Geotechnical Report
Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

December 17, 2008

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Excellence. Innovation. Service. Value.

Since 1954.

Submitted To:
Mr. Greg Moen, P.E.
HDR Engineering, Inc.
500 108th Avenue NE, Suite 1200
Bellevue, WA 98004-5549

By:
Shannon & Wilson, Inc.
400 N 34th Street, Suite 100
Seattle, Washington 98103

21-1-21000-001

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 SITE AND PROJECT DESCRIPTION 1

3.0 SUBSURFACE EXPLORATIONS 2

4.0 LABORATORY TESTING 3

5.0 SUBSURFACE CONDITIONS 3

6.0 CONCLUSIONS AND RECOMMENDATIONS 4
- 6.1 General 4
- 6.2 Seismic Design Considerations 4
  - 6.2.1 Ground Motions 4
  - 6.2.2 Earthquake-induced Geologic Hazards 5
- 6.3 Dike Design and Construction 6
  - 6.3.1 Construction Pad 6
  - 6.3.2 Global Stability Analyses 7
  - 6.3.3 Side Slopes 9
  - 6.3.4 Estimated Settlements 9
  - 6.3.5 Fill Reinforcement and Filter Geosynthetics 10
- 6.4 Soil Hydraulic Conductivity 11
- 6.5 Washington State Department of Transportation (WSDOT) Borrow Soil 11
- 6.6 Port of Grays Harbor Fill Soil Properties 11

7.0 CONSTRUCTION CONSIDERATIONS 12
- 7.1 Geosynthetic Placement 12
- 7.2 Fill Placement and Compaction 12
- 7.3 Wet Weather Considerations 13
- 7.4 Construction Observation 14

8.0 LIMITATIONS 14

9.0 REFERENCES 16

Page

LIST OF TABLES

Table No.

1 Summary of Global Stability Analyses.....8
2 Recommended Geosynthetic Strength Reduction Factors.....10

LIST OF FIGURES

Figure No.

1 Vicinity Map
2 Site and Exploration Plan
3 Estimated Subsurface Profile A-A'
4 Proposed Dike Cross-Section
5 Dike Stability Analysis, Geometry and Soil Properties
6 Settlement Plate Schematic

LIST OF APPENDICES

Appendix

A Subsurface Explorations
B Laboratory Testing Procedures and Results
C Important Information About Your Geotechnical/Environmental Report

GEOTECHNICAL REPORT
WASTEWATER TREATMENT FACILITY
BIOSOLIDS LAGOON PROJECT
HOQUIAM, WASHINGTON

1.0 INTRODUCTION

This report presents the results of Shannon & Wilson, Inc.'s (S&W's) geotechnical studies for the proposed dike to be constructed in the wastewater treatment facility biosolids lagoon in Hoquiam, Washington. This study was performed to evaluate the subsurface conditions at the project site and to provide geotechnical engineering recommendations to aid in the design and construction of the proposed dike. S&W's services included drilling one soil boring, performing four cone penetration tests (CPTs), performing laboratory and engineering analyses, and preparing this report. S&W performed its services in general accordance with its proposal, dated April 30, 2008, which was authorized by Mr. David Skinner of HDR Engineering on May 2, 2008.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located east of Bowerman Field, as shown in the Vicinity Map, Figure 1. The site is the City of Hoquiam wastewater treatment facility biosolids lagoon, which measures about 1,000 feet wide by 2,500 feet long. A pier with mounted runway approach lights for Bowerman Field extends from the southwest corner to near the center of the lagoon.

Based on drawings provided by HDR Engineering, two dikes are proposed. One dike will cordon off the northwest corner of the lagoon and will be about 650 feet long. The other dike will split the remainder of the lagoon in two and will be about 1,400 feet long, extending from the terminus of the west dike to near the end of the approach light pier, then south to the edge of the lagoon. Both dikes will be 12 feet wide and about 8 feet high, to match the elevation of the surrounding access road. The west dike will separate the facultative solids lagoon from the raw wastewater bypass storage lagoon. A weir will be placed on this dike so that the water levels will differ by no more than 1 foot. The layout of the dikes is shown in the Site and Exploration Plan, Figure 2.

Before dike construction begins, the lagoon water level will be lowered to within about 1 foot of the bottom, and the accumulated biosolids will be removed. Approximately 1 foot of water will remain in the lagoon during construction. After construction, the northeast portion of the lagoon will be decommissioned. Excavated borrow soils from a Washington Department of Transportation (WSDOT) construction site about 1½ miles to the east are being considered to fill the decommissioned portion of the lagoon. A soil stockpile at the Port of Grays Harbor, about 2 miles to the east, is being considered as a source for dike fill material.

3.0 SUBSURFACE EXPLORATIONS

S&W drilled and sampled one soil boring, performed four CPTs, and performed a series of vane shear tests in one location to obtain information about subsurface conditions at the site. The boring was drilled along the northern edge of the lagoon to a depth of 46.5 feet below the existing ground surface. The CPTs and vane shear test were performed from a barge on the lagoon. The CPTs were advanced to depths between about 25 and 30 feet below the lagoon mudline, which was about 4 to 5 feet below the surface of the water. The vane shear apparatus was advanced to a depth of about 12 feet below the mudline. Appendix A, Subsurface Explorations, describes the methods and procedures used for performing each of the explorations. Figures A-2 through A-7 in Appendix A show the exploration logs. Figure 2 shows the approximate locations of the explorations. A representative from S&W determined these locations by taping from existing landmarks and using triangulation with landmarks placed on shore.

S&W also reviewed three previous geotechnical studies: Shannon & Wilson (1993), CH2M Hill (2004), and Neil H. Twelker & Associates (1977). Shannon & Wilson (1993) drove 10 test timber piles into the lagoon for a proposed runway light expansion project. CH2M Hill (2004) drilled a soil boring, designated PB-1, at the southwest corner of the wastewater treatment facility, to a depth of 51.5 feet. Neil H. Twelker & Associates (1977) drilled 4 soil borings, designated DH-18 through DH-21, around the perimeter of the lagoon to depths of 14 to 29 feet. Figure 2 shows the approximate locations of the test piles and soil borings.

4.0 LABORATORY TESTING

S&W performed geotechnical laboratory tests on selected samples retrieved from the boring and used these tests to evaluate soil index and engineering properties. The S&W laboratory conducted the tests, which included visual classification, water content, grain size analyses, Atterberg limit determinations, and a one-dimensional consolidation test. Appendix B, Laboratory Testing Procedures and Results, describes the test methods and summarizes the test results. Figure B-1 summarizes the grain size analyses, Figure B-2 summarizes the Atterberg limits, and Figure B-3 summarizes the one-dimensional consolidation test. The boring logs in Appendix A show the water content and fines content measurements.

S&W also performed geotechnical laboratory tests on samples retrieved from the proposed dike fill stockpile about 2 miles east of the project site. These tests included grain size analyses and a modified Proctor test. Figure B-4 in Appendix B summarizes the grain size analyses, and Figure B-5 summarizes the modified Proctor test.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions below the lagoon consist of a sequence of very soft to soft organic silt, soft to stiff clay and silt, and medium dense silty sand. In probes CPT-1 through CPT-4, the organic silt was 6 to 17 feet thick, the silt/clay was 6 to 14 feet thick, and the silty sand was encountered to the termination depth of the explorations. The depth to the silty sand ranged from 21 to 26 feet. The depth of water in the lagoon ranged from 4 to 5 feet. Figure 3 shows the approximate subsurface profile beneath the east dike.

The test piles from Shannon & Wilson (1993) encountered a bearing layer 10 to 25 feet below the mudline, and penetrated 30 to 60 feet below the mudline. The depth of this bearing layer appears to correspond to the depth of the medium dense silty sand.

S&W's current boring and the 5 previous borings were each advanced around the perimeter of the lagoon. Borings B-1 and PB-1 encountered about 10 to 15 feet of loose silty sand, followed by a sequence of organic silt, silty clay, and silty sand, similar to the subsurface conditions below the lagoon. The 10 to 15 feet of loose silty sand appears to be fill placed during construction of the wastewater treatment plant and lagoon dikes. Borings DH-18 through DH-21 encountered varying thicknesses of silty sand overlying silt. In general, the explorations completed by CH2M

Hill and Neil H. Twelker were consistent with subsurface conditions encountered in S&W's explorations.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

S&W based its engineering studies for this project on its understanding of the proposed construction as described previously and on the results of field explorations and laboratory testing.

The base of the dikes will be constructed on very soft to soft soils under about 1 foot of water. To facilitate construction, the dike should incorporate a reinforcement geosynthetic. Quarry spalls should be placed above the geosynthetic to provide a stable construction pad for the dike fill to be placed above the waterline.

As suggested by HDR Engineering, S&W analyzed the stability of the dikes according to requirements in the Washington State Department of Ecology (DOE) Dam Safety Guidelines. S&W also estimated the consolidation settlement of the soft soils under the dikes.

The following sections provide recommendations for seismic design, dike design and construction, soil hydraulic conductivity, and potential borrow source soil properties.

6.2 Seismic Design Considerations

6.2.1 Ground Motions

The project is located in a moderately active seismic region. While the region has historically experienced moderate to large earthquakes (i.e., April 13, 1949, magnitude 7.1 Olympia Earthquake; April 29, 1965, magnitude 6.5 Seattle-Tacoma Earthquake; and February 28, 2001, magnitude 6.8 Nisqually Earthquake), geologic evidence suggests that larger earthquakes have occurred in the prehistoric past and will occur in the future (e.g., magnitude 8.5 to 9.0 Cascadia Subduction Zone Interplate events).

The project will comply with DOE Dam Safety Guidelines. These guidelines provide some latitude in choosing a design-level earthquake. In S&W's opinion, for the relatively small

height and relatively low risks associated with failure, the dikes can be designed for a 475-year return period seismic event.

Seismic design forces are calculated using peak ground acceleration (PGA). Based on regional probabilistic ground motion studies conducted by the United States Geological Survey (USGS), the mapped soft rock PGA value near the project site for a 475-year return period seismic event is 0.31 gravity (g).

To calculate the PGA at the ground surface, S&W calculated an amplification factor after Stewart et al. (2001). For Holocene soils and a soft rock PGA of 0.31g, Stewart et al. (2001) give a median amplification factor of 0.95. The ground surface PGA is the product of the amplification factor and the soft rock PGA. Therefore, the PGA at the ground surface is 0.29g.

6.2.2 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a site include landsliding, fault rupture, settlement, and liquefaction and associated effects (such as loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, and lateral spreading). Because of the flat site topography, the risk of landsliding at this site is low.

The potential for fault rupture is also low. The nearest mapped fault (USGS, 2006) is a northeast trace of the Saddle Hill Fault Zone, which consists of short, discontinuous traces that trend northeast-southwest for a distance of about 16 miles. The project site is located about 10 miles southeast of the most northeastern trace. Evidence of Holocene rupture (i.e., movement within the last 10,000 years) has not been reported along this trace of the Saddle Hill Fault Zone.

Liquefaction and related effects pose an earthquake-induced geologic hazard at the site. S&W calculated factors of safety (FSs) against liquefaction for boring Standard Penetration Test (SPT) N-values and equivalent N-values calculated from CPTs. S&W used design earthquake ground motions and empirical procedures established by the National Center for Earthquake Engineering Research (NCEER) (Youd et al. 2001) to calculate the FSs. S&W computed the FSs with a 0.29g PGA from a magnitude 8.0 source, located 20 miles away.

The silty sands below about 25 feet deep are susceptible to liquefaction. Liquefaction in these soils could cause settlement and lateral spreading.

S&W estimated post-liquefaction settlement using the methods of Tokimatsu and Seed (1987). S&W based its estimate on the FSs against liquefaction and soil relative density (using correlations with corrected SPT blow counts and equivalent SPT blow counts from CPTs). Post-liquefaction settlement of 1 to 1½ feet could occur at the site.

One of the major liquefaction-induced types of ground failure is lateral spreading. Lateral spreading movement of gently sloping ground occurs as a result of pore-pressure buildup or liquefaction in the underlying soil deposit. A lateral spread often contains a liquefied layer overlain by a non-liquefied layer at the ground surface that rides along the top of the liquefied soil. The non-liquefied layer is present because either it lies above the groundwater table or it is not susceptible to liquefaction.

Based on the empirical procedure by Youd, Hansen, and Bartlett (2002), lateral spreading displacement for the design level earthquake could be as much as 10 feet. The lateral spread displacements would generally be in a southerly direction (toward Grays Harbor). The lateral spread hazard is an existing site condition, and the proposed dike construction would not affect the geologic hazard. Lateral spread could be mitigated through ground improvement techniques such as stone columns. However, the lateral and vertical extents of ground improvement necessary to effectively mitigate lateral spreading hazards at this site would likely be cost-prohibitive.

6.3 Dike Design and Construction

6.3.1 Construction Pad

The very soft to soft subsurface soils, coupled with the lagoon water that will be present, will complicate construction of the dike. Without reinforcement and a firm work area, the dike could fail during construction. To increase dike stability during construction and to allow placement of dike fill, a reinforced construction pad should be constructed at the base of the dike before placing the rest of the dike fill.

The construction pad should consist of crushed rock or quarry spalls (as described in Section 7.2), which should be placed on geosynthetic reinforcement. Section 6.3.5 provides recommendations for the reinforcement. The pad should be built a minimum of 6 inches above the lagoon water level, over the proposed dike footprint. Once the construction pad is in place

and the geosynthetic has been wrapped over the pad (as described in Section 6.3.5), subsequent fill placement and compaction for the dike can progress.

To reduce seepage through the core of the dike, a central core of low hydraulic conductivity dike fill should extend from the bottom to the top of the dike. This will result in an 8-foot-wide center strip of the construction pad being composed of the same material as the dike fill. Figure 4 shows a typical cross-section through the dike, and illustrates the core concept.

To reduce migration of soil (piping) from the dike fill into the construction pad, a filter geotextile should be placed between the construction pad and the dike fill. Section 6.3.6 provides recommendations for the filtration geotextile. The location of the filter geotextile is shown in Figure 4.

6.3.2 Global Stability Analyses

S&W performed a series of slope stability analyses to estimate the dike FSs and to determine the required strength of the geogrid reinforcement. S&W used the computer program SLOPE/W Version 7.13 (Geo-Slope International, 2007) to perform the analyses. SLOPE/W estimates the FS for slope stability by analyzing the shear and normal forces acting on a failure surface. The FS is the ratio of resisting forces to driving forces. An FS of 1.0 means that the driving forces are equal to the resisting forces. An FS less than 1.0 means that the driving forces are greater than the resisting forces, and that the slope is not stable and would fail. The critical surface has the lowest FS for a given loading condition. DOE Dam Safety Guidelines specify minimum FS values for global stability of small dams, which are presented in Table 1.

S&W considered drained and undrained strength of the soft silt soils. A fine-grained soil exhibits undrained shear strength under short-term, rapid loading, and drained strength under long-term, slow loading. S&W developed an undrained shear strength profile for each of the CPTs using published correlations, as described in Appendix A. S&W estimated the drained shear strength of the soils using Atterberg limit correlations. Table 1 shows the type of soil strength considered for each loading case and minimum DOE Dam Safety FS values. Figure 5 shows the SLOPE/W geometry and the assumed soil strength profiles.

TABLE 1
SUMMARY OF GLOBAL STABILITY ANALYSES

Loading Case	Soil Strength	Minimum Factor of Safety
End of Construction	Drained/Undrained	1.3
Sudden Drawdown	Drained	1.0
Steady Seepage	Drained	1.5
Seismic Loading	Undrained	1.1*

* Washington State Department of Ecology Dam Safety Guidelines do not specify a minimum factor of safety for the seismic loading case. S&W chose this value based on its experience and typical Washington State Department of Transportation minimum factors of safety for seismic loading.

For each case, S&W first calculated a dike stability FS for the condition where reinforcement is not installed below the dike (unreinforced FS). If the unreinforced FS was below the minimum FS, S&W added reinforcement and increased the reinforcement capacity until the minimum FS was met. S&W assumed the following conditions for each dike section analyzed:

- **End of Construction**. Water level 2 feet above the ground surface on both sides of the dike.
- **Sudden Drawdown**. Water level 6 feet above the native ground surface in the dike and at the ground surface in the lagoon.
- **Steady Seepage**. Water level 6 feet above the native ground surface on one side of the dike, and at the ground surface on the other side.
- **Seismic Loading**. Water levels from the Steady Seepage case, with a seismic coefficient, k_h, equal to 0.15.

The magnitude of k_H is less than the PGA, accounting for the fact that the PGA is experienced only a few times within the record of earthquake shaking, and that the actual earthquake ground motion is cyclic in nature, as opposed to a static force. Based on Makdisi-Seed analysis for estimating seismically induced slope deformations (Makdisi and Seed, 1978), potential slope displacements of the dikes resulting from the 475-year ground motion (when lateral spread is discounted) would be less than about 6 inches. In S&W's opinion, this would not cause collapse and would constitute satisfactory performance.

S&W also considered the effect of the fill proposed to be placed in the decommissioned (east) part of the lagoon on the stability of the dike. S&W analyzed a 28-foot-high fill adjacent to the

dike for each loading case listed in Table 1. S&W varied the landfill side slopes and setback from the dikes to find a stable configuration. Section 6.5 provides recommendations for the landfill configuration.

6.3.3 Side Slopes

The dike side slopes should be no steeper than 2 Horizontal to 1 Vertical (2H:1V) on sides that will face the lagoon areas. The eastern side of the east dike, which will have borrow fill placed against it, could be steepened to 1.5H:1V. To protect the slopes from erosion due to wind and wave action on the lagoon, the water-side slope should be armored with quarry spalls or similar materials.

