GEOTECHNICAL ENGINEERING REPORT
Arlington Fire Station
172nd Street NE and 43rd Avenue NE
Arlington, Washington

January 7, 2008

Prepared for

City of Arlington

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January 7, 2008
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City of Arlington
238 North Olympic Avenue
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Attn.: Paul Ellis

Re: Geotechnical Engineering Evaluation
Proposed Arlington Fire Station
172nd Street NE and 43rd Avenue NE
Arlington, Washington

Dear Mr. Ellis,

As requested, GeoTest Services, Inc. is pleased to submit this preliminary report summarizing the results of our geotechnical engineering evaluation for the referenced project. The purpose of this evaluation was to establish general subsurface conditions beneath the site from which conclusions and recommendations for the proposed development could be formulated. Specifically, our scope of services included the following tasks:

- Borings: due to scheduling availability, exploration of soil and groundwater conditions underlying the site by advancing two to three test borings to evaluate subsurface conditions will take place at the end of January. Upon completion, an addendum report will be submitted which will include observations and recommendations with respect to seismic design considerations including liquefaction hazard potential.

- Exploration of soil and groundwater conditions underlying the site by excavating six exploratory test pits to evaluate subsurface conditions.

- Laboratory testing on representative samples in order to classify and evaluate the engineering characteristics of the soils encountered.

- Provide this written report containing a description of subsurface conditions, test pit logs, and findings and recommendations pertaining to site preparation and earthwork, fill and compaction, wet weather earthwork, foundation support and settlement, slab-on-grade construction, foundation and site drainage including infiltration rates for stormwater design, utilities, paved areas, and geotechnical consultation and construction monitoring.

PROJECT DESCRIPTION

We understand that the site is approximately 3.0 acres in size and is proposed to be entirely redeveloped into the new Arlington Fire Station. At this time the property is heavily forested with a mix of evergreen and deciduous trees. The proposed new building will most likely consist of a mix of concrete, masonry and steel construction with a concrete slab-on-grade floor. Final design of the building has not been decided as of the time of this report. The remaining property is planned to be fully developed with paved parking areas and heavy traffic lanes for truck traffic. We understand that some of the planned paved parking areas may incorporate permeable pavement.

SITE CONDITIONS

This section discusses the general surface and subsurface conditions observed at the project site at the time of our field investigation. Interpretations of the site conditions are based on the results of our review of available information, site reconnaissance, subsurface explorations, and laboratory testing.

General Geologic Conditions

Geologic information for the project site was obtained from the *Surficial Geologic Map of Port Townsend 30- by 60-Minute Quadrangle, Puget Sound Region, Washington* (Pessl, et. al 1989), published by the U.S. Geological Survey. According to Pessl, near-surface soils in the vicinity of the project site consist of glacial recessional-marine deposits of the Vashon Stade of the Fraser Glaciation. Recessional-marine deposits at the site are described by Pessl as a complex assemblage medium to well-sorted, massive to laminated sand, silt, and clay. Thicknesses of the unit typically range from 1 to 10 meters, with exceptional thicknesses observed to be up to 18 meters. Recessional-marine glacial outwash was deposited by meltwater flowing south from the stagnating and receding Vashon glacier. Site soils were relatively consistent with the mapped geology.

Surface Conditions

The site of the proposed improvements is a mostly undeveloped, heavily forested parcel approximately 3.0 acres in size and located on the northeast corner of 172nd Street NE and 43rd Avenue NE in Marysville, Washington, as shown on the Vicinity Map, Figure 1. No structures exist on the subject property. The site topography is nearly flat and vegetation consists mostly of evergreen and deciduous trees. The elevation of the property is situated at approximately the same elevation of 172nd Street NE. Surface water was not encountered within the areas of exploration at the time of our field investigation.

Subsurface Soil Conditions

Subsurface conditions within the areas of interest at the site were explored by excavating and sampling six exploratory test pits with a tracked excavator on December 13, 2007. The test pits (TP-1 through TP-6) were excavated to depths between approximately 9 and 10 feet below the existing ground surface (BGS). The test pits were advanced at locations throughout the project site. The approximate locations of the test pits are shown on the Site and Exploration Plan, Figure 2. A discussion of field exploration and laboratory test procedures, together with edited logs of the test pits, is presented in Appendix A.

