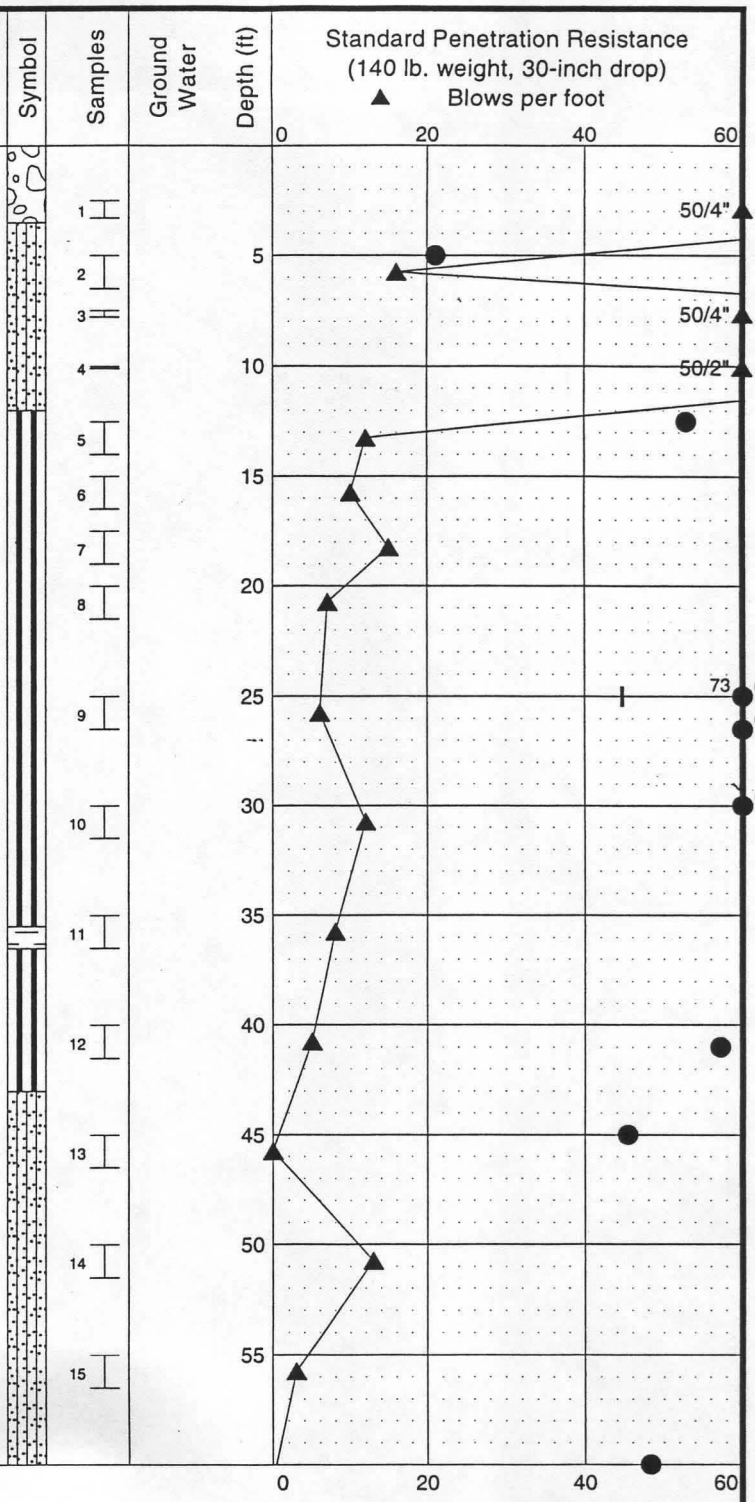

LEGEND

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

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-2

February 2001 | 21-1-09279-001

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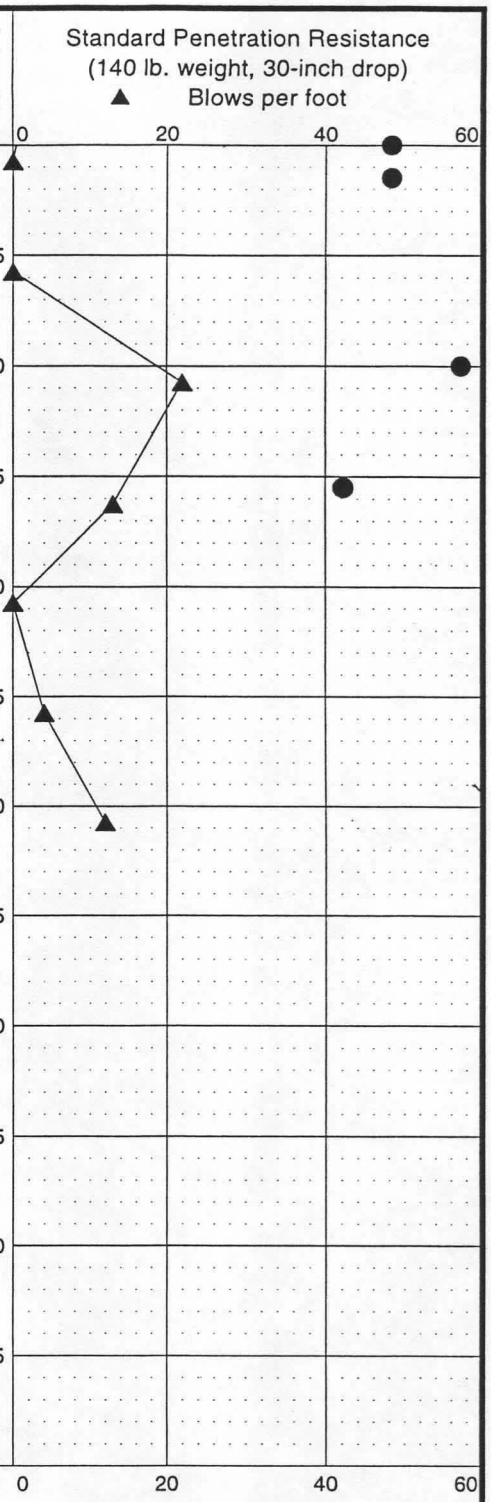
FIG. A-3
Sheet 1 of 2

DRAFT

MASTER LOG 21-09279.GPJ SHAN WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: LKD

SOIL DESCRIPTION Surface Elevation: 16.08 Ft.	Depth (ft)	Symbol	Samples	Ground Water	Depth (ft)
- becomes siltier at 65 feet			16, 17		65
Very stiff, gray, fine sandy SILT, trace of clay; moist; bedded, abundant brown organic seams, scattered light gray seams, scattered shells, increasing silt laminations towards bottom; (He) ML/SM.	68.0		18, 19		70, 75
Very soft to stiff, gray, fine sandy, slightly clayey SILT to slightly fine sandy SILT; moist; faintly bedded, scattered seams of silty fine sand, scattered to abundant organic fragments, micaceous; (He) MH.	78.5		20, 21, 22		80, 85, 90
BOTTOM OF BORING COMPLETED 1/31/2001	91.5				95, 100, 105, 110, 115



LEGEND

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

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

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
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Aberdeen, Washington

LOG OF BORING CB-2

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FIG. A-3
Sheet 2 of 2

DRAFT

MASTER LOG 21-09279.GPJ SHAN WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: LKD

SOIL DESCRIPTION

Surface Elevation: 15.62 Ft.

Medium dense to loose, gray and dark brown, silty gravelly SAND; moist; abundant wood/bark debris, scattered zones of bark and wood; (Hf) SM.

10.1

Very soft and medium stiff, dark gray, slightly clayey SILT, trace of fine sand to slightly fine sandy, slightly clayey SILT; moist to wet; massive to faintly bedded with scattered bioturbation, abundant organic fragments, scattered shell fragments, micaceous; (He) MH.

- scattered <1/2-inch fine sand seams between 12.8 and 13.2 feet
- abundant dark brown organic-rich seams between 17 and 28 feet
- gradational contact to silty fine sand at 28 feet

28.0

Loose, gray, silty fine to medium SAND; moist to wet; faintly bedded, abundant silt seams towards top, scattered silt pockets and seams towards bottom, abundant organic fragments, micaceous; (He) SM.

33.0

Very soft, gray, slightly clayey SILT, trace of fine sand; moist to wet; massive, abundant organic fragments, micaceous; (He) MH.

45.0

Very soft, gray, fine sandy SILT and very loose to medium dense, silty fine SAND; wet; interbedded to highly bioturbated, abundant organic fragments, abundant shells, scattered silt seams and pockets, micaceous; (He) ML/SM.

- scattered dark brown organic-rich seams at 50 and 65 feet

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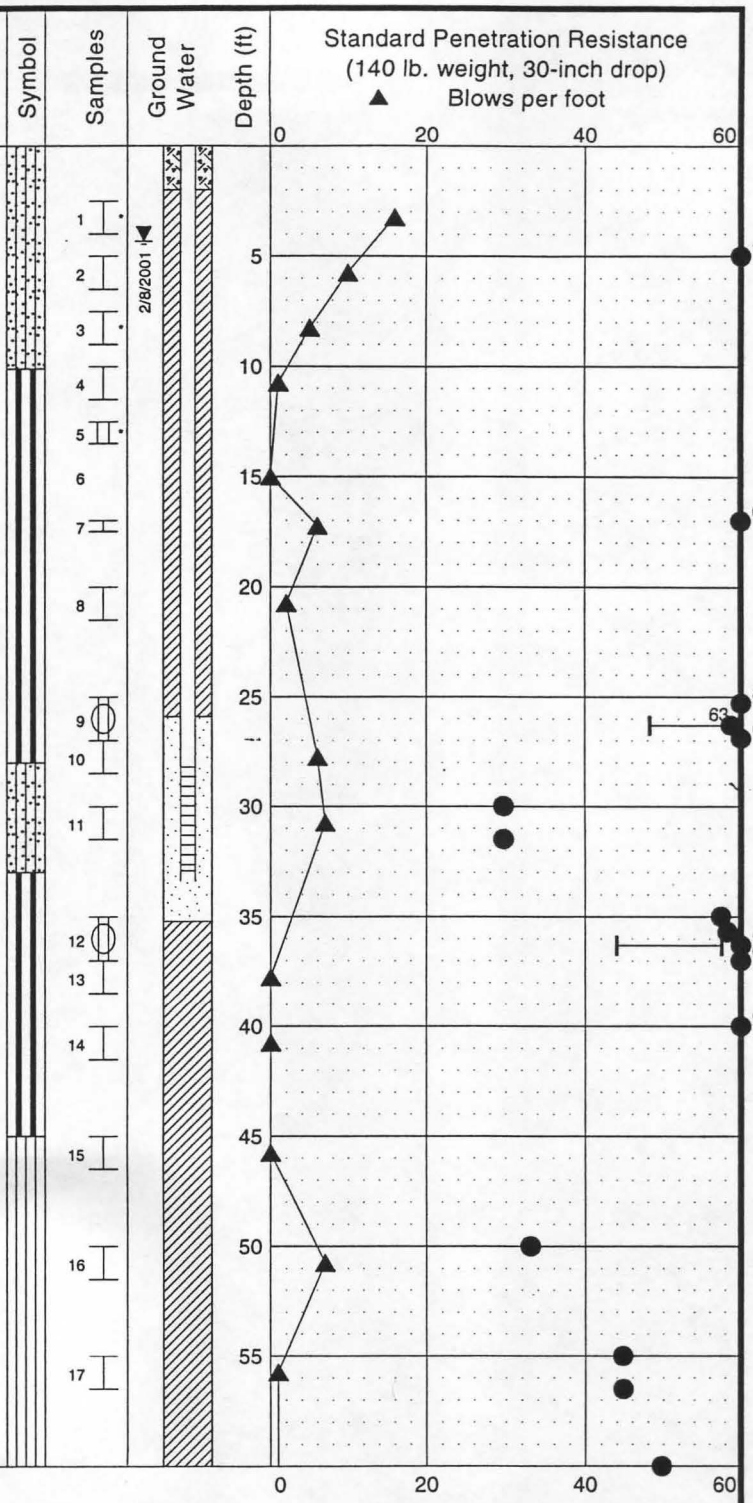

LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Ground Water Level ATD
- Ground Water Level in Well

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-3

February 2001 | 21-1-09279-001

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Geotechnical and Environmental Consultants

FIG. A-4
Sheet 1 of 3

DRAFT

MASTER LOG 21-09279.GPJ SHAN WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: LKD

SOIL DESCRIPTION Surface Elevation: 15.62 Ft.	Depth (ft)
- 4-inch layer of silt, trace of fine sand between 70.3 and 70.6 feet	
Soft to stiff, gray, trace of fine sand to slightly fine sandy, slightly clayey SILT; moist; massive, abundant organic fragments, scattered fine sand seams and pockets, micaceous; (He) MH.	75.9
NOTE: Osterberg sampler was lost in hole at 90 feet, boring abandoned and re-drilled 5 feet north to 90 feet. - possible sand layer between 91.5 and 97 feet, inferred from cuttings and slough on sample S-26.	
Hard, gray, trace of fine sand to fine sandy, slightly clayey, SILT; moist; massive, abundant organic fragments, scattered fine sand pockets towards bottom; (He) MH.	113.0

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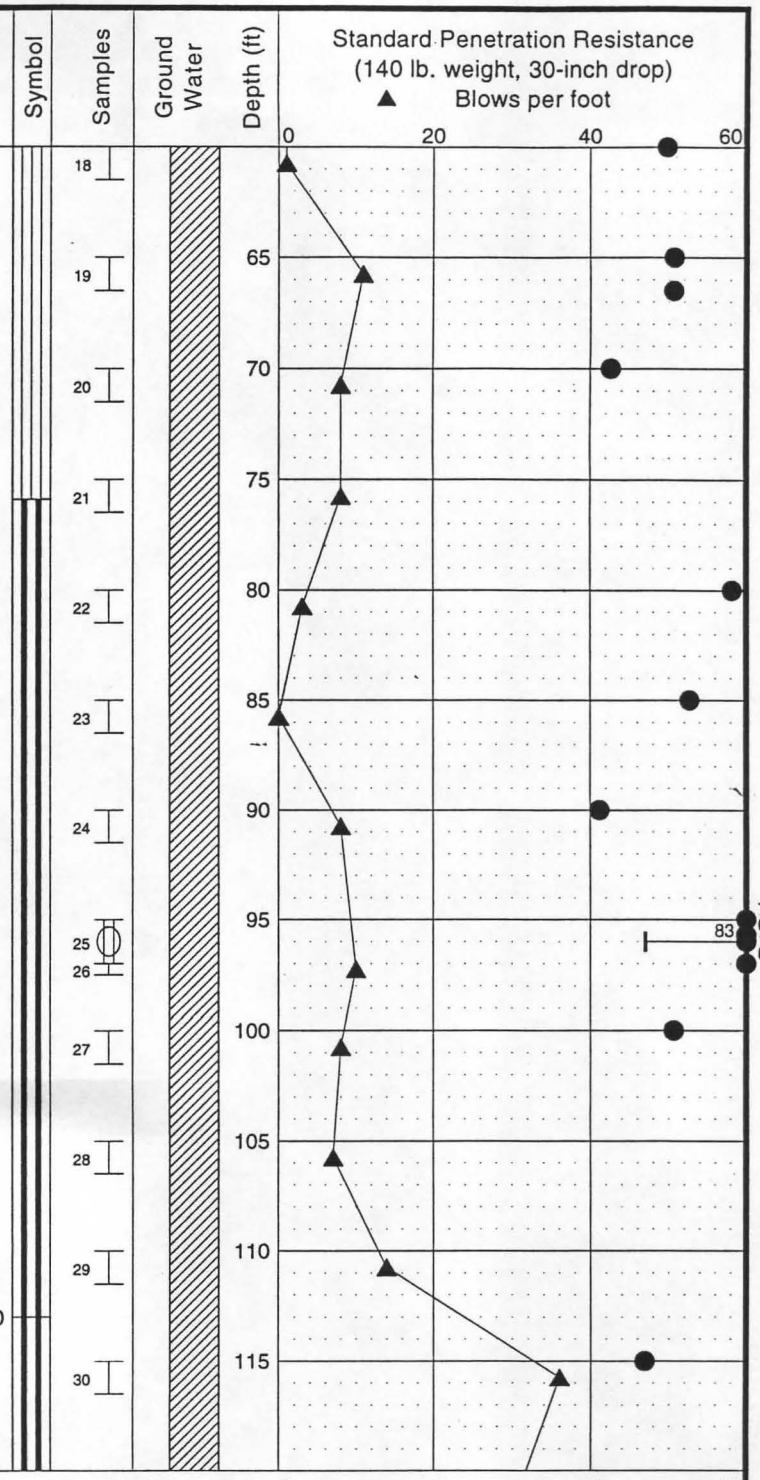

LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Ground Water Level ATD
- Ground Water Level in Well

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-3

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SHANNON & WILSON, INC.
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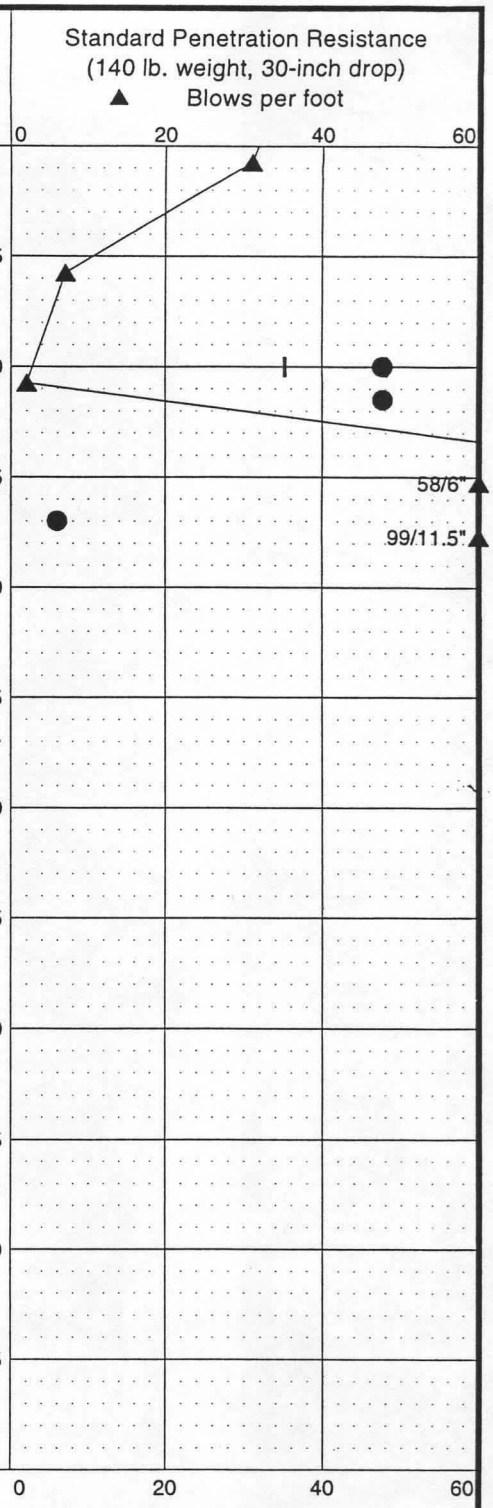
FIG. A-4
Sheet 2 of 3

DRAFT

MASTER LOG 21-09279.GPJ SHAN WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: LKD

SOIL DESCRIPTION Surface Elevation: 15.62 Ft.	Depth (ft)	Symbol	Samples	Ground Water	Depth (ft)
Dense, gray, silty fine SAND; moist; massive, scattered silt pockets, abundant organic fragments; (He) SM.	121.0		31		
Medium stiff to very soft, gray to gray-green, slightly clayey SILT; moist; massive, faint blocky texture, scattered shell fragments, abundant organic fragments; (He) ML/MH. - scattered fine sand at 130 feet	123.0		32 33 34		
Very dense, green-gray to gray, slightly silty to silty fine gravelly SAND; moist to wet; massive, weathered at top; (Ha) SM/SP-SM.	133.0		35 36		
BOTTOM OF BORING COMPLETED 2/2/2001	138.4				



● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

LEGEND

- * Sample Not Recovered
- 2-inch O.D. Split Spoon Sample
- 3-inch O.D. Shelby Tube Sample
- 3.0" O.D. Osterberg Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Ground Water Level ATD
- Ground Water Level in Well

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 the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of "Symbols" and definitions.
5. USCS designation is based on visual-manual classification and selected laboratory index testing.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

