GEOTECH
CONSULTANTS, INC.

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March 22, 2002

JN 02070

Village Community Services
18932 – 66th Avenue Northeast, Suite B
Arlington, Washington 98223

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Utilities Div.

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Office Building
Lot 9 – 166th Street Northeast
Arlington, Washington

Ladies and Gentlemen:

We are pleased to present this geotechnical engineering report for the proposed office building to be constructed in Arlington, Washington. The scope of our work consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations, retaining walls and pavements. This work was authorized by your acceptance of our proposal, P-5708, dated February 13, 2002.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Jeffrey M. Johnson
Geotechnical Engineer

cc: **Ed Linardic**—LDG Architects
via facsimile: (206) 283-1293

MRM/JMJ: esm

GEOTECHNICAL ENGINEERING STUDY
Proposed Office Building
Lot 9 – 166th Street Northeast
Arlington, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed office building to be located in Arlington, Washington.

We were provided with an undated site plan that was developed by LDG Architects. This plan indicated the building and parking lot locations relative to the property lines. Based on this plan and conversations with the architect, we anticipate that the lot will be developed with a two-story office building and on-grade parking. The lower floor will be near the existing site grade. A storm detention pond will likely be constructed for stormwater control.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site. The irregularly shaped site has approximately 175 feet of road frontage on the south side of the property along 166th Street Northeast and a depth of approximately 135 feet. The width along the north side of the property narrows to approximately 65 feet. The site was cleared of most vegetation prior to our field explorations. Short grass and the remnant bases of Scotch broom covered the ground surface. A medium-sized pine tree was standing in the southwestern corner. Topography of the site was nearly flat.

The adjacent lots to the north and east are developed with one-story, self-storage units and a parking area for trailer storage, respectively. The lots west of the site are undeveloped. Grading for new construction was being done on the lots to the south of the property across 166th Street Northeast. Development in the neighborhood to the east mostly consisted of on-grade, two-story, single-family residences.

SUBSURFACE

The subsurface conditions were explored by excavating four test pits at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The test pits were excavated on March 1, 2002 with a rubber-tired backhoe. A geotechnical engineer from our staff observed the excavation process, logged the test pits, and obtained representative samples of the soil encountered. "Grab" samples of selected subsurface soil were collected from the backhoe bucket. The Test Pit Logs are attached to this report as Plates 3 and 4.

Soil Conditions

Our explorations encountered similar conditions in the four test pits. We generally observed native, loose sands that became medium-dense below 2 to 3 feet in depth. Approximately one foot of loose fill soil was encountered over the native sands in the southwest site corner. No significant thickness of highly organic topsoil was encountered. Medium-dense sands were encountered to the maximum explored depth of 9 feet.

The Soil Survey of Snohomish County Area, Washington indicates that the soil is custer fine sandy loam (Type 13).

Groundwater Conditions

Groundwater seepage was observed below a depth of 5 to 6 feet in all of the test pits. The test pits were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. We anticipate that groundwater levels will vary seasonally with rainfall and other factors. However, because the test pits were conducted following an unusually wet fall and winter, the groundwater levels should be near their seasonal high.

The final logs represent our interpretations of the field logs. The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. The relative densities and moisture descriptions indicated on the test pit logs are interpretive descriptions based on the conditions observed during excavation.

The compaction of backfill was not in the scope of our services. Loose soil will therefore be found in the area of the test pits. If this presents a problem, the backfill will need to be removed and replaced with structural fill during construction.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test pits conducted for this study encountered loose sands that become medium-dense within 2 to 3 feet. Heavy groundwater seepage was observed below a depth of 5 to 6 feet. In our opinion, the proposed building can be supported on conventional foundations that bear on the medium-dense, native soils. The continuous foundations under the main bearing walls should be designed similar to grade beams, being theoretically able to span a distance of 10 feet without soil support. This creates a relatively rigid foundation system that would reduce differential settlement in the event of an earthquake. The footings should be excavated with a smooth bucket to reduce disturbance of the bearing surfaces. The footing subgrades should be recompacted with a jumping jack compactor before pouring concrete. Slab-on-grade floors may be supported on compacted, non-organic native soils or structural fill.