The subsurface soil profile generally consisted of topsoil overlying native glacial outwash deposits. At the surface of all explorations soft, dark brown, moist, organic, sandy silt (topsoil) was encountered to depths ranging between approximately 6 to 12 inches BGS. Below the topsoil generally a medium dense, reddish-brown silty sand (SM), weathered glacial outwash, was encountered to a depth of approximately 1.5 to 2 feet BGS. Generally below approximately 1.5 to 2 feet BGS we encountered a medium dense, light brown to gray, dry to moist, poorly graded, fine to coarse sand (SM to SP) to 9 to 10 feet BGS. At 9 to 10 feet BGS, we encountered medium dense, gray, wet, coarse sand (SP) with gravel or gravel with sand (GP) to the full depths of exploration. Please refer to the individual test pit logs, attached with this report, for more detail.

GROUNDWATER

At the time of our subsurface investigation on December 13, 2007, moderate to rapid groundwater seepage was observed at depths between approximately 9 and 10 feet BGS in all explorations. Evidence of an estimated average seasonal high water table elevation, indicated by a contact between upper mottled soil and lower clean soil or a heavily mottled ring or layer, was not observed in any of our explorations. Groundwater levels are not static, and vary due to surface runoff, precipitation, season, changes in site utilization (both on and off site) and other factors. In general, groundwater levels are higher during the wetter winter and spring months.

INFILTRATION ANALYSIS RESULTS

From the explorations excavated throughout the site, eight representative soil samples were selected and mechanically tested for grain size distribution and interpretation according to the United States Department of Agriculture (USDA) soil textural classification and Unified Soil Classification System (USCS) gradation testing approach. Subsurface infiltration rates corresponding to the United States Department of Agriculture (USDA) soil textural classification were determined via the 2005 Washington State Department of Ecology *Stormwater Management Manual for Western Washington* Table 3.7 and are reproduced in Table 1 on the following page. Subsurface infiltration rates corresponding to the D_{10} Size from ASTM D422 Soil Gradation Test were obtained from Table 3.8 in the Department of Ecology (DOE) manual and are reproduced in Table 2. Both systems are referenced in this report so that the designer can best fit the stormwater infiltration system to design criteria and specific site conditions.

TABLE 1 Test Pit Soil Sample Infiltration Rates Based On The 2005 DOE *Stormwater Management Manual for Western Washington* Table 3.7				
Test Pit Number	Sample Depth (ft)	Classification (USDA)	Short-Term Infiltration Rate (Inches/Hour)	Long-Term Infiltration Rate (Inches/Hour)
TP-1	1.5	Sand	8.0	2.0
TP-2	7.0	Sand	8.0	2.0
TP-3	1.5	Sand	8.0	2.0
TP-3	4.0	Sand	8.0	2.0
TP-4	2.0	Sand	8.0	2.0
TP-4	3.0	Sand	8.0	2.0
TP-5	2.5	Sand	8.0	2.0
TP-6	8.0	Sand	8.0	2.0
Note: Both the short term and long term design infiltration rates were listed for use in design as referenced in Table 3.7.				

TABLE 2 Test Pit Soil Sample Infiltration Rates Based On The 2005 DOE *Stormwater Management Manual for Western Washington* Table 3.8				
Test Pit Number	Sample Depth (ft)	USCS Soil Type	D_{10} from ASTM D422 Soil Gradation Test (mm)	Long-Term Infiltration Rate (Inches/hour)
TP-1	1.5	SM	0.06	0.8
TP-2	7.0	SP	0.15	2.0
TP-3	1.5	SM	0.06	0.8
TP-3	4.0	SP	0.20	3.5
TP-4	2.0	SP	0.18	2.0
TP-4	3.0	SP	0.22	3.5
TP-5	2.5	SP-SM	0.08	0.8
TP-6	8.0	SP	0.26	3.5
Notes: Listed infiltration rates are long term (design) rates as stated in Table 3.8.				

