Geotechnical Report
Wolfe Residence
363 Porter Lane
Port Townsend, Washington

March 21, 2006

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

At Shannon & Wilson, our mission is to be a progressive, well-managed professional consulting firm in the fields of engineering and applied earth sciences. Our goal is to perform our services with the highest degree of professionalism with due consideration to the best interests of the public, our clients, and our employees.

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TABLE OF CONTENTS

		Page
1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION	1
3.0	PROJECT DESCRIPTION	2
4.0	FIELD EXPLORATIONS	2
5.0	SUBSURFACE CONDITIONS	3
5.1	Geology	3
5.2	Groundwater Conditions	4
5.3	Landslide History	5
6.0	CONCLUSIONS AND RECOMMENDATIONS	5
6.1	General	5
6.2	Loose Test Pit Backfill	5
6.3	Slope Stability	6
6.4	Foundation Design	7
6.5	Floor Support	7
6.6	Lateral Resistance	8
6.7	Measures to Reduce the Slope Movement Risk	8
6.8	Building Setback	8
6.9	Drainage	9
6.10	Septic	9
6.11	Rockery Retaining Walls	10
6.11.1	Rockery Construction	10
6.11.2	Rockery Drainage	11
7.0	CONSTRUCTION CONSIDERATIONS	11
7.1	Site Preparation and Grading	11
7.2	Backfill Placement and Compaction	11
7.3	Wet Weather Earthwork	12
7.4	Erosion Hazard	12
7.5	Construction Observation	13
8.0	LIMITATIONS	13
9.0	REFERENCES	15

LIST OF FIGURES

Figure No.

1 Vicinity Map
2 Site and Exploration Plan
3 Soil Classification and Log Key (2 sheets)
4 Log of Test Pit TP-1
5 Log of Test Pit TP-2
6 Log of Test Pit TP-3
7 Log of Test Pit TP-4
8 Typical Cross Section
9 Foundation Subdrain Detail
10 Rockery Retaining Wall Detail

LIST OF APPENDICES

Appendix

A Photos
B Important Information About Your Geotechnical Report

GEOTECHNICAL REPORT
WOLFE RESIDENCE
363 PORTER LANE
PORT TOWNSEND, WASHINGTON

1.0 INTRODUCTION

This report presents Shannon & Wilson, Inc. observations, conclusions, and geotechnical recommendations regarding the single-family house site development for the Wolfe residence located at 363 Porter Lane in Port Townsend, Washington. The purpose of our studies was to evaluate the subsurface conditions at the proposed house and garage sites in order to provide geotechnical engineering recommendations for design of foundations, house and site drainage, and setback from the bluff.

The Coastal Zone Atlas for Jefferson County indicates that the landslide hazard rating of slopes adjacent to the site is listed as unstable. Since the proposed building location is within a landslide hazard area buffer, we have prepared this report in accordance with the Unified Development Code (UDC) for Jefferson County to evaluate the potential for slope movement and to provide recommendations for development of the site with respect to slope stability.

Our work was accomplished in general accordance with the scope of work outlined in our report dated February 2, 2006, and included two visits to the site, logging of four backhoe-excavated test pits, a geologic reconnaissance to evaluate slope stability conditions, and preparation of this report. Authorization to proceed was received from Mrs. Judith S. Wolfe on February 21, 2006.

2.0 SITE DESCRIPTION

The Wolfe residential property is located below Porter Lane, west of Port Townsend, Washington, along a bluff adjacent to the Strait of Juan de Fuca, as shown in the Vicinity Map, Figure 1. The property consists of an upper area of about 1.2 acres that slopes down to the east at about 10 degrees. The northern portion of the property slopes down toward the northwest and extends from the top of the bluff to the beach below. The top of this northwest-facing slope is very steep, about 75 to 90 degrees, and typically 10 to 20 feet high. The slope flattens to 40 to

50 degrees below the vertical exposures and adjacent slopes. The slope extends down to a near-vertical face of very dense silts, sands, and gravels approximately 15 feet above the beach elevation. The Site and Exploration Plan, Figure 2, shows the proposed location of the house and garage and the approximate topographic relief from Porter Lane to the top of the bluff. A cross section of the bluff slope topography and stratigraphy was developed following our geologic reconnaissance and is presented in Figure 8. At the time of our field visit, some of the large trees and the undergrowth had been cleared from the house and garage sites. Trees remaining on the upper portion of the lot consisted of large-diameter firs and cedars, and abundant alders. The lower slope is covered with alder, maple, and scattered evergreen, with dense undergrowth of sword fern, salal, and other native bushes.

3.0 PROJECT DESCRIPTION

The property will be developed for a single-family residence. The proposed house and garage will be located on the upper (south) portion of the lot at least 120 feet south of the top of the steep bluff slope. The house site will be accessed by a driveway that approaches the site from the south side, from Porter Lane. The house is planned to be one story and includes a deck on its north side. The total area of the house and deck will be about 3,000 square feet. A detached garage will occupy about 572 square feet.

A septic drain field has also been proposed for the property. We understand that the preferred location for the drain field is on the upland side of the determined setback north and west of the proposed residence and garage locations.

4.0 FIELD EXPLORATIONS

Geologic maps of the area indicate that the proposed site is likely underlain with advance glacial outwash sands and gravels. Locally, glacial outwash is an unsorted mixture of medium dense to very dense sand and gravel. In order to characterize subsurface conditions, four test pits (designated TP-1, through TP-4) were excavated and sampled at the approximate locations shown in the Site and Explorations Plan, Figure 2. The test pits were excavated on March 14, 2006, with a rubber-tired backhoe and ranged in depth from about 12 to 14 feet. The test pit logs are presented in Figures 4 through 7.

An experienced geologist from Shannon & Wilson, Inc. observed the test pit excavations and prepared a log describing the soil classification, relative density of the soil, and moisture conditions. The soil density was estimated by hand probing the pit bottom and sidewalls with a ½-inch-diameter steel probe rod and observing the ease or difficulty of excavation. Soil classifications shown on the test pit logs are based on the visual-manual procedures and follow the Soil Classification and Log Key presented in Figure 3.

Soil samples obtained in the field were classified, sealed in jars, and returned to our laboratory, where the classification of each sample was visually checked. The results of the classifications are summarized on the test pit logs.

5.0 SUBSURFACE CONDITIONS

5.1 Geology

Published geologic maps of the area indicate that the upland portion of the site is underlain by Pleistocene-age (13,500 to 17,000 years old) Vashon Advance Outwash. Vashon Advance Outwash typically consists of sand with lesser amounts of silt and gravel. The advance outwash was deposited on the pre-existing land surface in front of the continental Vashon Stade ice sheet that advanced from Canada across the Puget Sound region approximately 17,000 years ago. The ice sheet that overrode the advance outwash is estimated to have been about 3,000 to 4,000 feet thick in this area. Consequently, the till and the underlying advance outwash have been compacted to a dense or hard state. Since the retreat of the glaciers, the upper few feet of the very dense/hard soils have loosened and weathered, and topsoil and/or colluvium has developed at the ground surface and along the slopes. Colluvium is weathered material that has reached its present location due to the forces of water and gravity and is typically found on and at the base of slopes.

Our interpretation of the subsurface conditions on the Wolfe residence is based on the following information:

- The Coastal Zone Atlas of Washington, Volume Eleven, Jefferson County.
- Local geology maps.
- Four test pits excavated at the site on March 14, 2006.
- A geologic reconnaissance of the upper and lower portions of the property.

Since the glaciers retreated, weathering, soil creep, and mass movements have modified the surficial soil. Forest duff has accumulated over the top of glacial soils. The glacial soils encountered in the test pits include weathered and unweathered glacial outwash.

The following paragraphs describe soil units we observed during our geologic reconnaissance and/or encountered in the test pit excavations.

- **Forest Duff** consists of an accumulation of soft, organic silt, with decaying logs, numerous roots, wood fragments, and decomposed organic material. Forest duff is typically about 0.5 to 1 foot thick and covers the ground surface across the site. Forest duff was thinner at some locations due to recent clearing of the underbrush.
- **Weathered Glacial Outwash** consists of loose to medium dense, orange-brown, slightly silty, gravelly sand that has been subjected to weathering by the intrusion of surface and groundwater, growth of trees and other vegetation, and the processes of downslope erosion. Loose to medium dense, weathered, glacial outwash is generally 2 to 5 feet thick and may be deeper at locations where large trees have grown.
- **Advance Glacial Outwash (Qga and Qga_s)** is deposited directly at the base of the glacier and overridden by the advancing ice. Outwash consists of a poorly graded mixture of medium dense to very dense gravel and sand with minimal silt and varying amounts of cobbles. This soil was encountered at depth in all the test pits. Outwash sands and gravels are often minimally cohesive and thus, susceptible to caving in open pits, such as during our explorations.
- **Glaciomarine Drift (Qgp_p)** is locally indistinguishable from glacial till. It typically consists of hard, gray, clayey, sandy silt with varying amounts of gravel and cobbles. This soil was observed adjacent to the beach in a near-vertical exposure.