6.3.4 Estimated Settlements

S&W used the results of our laboratory testing to estimate dike settlement. For a 12-foot-wide, 8-foot-high dike with 2H:1V side slopes, the soft soils will consolidate and settle about 1 to 2 feet in response to dike construction. Settlement will begin as fill is placed and continue over several years. About 90 percent of the settlement will occur over two to six months. The dikes should be constructed to a minimum of 1½ feet above the design final grade to compensate for the anticipated settlement. Additional fill should be added if greater settlement occurs; excess material could be removed if less settlement occurs.

Based on the results of our settlement analyses, the toe of the dike should be located at least 50 feet from the existing runway approach lights pier. This distance will reduce the effects of settlement and downdrag on the existing pier piles.

S&W also estimated the settlement of the proposed WSDOT borrow soil landfill. For a 28-foot-high landfill, the soft soils will consolidate and settle 2 to 3 feet. Provided the recommendations of Section 6.5 are followed, the settlement induced by the proposed landfill on the proposed and existing dikes on the east side of the lagoon will be less than 1 foot. The settlement should be monitored and additional fill can be placed on the dikes as needed.

Dike settlement should be monitored during and after construction. Settlement data will be used to evaluate stability during construction and to determine when settlement is substantially complete. To monitor the dike settlement, settlement plates should be installed and monitored, and the results analyzed on a regular basis, at least twice per week during

construction. Settlement plates should be installed at a spacing of about 200 feet along each dike. Figure 6 shows a typical settlement plate detail.

6.3.5 Fill Reinforcement and Filter Geosynthetics

Geosynthetic reinforcement could be a woven polyvinyl chloride-coated polyester geogrid or a woven polyester or polypropylene geotextile. Because of its open structure, a geogrid may be more easily placed underwater; a woven geotextile may be more difficult to place in underwater.

The geosynthetic reinforcement should have a minimum allowable tensile strength of 4,000 pounds per foot for short-term loading (i.e., during and immediately after construction), and 1,000 pounds per foot for long-term loading. Table 2 shows our recommended reduction factors for short- and long-term loading.

TABLE 2
RECOMMENDED GEOSYNTHETIC STRENGTH REDUCTION FACTORS

Loading Case	Creep Reduction Factor	Installation Reduction Factor	Durability Reduction Factor
Short-term	Use 75 percent of manufacturer's recommended long-term value (minimum 1.1)	1.4	1.0
Long-term	Follow manufacturer's recommendations	1.4	1.5

The geosynthetic should extend beyond the edges of the dike, and should be wrapped back over the construction pad after the top of the pad is at least 6 inches above water level. The geosynthetic should overlap the construction pad a minimum of 3 feet on each side, as shown in Figure 4. Section 7.1 discusses geosynthetic placement considerations.

The filtration geotextile to be placed between the construction pad and dike fill should meet the WSDOT standard specification for moderate survivability filtration geotextiles (Section 9-33.2, Table 2, Class A), and should have a maximum apparent opening size of 0.2 millimeter.

6.4 Soil Hydraulic Conductivity

S&W estimated the native soil hydraulic conductivity using published correlations, the grain size analyses results and one-dimensional consolidation test results. S&W estimates that the silty sands in the boring and CPTs have a hydraulic conductivity of 1×10^{-3} to 5×10^{-3} centimeters per second (cm/sec). S&W estimates that the organic silts and silts/clays have a hydraulic conductivity of 1×10^{-6} to 1×10^{-8} cm/sec.

6.5 Washington State Department of Transportation (WSDOT) Borrow Soil

Based on the data contained in Landau Associates (2007), the proposed borrow material from the WSDOT construction site is mainly dark gray, soft silt with areas of medium dense, fine sand. The Landau Associates' boring logs indicate the presence of wood and organic debris in some of the borings. The average SPT blow count of the material is 2, and the average total unit weight is 125 pounds per cubic foot (pcf). Atterberg limit tests were performed on samples of the borrow material. The average water content was 65 percent; average liquid limit was 66 percent; average plastic limit was 41 percent; and the average plasticity index was 25 percent.

If the WSDOT borrow material is used as fill for the decommissioned lagoon, it will likely be difficult to compact and will compress and settle with time. If the WSDOT borrow material will be used as fill, organics and wood debris should be removed to the extent practical.

Based on S&W's slope stability analyses, the borrow soil landfill should have a maximum height of 8 feet within a distance of 50 feet of the proposed and existing dikes. The maximum landfill side slopes should be 3H:1V and the height can be up to 28 feet. To increase the storage volume of the landfill, these side slopes could be steepened by designing a geogrid-reinforced slope. Care should be taken to prevent the formation of a mud wave during fill placement.

6.6 Port of Grays Harbor Fill Soil Properties

The City of Hoquiam has proposed using fill material stockpiled at the Port of Grays Harbor to construct the dike. The fill stockpile is located about 2 miles east of the wastewater treatment lagoon. S&W visited the fill stockpile site on November 10, 2008, and used a hand shovel to obtain several samples for laboratory testing.

Based on soil samples collected by S&W, the proposed fill from the Port of Grays Harbor stockpile soil is gray-brown, slightly gravelly silty sand. Laboratory testing showed that the soil

had a fines content of about 40 percent. A modified Proctor compaction test on the soil showed a maximum dry density of 113 pcf at an optimum water content of 14.5 percent. The natural water content of the soil was about 30 percent. Because the natural water content of the soil is much higher than its optimum water content, the soil would be unsuitable for use as dike fill material unless it was moisture-conditioned or cement-treated. It may be used as landfill in the decommissioned lagoon.

6.7 Dike Fill Material

If imported, dike fill (above the construction pad) should meet the WSDOT specification for Common Borrow (Section 9-03.14[3]), except that the percent passing the No. 200 sieve should be no less than 30 percent. Dike fill should not contain organics or other deleterious material.

Either the WSDOT borrow soil or Port of Grays Harbor fill soil could be used as dike fill, provided the material was suitably moisture-conditioned or cement-treated to produce a stable, compactable mixture. Typical soil-cement amendment methods involve the use of 4 to 6 percent Portland cement (based on dry weight of soil).

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Geosynthetic Placement

The reinforcement geosynthetic panels should be oriented perpendicular to the dike centerline and placed with the machine (strong) direction oriented perpendicular to the dike centerline. The geosynthetic should be continuous from on side of the dike to the other (including wraps). Adjacent panels should overlap a minimum of 6 inches and be positively mechanically connected. The strength of the connections should meet or exceed that of the geosynthetic. The Contractor should protect the geosynthetic from damage during installation. The geosynthetic should be stretched tight using hand labor, stakes, or by weights placed on it prior to placement of backfill. Backfill placement should be performed so as to maintain tension in the geosynthetic.

7.2 Fill Placement and Compaction

Construction pad fill should consist of quarry spalls, crushed rock, or similar material that can be placed under water and interlock to form a stable pad. The pad fill should be compacted by tracking with construction vehicles.

Prior to the placement of dike fill, the construction pad should be completed to at least 6 inches above the water level. Dike fill should be placed in uniform lifts and compacted to at least 90 percent of the Modified Proctor maximum dry density (ASTM International D 1557-70, Method C or D). Lift thickness should not exceed 12 inches for heavy equipment compactors or 6 inches for hand-operated mechanical compactors. To reduce the potential for infiltration of the fine-grained native soils, heavy vibratory equipment compactors should not be used within 3 feet of the base of the dikes.

Before placing and compacting dike fill material in the dike core, water should be pumped from the core area (where there is no construction pad) using sumps.

Based on the subsurface conditions, S&W anticipates a bulge of soil, or mudwave, could form ahead of the advancing embankment. Therefore, construction pad and dike fill should be placed in a carefully controlled manner to minimize the potential for mudwave formation.

To reduce the potential for subgrade bulging or squeezing, the amount of fill placed in any location should be limited to 2 feet per day.

7.3 Wet Weather Considerations

In Western Washington, wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. The degree of compaction is highly dependent on soil water content. Wet soil tends to become unstable and difficult or impossible to compact if the moisture content significantly exceeds the optimum. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. Therefore, it would be prudent to schedule earthwork during the dry weather months of June through September. However, should wet weather/wet condition earthwork be unavoidable, S&W recommends the following.

- The dike surface should be sloped to promote runoff of precipitation away from work areas and to prevent ponding of water.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with drier materials or cement-treated fill materials.

- Placement of fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be done during periods of heavy, continuous rainfall.

S&W recommends that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

7.4 Construction Observation

Geotechnical recommendations that are used as a basis for design are developed from a limited number of explorations and tests. Consequently, there may be a need for adjustment in the field. S&W recommends that it be retained to observe the geotechnical aspects of construction such as geogrid and fill placement.

This observation would allow us to evaluate the subsurface conditions as they are exposed during construction and to observe that the work is accomplished in accordance with our recommendations. S&W also recommends that it review the geotechnical aspects of the plans and specifications to determine that they are also in accordance with our recommendations.

8.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the site subsurface conditions are not significantly different from those disclosed by the explorations. If, during construction, subsurface conditions different from those encountered in the explorations are encountered, or appear to be present, S&W should be advised at once so that S&W 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 construction at the site, or if conditions have changed due to natural forces or construction operations at or adjacent to the site, S&W recommends that S&W review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of 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. S&W makes no other warranty, either express or implied. These conclusions and recommendations are based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

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

This report was prepared for the exclusive use of HDR Engineering, the City of Hoquiam, and the design team for the Wastewater Treatment Facility Biosolids Lagoon Project. The factual data in this report should be provided to potential contractors for their information, but S&W's report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions at the site.

S&W's scope of our services did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site. Shannon & Wilson, Inc. has prepared and included in Appendix C, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.

Jeremy N. Butkovich
Geotechnical Engineer

Martin W. Page, P.E., L.E.G.
Associate

JNB:MWP:SRB/jnb

9.0 REFERENCES

ASTM International (ASTM), 2007, 2007 Annual book of standards, construction: West Conshohocken, Pa., volume 04.08 Soil and Rock.

CH2M Hill, 2004, Geotechnical data report, City of Hoquiam, waterline crossing, sewerline crossing & UV disinfection improvements project, Hoquiam, Washington, November.

Geo-Slope International, 2007, SLOPE/W computer program, version 703, Calgary, Alberta, Canada.

Landau Associates, 2007, Geotechnical design study, WSDOT Special Projects Construction Site, Grays Harbor, Washington: Report prepared by Landau Associates, Edmonds, Wash., for WSDOT, March.

Makdisi, F.I. and Seed, H.B., 1978, Simplified procedure for estimating dam and embankment earthquake-induced deformations: Journal of the Geotechnical Engineering Division, ASCE, v. 104, no. GT7, p. 849-867.

Neil H. Twelker & Associates, 1977, Soils and foundation investigation, Moon Island sewage treatment and outfall, Hoquiam, Washington, July 15.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of settlements in sands due to earthquake shaking: Journal of Geotechnical Engineering, v. 113, no. 8, p. 861-878.

Stewart, J.P., Liu, A.H., Choi, Y., and Baturay, M.B., 2001, Amplification factors for spectral acceleration in active regions: Pacific Engineering Earthquake Center Report 2001/10, December.

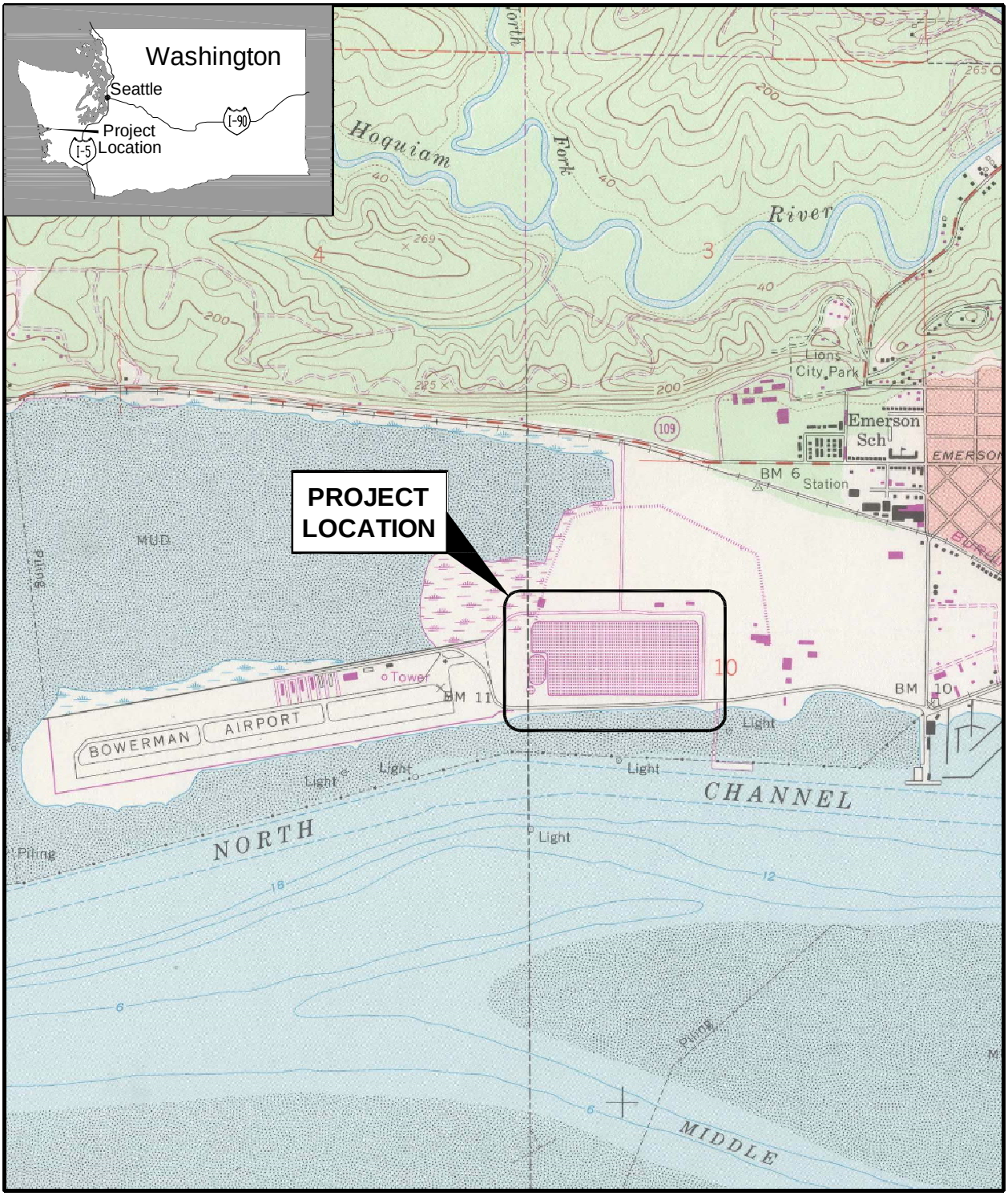
U.S. Geological Survey (and supporting agency if appropriate-see list below), 2006, Quaternary fault and fold database for the United States, accessed 18 June 2008, from USGS web site: http//earthquakes.usgs.gov/regional/qfaults/

Washington State Department of Ecology (DOE), 1993, Dam safety guidelines (92-55D): DOE.

Washington State Department of Transportation (WSDOT) and American Public Works Association (APWA), 2008, Standard specifications for road, bridge, and municipal construction (M41-10): WSDOT and APWA.

Youd, T.L., Hansen, C.M., and Bartlett, S.E., 2001, Revised multilinear regression equations for prediction of lateral spread displacement: Journal of Geotechnical and Geoenvironmental Engineering, v. 128, issue 12, p. 1007-1017, December.

Youd, T. L. and Idriss, I. M., 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: Journal of Geotechnical and Geoenvironmental Engineering, v. 127, no. 4, p. 297-313.



0 1/2 1

Scale in Miles

NOTE

Map adapted from 1:24,000 USGS topographic map of Hoquiam, WA quadrangle, dated 1957, photorevised 1983.