Because of the relatively shallow depth of groundwater seepage encountered in the test pits, subsurface infiltration of stormwater will not be feasible. Excavations for a stormwater detention system and other site utilities will be difficult because they extend close to, or below, groundwater seepage levels, and could require temporary dewatering or flattened cut slopes. Waterproofing will be necessary for portions of a storm detention vault that will be below the seasonal high groundwater level.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. While site clearing will expose a large area of bare soil, the erosion potential on the site is low due to the gentle slope of the ground and granular nature of the soils. Rocked construction access roads should be extended into the site to reduce the amount of mud carried off the property by trucks and equipment. Wherever possible, these roads should follow the alignment of planned pavements.

SEISMIC CONSIDERATIONS

The site is located within Seismic Zone 3, as illustrated on Figure No. 16-2 of the 1997 Uniform Building Code (UBC). In accordance with Table 16-J of the 1997 UBC, the site soil profile within 100 feet of the ground surface is best represented by Soil Profile Type S_D (Stiff Soil). The site soils have a low to moderate potential for seismic liquefaction due to the presence of near surface groundwater in the medium-dense soils.

Foundation recommendations provided in this report are intended to prevent catastrophic settlement of the foundations. By preventing catastrophic settlement of the foundations, the safety of the occupants should be protected. This conforms with the intent of Section 1626.1 of the 1997 UBC, which requires that the design "safeguards against major structural failures and loss of life." The intent is not to prevent damage or ensure continued function of the structure after the design seismic event.

CONVENTIONAL FOUNDATIONS

The proposed structure can be supported on conventional continuous and spread footings bearing on medium-dense, native sand or on structural fill placed above this competent, native soil. The ***GENERAL*** section should be reviewed for additional foundation considerations. See the section entitled ***GENERAL EARTHWORK AND STRUCTURAL FILL*** for recommendations regarding the placement and compaction of structural fill beneath structures. Adequate compaction of structural fill should be verified with frequent density testing during fill placement. Prior to placing structural fill beneath foundations, the excavation should be observed by the geotechnical engineer to document that adequate bearing soils have been exposed. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface. The local building codes should be reviewed to determine if different footing widths or embedment depths are required.

Depending on the final site grades, some overexcavation may be required below the footings to expose competent, native soil. Unless lean concrete is used to fill an overexcavated hole, the overexcavation must be at least as wide at the bottom as the sum of the depth of the overexcavation and the footing width. For example, an overexcavation extending 2 feet below the bottom of a 2-foot-wide footing must be at least 4 feet wide at the base of the excavation. If lean concrete is used, the overexcavation need only extend 6 inches beyond the edges of the footing.

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for footings supported on structural fill placed above medium-dense, native soil. A 3,000 psf bearing pressure

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for footings supported on structural fill placed above medium-dense, native soil. A 3,000 psf bearing pressure can be used if footings are placed directly on medium-dense, native soil. A one-third increase in these design bearing pressures may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent, native soil, or on structural fill up to 5 feet in thickness, will be less than one inch, with differential settlements on the order of one-half inch in a distance of 50 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level structural fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	300 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) passive earth pressure is computed using the equivalent fluid density.

We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PERMANENT FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	35 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.40
Soil Unit Weight	120 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) active and passive earth pressures are computed using the equivalent fluid pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The values given above are to be used to design permanent foundation and retaining walls only. The values for friction and passive resistance are ultimate values and do not include a safety factor. We recommend a safety factor of at least 1.5 for overturning and sliding, when using the above values to design the walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density.

Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction.