Generally, the ground surface is covered with a 6- to 12-inch-thick layer of topsoil consisting of very loose, forest duff with fine- to medium-sized roots that typically penetrate 2 feet deep. Underlying the topsoil, a layer of loose, weathered, glacial outwash 2 to 3 feet deep is present across the site. Medium dense to dense glacial outwash was found at all test pit locations and likely underlies the site at depth. Very loose colluvium was observed along the bluff slope.

5.2 Groundwater Conditions

Groundwater was not observed within the test pit excavations. Due to the slight to moderate slope of the upper portion of the lot, surface water likely flows toward the crest of the bluff slope. During our site visit, we looked for springs, seeps, and other evidence of near-surface

groundwater. We did not observe springs, seeps or other evidence of near-surface water on the upland portion of the site or the steep waterfront slope.

5.3 Landslide History

The soils on the slope are highly susceptible to surficial sliding owing to the steep terrain, presence of groundwater seepage, and geologic conditions. Bluff recession caused by these shallow slides within the colluvial layer is commonly known as mass wasting. After review of the Coastal Zone Atlas for Jefferson County and Coastal Erosion maps produced by the United States Geological Survey (USGS), it is our opinion that the potential for future landslides to occur on the subject property is relatively high. Average erosion rates for this reach of waterfront can range from 2 to 18 inches per year, but may recur in increments of several feet during single events sporadically spaced over a period of several years. The risks that these landslides pose to existing and future structures on the property can be mitigated through adequate setbacks and construction of effective landslide control measures, which are described in subsequent sections of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Our engineering studies for the Wolfe property were based on our understanding of the proposed residence as described herein and on the results of field explorations. In general, the site is underlain by loose to dense, gravelly sand (glacial outwash). Medium dense to dense outwash soils will provide competent foundation bearing materials for the proposed structures. The approximate elevations of bearing soils are indicated in parenthesis next to the test pit designations in Figure 2. The elevations represent our recommended minimum excavation depths. The following sections present geotechnical recommendations for design and construction of the proposed project.

6.2 Loose Test Pit Backfill

Test pits dug to explore the site were backfilled with the spoils and tamped. However, if a test pit location falls in an area that will not be overexcavated below the bottom of the test pit, the loose soil should be removed during construction and replaced with compacted structural fill.

6.3 Slope Stability

Geologic hazard maps indicate that recent slope movements have occurred along the steep waterfront slope, but do not identify the type of slope movement. Based on our experience in the Puget Sound region, instabilities on steep waterfront slopes can generally be categorized as either shallow or deep-seated slides. Shallow slides typically involve movement of the upper topsoil, colluvium, or weathered soil on or near a slope and are usually the result of an oversteepened condition (often caused by wave erosion at the toe of the slope) and saturation of the surficial soils. When deep-seated slides involve the underlying, very dense and/or hard soils, it is often the result of perched groundwater or thin sandy seams with relatively high groundwater pressures and gradients within the geologic unit.

There is some evidence of relatively shallow slope movements on scattered areas of the steep waterfront slope. However, based on the size and type of trees generally encountered on the waterfront slope, these shallow slope movements would appear to be relatively infrequent (e.g., several tens of years between recurrence of shallow slides at the same location); and given the distance between the proposed residence and the steep bluff slope (i.e., 120 feet, as proposed), it is our opinion that shallow slides on the steep waterfront slope present a relatively low risk to the proposed residence.

With regard to deep-seated slope instability, the Coastal Zone Atlas for Jefferson County indicates the presence of post-glacial but pre-historic landslide activity. Relatively large trees (and stumps) within and immediately down-slope of the property were observed without significant evidence of rapid movement during their growth period. As such, the potential for deep-seated soil movement on the slope of the subject property can be considered relatively low, in our opinion. It is likely that the topographic and/or groundwater conditions that may have caused the deep-seated slope movement are no longer present.

While it is our opinion that the risk posed by potential shallow or deep-seated slope instability is relatively low, please note that there is some risk of future instability (shallow or deep-seated) present on all hillsides, which the owner must be prepared to accept. Such instability could occur because of future water line breaks/leaks, uncontrolled drainage, imprudent development in adjacent areas, or other actions or events on a slope that may cause sliding. The following provides further discussion of risk reduction measures that may be effective at this site. If the

risk reduction measures discussed in this report are implemented, it is our opinion that the proposed development will not adversely impact the stability of the adjacent slope and properties.

6.4 Foundation Design

In our opinion, the proposed residence and garage should be supported on spread footings bearing in the unweathered, medium dense, glacial outwash. Unweathered glacial outwash was encountered at depths of about 2½ to 5 feet in the test pits. Footings bearing in these soils can be designed for an allowable bearing pressure of up to 2,000 pounds per square foot (psf). This pressure can be increased up to one-third for seismic and wind loads. Site preparation and subgrade evaluation measures outlined in Section 7.1, should be accomplished prior to layout of the footings.

Continuous footings should have a minimum width of 18 inches, and column footings should have a minimum width of 24 inches. The base of all footings should be located at least 18 inches below the adjacent exterior grade and at least 12 inches below the lowest adjacent interior grade.

Structure foundations designed and constructed as recommended in this report are estimated to undergo total settlement of about ½ inch. Differential settlement is estimated to be about one-half the total settlement. It is anticipated that the majority of the settlement would occur simultaneously as the loads are applied.

6.5 Floor Support

Prior to placing pea gravel and/or crushed rock, the site preparation and grading recommendations (Section 7.1) should be implemented. The exposed surface should then be compacted as needed to achieve a dense and unyielding condition.

As a capillary break, we recommend that a minimum 4-inch-thick layer of washed pea gravel (⅜-inch to No. 8 sieve size) and a vapor barrier consisting of plastic sheeting be placed beneath the floor slabs. An alternative to the pea gravel would be to place a 2-inch-thick layer of clean, ⅝-inch-minus crushed rock over a 2-inch-thick minimum layer of washed pea gravel, which would provide a firmer working surface on which to place the reinforcement. The crushed rock should be compacted with at least three complete passes of a vibrating plate compactor.

6.6 Lateral Resistance

Lateral forces would be resisted by passive earth pressures against the buried portions of the structure and friction against the bottom. In our opinion, passive earth pressures developed from compacted granular fill could be estimated using an equivalent fluid density of 350 pounds per cubic foot (pcf). This value is based on the assumptions that the structures extend at least 2 feet below the lowest adjacent exterior grade and are properly drained, and that the backfill around the structure is compacted in accordance with the recommendations for the structural fill outlined in Section 7.3. We recommend that a coefficient of friction of 0.4 be used between cast-in-place concrete and soil. The above design values include a factor-of-safety of 1.5.

6.7 Measures to Reduce the Slope Movement Risk

In general, the risk of soil movement on a slope can be reduced by not oversteepening a slope (e.g., not excavating the toe of a slope or placing sidecast fill at the top) and not increasing the weight on a slope (e.g., not placing yard debris or fill on or at the crest of a slope). The risk of soil movement on a slope can also be reduced by maintaining a slope as dry as possible (e.g., locate septic drain fields adequate distances, such as 100 feet, away from slopes; route roof downspouts and yard drains away from slopes; and minimize the amount of surface water that could flow down slope faces) and maintaining a vegetative cover on slopes.

6.8 Building Setback

One of the most cost-effective measures to reduce the potential impact of slope movement is to provide an adequate building setback so that if soil movement on the slope does occur, the hazard to the structure is minimal. An appropriate setback is a function of the rate or risk of slope movement (regression rate), the design life of the structure, and the risk the owner of the structure is willing to assume.

We understand that the proposed setback distance for the Wolfe residence is 120 feet from the top of the bluff slope. Based on the plans provided to us by Zimmerman Architecture, our explorations, evaluation of subsurface conditions and slope reconnaissance, and our experience with similar sites in the area, the residence setback at the current proposed location of 120 feet from the northwest-facing steep waterfront slope is adequate.

6.9 Drainage

In general, reducing the amount of water entering and discharging onto the slope can reduce the risk of slope movement. Drains should be constructed and maintained to collect water from impermeable surfaces on the property (e.g., roof, decks, patios, and driveways) and to direct it in a tightline to a suitable discharge point. Upon reviewing the site conditions and various options for discharge (including upland discharge), it is our opinion that a tightline located on the eastern half of the property over the bluff edge would provide a suitable discharge point without significantly impacting the stability of the slopes on the site or increasing the surface water discharge or sedimentation to adjacent properties beyond pre-development conditions. The tightline must continue to the base of the slope, i.e., to the beach. We understand that the proposed design setback for the main catch basin is approximately 25 feet from the bluff slope. A sump pump likely will not be required in order to convey the water collected in the vicinity of the building to the discharge point.

In addition to surface drainage, we recommend that footing drains be installed around the perimeter of the building to improve soil drainage in the immediate vicinity of the structure. Footing subdrains should consist of slotted, 4-inch-diameter minimum, plastic pipe bedded in washed, ⅜-inch pea gravel. Typical installation details for these drains are shown in Figure 9, which also includes subdrainage and foundation wall backfill recommendations. On-site soils would not be suitable for use as drainage sand and gravel. Note that the perimeter subdrain invert should be located at least 12 inches below the lowest adjacent grade at the highest slope elevation. The discharge from footing drains should be routed by means of a tightline to a suitable discharge point as previously discussed. All outside grades should slope away from the residence. Roof or other drains should not connect with foundation subdrains.