Based on the USDA textural classification and our interpretations of our soil logs, the near surface sand encountered within all explorations resulted in a short-term design infiltration rate of 8.0 inches per hour and a long-term design infiltration rate of 2.0 inches per hour using the DOE manual. Based on the D_{10} size from ASTM D422 Soil Gradation Test and our interpretations of our soil logs, the near surface silty sands (SM) and slightly silty sands (SP-SM) generally correlate to a long term design rate of 0.8 inches per hour and the clean sands (SP) correlate to a long-term design rate of 3.5 inches per hour.

Moderate to rapid groundwater seepage was encountered within all six of our explorations at depths ranging from 9 to 10 feet BGS. Evidence of a seasonal high water table, typically indicated by a distinct mottled horizon was not observed. At the request of Cascade Survey and Engineering, we intend to install two piezometers at the site during our subsequent boring exploration program in order to monitor the groundwater table through the wet season. Current DOE specifications require a minimum of 5 feet of separation between the base of an infiltration system and the top of the seasonal high watertable.

In addition, existing on-site native soil encountered at depths below approximately 3 feet BGS are not recommended for treatment purposes due to the relatively high infiltration rates associated with the samples collected below this depth. Therefore, we recommend that at least 18 inches of amended import soil, suitable for treatment purposes, be placed below the proposed infiltration system(s), or other appropriate methods of treatment be incorporated into the stormwater design.

If additional design parameters, such as a groundwater mounding analysis or cation exchange testing, are warranted, GeoTest Services would be pleased to assist in any additional testing and analysis necessary to complete a suitable infiltration design for the subject property.

CONCLUSIONS AND RECOMMENDATIONS

Based upon evaluation of the data collected during this investigation, it is our opinion that subsurface conditions at the site are suitable for the proposed construction, provided the recommendations contained herein are incorporated into the project design. Conventional shallow isolated and continuous footings and slab-on-grade floors are considered feasible for this project.

Site Preparation and Earthwork

The portions of the site to be occupied by foundations, floor slabs-on-grade, pavement, or sidewalk should be prepared by removing all topsoil, any existing fill (if applicable) and significant accumulations of organics from the area to be developed. Prior to placement of any structural fill, the exposed subgrade under all areas to be occupied by soil-supported floor slabs, spread or continuous foundations, and site pavement should be recompacted to a dense and unyielding condition and proof rolled with a loaded dump truck, large self-propelled vibrating roller, or equivalent piece of equipment applicable to the size of the excavation. The purpose of this effort is to identify possible loose or soft soil deposits and recompact the soil exposed during site preparation and excavation activities. We recommend that all topsoil be removed beneath the main entrance road, all areas providing building and/or interior slab-on-grade support and beneath all stormwater infiltration/dispersion areas.

Proof rolling should be carefully observed by qualified geotechnical personnel. Areas exhibiting significant deflection, pumping, or over-saturation that cannot be readily compacted should be overexcavated to firm soil. Overexcavated areas should be backfilled with compacted granular material placed in accordance with subsequent recommendations for structural fill. During periods of wet weather, proof rolling could damage the exposed subgrade. Under these conditions, qualified geotechnical personnel should observe subgrade conditions to determine if proof rolling is applicable.

Fill and Compaction

Structural fill used to obtain final elevations for footings and soil-supported floor slabs must be properly placed and compacted. In general, any suitable, non-organic, predominantly granular soil may be used for fill material, including portions of the existing onsite soil, provided the material is properly moisture conditioned prior to placement and compaction, and the specified degree of compaction is obtained. If the existing onsite soil is to be reused for structural fill, any cobbles or other material greater than about 6 inches in diameter should be removed. Excavated site material containing topsoil, wood, trash, organic material, or other debris will not be suitable for reuse as structural fill and should be properly disposed offsite or placed in nonstructural areas.