Retaining Wall Backfill

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. If the native sand is used as backfill, a minimum 12-inch width of free-draining gravel or a drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The drainage composites should be hydraulically connected to the foundation drain system. Free-draining backfill or gravel should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. The section entitled ***GENERAL EARTHWORK AND STRUCTURAL FILL*** contains recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls. The performance of subsurface drainage systems will degrade over time. Therefore, waterproofing should be provided where moist conditions or some seepage through the walls are not acceptable in the future. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. Applying a thin coat of asphalt emulsion is not considered waterproofing, but will only help to prevent moisture, generated from water vapor or capillary action, from seeping through the concrete. With any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that may be transmitted through concrete walls from the surrounding soil.

SLABS-ON-GRADE

The building floors may be constructed as slabs-on-grade atop compacted, non-organic native soils, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

All slabs-on-grade should be underlain by a capillary break or drainage layer consisting of a minimum 4-inch thickness of coarse, free-draining structural fill with a gradation similar to that discussed in ***PERMANENT FOUNDATION AND RETAINING WALLS***. As noted by the American Concrete Institute (ACI) in Section 3.2.3 of the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders*, such as 6-mil visqueen, are typically used. A vapor retarder is defined as a material with a permeance of less than 0.3 US perms per square foot (psf) per hour, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. However, if no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.00 perms per square foot per hour when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height cannot be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Flatter cut slopes, shoring, and/or dewatering will be needed for cuts extending close to, or below, the level of groundwater.

The above-recommended temporary slope inclination is based on what has been successful at other sites with similar soil conditions. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. The cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that sand or loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger.

DRAINAGE CONSIDERATIONS

Foundation drains should be used where (1) crawl spaces or basements will be below a structure, (2) a slab is below the outside grade, or (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock and then wrapped in non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space, and it should be sloped for drainage. All roof and surface water drains must be kept separate from the foundation drain system. A typical drain detail is attached to this report as Plate 5. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a building should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls.

PAVEMENT AREAS

The pavement section may be supported on competent, native soil or structural fill compacted to a 95 percent density. Because the site soils are silty and moisture sensitive, we recommend that the pavement subgrade must be in a stable, non-yielding condition at the time of paving. Granular structural fill or geotextile fabric may be needed to stabilize soft, wet, or unstable areas. To evaluate pavement subgrade strength, we recommend that a proof roll be completed with a loaded dump truck immediately before paving. In most instances where unstable subgrade conditions are encountered, an additional 12 inches of granular structural fill will stabilize the subgrade, except for very soft areas where additional fill could be required. The subgrade should be evaluated by Geotech Consultants, Inc., after the site is stripped and cut to grade. Recommendations for the compaction of structural fill beneath pavements are given in the section entitled ***GENERAL EARTHWORK AND STRUCTURAL FILL***. The performance of site pavements is directly related to the strength and stability of the underlying subgrade.

The pavement for lightly-loaded traffic and parking areas should consist of 2 inches of asphalt concrete (AC) over 6 inches of crushed rock base (CRB) or 3 inches of asphalt-treated base (ATB). We recommend providing heavily-loaded areas with 3 inches of AC over 9 inches of CRB or 4 inches of ATB. Heavily-loaded areas are typically main driveways, dumpster sites, or areas with truck traffic. The above-recommended CRB thicknesses are somewhat thicker than "typical" to deform under thin pavement sections.

The pavement section recommendations and guidelines presented in this report are based on our experience in the area and on what has been successful in similar situations. As with any pavements, some maintenance and repair of limited areas can be expected as the pavement ages. To provide for a design without the need for any repair would be uneconomical.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

Use of On-Site Soil

The site soils are generally suitable for use as structural fill. As with all fine-grained sands they can be difficult to compact to required compaction levels and may require reworking to obtain satisfactory compaction levels. If grading activities take place during periods of extended wet weather, the site soils may not be suitable for structural fill because of high moisture content, consequently site preparation costs may be higher because of delays due to rain and the potential need to import granular fill.