Based on our understanding of the limited, single-residence development of this property, it is our opinion that the anticipated discharge of roof and footing drains as outlined above will not significantly affect the pre-development drainage conditions on the adjacent properties.

6.10 Septic

We understand that the septic system will be designed by others, and that Shannon & Wilson, Inc., is to provide recommendations regarding setback distance for the system. We have shown a possible location for the drain field in Figure 2. We recommend that the drain field be located no

closer than 80 feet from the top of the 100-foot-high bluff. Location considerations should be based on all applicable local building codes.

6.11 Rockery Retaining Walls

In general, rockery walls may be used in cut or fill areas. In fill areas, rockeries may be up to 6 feet high with a level backslope. We recommend that rockery walls be founded on medium dense to dense native soils or on structural fill compacted to the recommended standard. A typical rockery detail is included in Figure 10.

6.11.1 Rockery Construction

The base of the rockery should be embedded at least one-half the thickness of the lowest base rocks, or 18 inches below the adjacent ground surface, which ever is deeper. The final rockery face should be constructed with a batter of 1 Horizontal to 4 Vertical (1H:4V) to 1H:5V. The base should be excavated approximately twice as wide as the base rock width. The minimum base rock width should be approximately one-third the height of the rockery.

The rockery rocks should be tabular and rectangular. Rocks should be hard, sound, durable and free of weathered portions, seams, cracks, and other weaknesses. The rock density should not be less than 160 pounds pcf. The lower 2 to 4 feet of the rockery should be constructed using 4- to 5-man rocks.

Rock selection and placement should be accomplished to reduce the number and size of the voids. In the exposed face of the wall, no openings greater than 6 inches in dimension in any direction should be permitted. Rock courses should be gradational in size from bottom to top with the largest course at the base; the lowest two courses should be uniform in size. The contact between rocks should slope downward to the backside of the rockery. Each course of rocks should be seated tightly and evenly on the course beneath. After each course or rock is seated, voids between the rocks should be chinked on the back with quarry spalls to reduce piping of backfill material. Backfill immediately behind the rockery should consist of quarry spalls. The spalls should be placed to form a graded chinking from front to back, with the smallest-diameter spalls against the backfilled on-site sands. The spalls should consist of well-graded ¾- to 4-inch crushed rock and should be durable, uncontaminated by soil or other debris, and not readily susceptible to weathering.

The quarry spall fill should be at least 18 inches wide from the rockery and the face of the cut. The spalls should be placed and compacted in lifts to a level approximately 2 inches below the top of each course of rocks as they are placed, until the uppermost course is placed. Backfill material falling onto the bearing surface of one rock course must be removed before setting the next course.

6.11.2 Rockery Drainage

A perforated drainpipe should be embedded in the backfill at the base of the rockery. This drain should discharge to the tightline system. A qualified contractor experienced in rockery construction should install rockeries. The construction should be monitored by a geotechnical engineer or geologist.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Site Preparation and Grading

We recommend that the topsoil and any other soil containing roots and organic matter be removed from beneath the building footprint. The depth of removal is estimated to be approximately 1 foot. Topsoil is not considered suitable for reuse as structural fill and should be removed from the site or stockpiled for reuse in landscape areas.

We anticipate that footing subgrades will consist of medium dense to dense, unweathered glacial outwash sand and gravel that generally will not require proof rolling or compaction prior to construction. Floor slab subgrade will consist of medium dense, weathered or unweathered glacial outwash. This assumes that the soil is not disturbed during excavation and is not subjected to softening in the wet weather. Any soft or spongy soil zones should be removed, and any loose soil should be compacted in accordance with the recommendations for structural fill presented below. Where necessary, the stripped soil should be replaced with structural fill to bring the surface to proper subgrade elevation.

7.2 Backfill Placement and Compaction

All fill soils placed beneath foundations, floor slabs, pavements, or any other areas where settlements are to be minimized should be structural fill. Imported structural fill soils should consist of a well-graded mixture of crushed sand and gravel free of organics and debris.

However, it is not anticipated that imported fill will be necessary due to the nature of the on-site, native soils. Native spoils generated from foundation excavation can be reused and recompacted in dry weather. Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition. The thickness of soil layers before compaction should not exceed 8 inches for heavy equipment compactors or 4 inches for hand-operated mechanical compactors.

Proper site preparation and subgrade evaluation methods presented in the previous section should be employed in areas to receive structural fill, and a geotechnical engineer or geologist should evaluate the prepared subgrade prior to fill placement.

7.3 Wet Weather Earthwork

Soils at the site are generally suitable for use as fill and are not particularly moisture sensitive based on their relatively low fines content. However, in general, soils are susceptible to changes in water content and can become difficult to compact. The following recommendations are applicable to reduce problems associated with rainwater, trafficability, and the handling of wet soils:

- Fill material should consist of clean, granular soil, of which not more than 5 percent by dry weight passes the No. 200 mesh sieve, based on wet-sieving the minus ¾-inch fraction. Any fines should be nonplastic.
- The ground surface in and surrounding the construction area should be sloped and sealed with a smooth-drum roller to promote runoff of precipitation away from work areas and to prevent water from ponding.
- Earthwork should be accomplished in small sections to reduce exposure to wet conditions. If there is to be vehicular traffic over the exposed subgrade during construction, the subgrade should be protected with a compacted layer (generally 8 inches or more) of clean crushed rock. The size or type of equipment may have to be limited to prevent soil disturbance.
- Where loosened, soil may be susceptible to moisture, or if it is uncompacted, a smooth drum vibratory roller, or equivalent, should be used to seal the surface where practicable. Soils that become too wet for compaction should be removed and replaced with clean, crushed rock.

7.4 Erosion Hazard

To reduce the potential for soil erosion and associated hazards, the following wet weather earthwork recommendations are presented. If these wet weather earthwork recommendations

and prudent construction practices are used, it is anticipated that the future earthwork for the proposed development will not significantly affect soil erosion and associated hazards on the site.

Covering work areas, soil stockpiles, or slopes with plastic; sloping; ditching; installing sumps; dewatering; and other measures should be employed, as necessary, to permit proper completion of the work. Straw bales and/or geotextile silt fences should be aptly located to control soil movement and erosion.

7.5 Construction Observation

With respect to implementing the risk reduction measures outlined in this report, we recommend that a geotechnical engineer/engineering geologist observe geotechnically related construction, including drainage installation, structural fill placement and compaction, building footing locations, and subgrade soils once they are excavated. The building footing locations should be observed to determine if foundation depths provide the minimum horizontal setback outlined in this report and to provide recommendations for additional excavation as needed.

8.0 LIMITATIONS

The conclusions in this report are based on site conditions visually observed during our site reconnaissance and inferred from published geologic, soils, topographic, and hazard maps and assume that observed conditions are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions are not significantly different from those inferred from the site reconnaissance or indicated on geologic maps. During subsequent site activities (e.g., construction), if subsurface conditions different from those inferred in this report are observed or appear to be present, we should be advised at once so that we can review those conditions and reconsider our conclusions where necessary.

Within the limitations of scope, schedule, and budget, the conclusions presented in this report were prepared in accordance with generally accepted geologic engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied.

This report was prepared for the use of Zimmerman Architecture and the owner in the evaluation of the stability of this site. With respect to possible future construction, it should be made available for information on factual data only and not as a warranty of subsurface conditions, such as those interpreted from the site visits and discussion of geologic conditions included in this report.

Please note that the scope of our services did not include any environmental assessments or evaluation regarding the presence or absence of wetlands or hazardous or toxic material in the soil, surface water, groundwater, or air, on or below or around this site. We are able to provide these services and would be pleased to discuss these with you if the need arises.

Shannon & Wilson, Inc., has prepared Appendix B, "Important Information About Your Geotechnical Report," to assist you in understanding the use and limitations of our report.

We appreciate the opportunity to provide geologic services to you, and we are available to answer any questions regarding our observations, conclusions or recommendations contained in this report.

SHANNON & WILSON, INC.