Reuse of Onsite Soil

GeoTest does not recommend reuse of the topsoil or any uncontrolled fill as structural fill under foundation elements. Native soils underlying the site, consisting of generally granular glacial outwash deposits, may be utilized for general site backfill if they are properly moisture conditioned and recompacted. Based on the results of our field classification and the laboratory testing performed on representative samples, native site soils have “fines” contents (percent passing the U.S. No. 200 sieve) generally between approximately 1 and 20 percent of the dry weights. Soils containing more than approximately 5 percent “fines” are considered moisture-sensitive, and can be very difficult to compact to a firm and unyielding condition when over the optimum moisture content by more than approximately 2 percent. The optimum moisture content is that which allows the greatest dry density to be achieved at a given level of compactive effort. The moisture contents of a native soil samples recovered from above the groundwater table generally ranged from 2 to 36 percent of the dry weight. The moisture content was estimated to be near to slightly below the optimum moisture content for the silty sands and sand soils encountered in our explorations.

Imported Structural Fill

We recommend that imported structural fill consist of clean, well-graded sandy gravel, gravelly sand, or other approved naturally occurring granular material (pit run) with at least 40 percent retained on the No. 4 sieve, or a well-graded crushed rock. Structural fill for dry weather construction may contain on the order of 10% fines (that portion passing the U.S. No. 200 sieve) based on the portion passing the U.S. No. 4 sieve. Soil containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is greater than optimum. Accordingly, we recommend that imported structural fill with less than 5% fines be used during wet weather conditions. Due to wet weather or wet site conditions, soil moisture contents could be high enough that it may be very difficult to compact even “clean” imported select granular fill to a firm and unyielding condition. Soils with over-optimum moisture contents should be either scarified and dried back to more suitable moisture contents during periods of dry weather or removed and replaced with fill soils at a more suitable range of moisture contents. We recommend that a geotechnical engineer familiar with the project specifications review the material proposed for use as structural fill prior to import to the site.

Backfill and Compaction

Structural fill should be placed in horizontal lifts approximately 8 to 10 inches in loose thickness and thoroughly compacted. All structural fill placed under building areas should be compacted to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. In paved areas, the fill should be compacted to at least 92 percent, except the upper 24 inches of subgrade, which should be compacted to a minimum of 95 percent of maximum dry density. The top of the compacted structural fill should extend outside the base of all foundations and other structural improvements a minimum distance equal to the thickness of the fill. We recommend that compaction be tested after placement of each lift in the fill pad.

Wet Weather Earthwork

As described above, the upper portion of the onsite soils are considered to be moisture sensitive up to two feet BGS. It is our experience that the existing near surface soils are particularly susceptible to degradation during wet weather. As a result, it may be difficult to control the moisture content of the site soils during the wet season. If construction is accomplished during wet weather, we recommend that structural fill consist of imported, clean, well-graded sand or sand and gravel as described above. If fill is to be placed or earthwork is to be performed in wet weather or under wet conditions, the contractor may reduce soil disturbance by:

- Limiting the size of areas that are stripped of topsoil and left exposed
- Accomplishing earthwork in small sections
- Limiting construction traffic over unprotected soil
- Sloping excavated surfaces to promote runoff
- Limiting the size and type of construction equipment used
- Providing gravel "working mats” over areas of prepared subgrade
- Removing wet surficial soil prior to commencing fill placement each day
- Sealing the exposed ground surface by rolling with a smooth drum compactor or rubber-tired roller at the end of each working day
- Providing upgradient perimeter ditches or low earthen berms and using temporary sumps to collect runoff and prevent water from ponding and damaging exposed subgrades.

Foundation Support and Settlement

Foundation support for the structure for the proposed improvements may be provided by continuous or isolated spread footings founded on the proof-rolled, undisturbed, medium dense to dense, native sand unit or on properly compacted structural fill placed directly over undisturbed inorganic native soils. To provide proper support, we recommend that any existing fill or organic topsoil beneath areas to be developed be removed and replaced with properly compacted structural fill as described above. All continuous and isolated spread footings should have minimum widths of 18 and 24 inches, respectively, and should be founded a minimum of 18 inches below the lowest adjacent final grade for freeze/thaw protection.