LOG OF BORING CB-3

February 2001 | 21-1-09279-001

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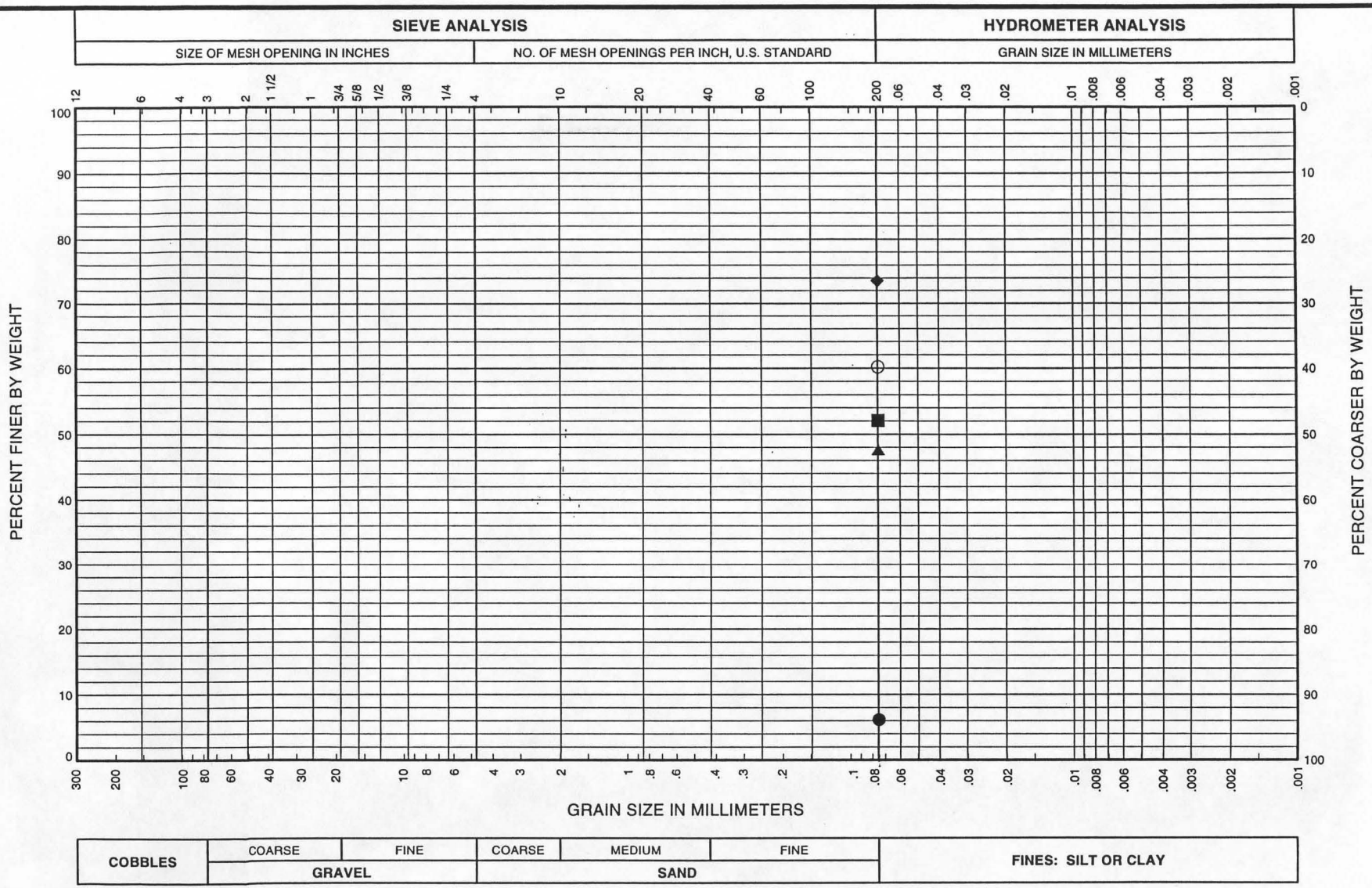
FIG. A-4
Sheet 3 of 3

DRAFT

MASTER LOG 21-09279.GPJ SHAN WIL.GDT 3/9/01 Log: WDN Rev: RM Typ: LKD

APPENDIX B

GEOTECHNICAL LABORATORY AND HYDROGEOLOGIC TESTING



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● CB-1, S-10	30.0	SP-SM	Gray, slightly silty SAND	6.2	19.5			
■ CB-1, S-15	50.0	ML	Gray-black, clayey, sandy SILT	52.1	40.3			
▲ CB-1, S-18	65.0	SM	Gray, clayey, silty SAND	47.6	50.7			
◆ CB-2, S-16	60.0	ML	Gray, clayey, sandy SILT	73.5	48.3			
○ CB-2, S-19	75.5	ML	Gray, slightly clayey, sandy SILT	60.2	42.2			

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

GRAIN SIZE DISTRIBUTION

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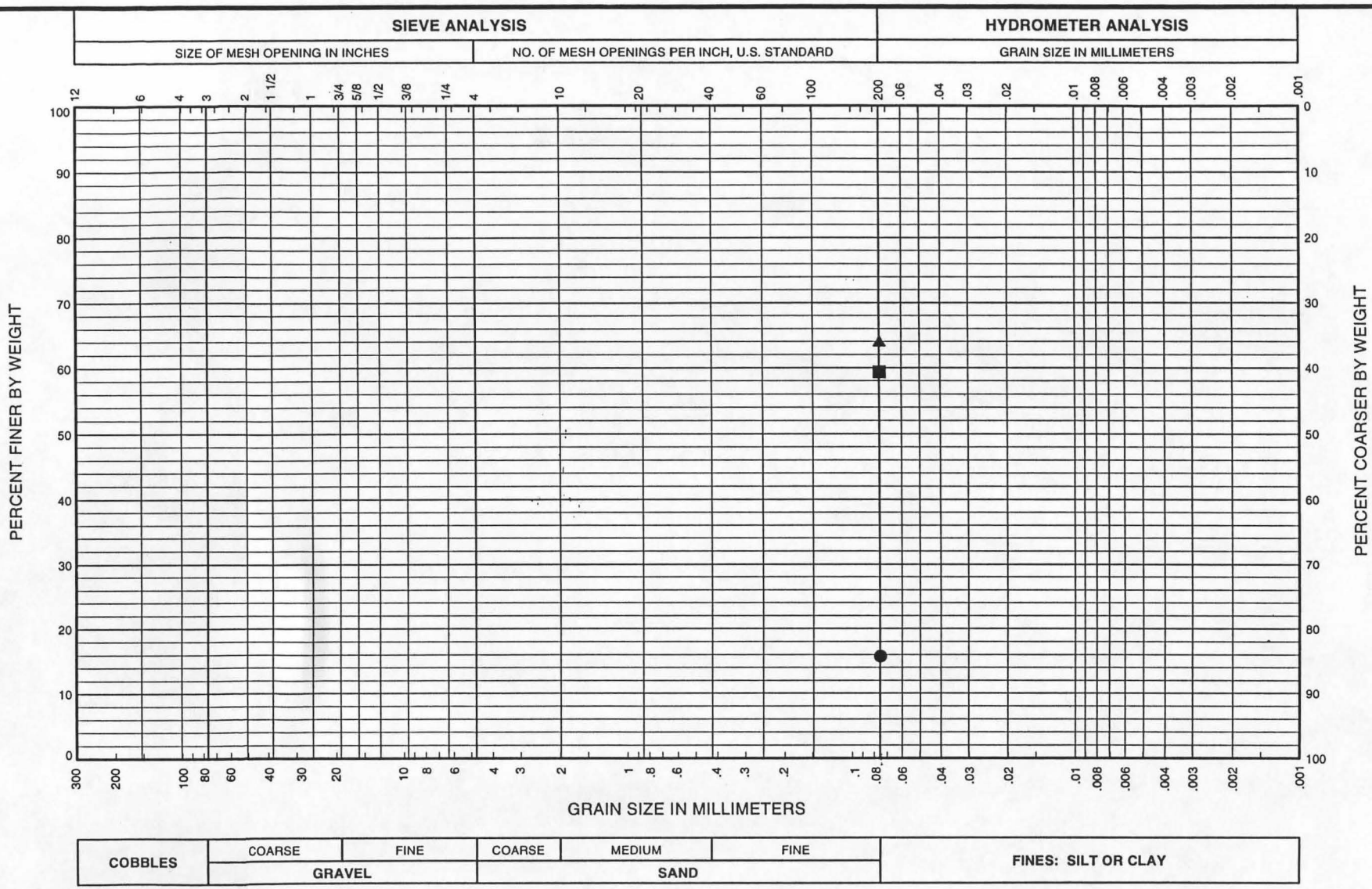
FIG. B-1a

GSAMAIN1 21-09279.GPJ SHAN WIL.GDT 3/9/01

FIG. B-1a



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● CB-3, S-11	30.0	SM	Gray, slightly clayey, slightly silty SAND, scattered organics	15.8	29.7			
■ CB-3, S-17	55.0	CL	Gray, sandy, silty CLAY	59.4	44.8			
▲ CB-3, S-19	65.0	ML	Gray, sandy, clayey SILT	64.1	50.6			

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

GRAIN SIZE DISTRIBUTION

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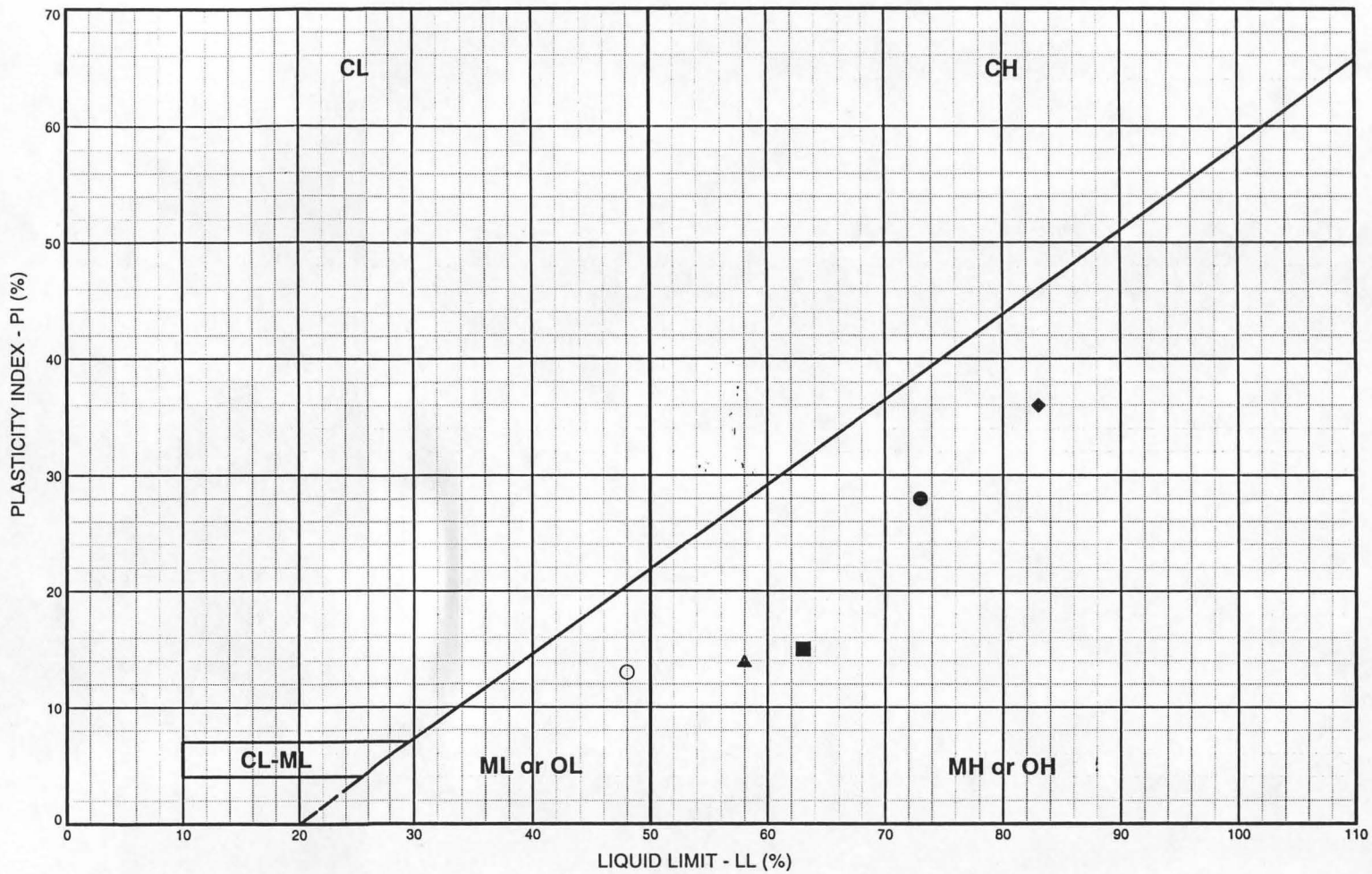
FIG. B-1b