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MET:NDM:TMG/met

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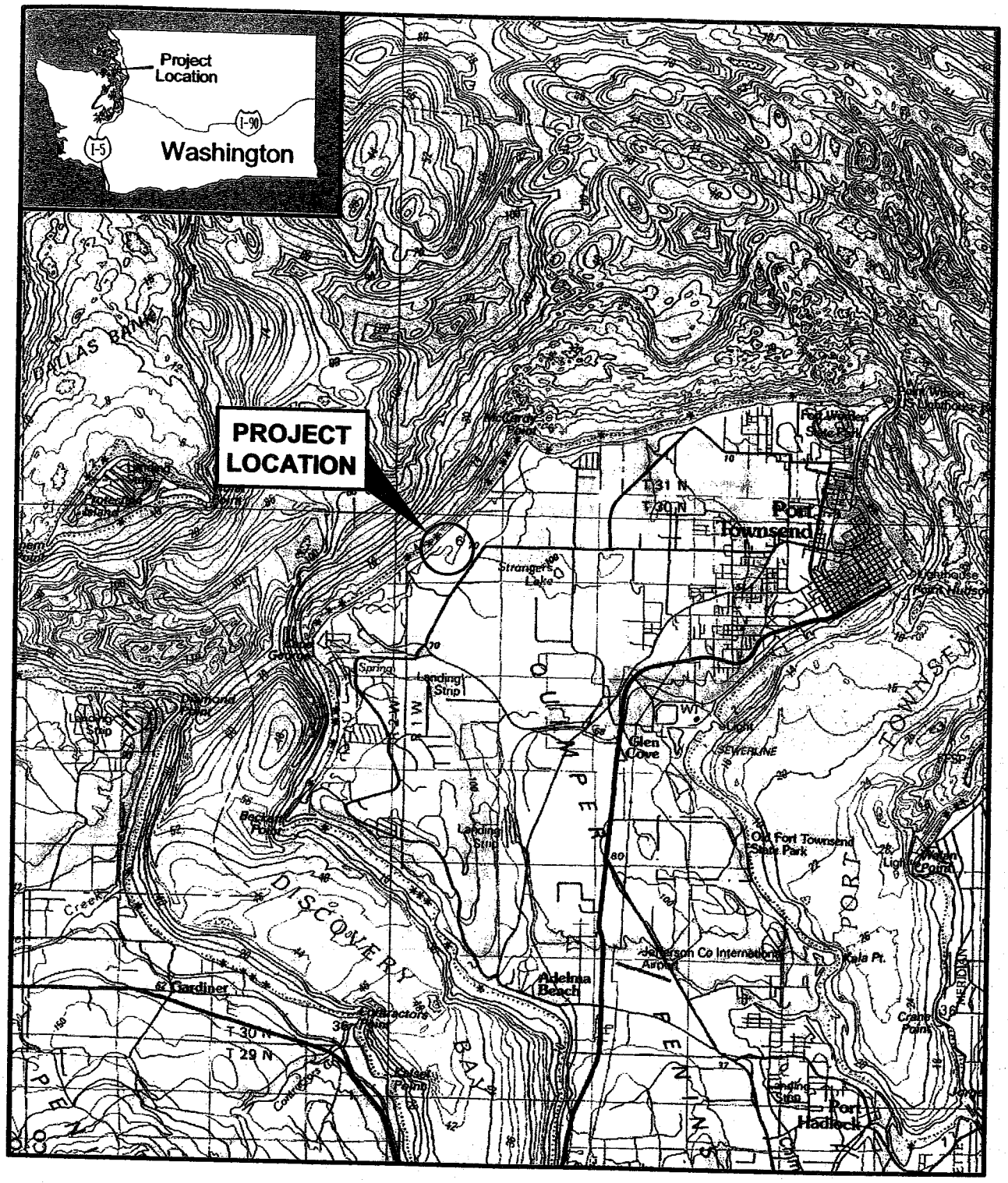

0 1.5 3

Scale in Miles

NOTE

Map adapted from electronic CD ROM USGS topographic map by TOPO! ©2000 National Geographic Holdings.

Wolfe Residence
Port Townsend, Washington

VICINITY MAP

March 2006 21-1-20457-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

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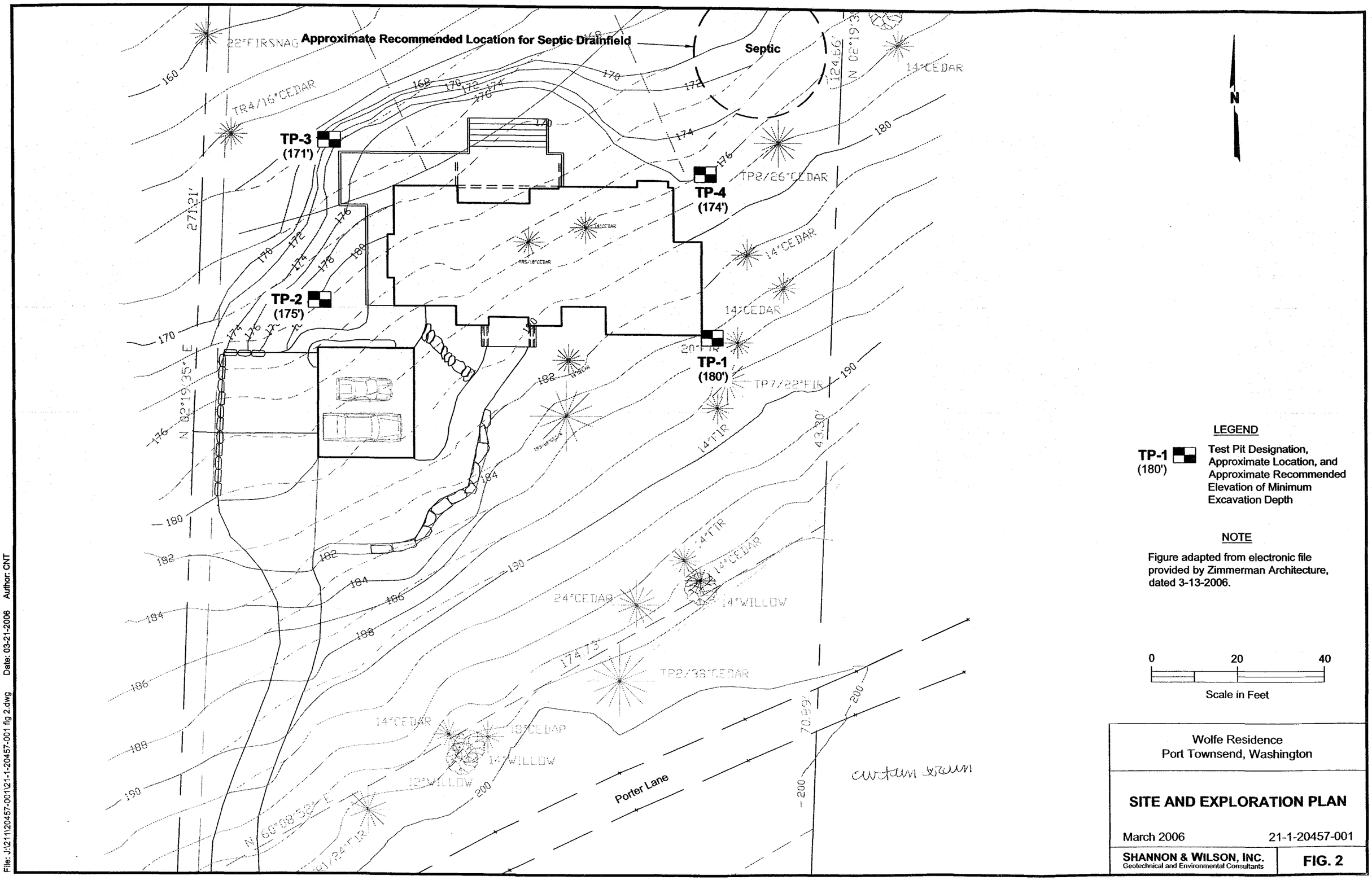

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Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual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

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363 Porter Lane
Port Townsend, Washington

SOIL CLASSIFICATION AND LOG KEY

March 2006 21-1-20457-001

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FIG. 3
Sheet 1 of 2

BORING CLASS1 21-20457.GPJ SWNEW.GDT 3/21/06

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.

Wolfe Residence
363 Porter Lane
Port Townsend, Washington

SOIL CLASSIFICATION AND LOG KEY

March 2006 | 21-1-20457-001

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FIG. 3
Sheet 2 of 2

BORING CLASS2 21-20457.GPJ SWNEW.GDT 3/21/06

File: J:\211\20457-001\21-1-20457-001 fig 4-7 TPs.dwg Date: 03-16-2006 Author: CNT