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

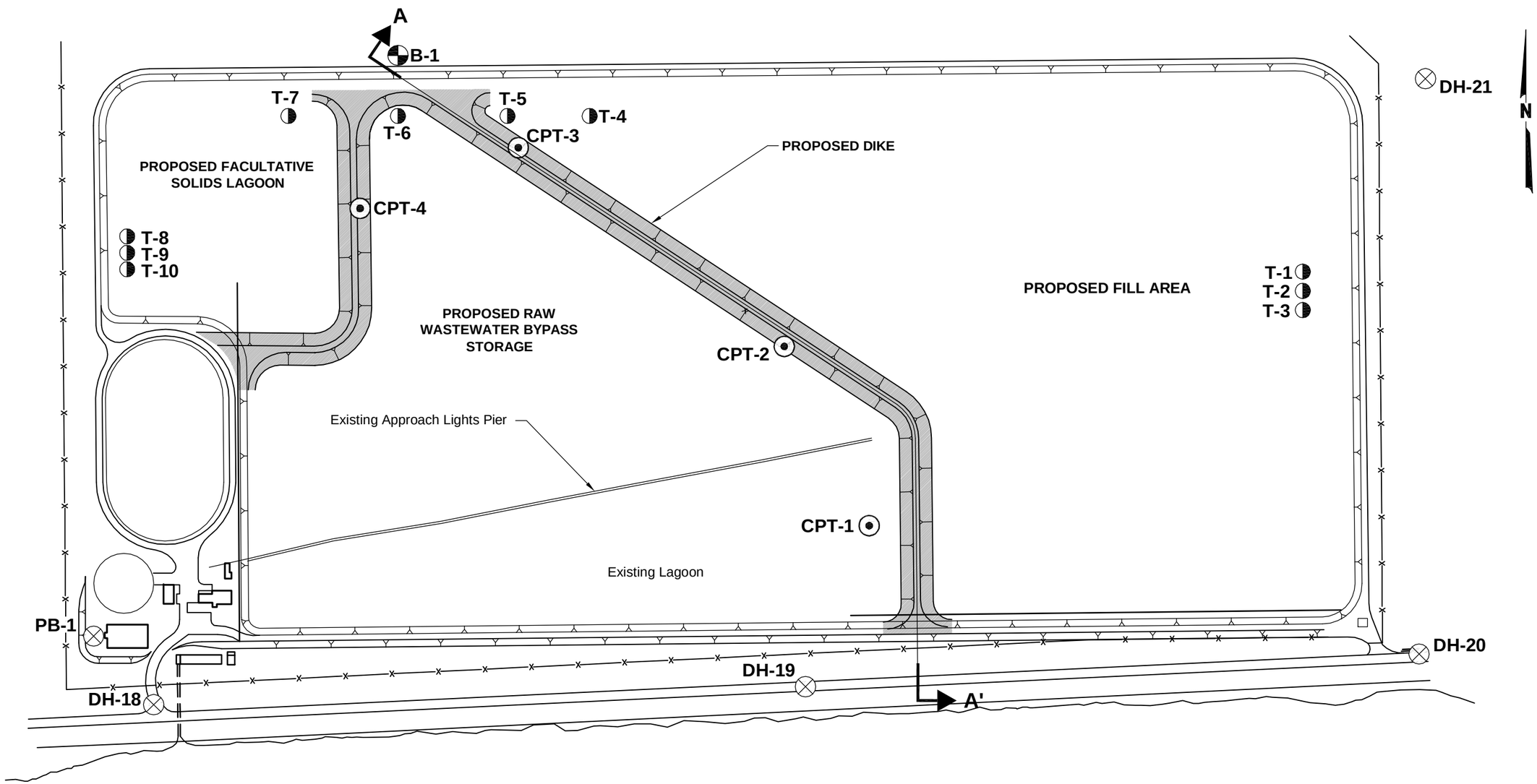
VICINITY MAP

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

Filename: J:\211\21000-001\2008-December Report\21-1-21000-001 fig 1.dwg Date: 12-12-2008 Login: sac



0 200 400

Scale in Feet

NOTE

Figure adapted from file *00C-BP01-01.dwg* and *00C-BP01-02.dwg* received 12/12/2008.

LEGEND

B-1	Boring Designation and Approximate Location
CPT-1	Cone Penetration Test Designation and Approximate Location
PB-1	Previous Boring Designation and Approximate Location (by Others)
T-1	Previous Test Pile Designation and Approximate Location (Shannon & Wilson, 1993)
A	Generalized Subsurface Profile (See Figure 3)

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

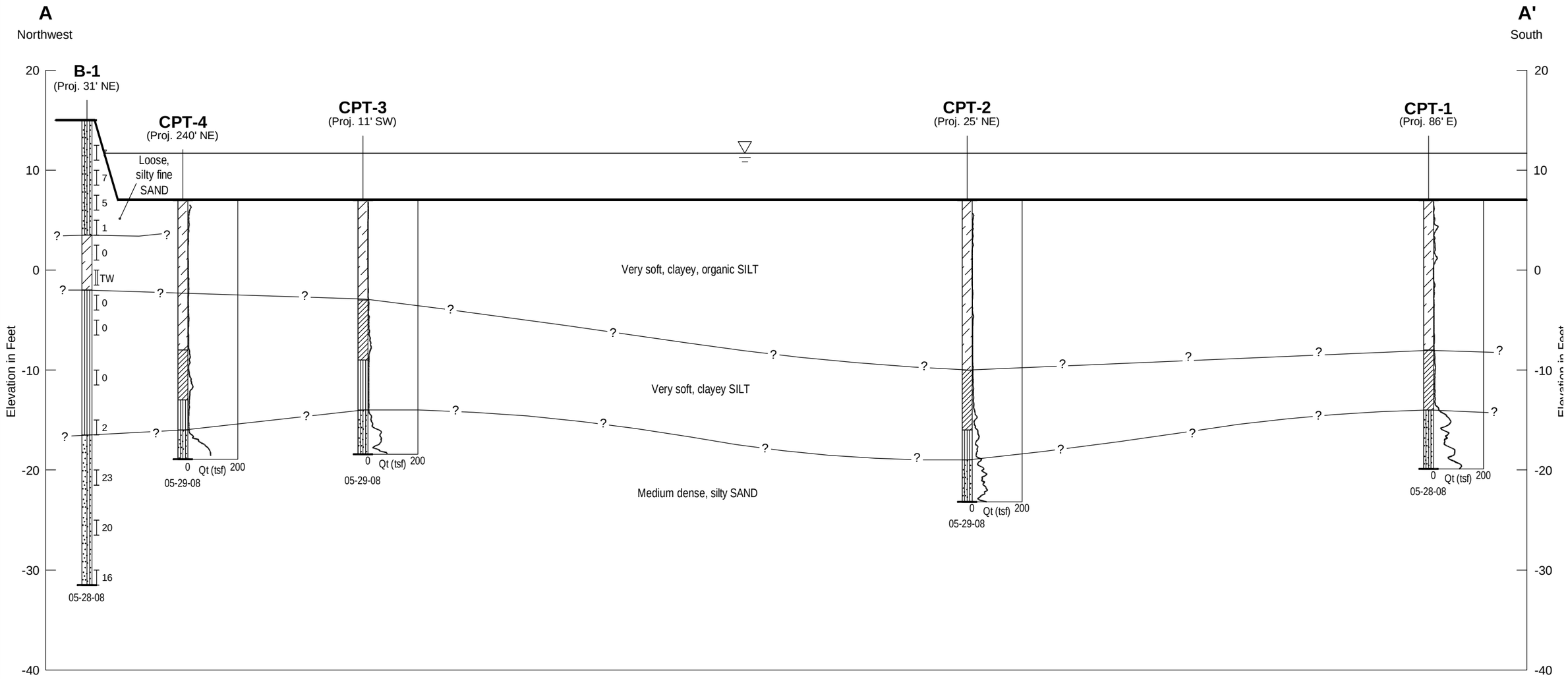
SITE AND EXPLORATION PLAN

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

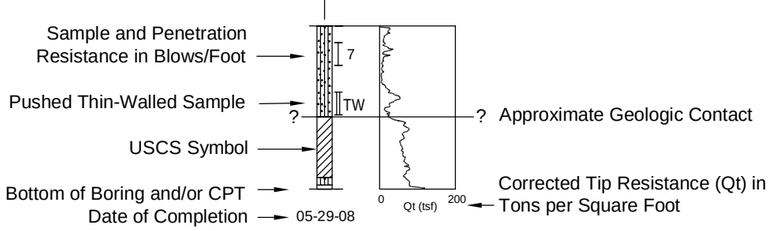
FIG. 2

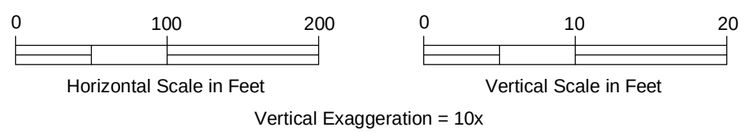
Filename: J:\211\21000-001\2008-December Report\21-1-21000-001 fig 2 (Current).dwg Date: 12-18-2008 Login: BAC



LEGEND

B-1 ← Designation of Boring and/or
CPT-1 ← Cone Penetration Test (CPT)
(Proj. 250' NE) ← Offset (Distance/Direction)





NOTE

This profile is generalized and based on interpretations of field explorations. Variations between conditions depicted on this profile and the actual conditions may exist.

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

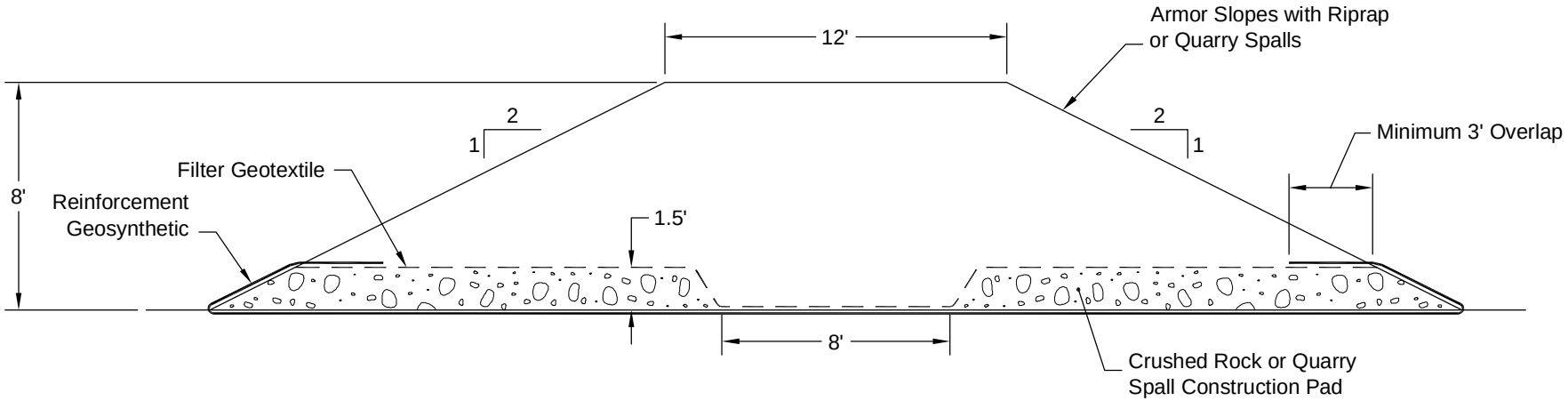
December 2008 | 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 3

Filename: J:\21\21000-001\2008-December Report\21-1-21000-001 Fig 3 (Current).dwg Date: 12-18-2008 Login: BAC

Filename: J:\211\21000-001\2008-December Report\21-1-21000-001 fig 4.dwg Date: 12-15-2008 Login: cnt



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

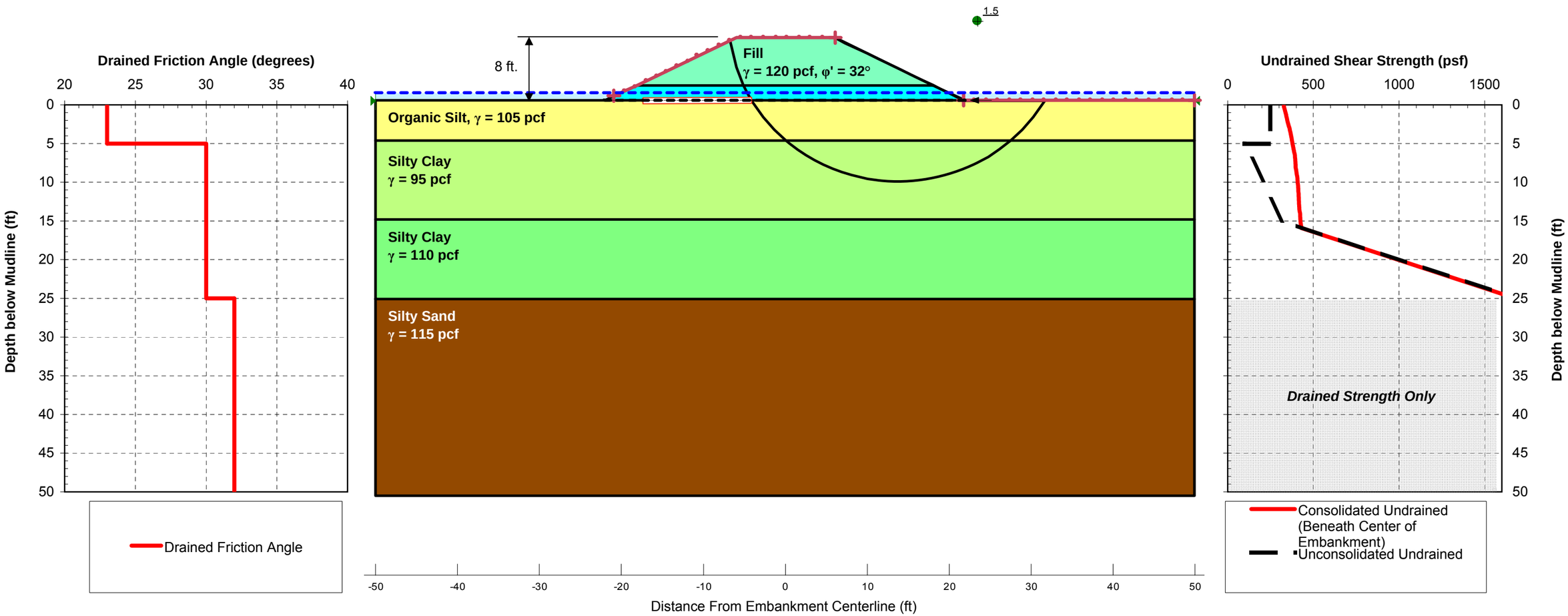
PROPOSED DIKE CROSS-SECTION

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4

FIG. 4



NOTES

1. The drained soil strength was used for fine-grained soils under long-term loading conditions and for granular soils under all loading conditions. The undrained soil strength was used for fine-grained soils under short-term loading conditions. The Unconsolidated Undrained shear strength was used for soil outside the dike footprint; the Consolidated Undrained shear strength was used for soil under the dike.

2. Soil strength and unit weight parameters were estimated from the results of SPT and CPT testing, laboratory testing, and published correlations.

3. γ = total unit weight
   φ' = drained angle of internal friction
   pcf = pounds per cubic foot
   ft = feet

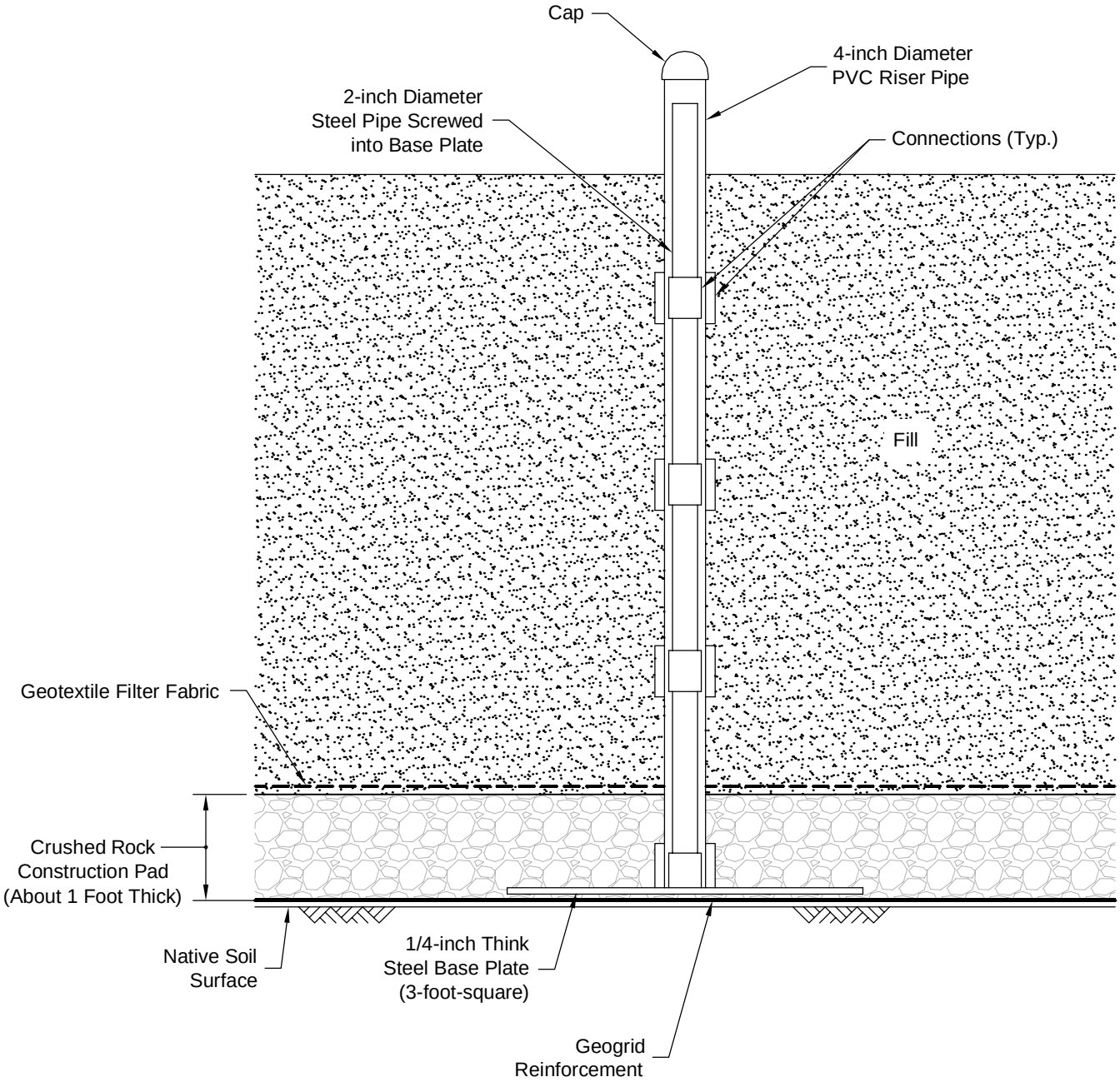
Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

DIKE STABILITY ANALYSIS
GEOMETRY AND SOIL PROPERTIES

December 2008 | 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 5



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

SETTLEMENT PLATE SCHEMATIC

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6

File: J:\211\21000-001\2008-December Report\21-1-21000-001 Fig 6.dwg Date: 12-12-2008 Author: sac

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

TABLE OF CONTENTS

Page

A.1 INTRODUCTION A-1

A.2 BORING A-1

A.3 CONE PENETRATION TEST (CPT) A-2
- A.3.1 Field Equipment A-2
- A.3.2 Testing Procedures A-3
- A.3.3 Interpretation A-4

A.4 VANE SHEAR TEST A-4

A.5 REFERENCE A-5

LIST OF FIGURES

Figure No.