FIG. B-1b

GSAMAIN1 21-09279.GPJ SHAN WIL.GDT 3/9/01



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● CB-2, S-9	25.0	MH	Gray, clayey SILT	73	45	28	68.0	
■ CB-3, S-9	26.3	MH	Gray, slightly clayey SILT; abundant organics	63	48	15	58.7	
▲ CB-3, S-12	36.3	MH	Gray, slightly sandy, clayey SILT; abundant organics	58	44	14	65.5	
◆ CB-3, S-25	96.0	MH	Gray, slightly clayey SILT;abundant organics	83	47	36	70.1	
○ CB-3, S-33	130.0	ML	Gray, clayey SILT; scattered shell fragments and organics, trace of vivianite	48	35	13	47.6	

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

PLASTICITY CHART

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FIG. B-2

FIG. B-2

ATTMAIN1 21-09279.GPJ SHAN WIL.GDT 3/9/01

UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 2

SUMMARY OF TEST DATA

Boring CB-3
Sample S-12
Depth, ft. 36.3-36.8

Tested By / Date WLB 2/16/01
Calc. By / Date WLB 2/20/01
Check By / Date _____

CLASSIFICATION:
Medium Stiff, gray, slightly clayey SILT, trace of sand; abundant organics; scattered sand seams; (MH/OH)
SAMPLE DATA:
Spec. Grav. (est.) : 2.74
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test
Height, inches :	6.15
Diameter, inches :	2.87
Height/Dia. Ratio :	2.14
Weight, grams :	1046.3
Water Content, % :	65.5
Wet Density, pcf :	100.1
Dry Density, pcf :	60.5
Saturation, % :	98

TEST DATA:

Defl. Constant, in./div. :	.001
Load Constant, kg/div. :	1.973
Conf. Press., tsf :	1.92

Axial Defl. Read. div	Axial Load Read. div	Axial Strain %	Corr. Area sq. in.	Axial Load lbs	Principal Stress Diff tsf	p tsf	q tsf
5.0	1.1	.1	6.47	4.8	.05	1.95	.03
10.0	1.9	.2	6.48	8.3	.09	1.97	.05
15.0	2.1	.2	6.49	9.1	.10	1.97	.05
20.0	2.3	.3	6.49	10.0	.11	1.98	.06
30.0	3.0	.5	6.50	13.1	.14	1.99	.07
40.0	3.8	.7	6.51	16.5	.18	2.01	.09
50.0	4.4	.8	6.52	19.1	.21	2.03	.11
60.0	5.0	1.0	6.53	21.8	.24	2.04	.12
70.0	5.6	1.1	6.54	24.4	.27	2.06	.13
80.0	6.7	1.3	6.55	29.1	.32	2.08	.16
100.0	7.1	1.6	6.58	30.9	.34	2.09	.17
125.0	8.1	2.0	6.60	35.2	.38	2.11	.19
150.0	9.2	2.4	6.63	40.0	.43	2.14	.22
175.0	9.9	2.8	6.66	43.1	.47	2.16	.23
200.0	10.5	3.3	6.69	45.7	.49	2.17	.25
250.0	11.3	4.1	6.74	49.2	.52	2.18	.26
300.0	11.7	4.9	6.80	50.9	.54	2.19	.27
350.0	12.0	5.7	6.86	52.2	.55	2.20	.27
400.0	12.3	6.5	6.92	53.5	.56	2.20	.28
450.0	12.6	7.3	6.98	54.8	.57	2.21	.28
500.0	12.9	8.1	7.04	56.1	.57	2.21	.29
550.0	13.1	8.9	7.10	57.0	.58	2.21	.29
600.0	13.3	9.8	7.17	57.9	.58	2.21	.29
700.0	13.8	11.4	7.30	60.0	.59	2.22	.30
800.0	14.1	13.0	7.44	61.3	.59	2.22	.30
900.0	14.3	14.6	7.58	62.2	.59	2.22	.30
950.0	14.7	15.4	7.65	63.9	.60	2.22	.30

FAILURE SKETCH:

Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

UU TRIAXIAL TEST

February 2001 21-1-09279-001

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FIG. B-3a

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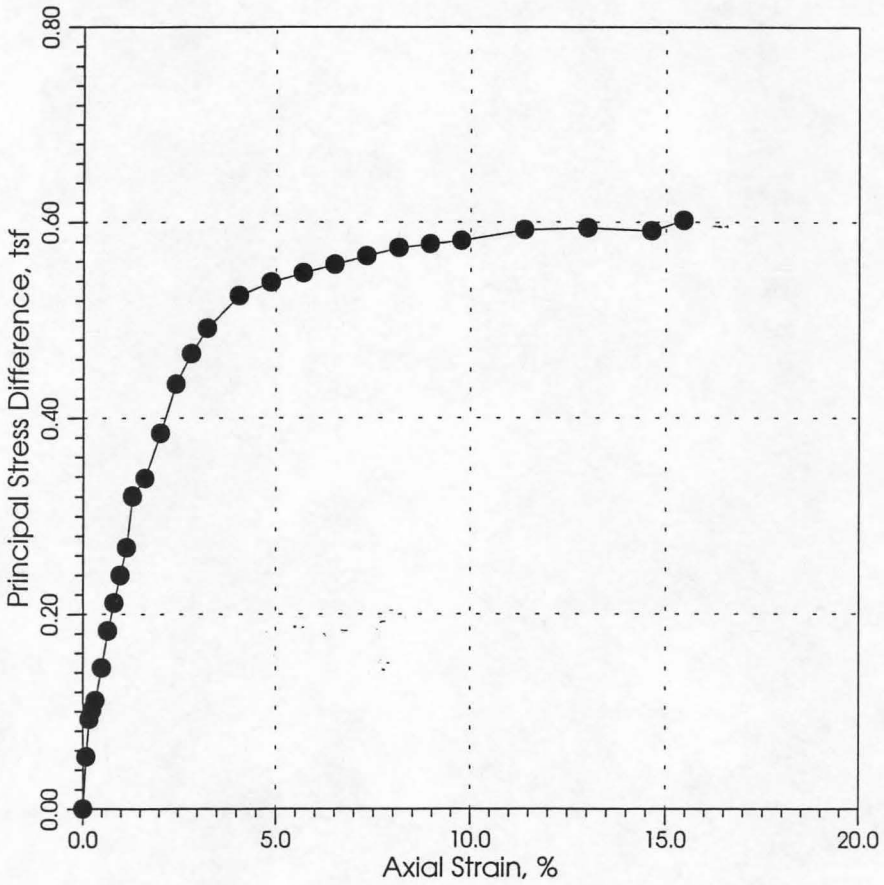
UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 2
STRESS VS STRAIN

Boring	CB-3	Tested By / Date	WLB 2/16/01
Sample	S-12	Calc. By / Date	WLB 2/20/01
Depth, ft.	36.3-36.8	Check By / Date	_____

Confining Pressure, tsf = 1.92



Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

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February 2001 | 21-1-09279-001

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FIG. B-3b

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UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 3

SUMMARY OF TEST DATA

Boring CB-3
Sample S-25
Depth, ft. 96-96.5

Tested By / Date WLB 2/16/01
Calc. By / Date WLB 2/20/01
Check By / Date _____

CLASSIFICATION:
Medium stiff to stiff, dark brown, slightly clayey SILT, trace of fine sand; abundant organics; (OH/MH)
SAMPLE DATA:
Spec. Grav. (est.) : 2.74
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test
Height, inches :	6.08
Diameter, inches :	2.87
Height/Dia. Ratio :	2.12
Weight, grams :	988.5
Water Content, % :	70.1
Wet Density, pcf :	95.7
Dry Density, pcf :	56.3
Saturation, % :	94

TEST DATA:

Defl. Constant, in./div. :	.001
Load Constant, kg/div. :	1.973
Conf. Press., tsf :	5.31

Axial Defl. Read. div	Axial Load Read. div	Axial Strain %	Corr. Area sq. in.	Axial Load lbs	Principal Stress Diff tsf	p tsf	q tsf
5.0	2.8	.1	6.47	12.2	.14	5.38	.07
10.0	5.1	.2	6.48	22.2	.25	5.44	.12
15.0	7.0	.2	6.49	30.5	.34	5.48	.17
20.0	8.3	.3	6.49	36.1	.40	5.51	.20
30.0	10.5	.5	6.50	45.7	.51	5.57	.25
40.0	12.5	.7	6.51	54.4	.60	5.61	.30
50.0	14.4	.8	6.52	62.6	.69	5.66	.35
60.0	17.0	1.0	6.53	74.0	.81	5.72	.41
70.0	18.5	1.2	6.54	80.5	.89	5.76	.44
80.0	20.0	1.3	6.56	87.0	.96	5.79	.48
90.0	21.5	1.5	6.57	93.5	1.03	5.83	.51
100.0	23.5	1.6	6.58	102.2	1.12	5.87	.56
125.0	27.8	2.1	6.61	120.9	1.32	5.97	.66
150.0	30.5	2.5	6.63	132.7	1.44	6.03	.72
175.0	33.1	2.9	6.66	144.0	1.56	6.09	.78
200.0	35.1	3.3	6.69	152.7	1.64	6.14	.82
250.0	37.4	4.1	6.75	162.7	1.74	6.18	.87
300.0	38.6	4.9	6.81	167.9	1.78	6.20	.89
350.0	38.9	5.8	6.86	169.2	1.77	6.20	.89
400.0	38.9	6.6	6.92	169.2	1.76	6.19	.88
450.0	38.8	7.4	6.99	168.8	1.74	6.18	.87
500.0	38.3	8.2	7.05	166.6	1.70	6.16	.85
550.0	37.8	9.0	7.11	164.4	1.66	6.15	.83
600.0	37.2	9.9	7.18	161.8	1.62	6.13	.81
650.0	36.9	10.7	7.24	160.5	1.60	6.11	.80
700.0	36.7	11.5	7.31	159.6	1.57	6.10	.79
800.0	36.3	13.2	7.45	157.9	1.53	6.08	.76
900.0	36.0	14.8	7.59	156.6	1.48	6.06	.74
980.0	35.7	16.1	7.71	155.3	1.45	6.04	.72