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LOG OF TEST PIT TP-1

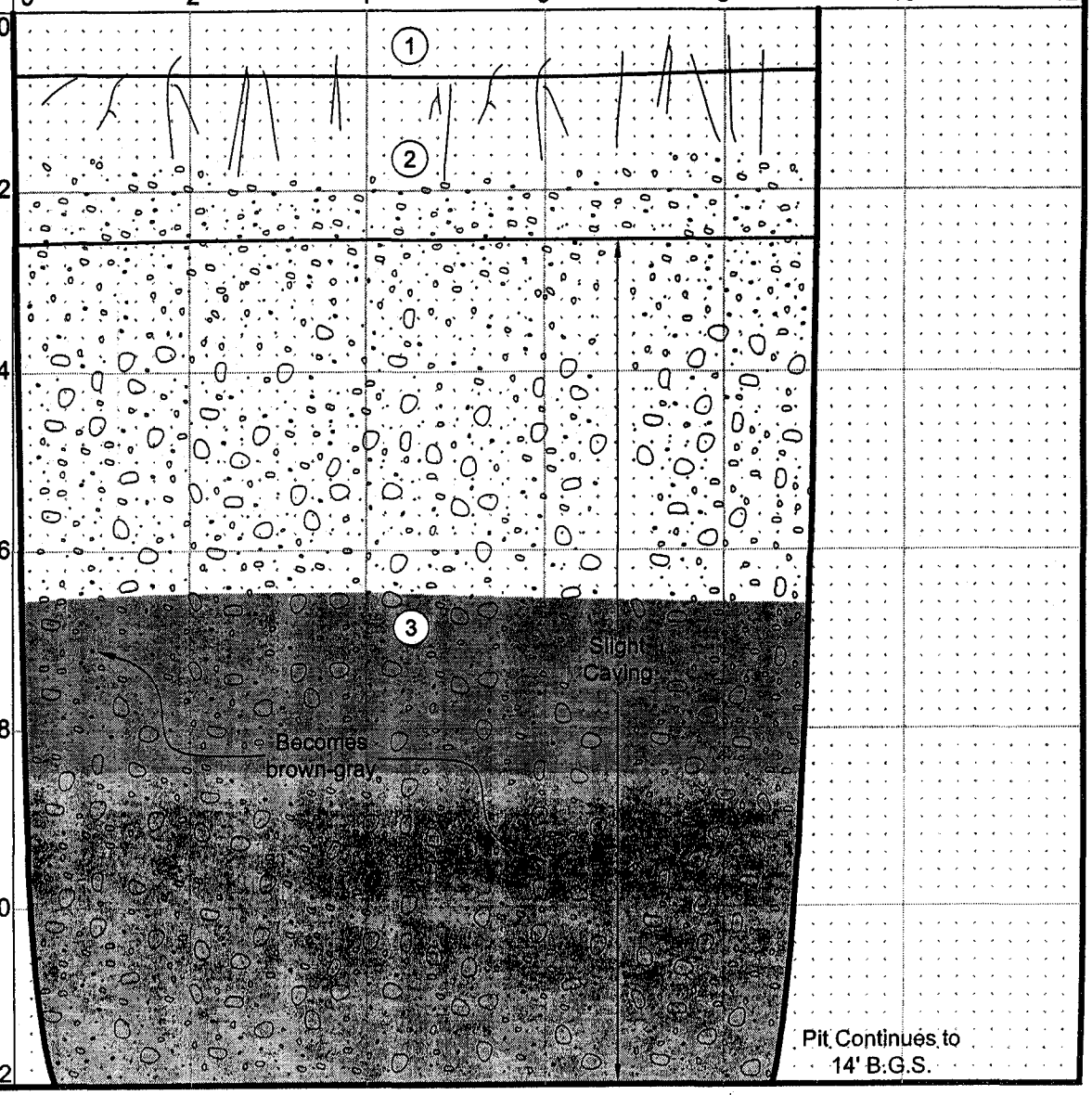
JOB NO: 21-1-20457-001 DATE: 3-14-2006 LOCATION: See Site and Exploration Plan

PROJECT: Wolfe Residence; Port Townsend, Washington

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) 6" Forest Duff.	None Observed			0
(2) Loose, orange-tan, slightly fine gravelly, fine to medium SAND, trace of silt; moist; (Qga_s) SP.				2
(3) Medium dense, brown, fine to medium sandy GRAVEL; moist; becomes brown-gray near 6.5 feet; (Qga) GP.			S-1	4
				6
			S-2	8
				10
				12

Sketch of West Pit Side

Surface Elevation: 184'



NOTES

1. Test Pit completed at 14 feet below ground surface (bgs).
2. No groundwater seepage observed.
3. Slight caving.

FIG. 4

File: J:\211\20457-001\21-1-20457-001 fig 4-7 TPs.dwg Date: 03-16-2006 Author: CNT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

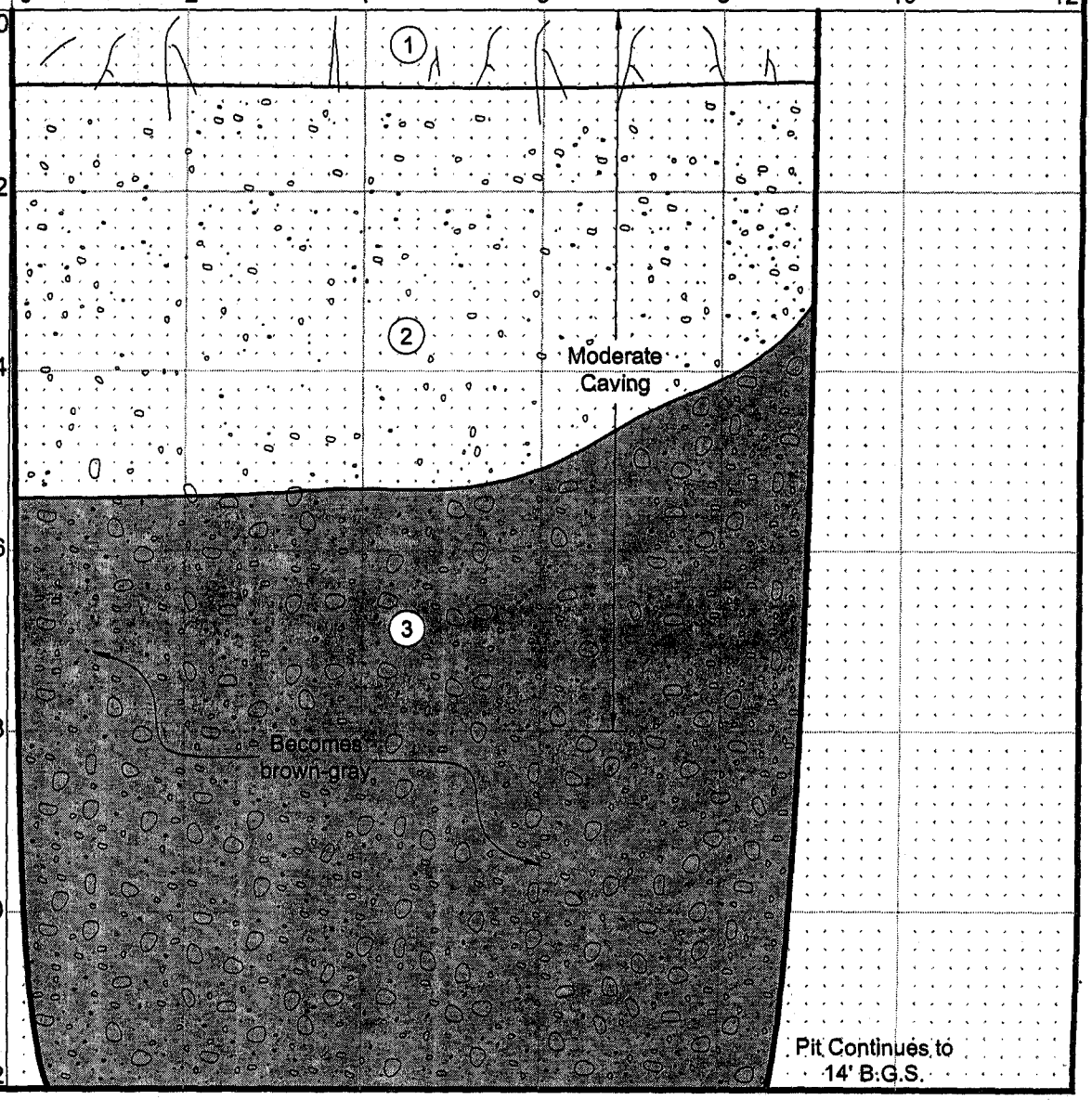
LOG OF TEST PIT TP-2

JOB NO: 21-1-20457-001 DATE: 3-14-2006 LOCATION: See Site and Exploration Plan

PROJECT: Wolfe Residence; Port Townsend, Washington

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) 8" Topsoil. Dark brown, silty, fine to medium SAND; moist; occasional roots and rootlets; SM.	None Observed		S-1	0
(2) Loose to medium dense, orange-tan, slightly silty, slightly gravelly, fine to medium SAND; moist; slight oxidation; (Qga_s) SP-SM.				2
(3) Becomes brown-gray, slightly silty, slightly gravelly to gravelly, fine to medium SAND; moist; (Qga_s) SP-SM.			S-2	4
				6
			S-3	8
				10
				12

Sketch of West Pit Side Surface Elevation: 180'



NOTES

1. Test Pit completed at 14 feet below ground surface (bgs).
2. No groundwater seepage observed.
3. Moderate caving to 8 feet.

FIG. 5

File: J:\211\20457-001\21-1-20457-001 fig 4-7 TPs.dwg Date: 03-16-2006 Author: CNT

SHANNON & WILSON, INC.
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LOG OF TEST PIT TP-3

JOB NO: 21-1-20457-001 DATE: 3-14-2006 LOCATION: See Site and Exploration Plan

PROJECT: Wolfe Residence; Port Townsend, Washington

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) 9" Topsoil. Dark brown, silty, fine to medium SAND; moist; occasional roots and rootlets; SM.	None Observed			0
(2) Loose, rust brown, slightly gravelly, silty SAND; moist; SM.			S-1	2
(3) Medium dense, orange-tan, slightly silty, slightly gravelly, fine to medium SAND; moist; (Qga$_s$) SP-SM.				4
(4) Medium dense to dense, gray-brown, fine to medium sandy GRAVEL, trace of silt; moist; (Qga) GP.				6
(5) Dense, light tan/gray, silty, fine SAND; moist; (Qga$_s$) SM.				8
(6) Medium dense to dense, light gray, slightly gravelly, fine to medium SAND; moist; (Qga$_s$) SP.			S-2	10
			S-3	12

NOTES

1. Test Pit completed at 12 feet below ground surface (bgs).
2. No groundwater seepage observed.
3. Top 4 feet caved upon inital excavation. Moderate to severe caving to 8 feet.