Allowable Bearing Capacity

Assuming the above foundation support criteria are satisfied, continuous or isolated spread footings founded directly on the inorganic, native, brown to gray, medium dense to dense, sand unit (glacial outwash) or on compacted structural fill placed directly over suitably prepared native soils may be proportioned using a maximum net allowable soil bearing pressure of 2,000 pounds per square ft (psf). If footings are constructed on a minimum of 18 inches of properly placed and compacted granular structural fill, as described in this report, the allowable soil bearing pressure may be increased to 2,500 psf. The term "net allowable bearing pressure" refers to the pressure that can be imposed on the soil at foundation level resulting from the total of all dead plus live loads, exclusive of the weight of the footing or any backfill placed above the footing. The net allowable bearing pressure may be increased by one-third for transient wind or seismic loads.

Foundation Settlement

Settlement of shallow foundations depends on foundation size and bearing pressure, as well as the strength and compressibility characteristics of the underlying soil. Assuming construction is accomplished as previously recommended and for the maximum allowable soil bearing pressure recommended above, we estimate the total settlement of building foundations should be less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil should be approximately ½ the total settlement. The soil response to applied stresses caused by building and other loads is expected to be mostly elastic in nature, with most of the settlement occurring during construction as loads are applied.

Resistance to Lateral Loads

Passive earth pressures developed against the sides of building foundations, in conjunction with friction developed between the base of the footings and the supporting subgrade, will resist lateral loads transmitted from the structure to its foundation. For design purposes, the passive resistance of well-compacted fill placed against the sides of foundations may be considered equivalent to a fluid with a density of 250 pounds per cubic ft. The recommended value includes a safety factor of about 1.5 and is based on the assumption that the ground surface adjacent to the structure is level in the direction of movement for a distance equal to or greater than twice the embedment depth. The recommended value also assumes drained conditions that will prevent the buildup of hydrostatic pressure in the compacted fill. In design computations, the upper 12 inches of passive resistance should be neglected if the soil is not covered by floor slabs or pavement. If future plans call for the removal of the soil providing resistance, the passive resistance should not be considered.

An allowable coefficient of base friction of 0.30, applied to vertical dead loads only, may be used between the underlying imported granular structural fill and the base of the footing. The coefficient of base friction should be reduced to 0.25 if the footings will be bearing on the native near surface silty sand soils. If passive and frictional resistance are considered together, one half the recommended passive soil resistance value should be used since larger strains are required to mobilize the passive soil resistance as compared to frictional resistance. A safety factor of about 1.5 is included in the base friction design value. We do not recommend increasing the coefficient of friction to resist seismic or wind loads.

Concrete Slabs-on-Grade

Conventional slab-on-grade floor construction is considered feasible for the planned site improvements. Floor slabs may be supported on properly prepared native subgrade or on compacted structural fill placed over properly prepared native subgrade. Prior to placement of the structural fill, the subgrade should be proof-rolled as recommended in the *Site Preparation and Earthwork* section of this report.

We recommend that interior concrete slab-on-grade floors be underlain by a minimum of 4 inches of compacted, clean, free-draining gravel with less than 5 percent passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The purpose of this layer is to provide uniform support for the slab, provide a capillary break, and act as a drainage layer. To help reduce the potential for water vapor migration through floor slabs, a continuous impermeable membrane or a 6 to 10-mil polyethylene sheeting with tape-sealed joints can be installed below the slab. The American Concrete Institute (ACI) guidelines suggest that the slab may either be poured directly on the vapor retarding membrane or on a granular curing layer placed over the vapor retarding membrane depending on conditions anticipated during construction. We recommend that the architect or structural engineer specify if a curing layer should be used. If moisture control within the building is critical, we recommend an inspection of the vapor retarding membrane to verify that all openings have been properly sealed.

Exterior concrete slabs-on-grade, such as sidewalks, may be supported directly on undisturbed native or on properly placed and compacted structural fill; however, long-term performance will be enhanced if exterior slabs are placed on a layer of clean, durable, well-draining granular material.

Foundation and Site Drainage

To reduce the potential for groundwater and surface water to seep into interior spaces we recommend that an exterior footing drain system be constructed around the perimeter of new building foundations as shown in the Typical Footing and Wall Drain Section, Figure 3. The drain should consist of a minimum 4-inch diameter perforated pipe, surrounded by a minimum 12 inches of filtering media with the discharge sloped to carry water to a suitable collection system. The filtering media may consist of open-graded drain rock wrapped by a nonwoven geotextile fabric (such as Mirafi 140N, Synthetic Industries 351, or equivalent) or a graded sand and gravel filter. The drainage backfill should contain less than 3 percent by weight passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The invert of the footing drain pipe should be placed at approximately the same elevation as the bottom of the footing or 12 inches below the adjacent floor slab grade, whichever is deeper, so that water will not seep through walls or floor slabs. The footing drain should discharge to an approved drain system and include cleanouts to allow periodic maintenance and inspection.