A-1 Soil Classification and Log Key (2 sheets)
A-2 Log of Boring B-1
A-3 Cone Penetration Test Plot CPT-1
A-4 Cone Penetration Test Plot CPT-2
A-5 Cone Penetration Test Plot CPT-3
A-6 Cone Penetration Test Plot CPT-4
A-7 Log of Vane Shear Test VS-1

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 INTRODUCTION

The field explorations performed for this study consisted of drilling one boring on May 28, 2008, performing four cone penetration tests (CPTs) on May 28 and 29, 2008, and performing a series of vane shear tests in one location on May 29, 2008.

The boring was drilled on the north edge of the lagoon. The boring location was determined by taping from existing landmarks, and is shown in Figure 2 within the main text of this report. The Unified Soil Classification System (USCS), as described in Figure A-1, was used to classify the soils encountered in the boring.

The CPTs and vane shear tests were performed from a barge in the lagoon. Quigg Brothers Construction of Hoquiam, Washington, supplied and operated the barge. Quigg Brothers used a small crane to lower the barge into the lagoon. A small outboard boat towed the barge to each test location, where two cylindrical steel anchors were lowered into the lagoon bottom to secure the barge. Both the CPT and vane shear tests were performed through a gap about 1 foot wide at the middle of the barge. The test locations were determined by triangulation with landmarks placed on shore, shown in Figure 2 within the main text of this report. The depth of the water at each location was determined by lowering a weighted tape measure into the water.

A.2 BORING

One boring, drilled to a depth of 45 feet and designated B-1, was drilled to evaluate the subsurface conditions at the north edge of the lagoon, and to supplement the other subsurface explorations. The boring log is presented as Figure A-2.

Gregory Drilling, Inc., under subcontract to Shannon & Wilson, Inc., drilled the boring using a CME 85 truck-mounted drill rig and mud-rotary drilling techniques. The mud-rotary method consists of drilling the subsurface soils and removing the cuttings by circulation of abentonite/ water mix drilling mud. A settling tank at the ground surface collected the cuttings while the

mud was recirculated into the boring. Gregory Drilling, Inc. spread the drill cuttings on the ground next to the boring location.

The samples from the boring were obtained in conjunction with the Standard Penetration Test (SPT). SPTs were performed in general accordance with the ASTM International (ASTM) Designation: D 1586, generally at 2.5-foot intervals. This test consists 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 number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). When penetration resistances exceeded 50 blows for 6 inches or less of penetration, the test was terminated. The penetration resistances were recorded by our field representative and are plotted on the boring logs. These values provide a means for evaluating the relative density or compactness of cohesionless (granular) soils and the consistency (stiffness) of cohesive soils as described in Figure A-1.

The split-spoon sampler used during the penetration testing recovers a disturbed sample of the soil, which is useful for identification purposes. The samples were sealed in jars and returned to our laboratory for testing.

A.3 CONE PENETRATION TEST (CPT)

Four CPT probes, designated CPT-1 through CPT-4, were performed to obtain subsurface information in the lagoon area. The electric piezocone test develops a continuous subsurface profile of soil conditions at a particular location. The testing was performed by In Situ Engineering, Inc. (In Situ) in general accordance with procedures outlined in ASTM Designation: D 5778-07, Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils. Soil samples are not obtained in this test method. Each probe was pushed until refusal was met, generally at a depth of 25 to 30 feet below the mudline. Logs of the CPT probes are presented in Figures A-3 through A-6.

A.3.1 Field Equipment

The piezocone apparatus used by In Situ is a Hogentogler system. In this test, steel rods with a cone tip on the end are pushed hydraulically into the soil at a relatively constant rate of approximately 2 centimeters per second (cm/sec) (0.8 inch per second [in/sec]). Readings are

recorded every 5 cm (2 inches [in]). The cone tip is connected to a stationary friction sleeve and has a cross-sectional area of 10 square centimeters (cm^2) (1.5 square inches [in^2]), a surface area of 15 cm^2 (2.3 in^2), and an angle of 30 degrees from the axis. The area ratio for the tip was 0.8. The stationary friction sleeve had the same diameter as the cone tip but a surface area of 150 cm^2 (23 in^2). The cone tip and friction sleeve assembly is about 50 cm (18 in) long and is pushed into the ground by steel rods about 1 meter (3 feet) long. An electronic cable is prestrung through the rods. The cable provides power to the instruments and communication between the instrument and a computer. The entire system was powered by a gasoline-powered generator.

The CPT instrument is capable of recording tip resistance, sleeve friction, pore pressure, and inclination as it penetrates into the ground. The cone has a tip capacity of 10 tons, or approximately 1,000 tons per square foot (tsf). Tip accuracy is approximately 0.1 tsf. The friction sleeve has a capacity of 10 tsf with an accuracy of 0.01 tsf. The cone is a subtraction-type cone, which senses the tip resistance on one set of strain gauges and senses tip resistance plus side friction on another set of gauges. The frictional reading is determined by subtracting the tip reading from the combined reading. The pore pressure sensor has a capacity of 500 pounds per square foot, with an accuracy of 0.1 pound per square inch. The inclinometer has a full range capability of 10 degrees, with an accuracy of about 0.1 degree.

The pore pressure filter element, located behind the cone tip, is a high-air-entry polypropylene disk that is discarded and replaced after every test hole. Disks are presaturated by subjecting them to a vacuum of 25 to 28 inches of mercury for 30 minutes while submerged in a 50 percent solution of glycerin and water. This filter element transmits pore pressures to the pressure transducer located within the cone tip.

A.3.2 Testing Procedures

The cone system rods were hydraulically pushed into the ground under the action of the barge. As the cone penetrated through the soil, measurements of tip resistance, sleeve friction, pore pressure, and inclination were electronically transmitted through the electronic cable to the ground surface, and then displayed and recorded on a portable computer.

The cone tip, filter element, and friction sleeve assemblies were disassembled and cleaned between holes. A new pore pressure filter element was placed in the assembly prior to each hole, and pore pressure cavities were filled with a 50 percent solution of glycerin and water.

A syringe was used in filling void spaces to assist in removing air bubbles and increasing saturation.

A.3.3 Interpretation

The CPT data consist of cone tip resistance, sleeve friction, friction ratio (ratio of sleeve friction to cone tip resistance), and pore pressure versus depth. These data were processed and interpreted by Shannon & Wilson, Inc. and In Situ. Soil parameters were estimated based on published correlations, as shown in the following table.

SOIL PARAMETERS ESTIMATED FROM CONE PENETRATION TESTS

Soil Parameter	Published Reference and Year
Soil Behavior Type (Classification)	Robertson and Campanella, 1989 Robertson and Campanella, 1983
Angle of Internal Friction	Kulhawy, 1994
Equivalent SPT N-value (uncorrected)	Robertson and Campanella, 1989
Undrained Shear Strength	Robertson and Campanella, 1989

Note: SPT = Standard Penetration Test

A.4 VANE SHEAR TEST

A vane shear test, designated VS-1, was performed at the location of CPT-2 to obtain information about the undrained shear strength of the lagoon soils. The vane shear test consists of placing a four-bladed vane into the undisturbed soil and rotating it from the surface to determine the torsional force required to cause a cylindrical surface to be sheared by the vane. The torsional force is measured with a torque wrench and is then converted to a unit shearing resistance of the cylindrical surface. The testing was performed by Shannon & Wilson, Inc. personnel in general accordance with procedures outlined in ASTM Designation: D 2573-94, Standard Test Method for Field Vane Shear Test in Cohesive Soil. Soil samples are not obtained in this test method. The vane was advanced until refusal was met at a depth of 12 feet below the mudline; tests were performed at 1- to 3-foot depth intervals. A log of the vane shear testing is presented in Figure A-7.

A.5 REFERENCE

ASTM International (ASTM), 2007, 2007 Annual book of standards, construction: West Conshohocken, Pa., volume 04.08 Soil and Rock.

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 consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., 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
FeO	Iron Oxide
MgO	Magnesium Oxide
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
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

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

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 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

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

SOIL CLASSIFICATION AND LOG KEY

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

BORING_CLASS1 21-21000.GPJ SWNEW.GDT 6/18/08

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW		Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with 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 *(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 Fines *(more than 12% fines)*	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sand-clay 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, sandy silts, gravelly silts, 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	MH		Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH		Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		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 ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

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.

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

SOIL CLASSIFICATION AND LOG KEY

June 2008 | 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 2 of 2

BORING_CLASS2 21-21000.GPJ SWNEW.GDT 6/18/08

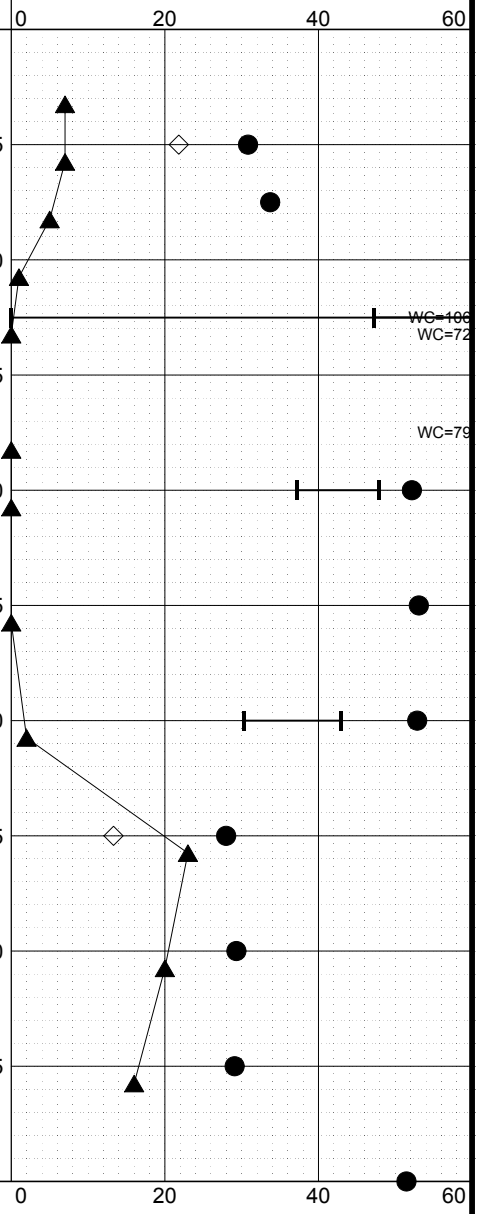
Total Depth:	46.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	4 in.
Top Elevation:	~ 10 ft.	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Truck Rig	Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION	Depth, ft.
Loose to very loose, gray, silty, fine SAND; wet; laminated, scattered wood fragments; SM.	
Very soft, black to dark gray, clayey, organic SILT; moist; bedded, scattered wood and shell fragments, abundant organics; OH.	11.5
Very soft, dark to light gray, trace to slightly fine sandy, clayey silt; moist; bedded, scattered wood and shell fragments, increasing frequency of sand seams with depth; ML.	17.0
Medium dense, dark gray, slightly silty to silty, fine SAND; wet; widely scattered silt seams, wood fragments, and shell fragments; SP-SM/SM.	31.5
BOTTOM OF BORING COMPLETED 5/28/2008	46.5

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

Ground Water: None Observed During Drilling



LEGEND

* Sample Not Recovered

Standard Penetration Test

Thin Wall Sample

NOTES

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

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

LOG OF BORING B-1

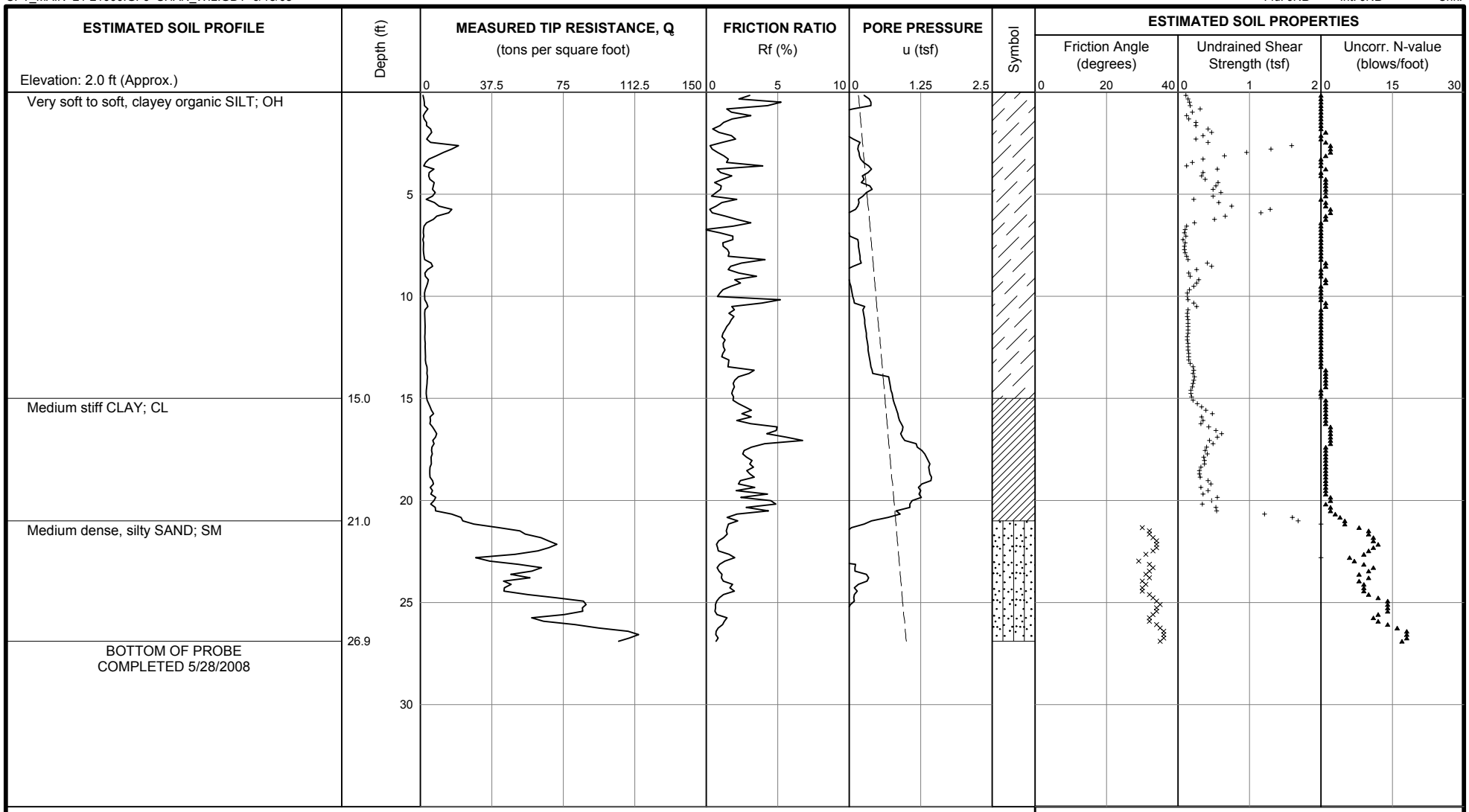
June 2008 | 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

Log: JNB Rev: JNB Typ: LKD

MASTER_LOG_E_21-21000.GPJ SHAN_WIL.GDT 6/18/08



NOTES:

1. The stratification lines represent the approximate boundaries between soil types; the transition may be gradual.
2. The estimated soil properties are based on analyses performed using published correlations and equations. The method used for estimating the properties listed above are:

Property	Method
Friction Angle	Durgunoglu & Mitchell
Uncorrected N-Value (N60)	Robertson & Campanella
Undrained Shear Strength	$\frac{Qt - \sigma v}{Nkt}$ where: Qt = Corrected Cone Bearing; Nkt = 12.5; σv = Total Overburden Stress

3. Log of probe is based on piezocone probe data provided by Northwest Cone Exploration.
4. The pore pressure was measured behind the tip of the penetrometer. Hydrostatic pore pressure based on the estimated groundwater depth is also shown above (dashed line).