The moisture content of the site soil must be at, or near, the optimum moisture content, as the soil cannot be consistently compacted to the required density when the moisture content is significantly greater than optimum. During excessively dry weather, however, it may be necessary to add water to achieve the optimum moisture content.

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test pits are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples in test pits. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Village Community Services, and its representatives, for specific application to this project and site. Our recommendations and conclusions are based on observed site materials. Our conclusions and recommendations are professional opinions derived in accordance with current standards of practice within the scope of our services and within budget and time constraints. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 4	Test Pit Logs
Plate 5	Typical Footing Drain

We appreciate the opportunity to be of service on this project. If you have any questions, or if we may be of further service, please do not hesitate to contact us.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Jeffrey M. Johnson
Geotechnical Engineer

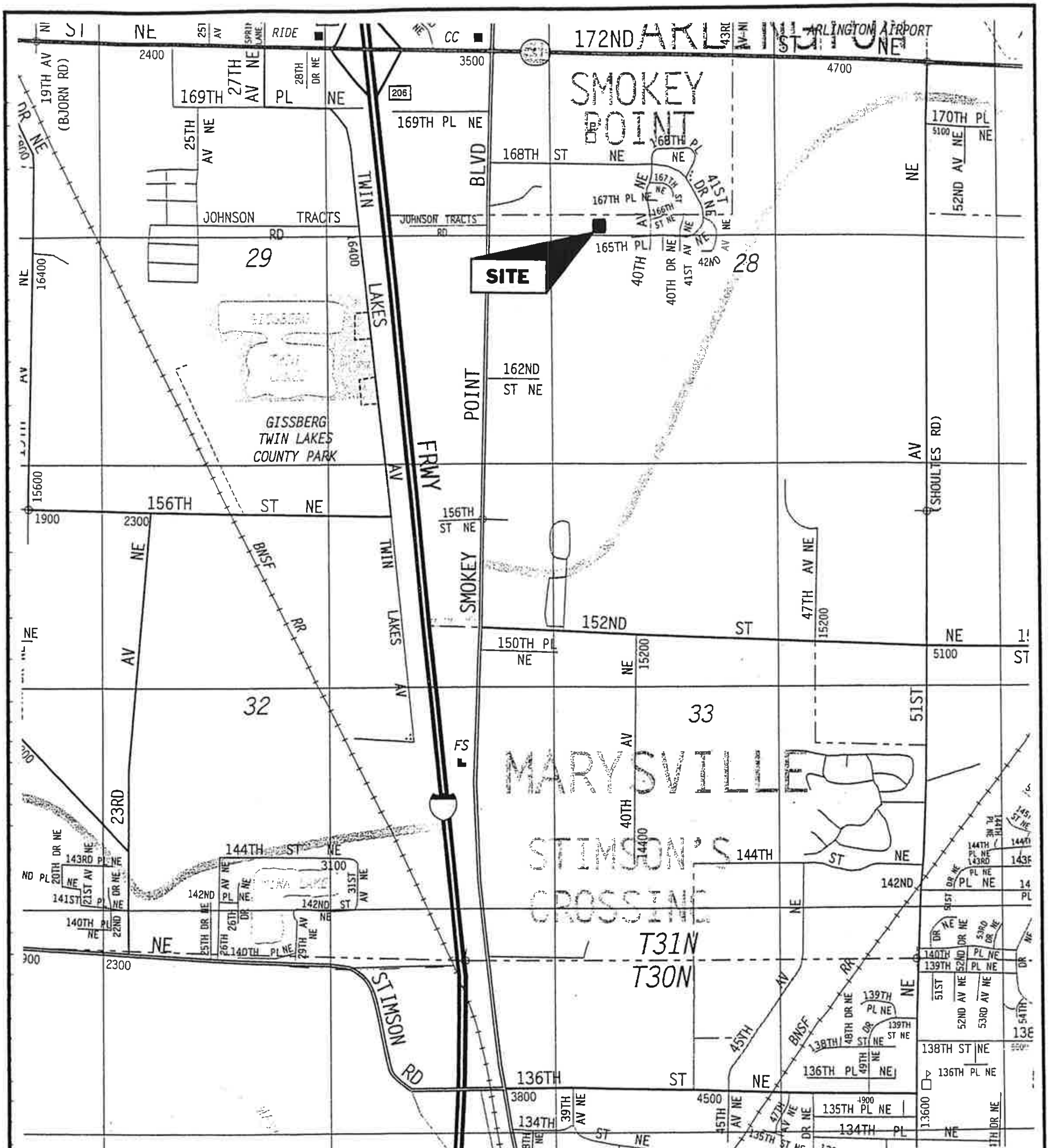
MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