FAILURE SKETCH:

Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

UU TRIAXIAL TEST

February 2001 21-1-09279-001

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FIG. B-4a

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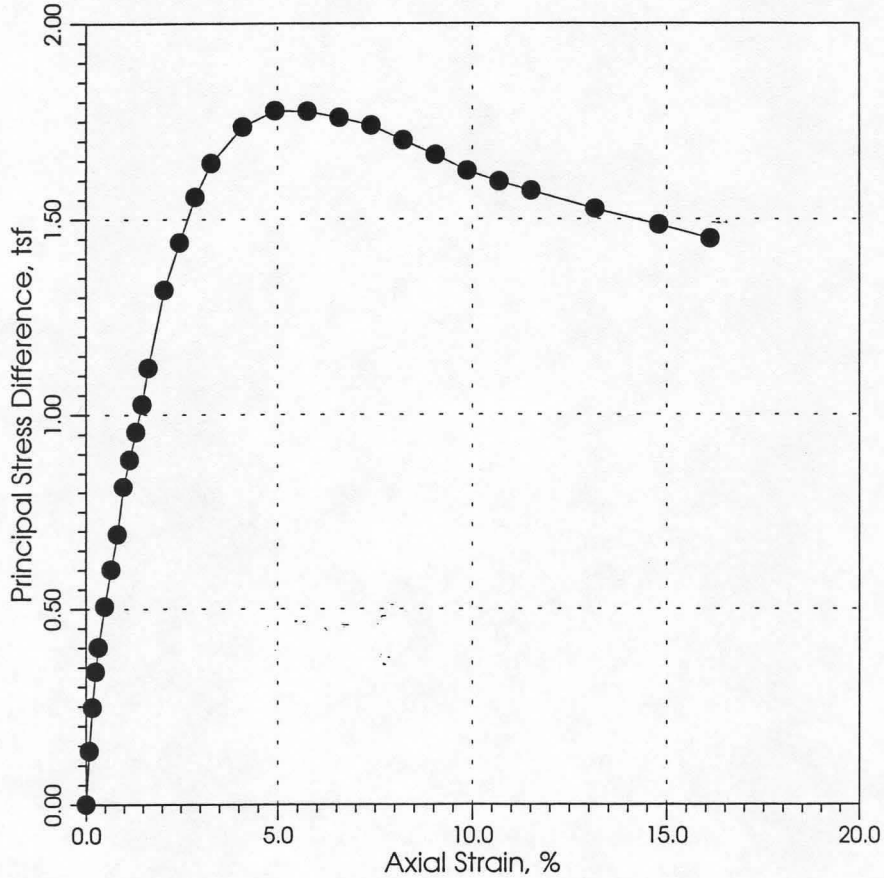
UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 3
STRESS VS STRAIN

Boring	CB-3	Tested By / Date	WLB 2/16/01
Sample	S-25	Calc. By / Date	WLB 2/20/01
Depth, ft.	96-96.5	Check By / Date	_____

Confining Pressure, tsf = 5.31



Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

UU TRIAXIAL TEST

February 2001 21-1-09279-001

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FIG. B-4b

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UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 1

SUMMARY OF TEST DATA

Boring	CB-3	Tested By / Date	SAM 2/16/01
Sample	S-9	Calc. By / Date	WLB 2/20/01
Depth, ft.	26.3-26.9	Check By / Date	_____

CLASSIFICATION:
Soft, gray-brown, slightly clayey SILT, trace of fine sand; abundant organics; scattered sand seams; (MH/OH)

SAMPLE DATA:
Spec. Grav. (est.) : 2.74
Specimen : UNDISTURBED

SPECIMEN DATA:	Before Test
Height, inches :	5.92
Diameter, inches :	2.87
Height/Dia. Ratio :	2.06
Weight, grams :	1023.7
Water Content, % :	58.7
Wet Density, pcf :	101.8
Dry Density, pcf :	64.1
Saturation, % :	97
TEST DATA:	
Defl. Constant, in./div. :	.001
Load Constant, kg/div. :	1.973
Conf. Press., tsf :	1.87

Axial Defl. Read. div	Axial Load Read. div	Axial Strain %	Corr. Area sq. in.	Axial Load lbs	Principal Stress Diff tsf	p tsf	q tsf
5.0	.3	.1	6.47	1.3	.01	1.88	.01
10.0	.6	.2	6.48	2.6	.03	1.89	.01
25.0	1.5	.4	6.50	6.5	.07	1.91	.04
30.0	2.0	.5	6.50	8.7	.10	1.92	.05
35.0	2.5	.6	6.51	10.9	.12	1.93	.06
40.0	2.7	.7	6.51	11.7	.13	1.94	.06
50.0	3.0	.8	6.52	13.1	.14	1.94	.07
55.0	3.5	.9	6.53	15.2	.17	1.96	.08
60.0	4.0	1.0	6.54	17.4	.19	1.97	.10
70.0	4.5	1.2	6.55	19.6	.22	1.98	.11
85.0	5.0	1.4	6.56	21.8	.24	1.99	.12
95.0	5.2	1.6	6.57	22.6	.25	2.00	.12
105.0	6.0	1.8	6.59	26.1	.29	2.01	.14
115.0	7.0	1.9	6.60	30.5	.33	2.04	.17
130.0	7.5	2.2	6.61	32.6	.36	2.05	.18
150.0	8.0	2.5	6.64	34.8	.38	2.06	.19
170.0	8.5	2.9	6.66	37.0	.40	2.07	.20
190.0	9.0	3.2	6.68	39.2	.42	2.08	.21
210.0	9.5	3.5	6.71	41.3	.44	2.09	.22
230.0	10.0	3.9	6.73	43.5	.47	2.10	.23
250.0	10.5	4.2	6.75	45.7	.49	2.12	.24
270.0	11.0	4.6	6.78	47.9	.51	2.13	.25
315.0	11.5	5.3	6.83	50.0	.53	2.14	.26
410.0	12.5	6.9	6.95	54.4	.56	2.15	.28
490.0	13.5	8.3	7.05	58.7	.60	2.17	.30
570.0	14.0	9.6	7.16	60.9	.61	2.18	.31
690.0	15.0	11.7	7.32	65.3	.64	2.19	.32
820.0	15.5	13.9	7.51	67.4	.65	2.20	.32

FAILURE SKETCH:

Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

UU TRIAXIAL TEST

February 2001 21-1-09279-001

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FIG. B-5a

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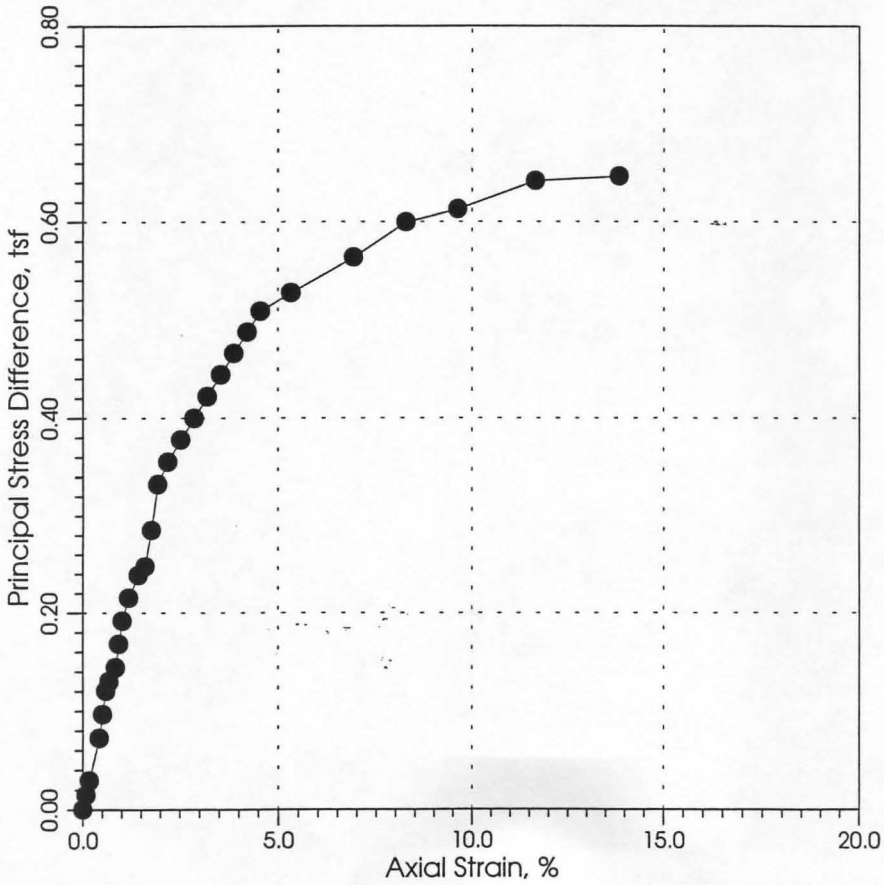
UNCONSOLIDATED, UNDRAINED TRIAXIAL COMPRESSION TEST NO. 1
STRESS VS STRAIN

Boring	CB-3	Tested By / Date	SAM 2/16/01
Sample	S-9	Calc. By / Date	WLB 2/20/01
Depth, ft.	26.3-26.9	Check By / Date	_____

Confining Pressure, tsf = 1.87



Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

UU TRIAXIAL TEST

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FIG. B-5b

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ONE DIMENSIONAL CONSOLIDATION TEST NO. 1

SUMMARY OF TEST DATA

Boring	CB-3	Tested By / Date	WLB 2/13/01
Sample	S-25	Calc. By / Date	WLB 2/20/01
Depth, ft.	95.1	Check By / Date	_____

CLASSIFICATION:
Medium stiff, dark brown, slightly clayey SILT, trace of fine sand; abundant organics; (MH/OH)

SAMPLE DATA:
Spec. Grav. (est.) : 2.74
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	.788	.680
Diameter, inches :	2.501	2.501
Wet Density, pcf :	94.4	104.5
Dry Density, pcf :	55.2	64.0
Water Content, % :	71.0	63.4
Void Ratio :	2.096	1.673
Saturation, % :	93	104