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

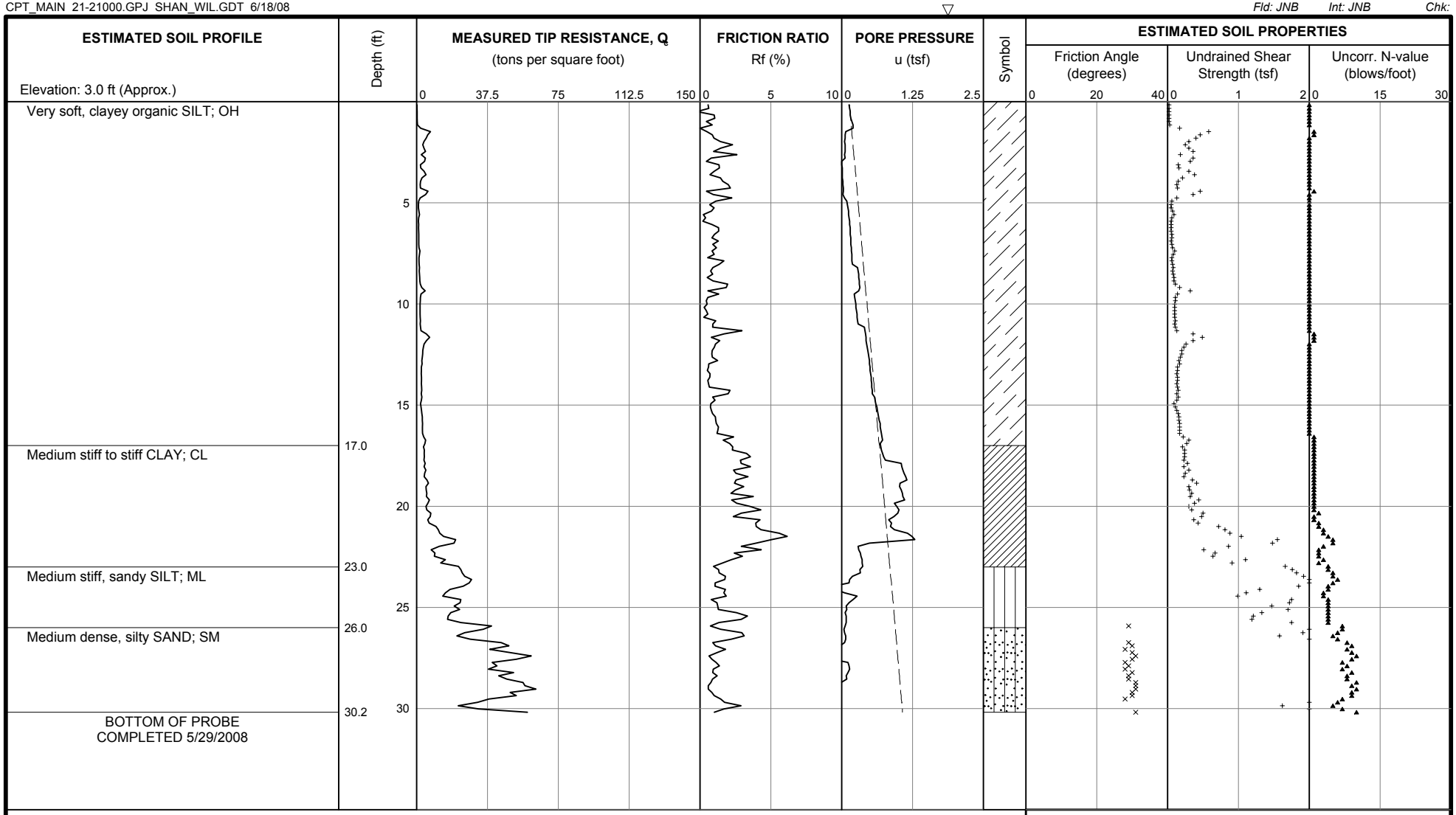
LOG OF PROBE CPT-1

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3

FIG. A-3



NOTES:

1. The stratification lines represent the approximate boundaries between soil types; the transition may be gradual.
2. The estimated soil properties are based on analyses performed using published correlations and equations. The method used for estimating the properties listed above are:

Property	Method
Friction Angle	Durgunoglu & Mitchell
Uncorrected N-Value (N60)	Robertson & Campanella
Undrained Shear Strength	$\frac{Qt - \sigma v}{Nkt}$ where: Qt = Corrected Cone Bearing; Nkt = 12.5; σv = Total Overburden Stress

3. Log of probe is based on piezocone probe data provided by Northwest Cone Exploration.
4. The pore pressure was measured behind the tip of the penetrometer. Hydrostatic pore pressure based on the estimated groundwater depth is also shown above (dashed line).

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

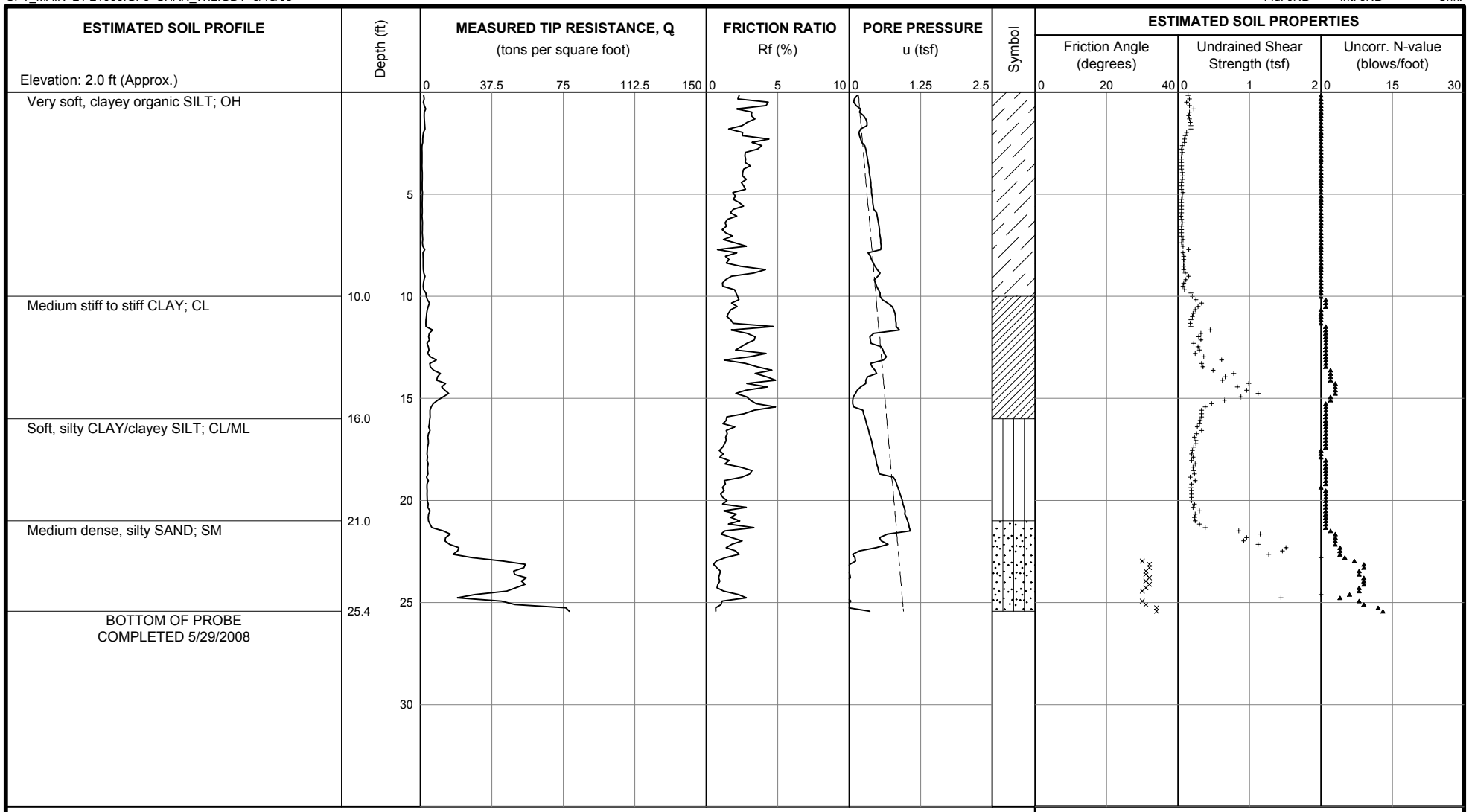
LOG OF PROBE CPT-2

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-4

FIG. A-4



NOTES:

1. The stratification lines represent the approximate boundaries between soil types; the transition may be gradual.
2. The estimated soil properties are based on analyses performed using published correlations and equations. The method used for estimating the properties listed above are:

Property	Method
Friction Angle	Durgunoglu & Mitchell
Uncorrected N-Value (N60)	Robertson & Campanella
Undrained Shear Strength	$\frac{Qt - \sigma v}{Nkt}$ where: Qt = Corrected Cone Bearing; Nkt = 12.5; σv = Total Overburden Stress

3. Log of probe is based on piezocone probe data provided by Northwest Cone Exploration.
4. The pore pressure was measured behind the tip of the penetrometer. Hydrostatic pore pressure based on the estimated groundwater depth is also shown above (dashed line).

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

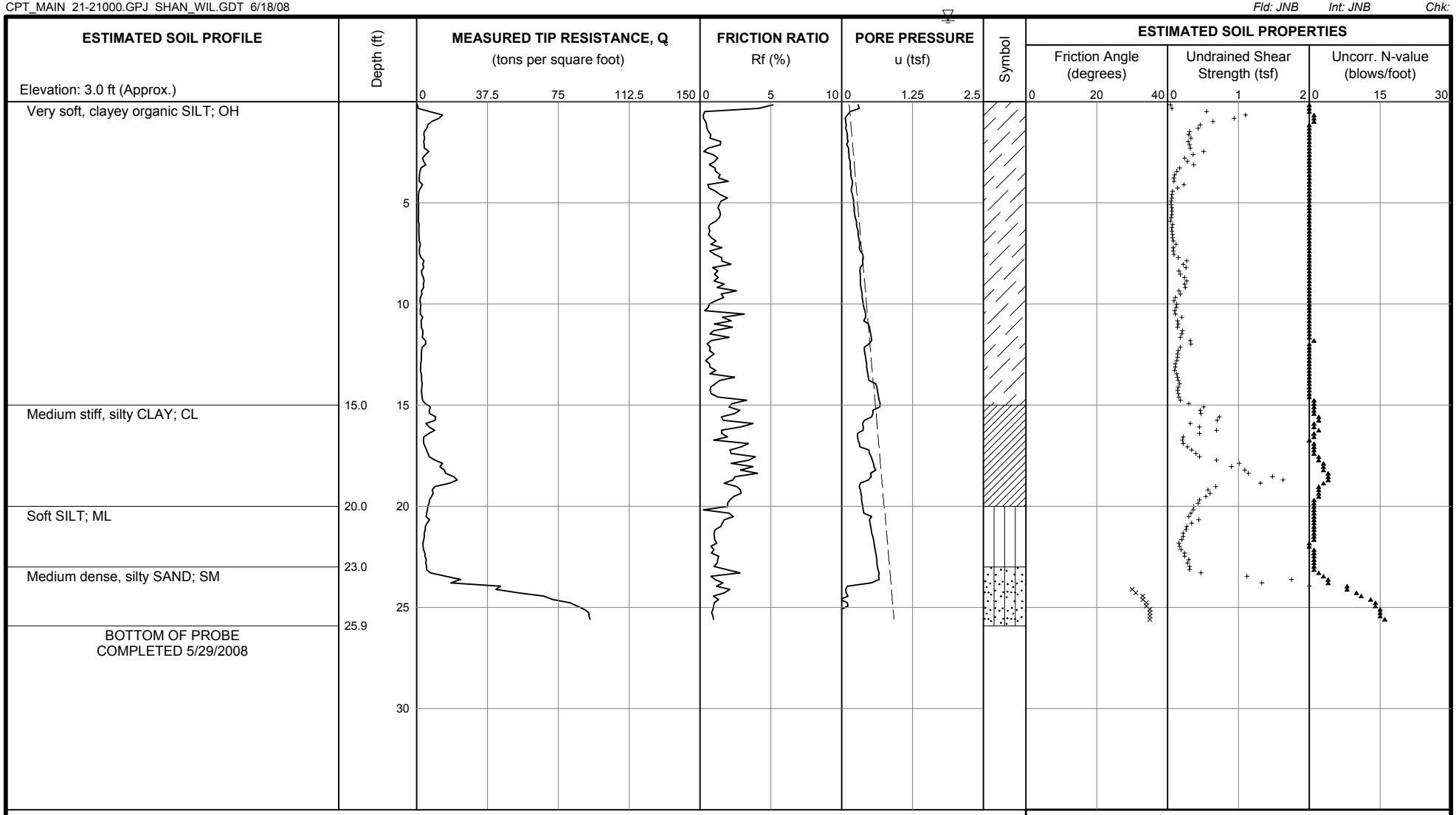
LOG OF PROBE CPT-3

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5

FIG. A-5



NOTES:

1. The stratification lines represent the approximate boundaries between soil types; the transition may be gradual.
2. The estimated soil properties are based on analyses performed using published correlations and equations. The method used for estimating the properties listed above are:

Property	Method
Friction Angle	Durgunoglu & Mitchell
Uncorrected N-Value (N60)	Robertson & Campanella
Undrained Shear Strength	$\frac{Qt - \sigma v}{Nkt}$ where: Qt = Corrected Cone Bearing; Nkt = 12.5; σv = Total Overburden Stress

3. Log of probe is based on piezocone probe data provided by Northwest Cone Exploration.
4. The pore pressure was measured behind the tip of the penetrometer. Hydrostatic pore pressure based on the estimated groundwater depth is also shown above (dashed line).

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

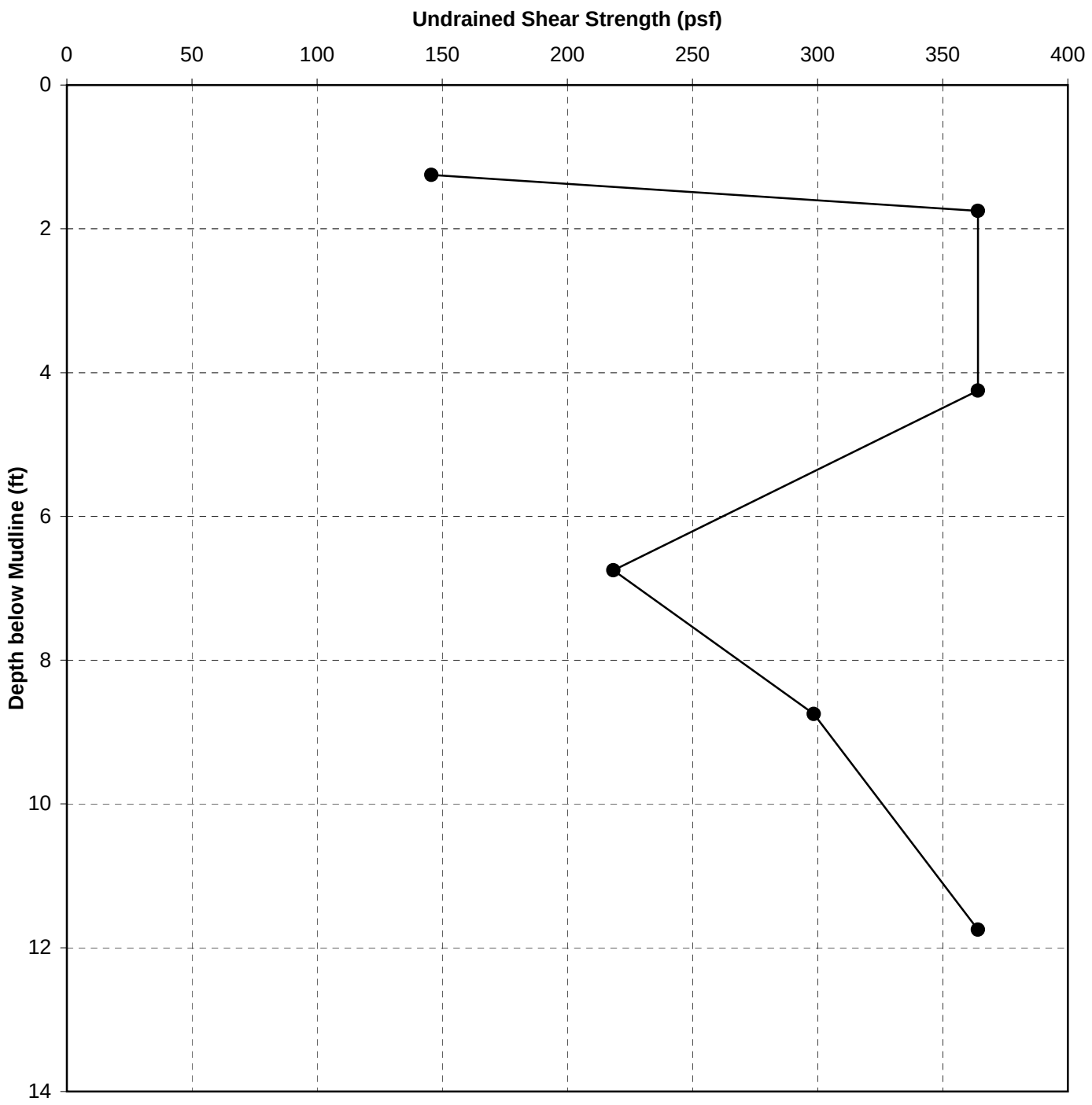
LOG OF PROBE CPT-4

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-6

FIG. A-6



NOTES:

1. The test was conducted with a vane 5 inches high by 3.5 inches wide.
2. The depth to the mudline was 4.3 feet.
3. ft - feet
   psf = pounds per square foot

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

LOG OF VANE SHEAR TEST VS-1

June 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-7

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

TABLE OF CONTENTS

Page

B.1 INTRODUCTION B-1

B.2 VISUAL CLASSIFICATION B-1

B.3 WATER CONTENT DETERMINATION B-1

B.4 GRAIN SIZE ANALYSIS B-2

B.5 ATTERBERG LIMIT DETERMINATION B-2

B.6 ONE-DIMENSIONAL CONSOLIDATION TEST B-2

B.7 MOISTURE-DENSITY TEST B-3

B.8 REFERENCES B-3

LIST OF FIGURES

Figure No.