3/22/02

EXPIRES 10/25/03

Marc R. McGinnis, P.E.
Principal

JMJ/MRM: esm



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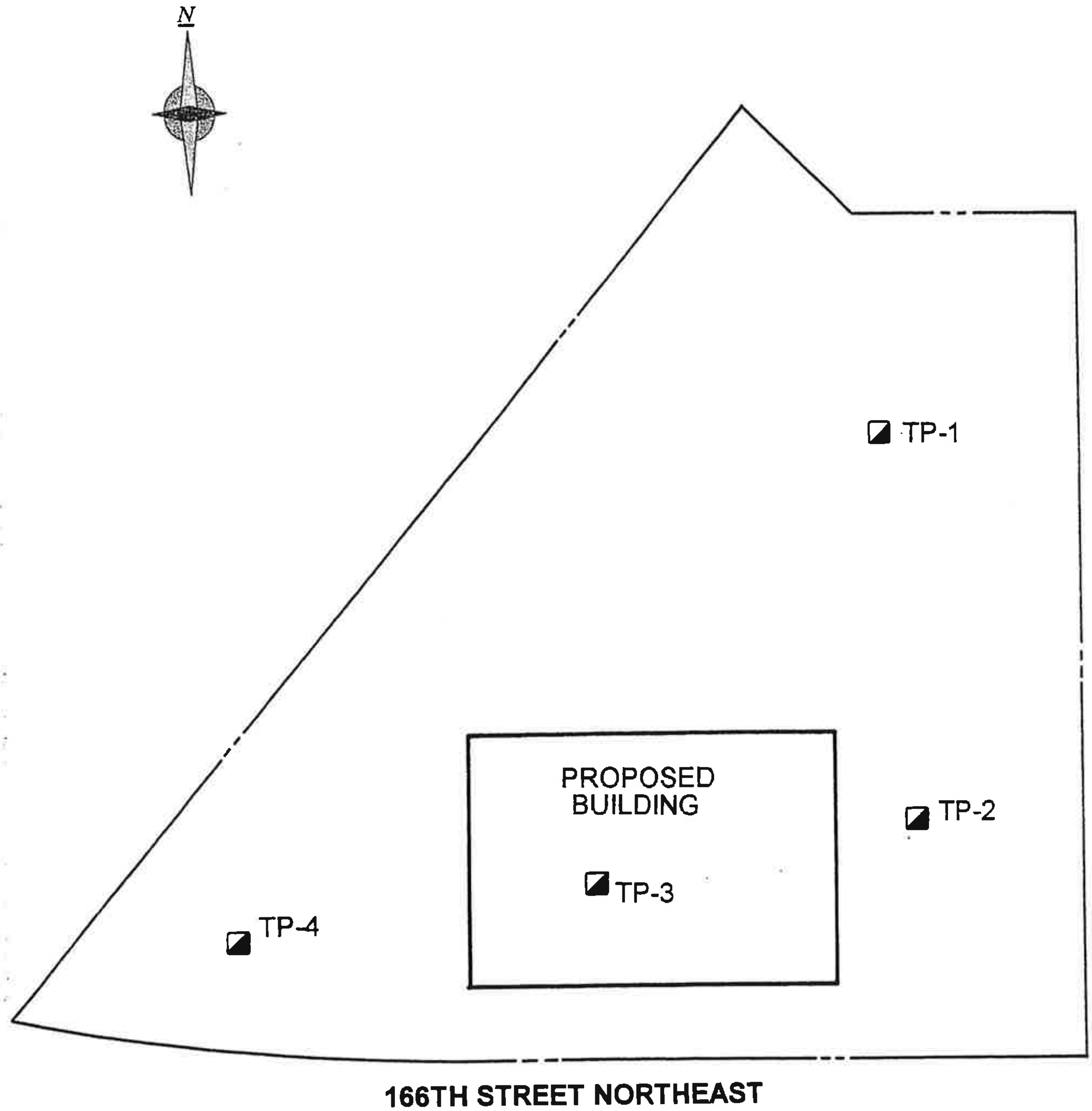
VICINITY MAP