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
.3	.5	.3	.10	.0	2.096				
1.5	14.8	2.1	.50	.6	2.077	.2	12.5	1.63E-02	2.54E-07
3.1	34.9	3.9	1.00	1.5	2.048	.3	31.4	1.07E-02	1.95E-07
4.7	49.5	5.1	1.52	2.2	2.027	.2	46.9	1.58E-02	2.04E-07
7.0	70.5	6.8	2.26	3.2	1.998	.5	65.9	6.19E-03	8.04E-08
10.5	108.2	8.2	3.39	5.0	1.941	.4	100.7	7.48E-03	1.20E-07
15.5	177.7	10.3	5.01	8.4	1.837	2.3	158.7	1.23E-03	2.55E-08
23.2	298.6	12.7	7.50	14.3	1.654	.3	289.1	8.14E-03	1.94E-07
23.2	364.5	12.7	7.50	17.6	1.552				
15.5	359.8	11.3	5.01	17.4	1.557	.2	360.8	1.12E-02	
7.0	347.9	8.8	2.26	16.9	1.572	.1	351.0	2.26E-02	
3.1	321.6	7.0	1.00	15.7	1.610	1.3	328.6	1.78E-03	
.3	277.6	3.8	.10	13.7	1.673	1.4	293.5	1.72E-03	

Improvements to the Waste Water Treatment Plant, Phase One
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CONSOLIDATION TEST

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FIG. B-6a

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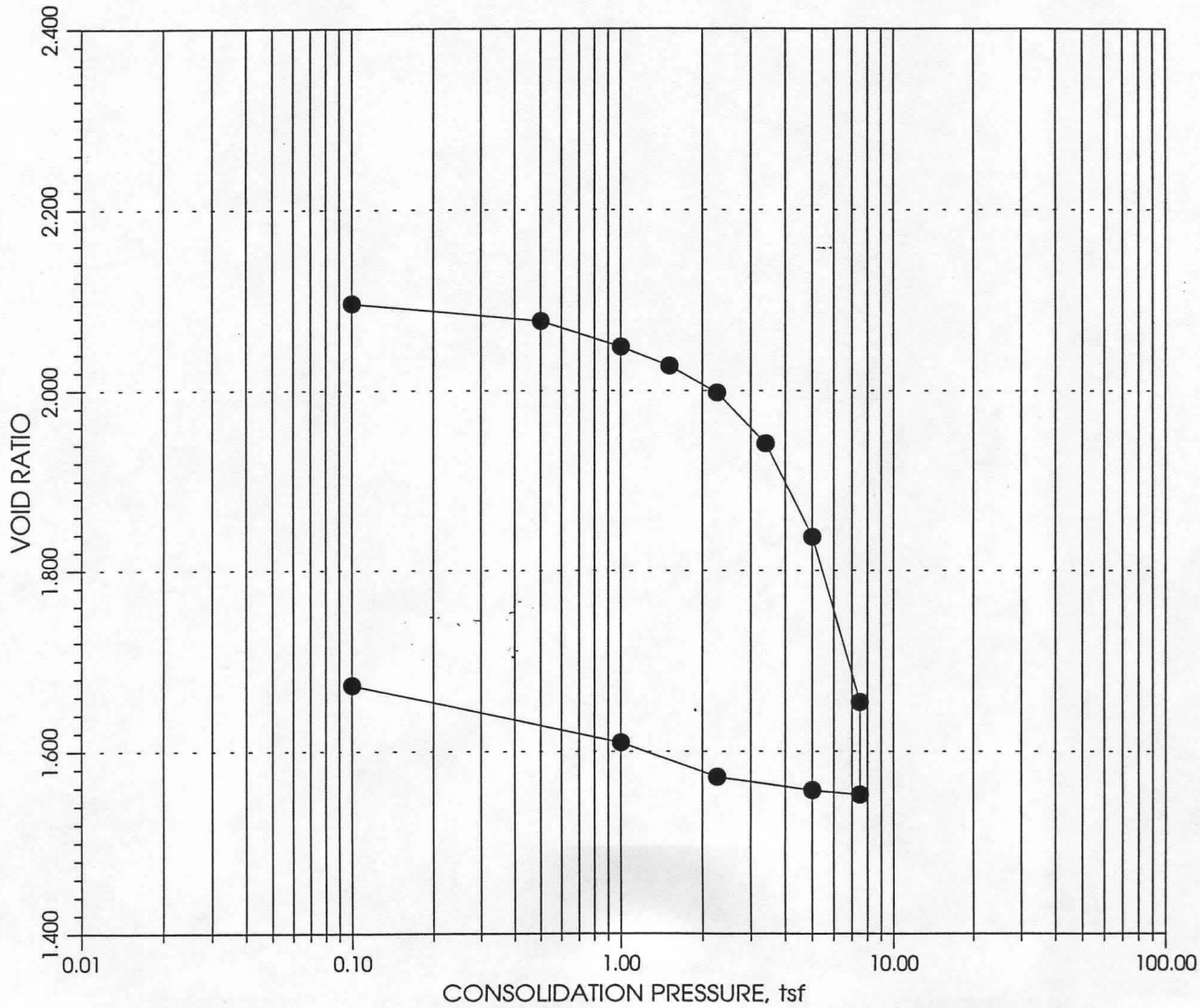
ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	CB-3	Tested By / Date	WLB 2/13/01
Sample	S-25	Calc. By / Date	WLB 2/20/01
Depth, ft.	95.1	Check By / Date	_____



Improvements to the Waste Water Treatment Plant, Phase One
Aberdeen, Washington

CONSOLIDATION TEST

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FIG. B-6b

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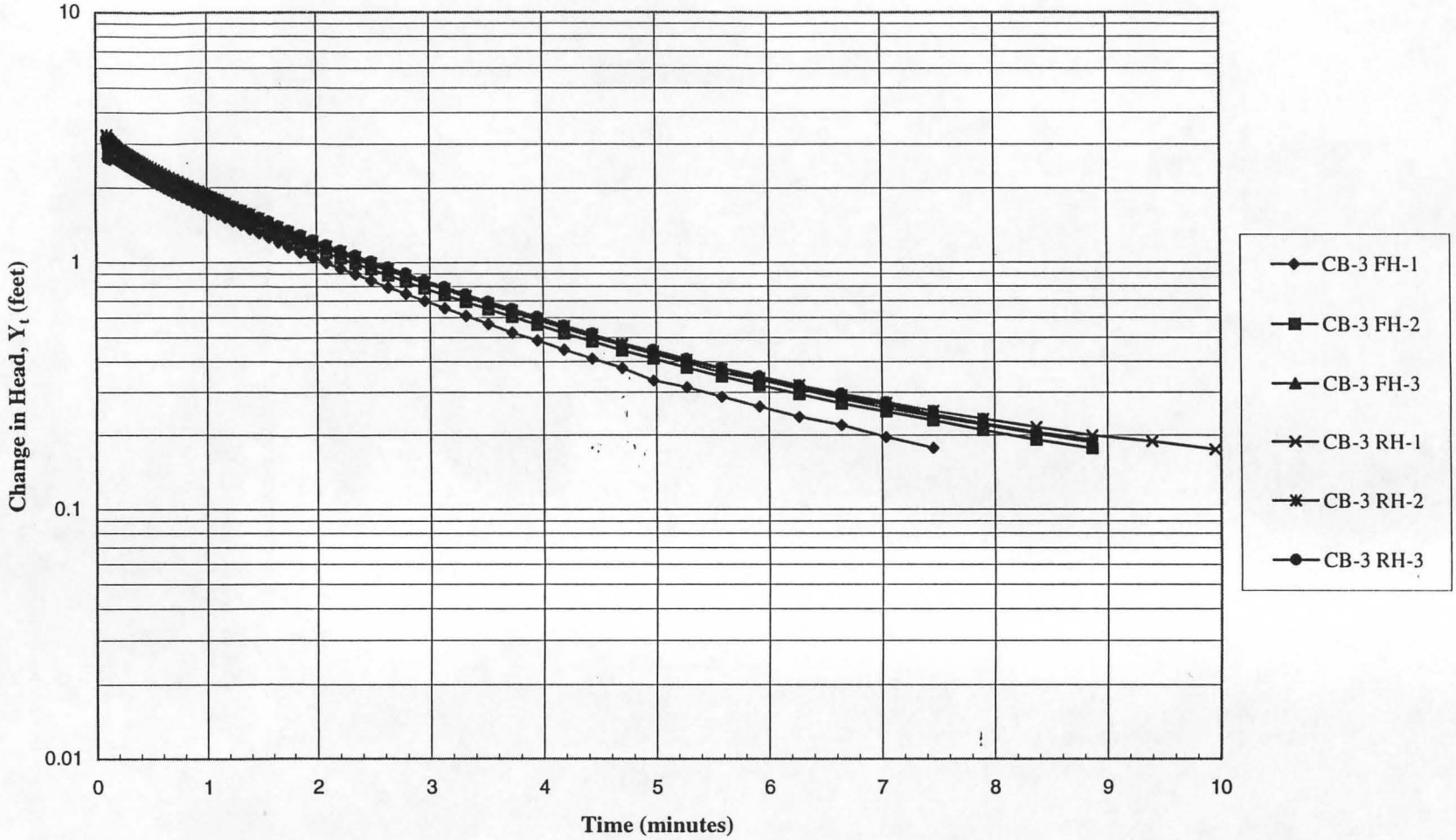

Notes:

1. CB-3 FH-3 denotes boring designation and method of slug testing, either Falling Head (FH) or Rising Head (RH). Multiple tests are designated by the digit placed at the end.
2. In accordance with the procedures developed by Bouwer & Rice (1976), "fit" represents a linear approximation of the data used for calculation of soil hydraulic conductivity.

Imrovements to the Wastewater Treatment Plant - Phase One
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SLUG TEST DATA
BORING CB-3

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FIG. B-7

FIG. B-7

APPENDIX C

PREVIOUS BORINGS

BORING 203

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
0				GP	GRAY FINE GRAVEL WITH SOME SAND (LOOSE, MOIST) (FILL)
				SM	
			12		BROWN, FINE SAND WITH SOME SILT, ORGANICS, BRICKS AND BUILDING DEBRIS (LOOSE, MOIST) (FILL)
5					
10					SOME VOIDS
			4		GRAY (VERY LOOSE, WET)
15					
			4	ML	GRAY SILT WITH A TRACE OF ORGANICS (SOFT, WET)
20					
25	64.0	57.3	3		
30	63.0	61.3	PUSH		
			17	SM	GRAY SILTY SAND WITH A TRACE OF WOOD (LOOSE, WET)
35					
			5	ML	GRAY SILT WITH SOME FINE SAND AND A TRACE OF ORGANICS (MEDIUM STIFF, WET)
40					

BORING 203 CONTINUED

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
40				ML	
45	58.0	65.3	6		
50			10	SM	GRAY, SILTY FINE SAND AND FINE SANDY SILT (LOOSE, WET)
55	41.0	76.7	10		
60			7		VERY LOOSE
65	54.9	62.8	7		LESS SILT
70			PUSH		
75			23	SP	GRAY FINE SAND WITH SOME SILT AND A TRACE OF ORGANICS (MEDIUM DENSE, WET)
80			9	ML	GRAY SILT WITH SOME ORGANICS AND A TRACE OF SAND (MEDIUM STIFF, WET)

BORING 203 CONTINUED

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
80				ML	
	65.0	60.6	13		STIFF
85					
			15		
90					
			38		VERY STIFF
95					
100					
			12		STIFF
105					
			21		TRACE OF ORGANICS, VERY STIFF
110	75.7	53.5			
			21		
115					
			20		
120					

BORING 203 CONTINUED

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
120				ML	
	50.0	69.4	22		LENSES OF FINE SAND
125					
			26		
130					
			87/9"	GP	DARK GRAY SANDY GRAVEL WITH SOME SILT (VERY DENSE, WET)
135				SP	DARK GRAY GRAVELLY SAND WITH SOME SILT (VERY DENSE, WET)
			100/9"		
140					

BORING TERMINATED AT 139.5 FEET

GROUNDWATER OBSERVATION WELL INSTALLED TO 20-FOOT DEPTH.