Sketch of South Pit Side Surface Elevation: 174'

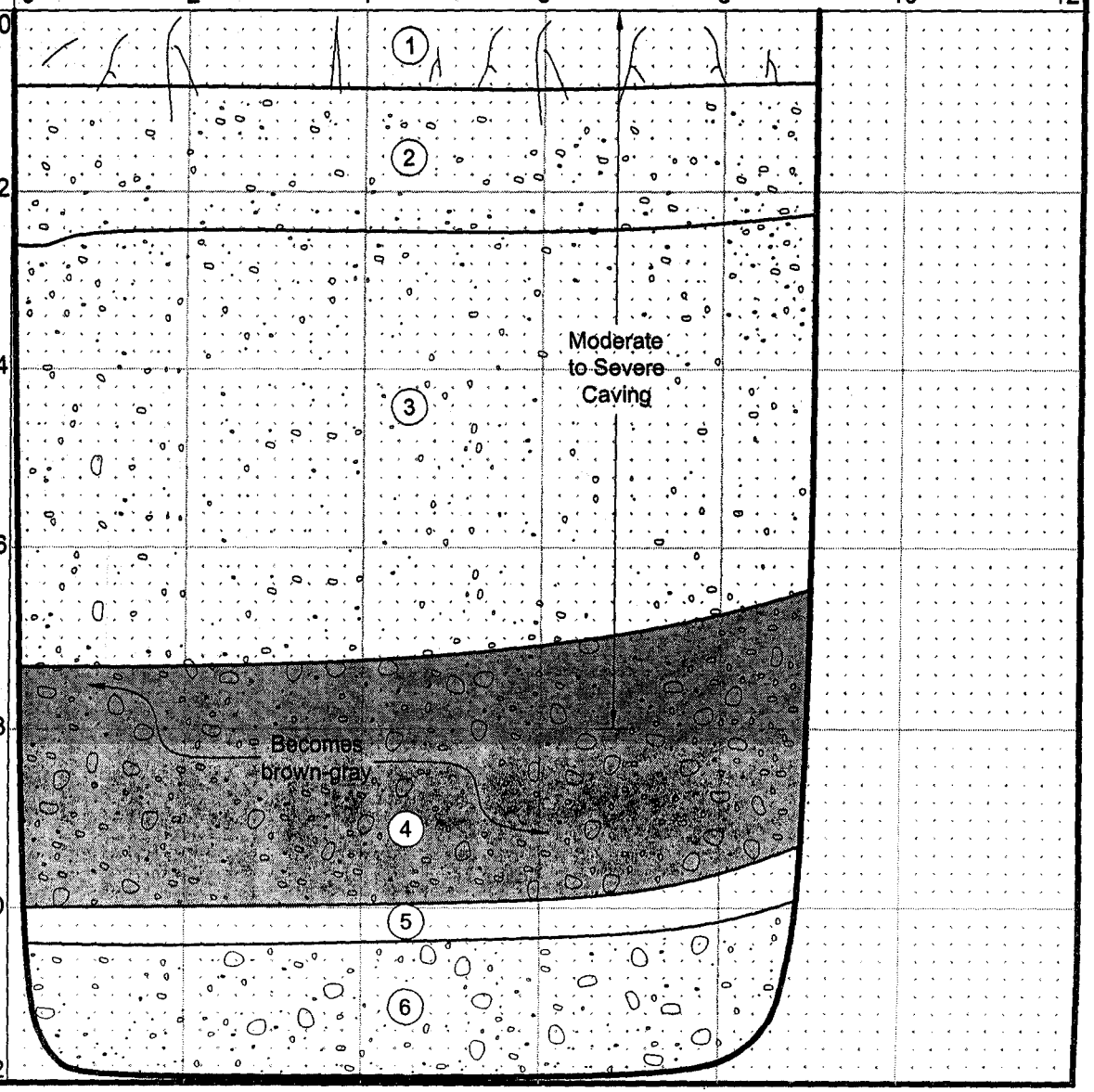

FIG. 6

File: J:\211\20457-001\21-1-20457-001 fig 4-7 TPs.dwg Date: 03-21-2006 Author: CNT

SHANNON & WILSON, INC.
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LOG OF TEST PIT TP-4

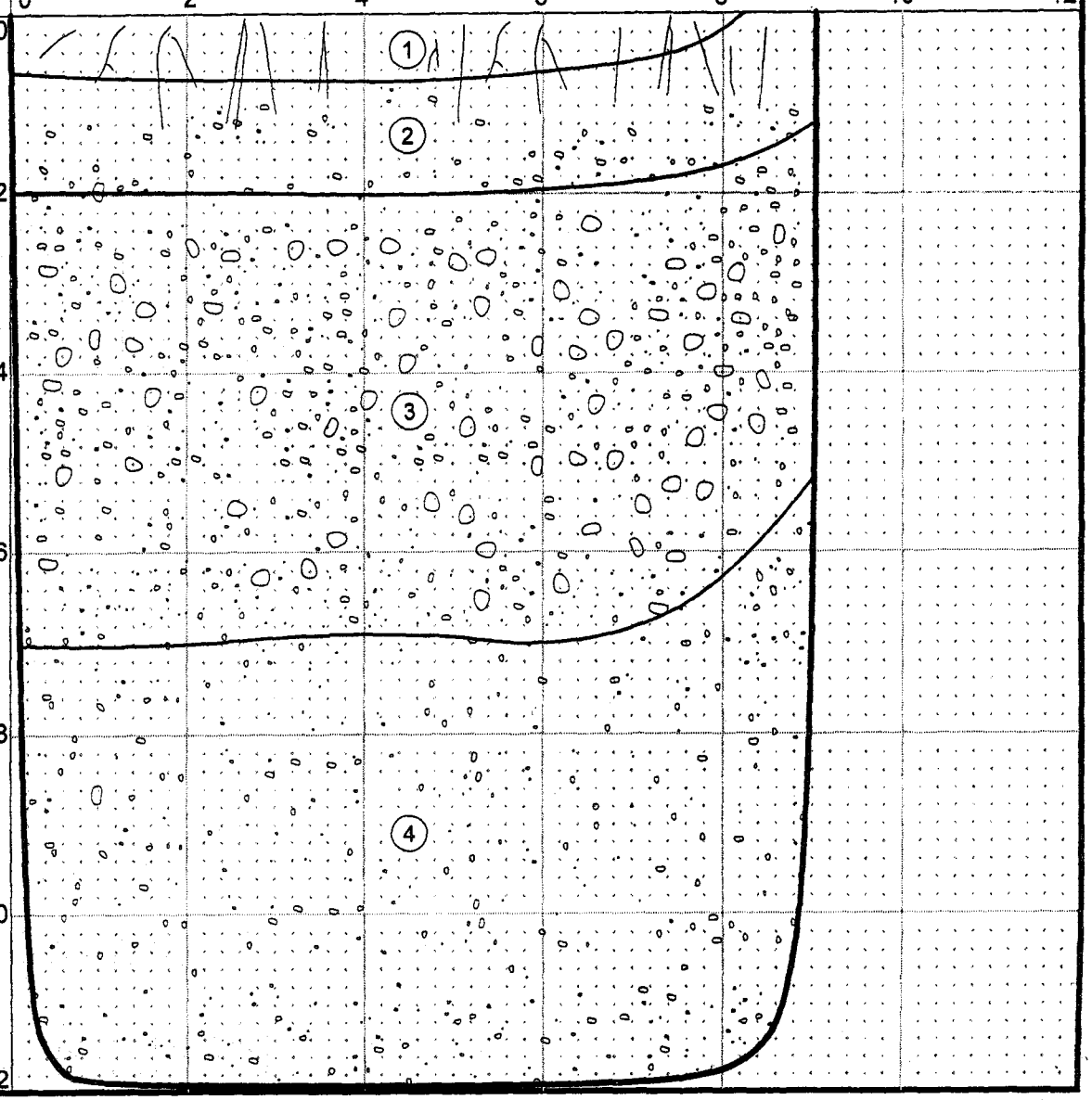
JOB NO: 21-1-20457-001 DATE: 3-14-2006 LOCATION: See Site and Exploration Plan

PROJECT: Wolfe Residence; Port Townsend, Washington

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) 6-10" Forest Duff; numerous roots and rootlets.	None Observed			0
(2) Loose to medium dense, rust brown, slightly gravelly, silty, fine to medium SAND; moist; SM.			S-1	2
(3) Medium dense, orange-tan, firm sandy GRAVEL, trace of silt; moist; (Qga) GP.				4
(4) Medium dense to dense, gray-brown, gravelly, fine to medium SAND, trace of silt; moist; (Qga_s) SP.				6
				8
			S-2	
				10
			S-3	12

Sketch of North Pit Side Surface Elevation: 177'