B-1 Grain Size Distribution, Boring B-1
B-2 Plasticity Chart, Boring B-1
B-3 Consolidation Test, Boring B-1 (8 pages)
B-4 Grain Size Distribution, Proposed Fill Material
B-5 Moisture-Density Test, Proposed Fill Material

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

B.1 INTRODUCTION

This appendix contains descriptions of the procedures and the results of the geotechnical laboratory tests performed on soil samples obtained from the boring performed for the Hoquiam Wastewater Treatment Facility Biosolids Lagoon Project. The samples were tested to evaluate the basic index and physical properties of the soils. The laboratory testing was performed by an engineer or experienced technician at the Shannon & Wilson, Inc. laboratory in Seattle in June 2008.

B.2 VISUAL CLASSIFICATION

All of the soil samples recovered from the borings were visually reclassified in our laboratory using a system based on ASTM International (ASTM) Designation: D-2487, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D-2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). This visual classification method allows for convenient and consistent comparison of soils from widespread geographic areas. Using this method, the soils can be classified by using the Unified Soil Classification System. The individual sample classifications have been incorporated into the boring logs presented in Appendix A.

B.3 WATER CONTENT DETERMINATION

The natural water content of all soil samples recovered from the boring was determined in general accordance with ASTM Designation: D-2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. The water content is plotted on the boring logs presented in Appendix A.

B.4 GRAIN SIZE ANALYSIS

Grain size analyses were performed on selected samples of granular soil in general accordance with ASTM Designation: D-422, Standard Method for Particle-Size Analysis of Soils. The general procedures to determine the grain size distribution of a soil include sieve analysis, hydrometer analysis, and combined analysis. For this project, only sieve analyses were performed.

Grain size distribution is used to assist in classifying soils and evaluating their potential to be used as backfill and to provide correlation with soil properties. The results of the grain size analyses are plotted on the grain size distribution curves presented in Figures B-1 and B-4.

B.5 ATTERBERG LIMIT DETERMINATION

Atterberg Limits were determined on selected samples of fine-grained soils obtained in the borings and test pits in general accordance with ASTM D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg Limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (LL – PL = PI). They are generally used to assist in classification of soils, to indicate soil consistency (when compared with natural water content), and to provide correlation to soil properties including compressibility and strength. The results of the Atterberg Limits determinations are shown in the boring log and are shown graphically on the plasticity chart presented in Figure B-2.

B.6 ONE-DIMENSIONAL CONSOLIDATION TEST

A one-dimensional consolidation test was performed on a relatively undisturbed, 3-inch outside-diameter, thin-walled (Shelby) tube sample retrieved from boring B-1. The test was performed in general accordance with ASTM D 2435, Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Incremental Loading. The soil specimen was incrementally loaded using a fixed-ring consolidometer, with each load increment approximately doubling the previous load. Drainage was allowed from both the top and bottom of the specimen. The specimen was saturated with water before any load increments were applied. Additional load increments were applied at the end of primary consolidation, or at the end of an approximately 24-hour period during testing, whichever occurred first. Upon reaching the maximum normal stress, the specimen was unloaded in increments of about one-fourth of the previous load. During each loading period, the change in sample height with time was recorded.

The test data were analyzed to evaluate the consolidation properties of the soil specimen in general accordance with the methods established by Casagrande and others (Casagrande, 1936, and Casagrande and Fadum, 1940). The results of the one-dimensional consolidation test are presented in Figure B-3.

B.7 MOISTURE-DENSITY TEST

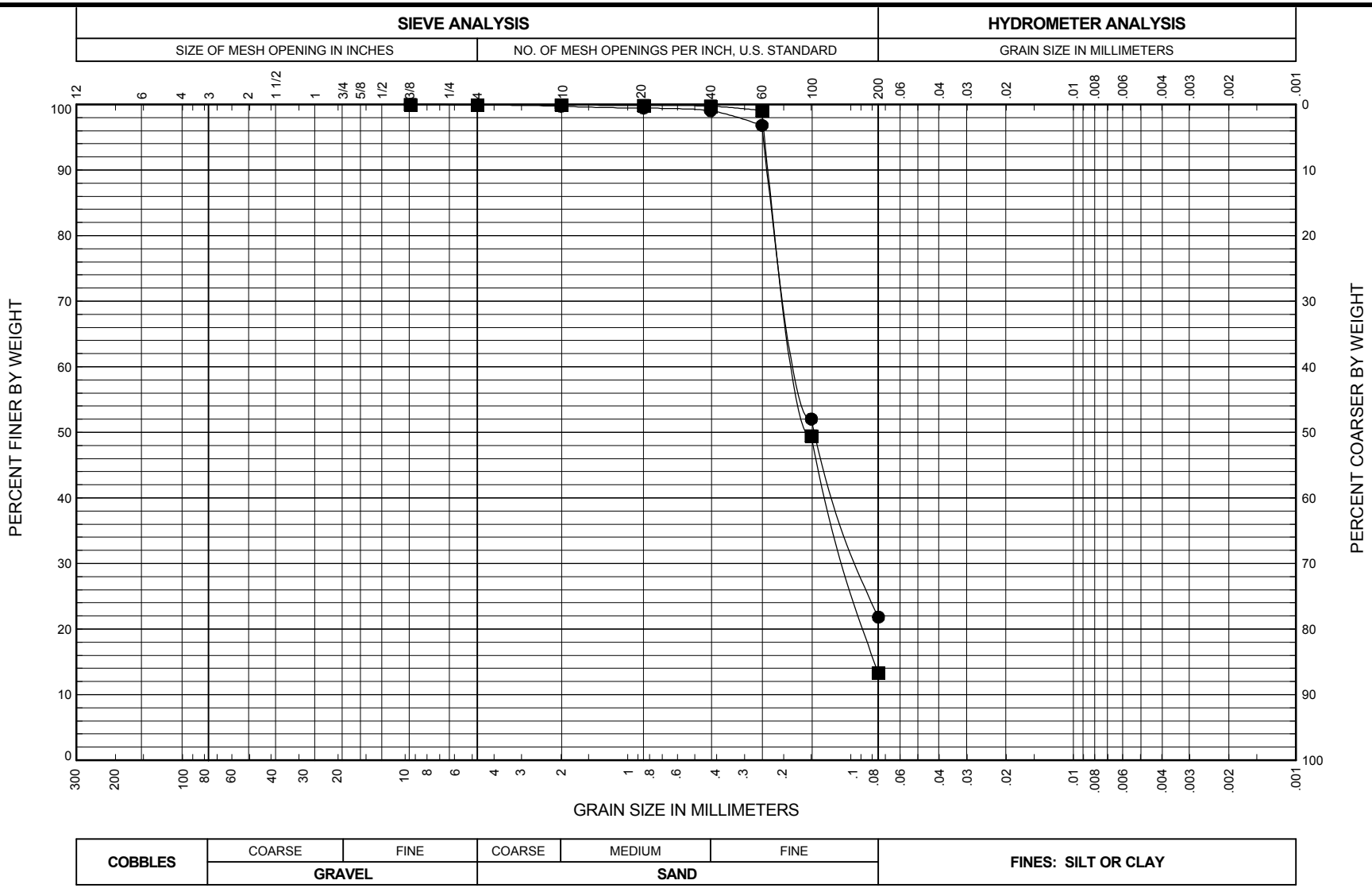
Laboratory compaction tests were conducted in accordance with ASTM D 1557 (modified Proctor) to determine the relationship between water content and dry unit weight of soils. A compaction curve was plotted based on several trials of compacting different samples of the same soil at different levels of moisture content. The values of optimum water content and maximum dry unit weight are determined from the compaction curve. The results of the moisture-density test are presented in Figure B-5.

B.8 REFERENCES

ASTM International (ASTM), 2006, Annual book of standards, Construction, v. 04.08, Soil and rock (I): D 420 – D 5611: West Conshohocken, Pa.

Casagrande, Arthur, 1936, The determination of pre-consolidation load and its practical significance, *in* International Conference on Soil Mechanics and Foundation Engineering, Harvard University, Mass., 1936, Proceedings: Cambridge, Mass., p. 60-64.

Casagrande, Arthur, and Fadum, R.E., 1940, Notes on soil testing for engineering purposes: Cambridge, Mass., Harvard University Graduate School of Engineering, Soil Mechanics Series no. 8, 74 p.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-1, S-2	5.0	SM	Gray, silty SAND, trace of gravel	21.8	30.8			
■ B-1, S-11	35.0	SM	Gray, silty SAND, trace of gravel; trace of organics	13.3	28.0			

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

GRAIN SIZE DISTRIBUTION
BORING B-1

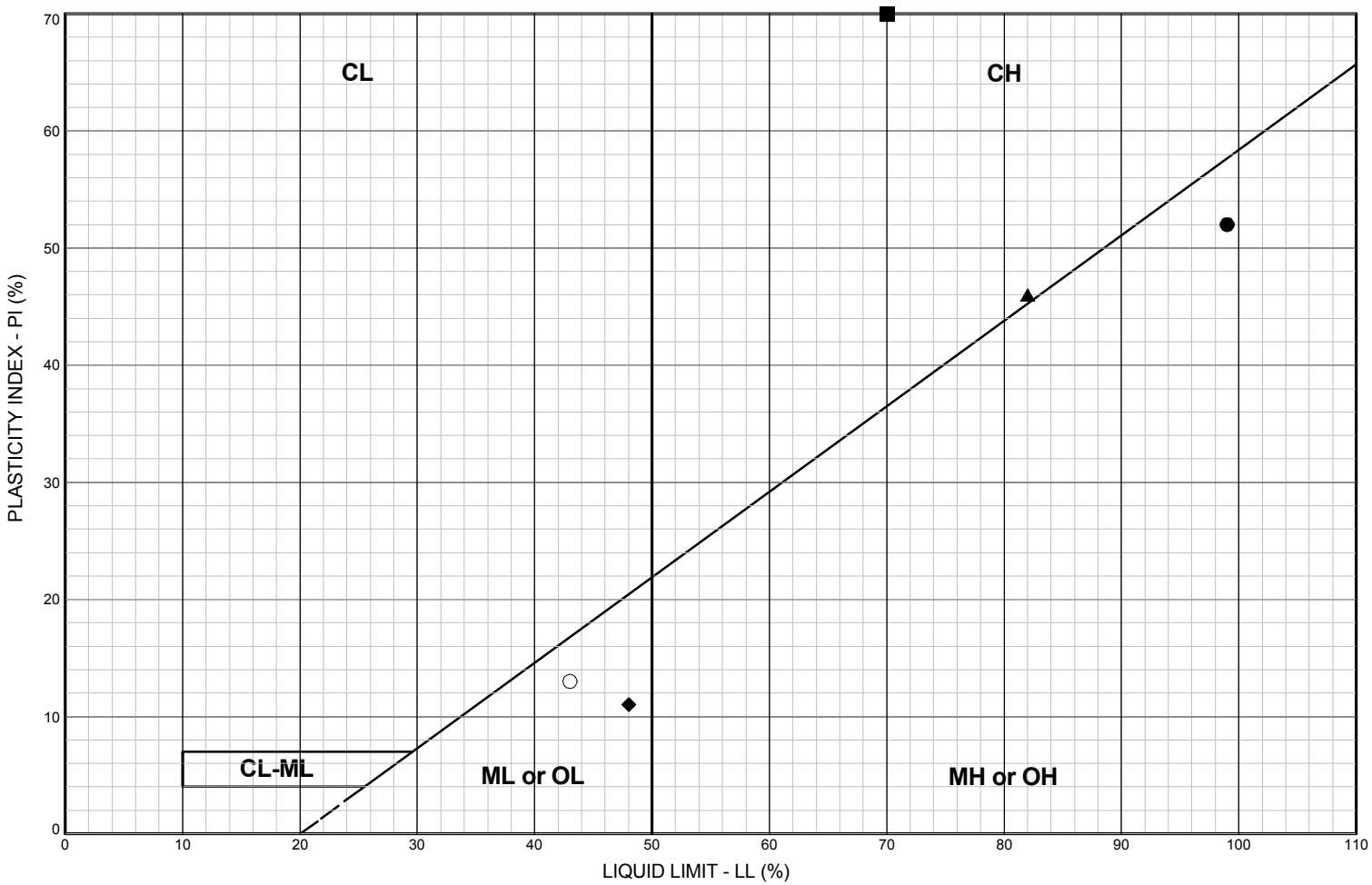
December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-1

GSA_MAIN 21-21000.GPJ SHAN_WIL.GDT 12/10/08

FIG. B-1



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, %
● B-1, S-5	12.5	MH/OH	Dark gray-brown, clayey, organic SILT; abundant organics	99	47	52	105.9	
■ B-1, S-5	12.5	OH	Organic limit- oven dried liquid limit 71% of natural liquid limit	70	NP	70		
▲ B-1, S-6	15.8	CH/OH	Black, silty, organic CLAY	82	36	46	88.1	
◆ B-1, S-8	20.0	ML	Gray, clayey SILT, trace of fine sand	48	37	11	52.2	
○ B-1, S-10	30.0	ML	Gray, slightly fine sandy, clayey SILT; trace of wood fragments	43	30	13	52.9	

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

PLASTICITY CHART
BORING B-1

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2

FIG. B-2

ATT_MAIN 21-21000.GPJ SHAN WIL.GDT 12/10/08

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1

SUMMARY OF TEST DATA

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AJC 6/23/08

CLASSIFICATION:
Soft, black, silty, organic CLAY; moist; abundant organics; micaceous; CH/OH. (Estimated Specific Gravity does not take organics into account)

SAMPLE DATA:

Spec. Grav. (est.) :	2.65
Liquid Limit :	82
Plastic Limit :	36
Plasticity Index :	46
Specimen :	UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	.785	.616
Diameter, inches :	2.508	2.508
Wet Density, pcf :	95.1	105.2
Dry Density, pcf :	52.8	67.3
Water Content, % :	80.1	56.4
Void Ratio :	2.129	1.458
Saturation, % :	100	102

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
.1	3.0	.4	.03	.1	2.129	.3	2.7	1.09E-02	
.2	7.2	1.6	.06	.3	2.124	.4	6.2	8.12E-03	3.80E-07
.4	16.7	4.0	.13	.6	2.113	.3	14.1	1.08E-02	5.99E-07
.8	30.4	7.7	.26	1.1	2.098	.3	26.3	1.07E-02	4.16E-07
1.6	51.5	15.7	.51	1.8	2.077	.3	45.2	1.06E-02	2.70E-07
3.2	87.1	25.1	1.03	3.1	2.036	.4	75.1	7.75E-03	1.98E-07
6.4	262.6	33.2	2.06	11.5	1.773	1.4	182.3	2.00E-03	1.63E-07
1.6	243.2	28.0	.51	10.8	1.795	.2	250.2	1.29E-02	
.4	204.2	22.2	.13	9.1	1.847	1.4	221.2	1.89E-03	
.1	167.3	17.7	.03	7.5	1.898	3.7	185.9	7.39E-04	
.4	170.7	18.9	.13	7.6	1.895	.4	169.4	6.97E-03	7.98E-08
1.6	204.7	24.3	.51	9.1	1.850	.9	190.0	3.05E-03	1.13E-07
6.4	287.8	33.4	2.06	12.8	1.733	.6	251.4	4.31E-03	1.04E-07
12.8	439.2	39.7	4.11	20.0	1.505	1.4	368.0	1.63E-03	5.76E-08
25.6	585.0	46.5	8.23	27.0	1.287	1.2	515.0	1.59E-03	2.70E-08
6.4	563.2	40.3	2.06	26.2	1.311	.4	569.2	4.40E-03	
1.6	519.0	30.3	.51	24.5	1.365	1.7	537.0	1.07E-03	
.4	457.7	27.8	.13	21.6	1.458	3.5	488.0	5.52E-04	

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 21-1-21000-001

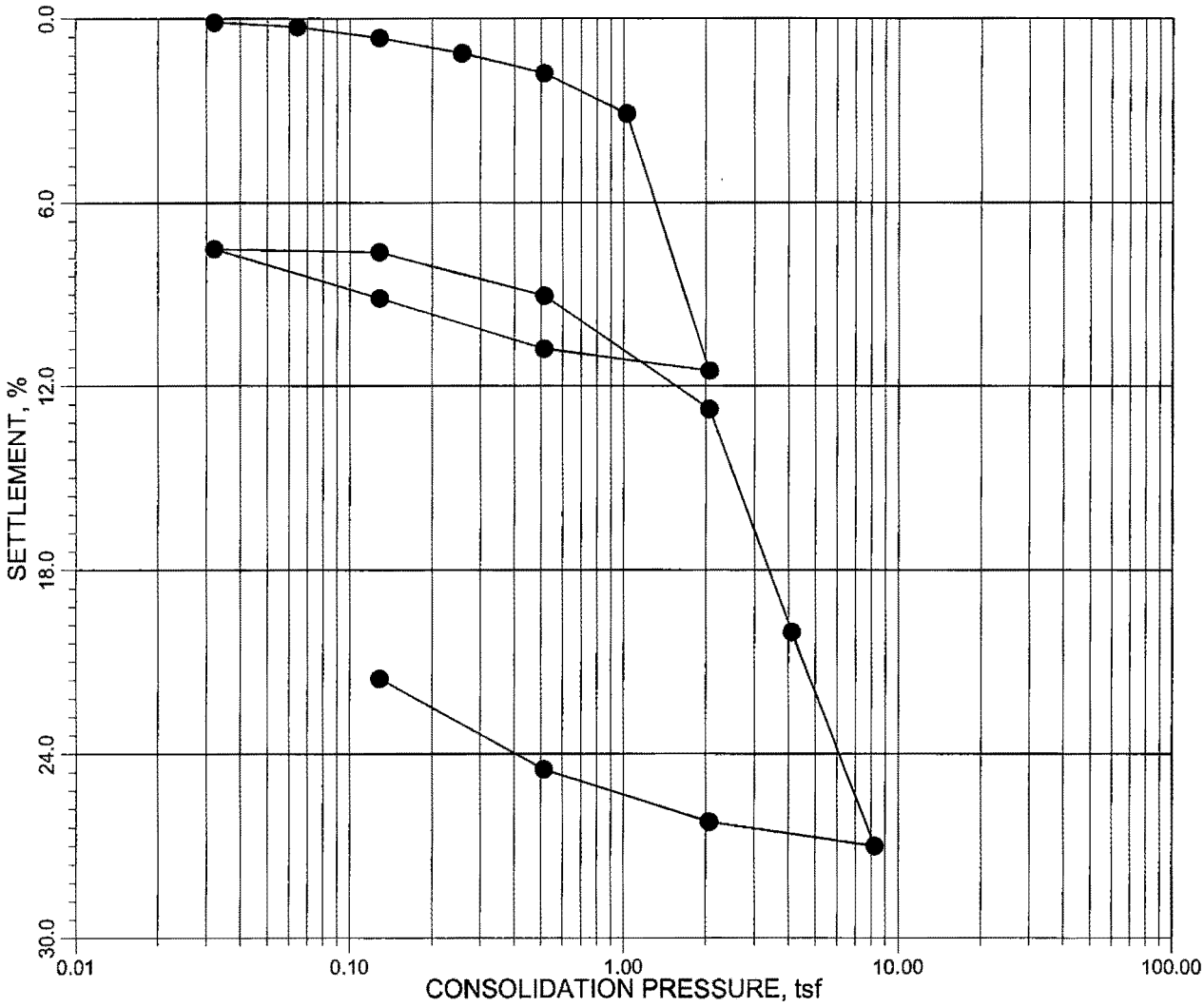
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(1/8)