Lot 9, 166th Street NE
Arlington, Washington

Job No:	Date:	Scale:	Plate:
02070	March 2002.	Not to Scale	1



LEGEND:

◪ APPROXIMATE BORING LOCATIONS

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SITE EXPLORATION PLAN

Lot 9, 166th Street NE
Arlington, Washington

Job No:	Date:	Scale:	Plate:
02070	March 2002.	Not to Scale	2

TEST PIT 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SP	Orange-gray SAND, fine- to medium-grained, moist, loose
				-becomes gray-brown with trace of gravel, medium-dense
5		▼		-becomes very moist
				-becomes wet
				-becomes gravelly and coarse-grained
10				
15				

* Test Pit was terminated at 9 feet due to caving soil on March 1, 2002.
* Heavy groundwater seepage was observed below 5 feet during excavation.
* Heavy caving was observed below 5 feet during excavation.

TEST PIT 2

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SP	Orange-gray SAND, fine- to medium-grained, moist, loose
				-becomes gray-brown with trace of gravel, medium-dense
				-gravel pocket
5		▼		-becomes gray, wet
10				
15				

* Test Pit was terminated at 9 feet due to caving soil on March 1, 2002.
* Heavy groundwater seepage was observed below 5 feet during excavation.
* Heavy caving was observed below 5 feet during excavation.

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TEST PIT LOG

Lot 9, 166th Street NE
Arlington, Washington

Job No:	Date:	Logged by:	Plate:
02070	March 2002	JMJ	3

TEST PIT 3

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SP	Orange-gray SAND, fine- to medium-grained, moist, loose
				-becomes gray-brown with trace of gravel, medium-dense
				-becomes very moist
5		▼		-becomes wet
				-becomes gravelly and coarse-grained
10				
15				

* Test Pit was terminated at 9 feet due to caving soil on March 1, 2002.
* Heavy groundwater seepage was observed below 5 feet during excavation.
* Heavy caving was observed below 5 feet during excavation.

TEST PIT 4

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Orange-brown slightly silty SAND, fine- to medium-grained, moist, loose
			SP	Orange-gray SAND, fine- to medium-grained, moist, loose
				-becomes gray-brown, medium-dense
5		▼		-becomes wet
				-becomes coarse-grained
10				
15				

* Test Pit was terminated at 8 feet on March 1, 2002.
* Heavy groundwater seepage was observed below 6 feet during excavation.
* Heavy caving was observed below 3 feet during excavation.