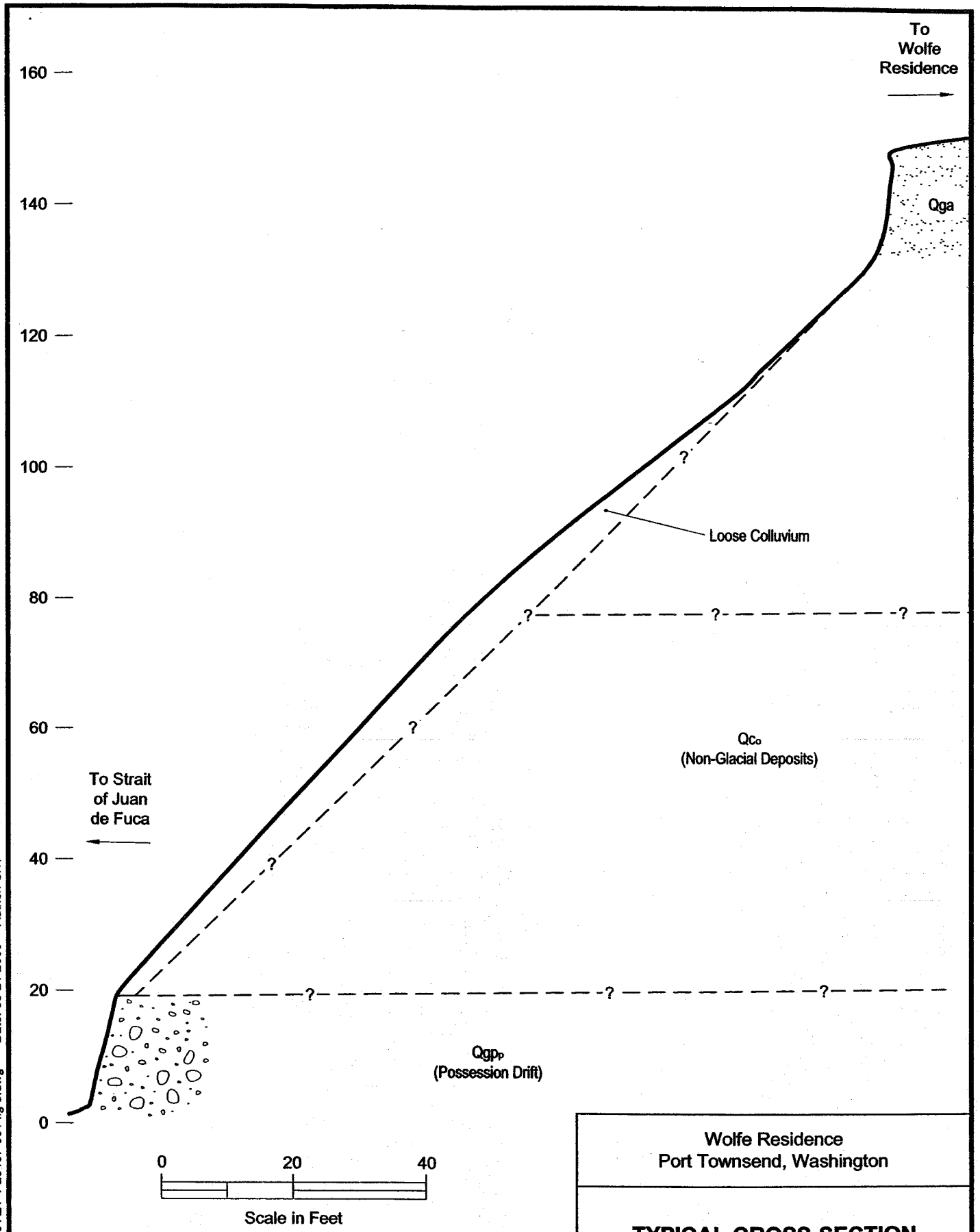
Horizontal Distance in Feet



NOTES

1. Test Pit completed at 12 feet below ground surface (bgs).
2. No groundwater seepage observed.
3. Slight caving.

FIG. 7



NOTE

These measurements were made in the field using hand instruments and should be considered approximate.

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TYPICAL CROSS-SECTION

March 2006 | 21-1-20457-001

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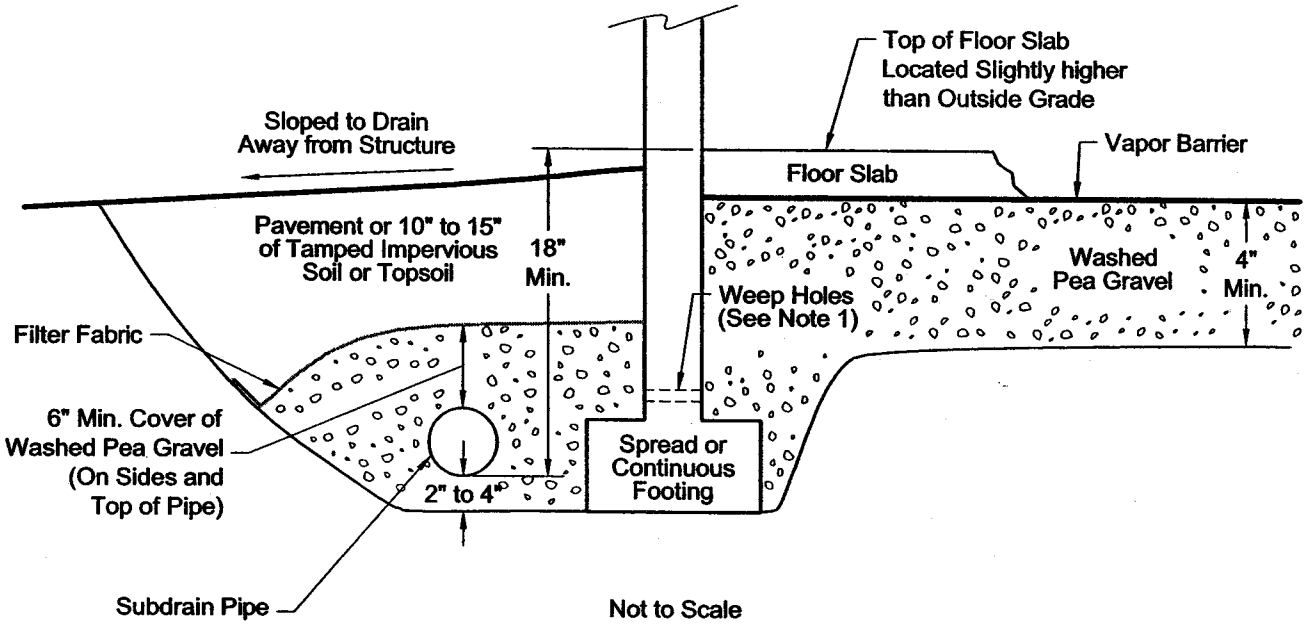
FIG. 8

File: J:\21\20457-001\21-1-20457-001 fig 8.dwg Date: 03-21-2006 Author: CNT



NOTES

1. Washed pea gravel beneath floor slab should be hydraulically connected to subdrain pipe. Use of 2-inch diameter weep holes as shown is one applicable method. Weep holes should be spaced not more than 15 feet apart.

2. If floor slab is located below outside grade, place drainage sand and gravel against wall (18" out from wall) extending up to pavement or impervious zone.

3. Do not wrap subdrain pipe with filter geotextile.

4. To provide a more firm working surface, 2 inches of clean crushed rock conforming to WSDOT 9-03.9 (3) top course may replace upper 2 inches of washed pea gravel. Compact the crushed rock with 3 complete coverages of a vibrating plate compactor.

SUBDRAIN PIPE

4" minimum diameter perforated or slotted, concrete, metal, or plastic pipe; tight joints; sloped to drain (4"/100' min. slope); provide clean-outs.

Perforated pipe holes (3/16" to 3/8" dia.) to be in lower half of the pipe with lower quarter segment unperforated for water flow.

Slotted pipe to have 1/8" maximum width slots.