BORING 204

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
0				GP	BROWN SANDY GRAVEL (LOOSE, MOIST) (FILL)
				ML	GRAY CLAYEY SILT WITH SOME SAND AND FINE GRAVEL (MEDIUM STIFF, MOIST) (FILL)
			11	SP	GRAY GRAVELLY SAND WITH SOME SILT (LOOSE, MOIST) (FILL)
5				ML	GRAY SILT WITH SOME SAND AND A TRACE OF ORGANICS (MEDIUM STIFF, WET)
			7		
10					SOFT
			4		
15					
	74.0	56.6	5		TRACE OF SAND (MEDIUM STIFF)
20					
				SM	GRAY FINE SAND WITH SOME SILT AND ORGANIC LENSES (LOOSE, WET)
	46.7	76.2	8		
25					
			40	SP	GRAY FINE SAND WITH SOME COARSE SAND AND A TRACE OF SILT (MEDIUM DENSE, WET)
	29.9	92.5			
30					
				ML	GRAY SILT WITH A TRACE OF SAND AND ORGANICS (MEDIUM STIFF, WET)
			5		
35					
			5		
40	61.0	62.1			

ROGER LOWE ASSOCIATES | LOG OF EXPLORATION

BORING 204 CONTINUED

DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
40				ML	
			6		SOME SAND
45				SM	GRAY SILTY FINE SAND (LOOSE, WET)
50	35.6	82.8	PUSH		
					TRACE OF SHELLS
			8		
55					
	56.0	65.4	7		
60					
			60		DENSE
65					
			16		LOOSE
70					
	46.2	72.9	58		MEDIUM DENSE
75					
			43		
80					

BORING 204 CONTINUED

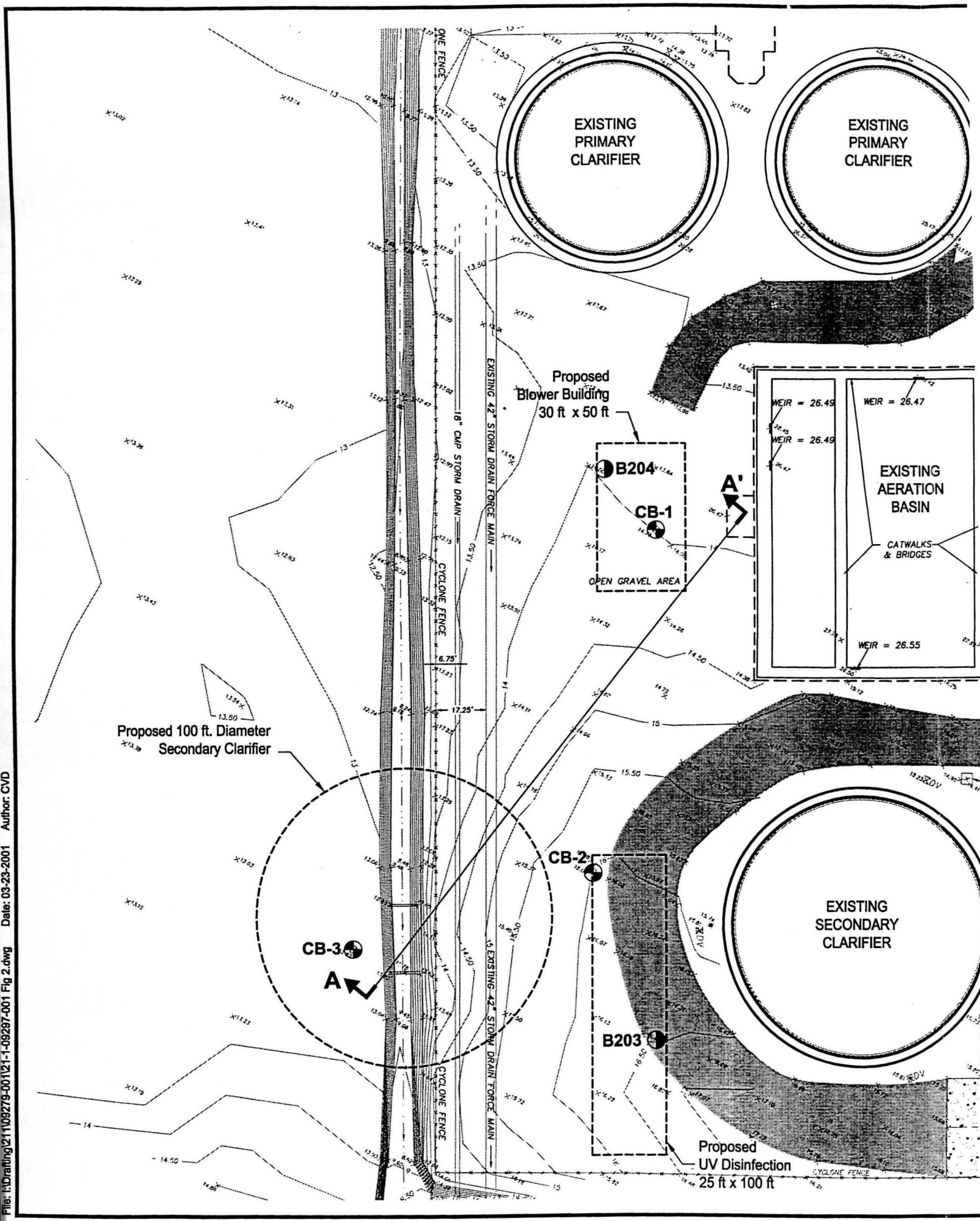
DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
80				SM	
				ML	GRAY FINE SANDY SILT WITH A TRACE OF ORGANICS (VERY STIFF, WET)
			26		
85					
			19		
90	57.2	64.6			
95					
			23		
100					
105					
			20		
110	48.7	68.1			
115					
			13		STIFF
120					

BORING 204 CONTINUED

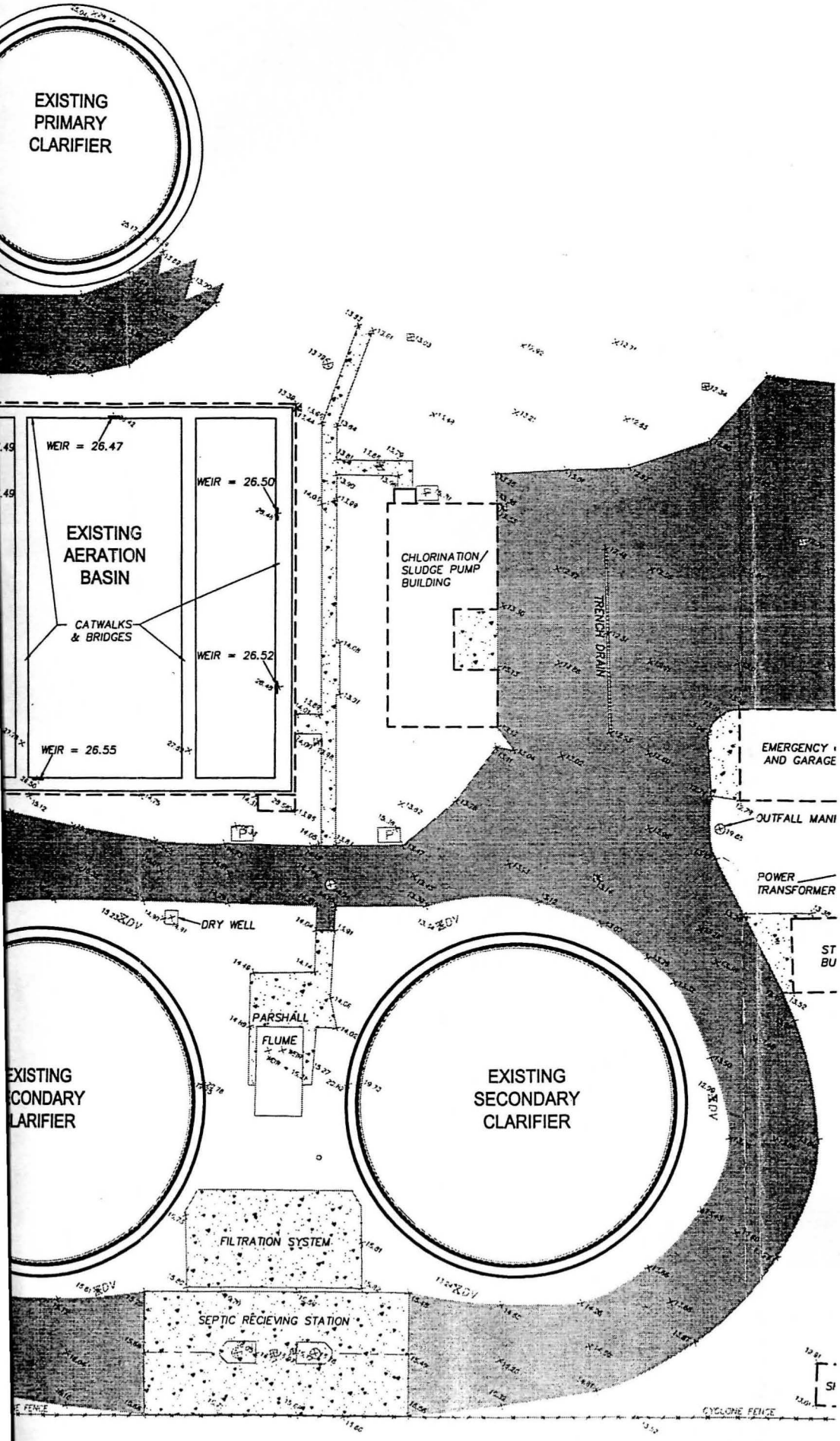
DEPTH IN FEET	MOISTURE CONTENT	DRY DENSITY (PCF)	BLOWS PER FOOT	GRAPHIC LOG	SOIL DESCRIPTION
120					
					SANDY
125					
			22		VERY STIFF
130					
				SP	DARK GRAY GRAVELLY SAND WITH SOME SILT (VERY DENSE, WET)
			146/11"		
135					

BORING TERMINATED AT 134.5 FEET

GROUNDWATER OBSERVATION WELL INSTALLED TO 20-FOOT DEPTH.