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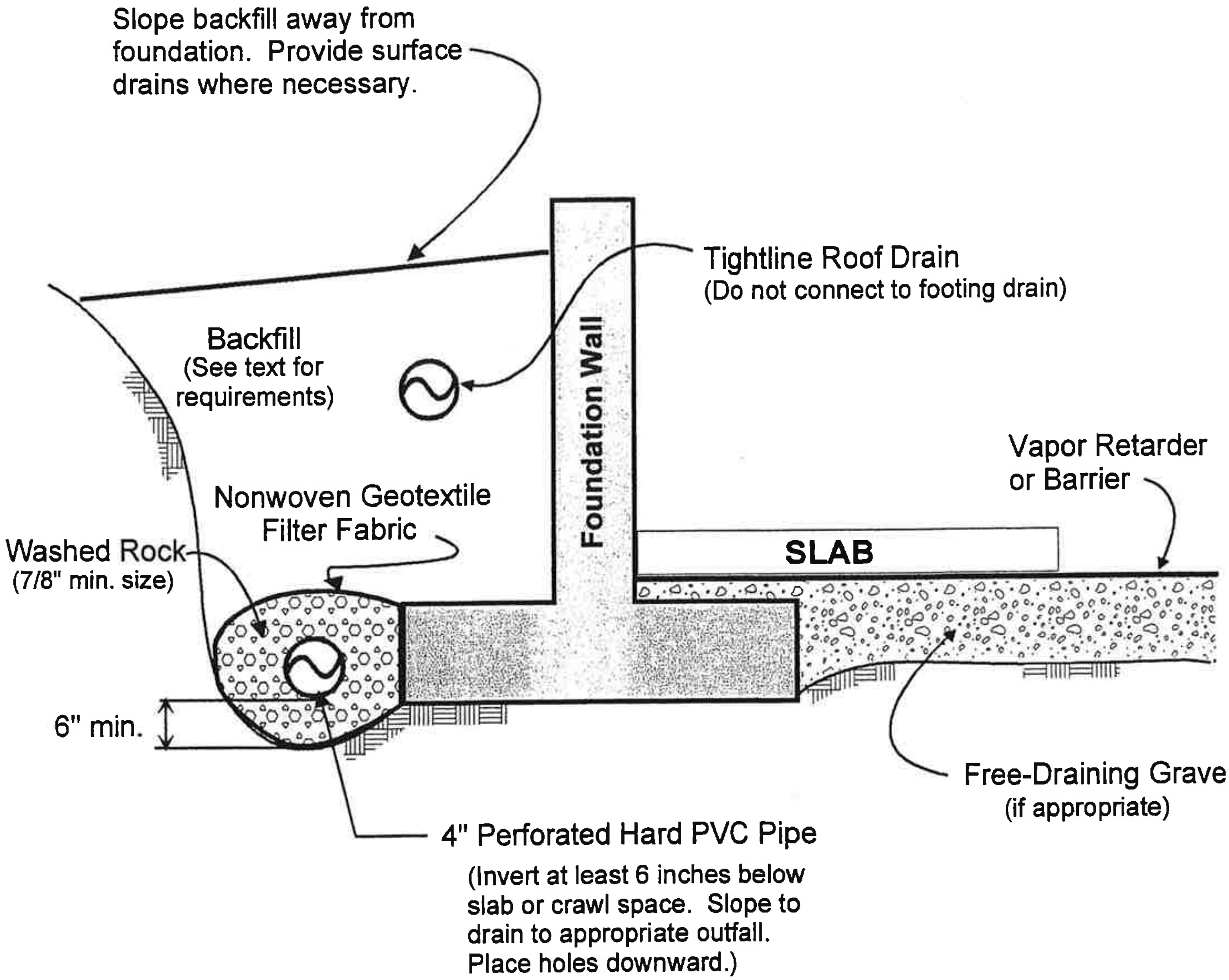
TEST PIT LOG

Lot 9, 166th Street NE
Arlington, Washington

Job No:	Date:	Logged by:	Plate:
02070	March 2002	JMJ	4



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage and waterproofing considerations.

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TYPICAL FOOTING DRAIN

Lot 9, 166th Street NE
Arlington, Washington

Job No:	Date:	Scale:	Plate:
02070	March 2002	Not to Scale	5