Positive surface gradients should be provided adjacent to proposed buildings to direct surface water away from the foundation and toward suitable discharge facilities. Roof drainage should not be introduced into the perimeter footing drains, but should be separately discharged directly to the stormwater collection system or other appropriate outlet. Pavement and sidewalk areas should be sloped and drainage gradients should be maintained to carry all surface water away from the building towards the local stormwater collection system. Surface water should not be allowed to pond and soak into the ground surface near buildings areas during or after construction. Where applicable, construction excavations should be sloped to drain to sumps where water from seepage, rainfall, and runoff can be collected and pumped to a suitable discharge facility.

Utilities

It is anticipated that excavations for new underground utilities will generally be in medium dense to dense, native, sandy glacial soils. A tracked hydraulic excavator or rubber-tired backhoe with sufficient reach should be able to excavate to the required trench depths without difficulty.

Temporary Excavations

Temporary excavations in excess of 4 ft should be shored or sloped in accordance with Safety Standards for Construction Work Part N, WAC 296-155-657. Temporary unsupported excavations in the sandy onsite native soils are classified as a Type C soil according to WAC 296-155-657 and may be sloped as steep as 1½H:1V. Flatter slopes or temporary shoring may be required in areas where groundwater flow is present and unstable conditions develop.

Surcharge loads on trench support systems due to construction equipment, stockpiled material, and vehicle traffic should be included in the design of any anticipated shoring system. In addition, the contractor should implement measures to prevent surface water runoff from entering trenches and excavations. Vibration as a result of construction activities and traffic may cause caving of the trench walls.

Actual construction trench configurations and maintenance of safe working conditions, including temporary excavation stability, should be the responsibility of the contractor, who is able to monitor the construction activities and has direct control over the means and methods of construction. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored during and after excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring.

Pipe Foundation Support

Provided the trench excavations are properly dewatered and maintained in an undisturbed condition, the native glacial outwash deposits expected at the invert elevations of the proposed pipes will generally provide suitable foundation support for the site utilities. Loosened and/or disturbed native soil will generally provide poor foundation support for pipelines and should be either recompacted, removed, or stabilized.

Utility Trench Backfill

The onsite soil within the depths of the proposed utility trenches is expected to consist primarily of sand. These soils are expected to be suitable for use as trench backfill provided the moisture content is maintained near optimum. The moisture content of the soil observed in the upper portions of our explorations generally appeared to be close to or above optimum; however, the moisture content would be expected to increase to percentages possibly well above optimum if construction is conducted during the wetter months. Any silt or organic-rich soil that is encountered within the proposed utility trenches will not be suitable for use as trench backfill beneath paved roads or where several inches of post-construction settlement is not tolerable. This fine-grained soil could be used as backfill above the proposed pipes in unimproved or landscaped areas where large settlements are not objectionable.

It is important that each section of utility line be adequately supported in the bedding material. Bedding material should be hand tamped to ensure support is provided around the pipe haunches. Fill should be carefully placed and hand tamped to about twelve inches above the pipe crown before heavy compaction equipment is used.

If earthwork must be performed during periods of wet weather, and appropriate levels of compaction cannot be achieved with onsite soil, or if sufficient fill material is not available onsite, we recommend fill material be imported to backfill trenches. Imported trench backfill should meet the requirements for Bank Run Gravel for Trench Backfill in Section 9-03.19 of the 2006 WSDOT/APWA Standard Specifications. The fines content of the trench backfill may be increased somewhat during dry weather conditions, provided the soil is not too wet for proper compaction. If wet weather construction is anticipated, the amount of fines should be limited to 5 percent or less, based on a wet sieve analysis of that portion passing the US No. 4 sieve.