I:\Groups\Lab\LabProgm\Projects\21-21000\B1S7158.ODC :: 6/20/2008 2:06:58 PM

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

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AJZ 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

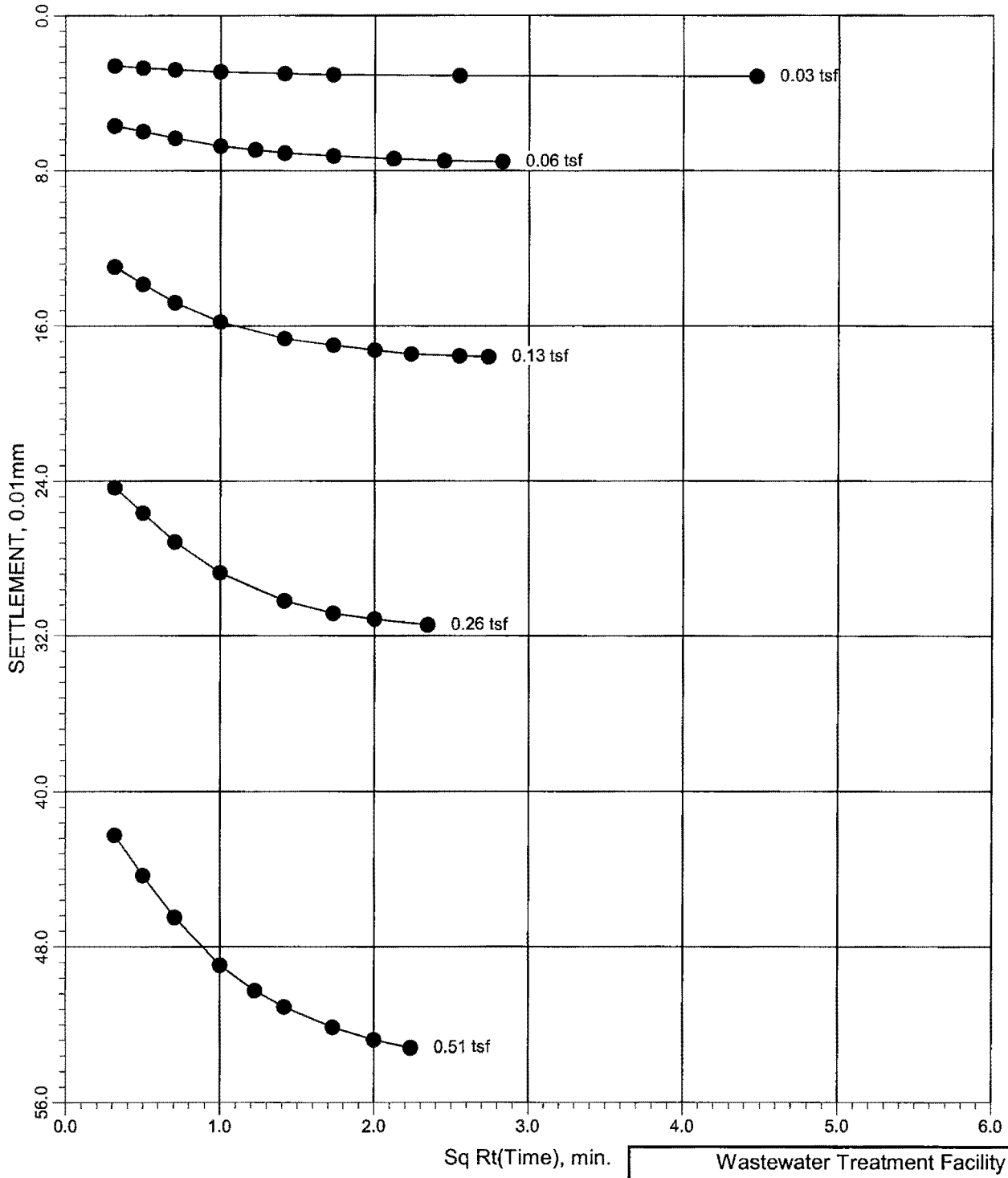
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(2/8)

I:\Groups\Lab\LabFrom\Projects\21-21000\B1S7158.ODC : 6/20/2008 2:58:23 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	ATR 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

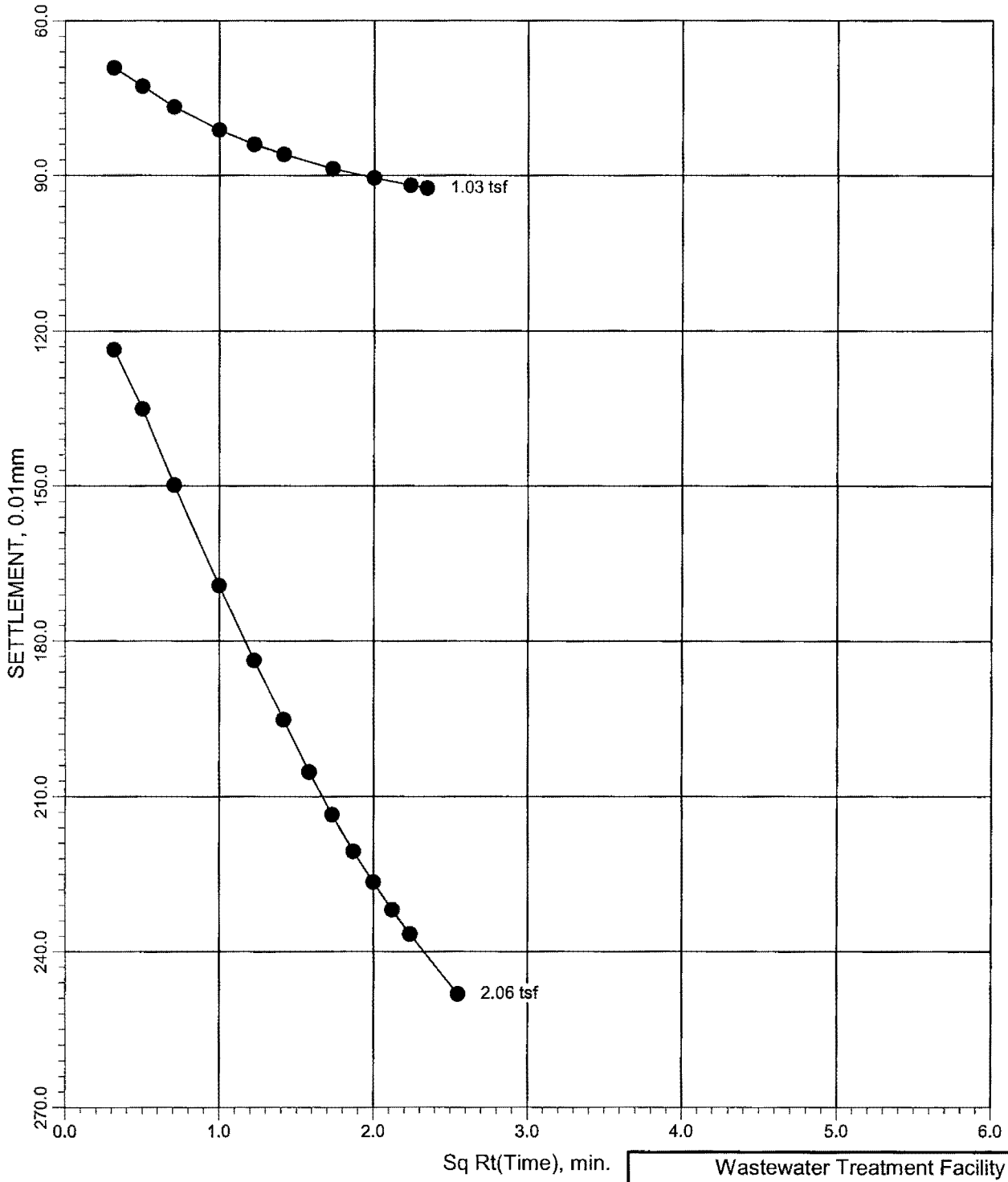
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(3/8)

I:\Groups\Lab\LabProgm\Projects\21-21000\B1\S7158.ODC :: 6/20/2008 2:58:39 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AJZ 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

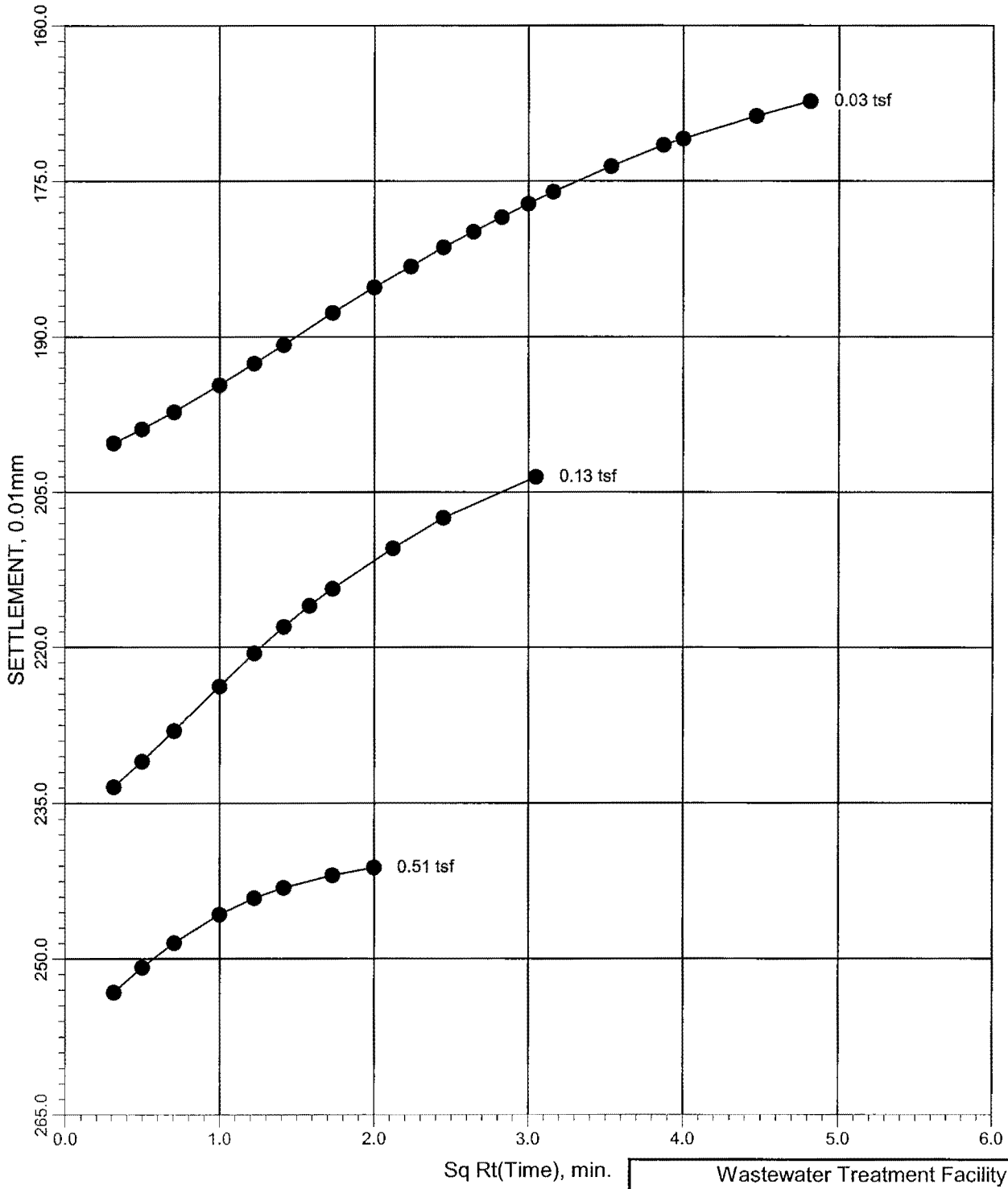
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(4/8)

I:\Groups\Lab\LabProgm\Projects\21-21000\B1S7158.ODC :: 6/20/2008 2:58:54 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AJR 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

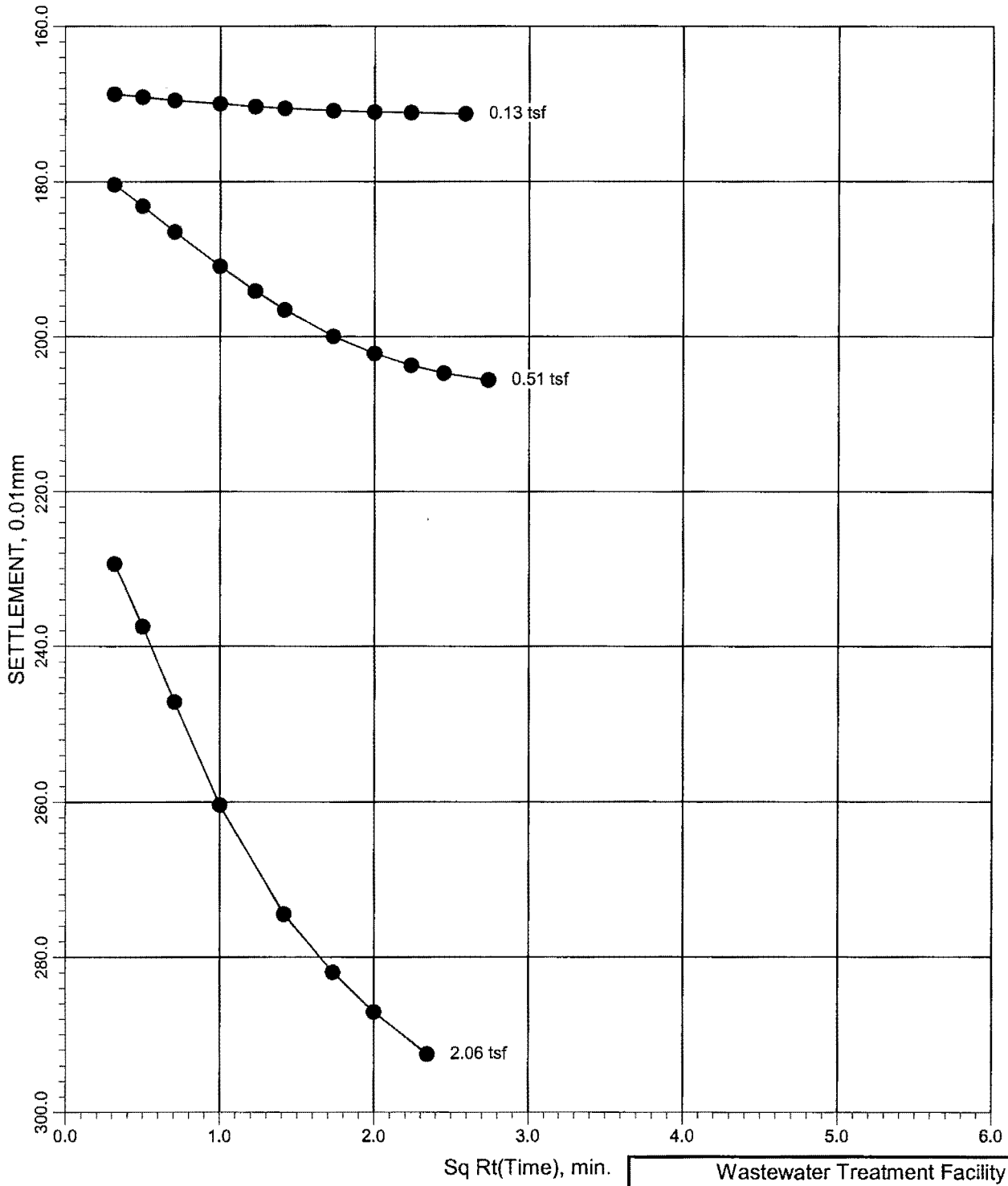
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(5/8)