Wolfe Residence
Port Townsend, Washington

FOUNDATION SUBDRAIN DETAIL

March 2006 | 21-1-20457-001

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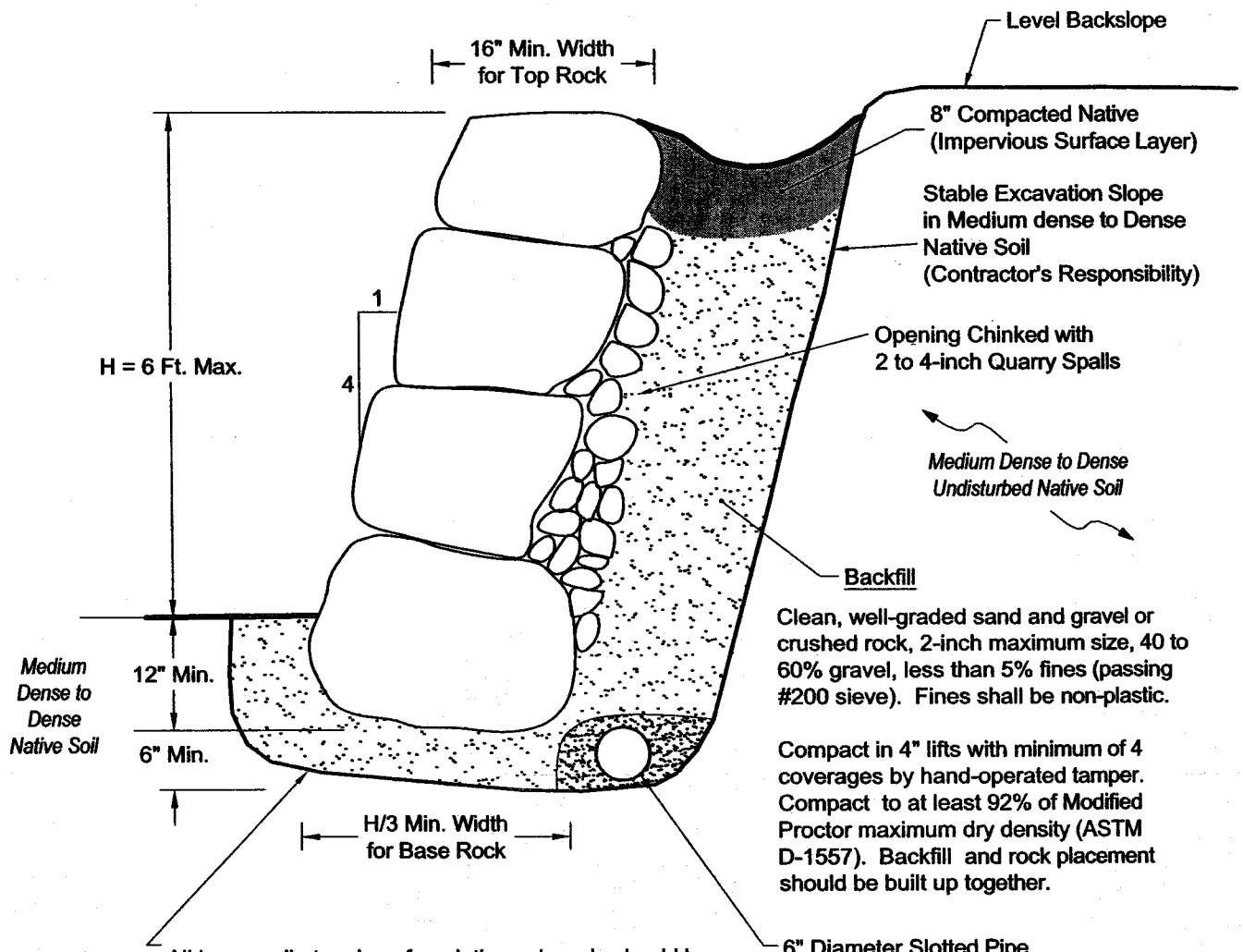
FIG. 9

File: J:\211\20457-001\21-1-20457-001 fig 9.dwg Date: 03-21-2006 Author: CNT



Clean, well-graded sand and gravel or crushed rock, 2-inch maximum size, 40 to 60% gravel, less than 5% fines (passing #200 sieve). Fines shall be non-plastic.

Compact in 4" lifts with minimum of 4 coverages by hand-operated tamper. Compact to at least 92% of Modified Proctor maximum dry density (ASTM D-1557). Backfill and rock placement should be built up together.

All loose soil at rockery foundation subgrade should be overexcavated down to medium dense to dense soil and replaced with compacted backfill as described above. The excavation shall be kept free of water. The prepared foundation subgrade shall be evaluated by a soils engineer prior to placement of rock.

6" Diameter Slotted Pipe
Bedded in washed 3/8" to No.8 sieve size pea gravel (6" cover around pipe), sloped to drain and connected by tightline to storm drain outfall or other appropriate outlets.
No fabric around pipe.
Maximum slot width is 1/8".

Not to Scale

MINIMUM WEIGHT OF ROCK

Rock shall be sound and have a minimum density of 160 pounds per cubic foot.

Wolfe Residence
Port Townsend, Washington

TYPICAL ROCKERY DETAIL

March 2006 | 21-1-20457-001

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FIG. 10

File: J:\211\20457-001\21-1-20457-001 fig 10.dwg Date: 03-21-2006 Author: CNT

APPENDIX A

PHOTOS

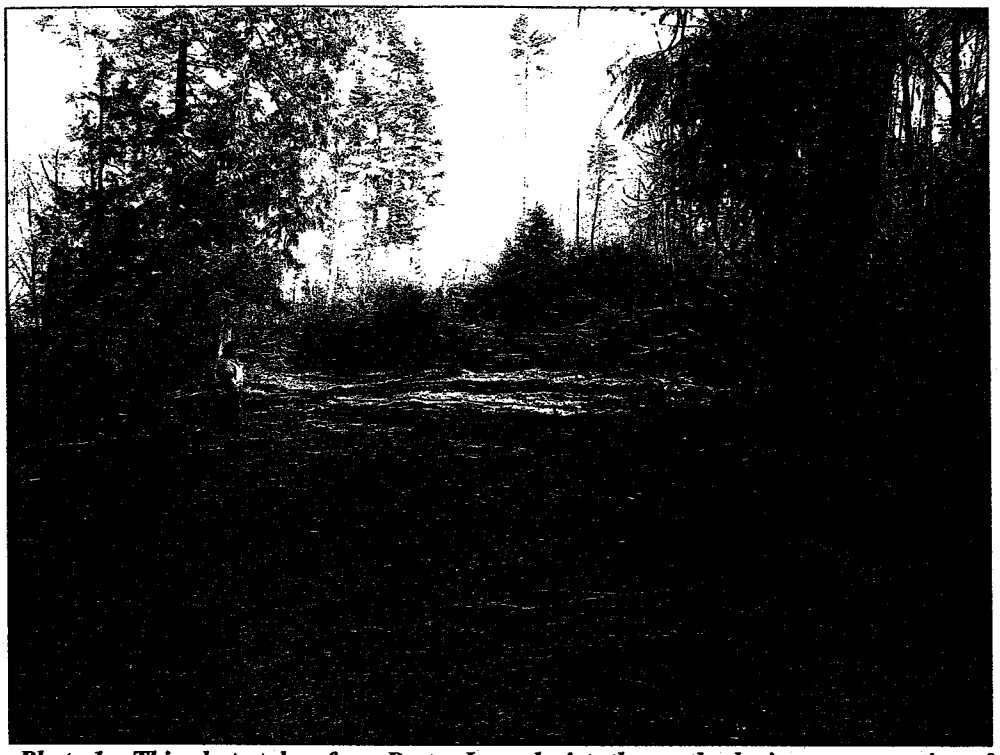

Photo 1 – This photo taken from Porter Lane depicts the gently-sloping upper portion of the Wolfe property overlooking the Strait of Juan de Fuca.

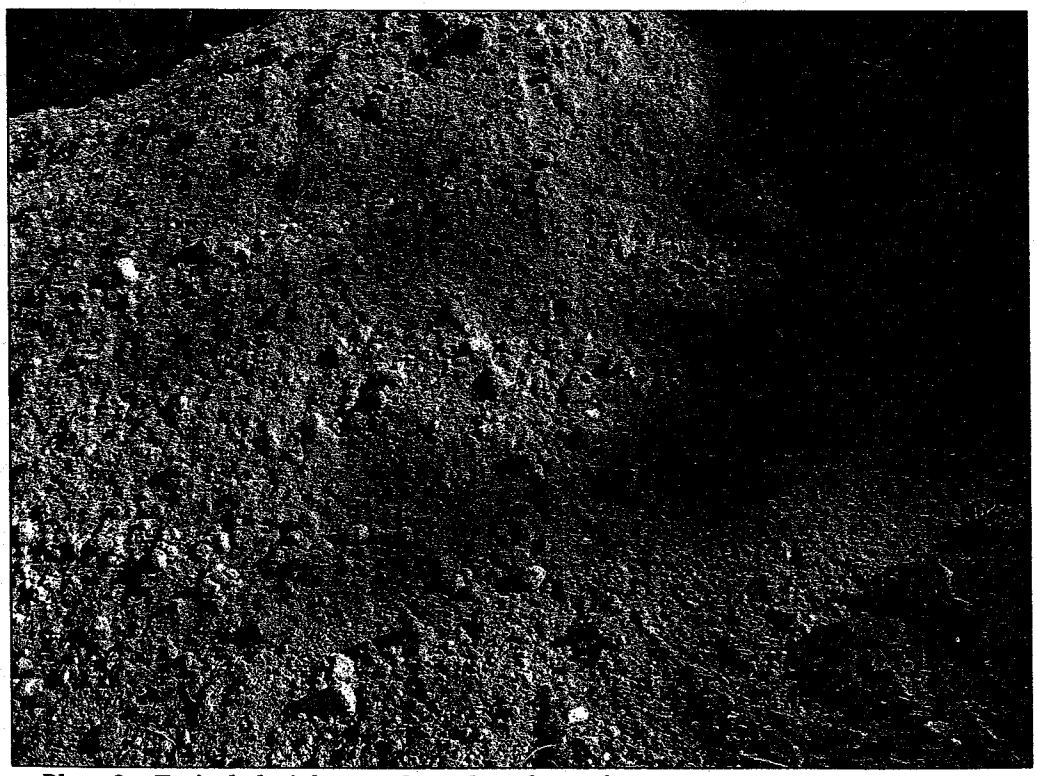

Photo 2 – Typical glacial outwash sands and gravels.

Photo 3 – This photo depicts active erosional processes; sand and gravel are sloughing off of a near-vertical exposure (not shown) onto the bluff slope.

Photo 4 – This photo shows the 40- to 50-degree-angle bluff slope. Slight butt-bowing, evident at base of the conifer, is a result of slope creep (slow, gravity-induced movement) in loose colluvial soils.

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Date: March 21, 2006
To: Ms. Priscilla Zimmerman
Zimmerman Architecture

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