File: I:\Drafting\211\09279-001\21-1-09297-001 Fig 2.dwg Date: 03-23-2001 Author: CVD



LEGEND

CB-1 Soil Boring Designation and Approximate Location (Current Study 2001)

B203 Soil Boring Designation and Approximate Location (Previous Study 1977)

AA' Cross Section Location and Designation

NOTES

1. Figure based on drawing provided by Berglur Schmidt, & Assoc., Inc. entitled "Topographic Survey Base Map", dated 2-23-01.
2. Locations of previous borings are approximat and based on the 1977 geotechnical report by Roger Lowe Associate

N

0 40 80

Scale in Feet

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

SITE AND EXPLORATION PLAN

March 2001 21-1-09279

SHANNON & WILSON, INC.
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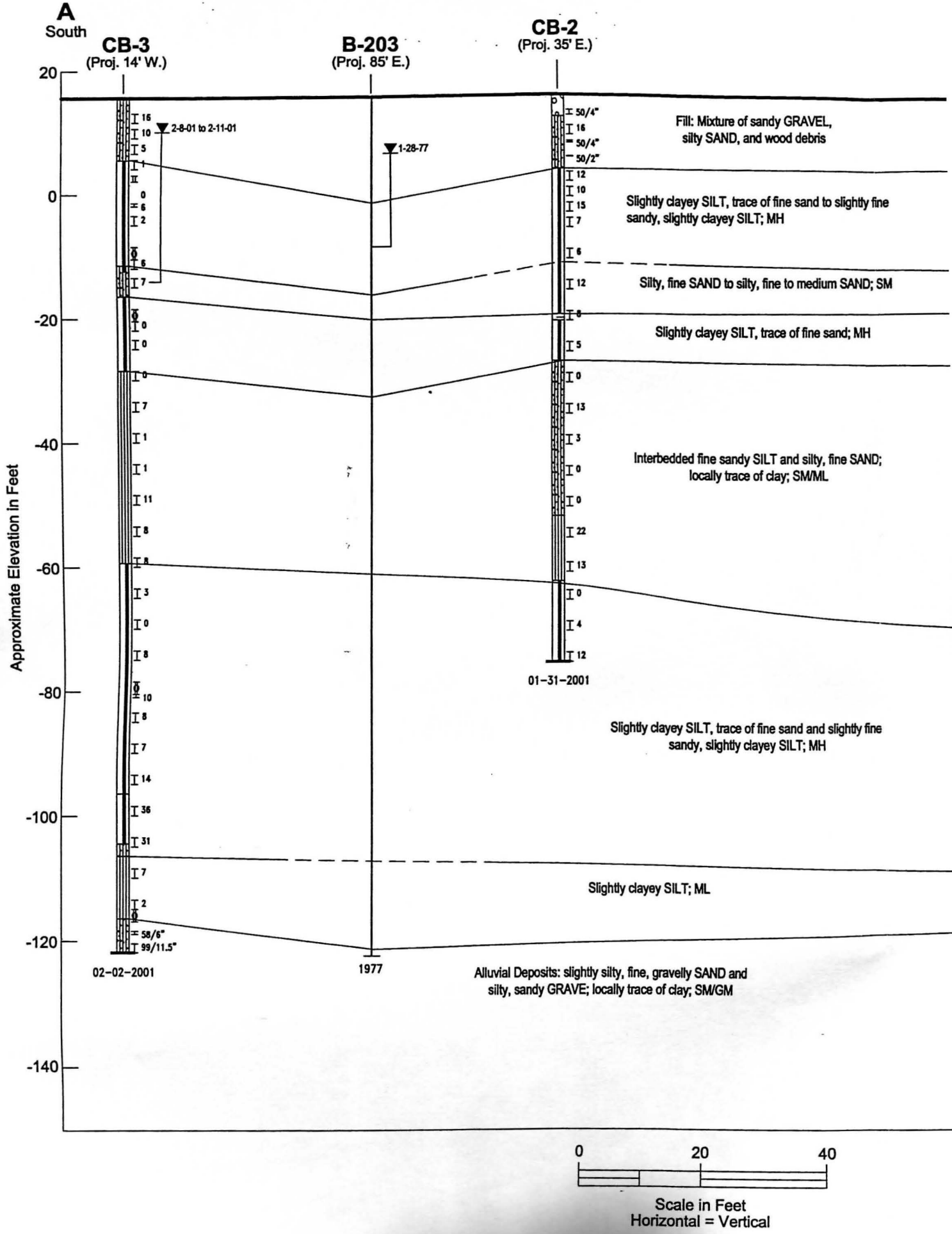
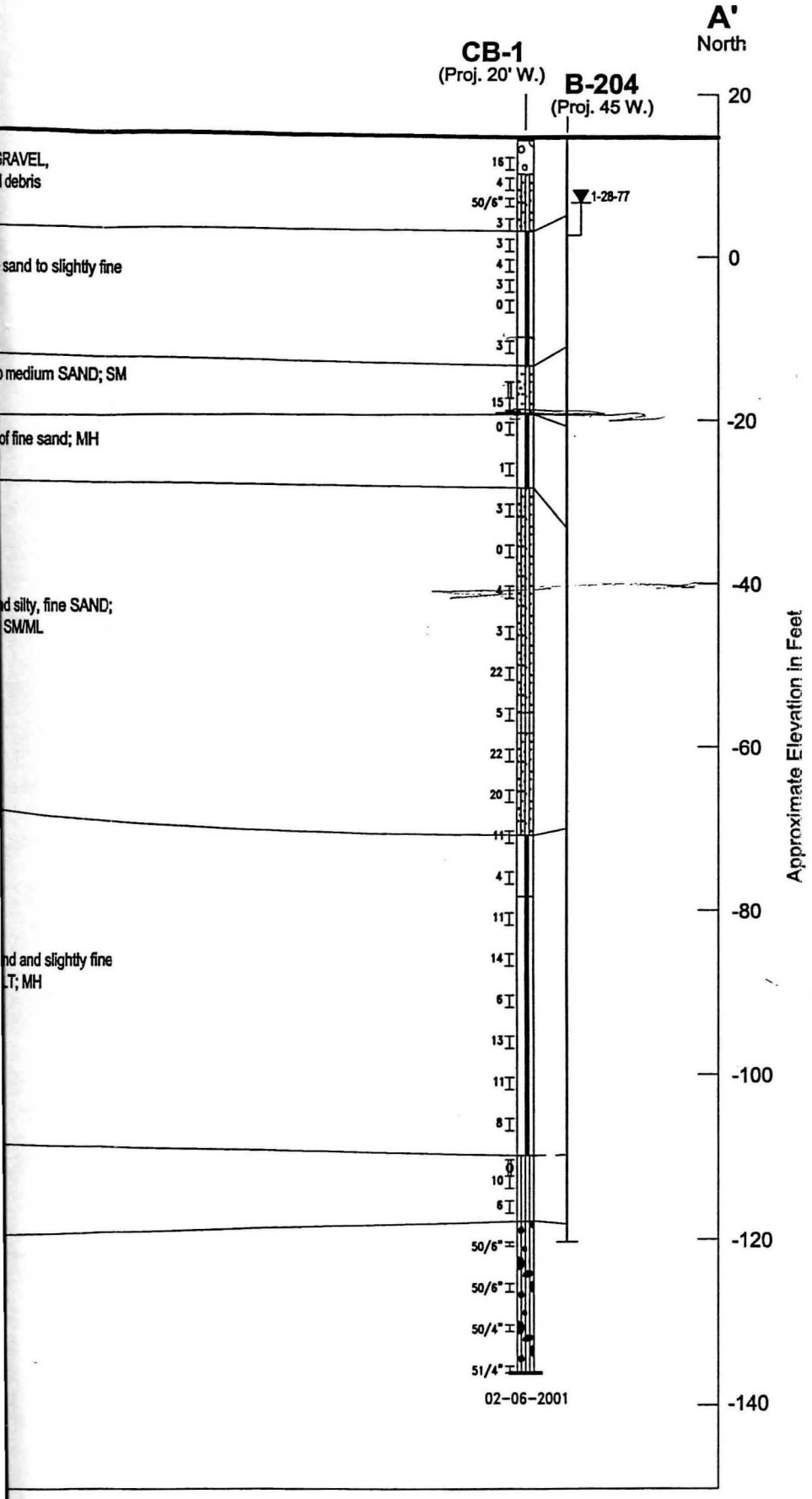
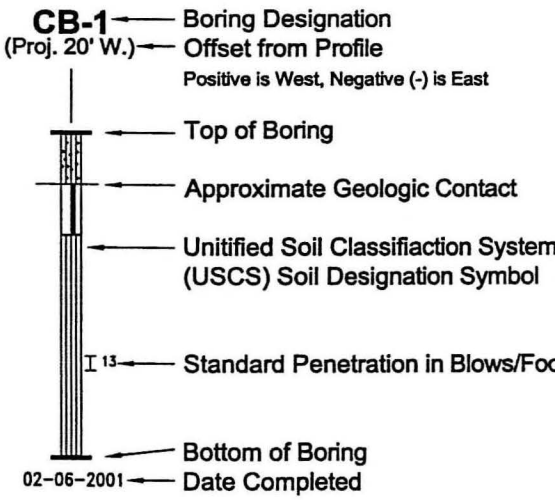
FIG.


A
South
CB-3
(Proj. 14' W.)
B-203
(Proj. 85' E.)
CB-2
(Proj. 35' E.)
Approximate Elevation in Feet
20
0
-20
-40
-60
-80
-100
-120
-140
2-8-01 to 2-11-01
1-28-77
Fill: Mixture of sandy GRAVEL, silty SAND, and wood debris
Slightly clayey SILT, trace of fine sand to slightly fine sandy, slightly clayey SILT; MH
Silty, fine SAND to silty, fine to medium SAND; SM
Slightly clayey SILT, trace of fine sand; MH
Interbedded fine sandy SILT and silty, fine SAND; locally trace of clay; SM/ML
Slightly clayey SILT, trace of fine sand and slightly fine sandy, slightly clayey SILT; MH
Slightly clayey SILT; ML
Alluvial Deposits: slightly silty, fine, gravelly SAND and silty, sandy GRAVE; locally trace of clay; SM/GM
02-02-2001
1977
01-31-2001
0
20
40
Scale in Feet
Horizontal = Vertical



LEGEND



NOTE

This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

Improvements to the Wastewater Treatment Plant
Phase One
Aberdeen, Washington

SUBSURFACE PROFILE A-A'

March 2001 | 21-1-09279-001

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FIG. 3

SHANNON & WILSON, INC.
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Attachment to and part of Report

Date: March 26, 2001
To: Tetra-Tech/KCM, Inc.
Seattle, Washington

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