I:\Groups\Lab\LabProgs\Projects\21-21000\B1S7155.ODC - 6/20/2008 2:59:01 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AIZ 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

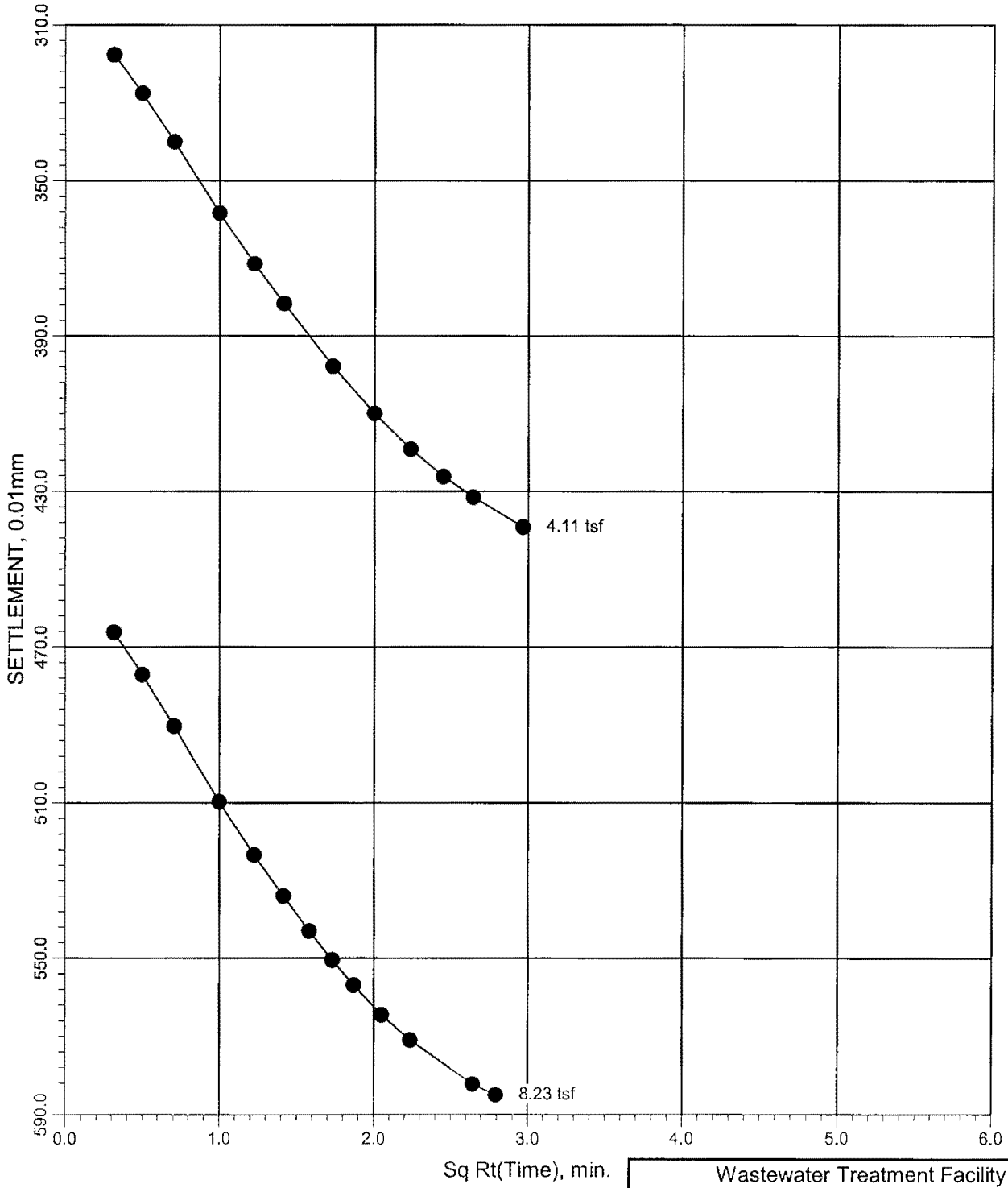
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(6/8)

I:\Groups\Lab\LabPrgm\Projects\21-21000\B1S7158.ODC :: 6/20/2008 2:59:06 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AFR 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 21-1-21000-001

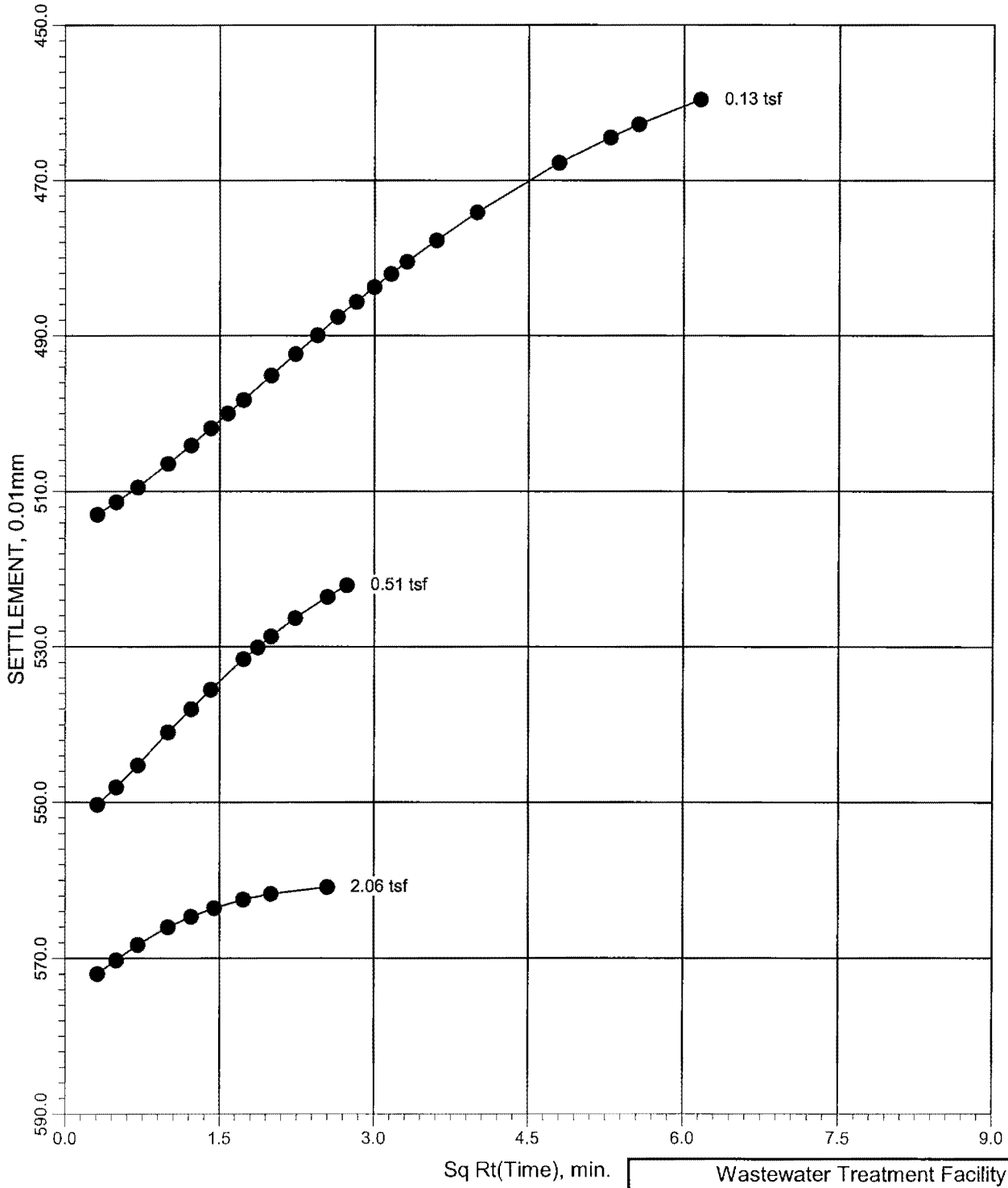
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(7/8)

I:\Groups\Lab\LabPrgm\Projects\21-21000\B1S7158.ODC :: 6/20/2008 2:59:11 PM

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS SQR ROOT(TIME)

Boring	B-1	Tested By / Date	JFL 6/17/08
Sample	S-7	Calc. By / Date	JFL 6/19/08
Depth, ft	15.8	Check By / Date	AJZ 6/23/08



Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

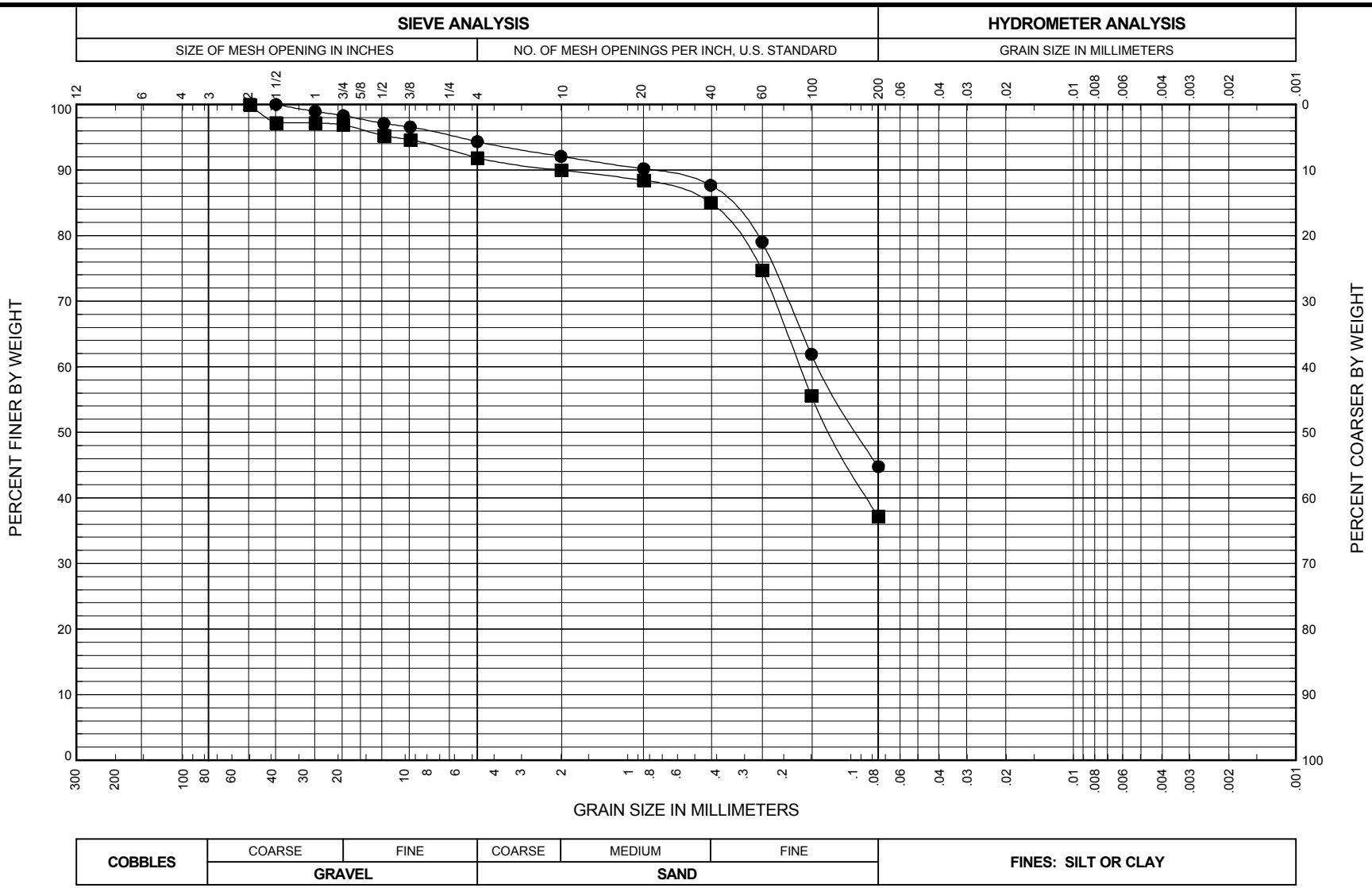
CONSOLIDATION TEST
Boring B-1, Sample S-7, Depth 15.8'

June 2008 | 21-1-21000-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. B-3(8/8)

I:\Groups\Lab\LabPrgm\Projects\21-21000\B1\S7158.ODC :: 6/20/2008 2:59:26 PM



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● 11/10/2008 S-2	0.0	SM	Gray-brown, slightly gravelly, silty SAND; numerous organics	44.8	32.5			
■ 11/10/2008 S-3	0.0	SM	Gray-brown, slightly gravelly, silty SAND; numerous organics	37.2	28.0			

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

GRAIN SIZE DISTRIBUTION
PROPOSED FILL MATERIAL

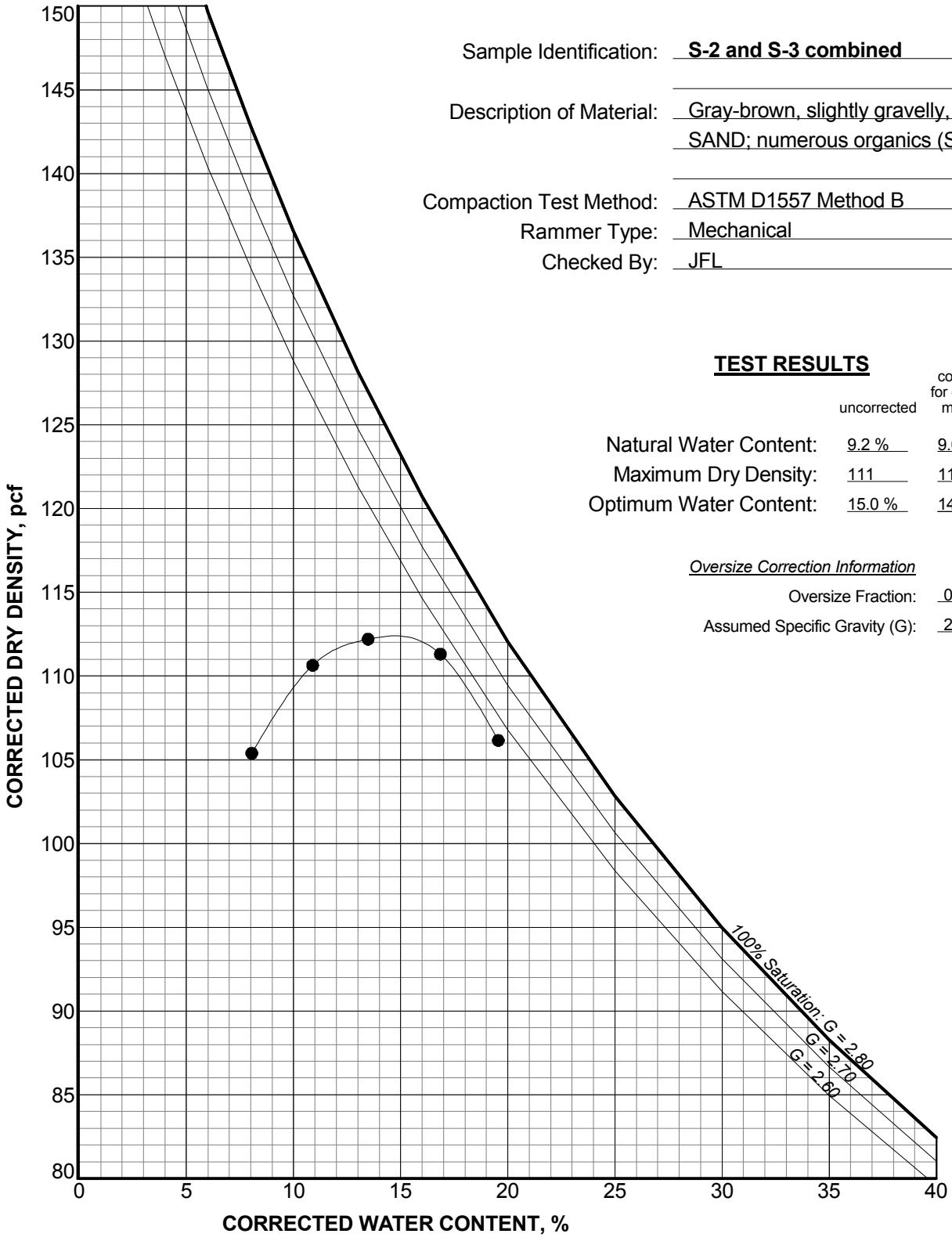
December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-4

GSA_MAIN 21-21000.GPJ SHAN_WIL.GDT 12/10/08

FIG. B-4



Sample Identification: **S-2 and S-3 combined**

Description of Material: Gray-brown, slightly gravelly, silty SAND; numerous organics (SM)

Compaction Test Method: ASTM D1557 Method B

Rammer Type: Mechanical

Checked By: JFL

TEST RESULTS

	uncorrected	corrected for oversize material
Natural Water Content:	9.2 %	9.0 %
Maximum Dry Density:	111	113
Optimum Water Content:	15.0 %	14.5 %

Oversize Correction Information

Oversize Fraction: 0.05

Assumed Specific Gravity (G): 2.7

OTHER TEST RESULTS

GRAIN SIZE ANALYSES:

% Gravel: 5.7

% Sand: 49.6

% Fines: 44.8

Wastewater Treatment Facility
Biosolids Lagoon Project
Hoquiam, Washington

MOISTURE-DENSITY TEST
PROPOSED FILL MATERIAL

December 2008 21-1-21000-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-5

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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