It is recommended that the trench backfill be placed in 8 to 10 inch loose lifts, and compacted using mechanical equipment to at least 95 percent of the maximum dry density as determined by test method ASTM D 1557. In areas where several inches of backfill settlement can be tolerated, such as unimproved or landscaped areas, backfill compaction could be reduced to 85 percent. Figure 4, attached with this report, provides typical utility trench section criteria.

Paved Areas

Selection of a pavement section is typically a compromise between higher initial cost and lower maintenance on one side, and lower initial cost, with more frequent maintenance and less time before an overlay or other maintenance if necessary, on the other. For this reason, we recommend that the Owner participate in the selection of a pavement section for the site. Site grading plans should include provisions for sloping of the native subgrade soils in proposed pavement areas, so that passive drainage of the pavement section(s) can proceed uninterrupted during the life of the project.

Any new pavement sections must be installed over firm subgrade as outlined in the *Site Preparation and Earthwork* section of this report. Following excavation or filling to establish subgrade elevation, but immediately prior to paving, the subgrade surface should be proof-rolled with a loaded dump truck, heavy roller, or equivalent piece of equipment. Soft spots or areas exhibiting excessive movement or pumping exposed by this proof-rolling, which cannot be easily stabilized, should be overexcavated and replaced with suitable granular fill, placed and compacted as previously described. If the subgrade is particularly loose or disturbed by construction equipment during wet weather, a thicker sub-base layer or the use of a geotextile in combination with a granular base material may be needed to achieve suitable conditions for the proposed pavement section.

Pavement sections should be constructed in accordance with the Civil Engineers design or the City of Marysville's Development Standards. In areas of wet subgrade conditions, it may be necessary to separate the granular fill from any encountered native subgrade soils by a suitable geotextile fabric, such as Mirafi 600X or approved equivalent. Pavement grades should be set at 2% minimum to accommodate potential long-term settlement for pavement sections founded in fill areas.

Geotechnical Consultation and Construction Monitoring

We recommend that geotechnical construction monitoring services be provided. These services should include observation by geotechnical personnel during fill placement/compaction activities and subgrade preparation operations to verify that design subgrade conditions are obtained beneath the proposed building and other site improvement areas. We also recommend that periodic field density testing be performed to verify that the appropriate degree of compaction is obtained. The purpose of these services would be to observe compliance with the design concepts, specifications, and recommendations of this report, and in the event subsurface conditions differ from those anticipated before the start of construction, provide revised recommendations appropriate to the conditions revealed during construction. GeoTest Services would be pleased to provide these services for you.

GeoTest Services is also available to provide a full range of materials testing and special inspection during construction as required by the local building department and the International Building Code. This may include specific construction special inspections on materials such as reinforced concrete, reinforced masonry, and structural steel. These services are supported by our fully accredited materials testing laboratory.

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of the City of Arlington and their design consultants for specific application to the design of the proposed Arlington Fire Station project. Use of this report by others or for another project is at the user's sole risk. Within the limitations of scope, schedule, and budget, our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, either express or implied, is made as to the professional advice included in this report.

Our site explorations indicate subsurface conditions at the dates and locations indicated. It is not warranted that they are representative of subsurface conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth of our explorations at the time of our exploration program, a brief geological reconnaissance of the area, and review of published geological information for the site. We assume that the explorations are representative of the subsurface conditions throughout the site during the preparation of our recommendations. If variations in subsurface conditions are encountered during construction, we should be notified for review of the recommendations of this report, and revision of such if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

The earthwork contractor is responsible to perform all work in conformance with all applicable WISHA/OSHA regulations. GeoTest Services, Inc. should not be assumed to be responsible for job site safety on this project, and this responsibility is specifically disclaimed.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the construction phase. If you have any questions regarding the information contained in this report, or if we may be of further service, please contact the undersigned.

Respectfully Submitted,
GeoTest Services, Inc.

State of Washington
Engineering Geologist
950
Licensed Geologist
David P. Bufalini

1/7/08

David Bufalini, L.E.G.
Engineering Geologist

Dong-Soo Lee
State of Washington
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Registered
Professional Engineer
EXPIRES 12/20/08

1/7/08

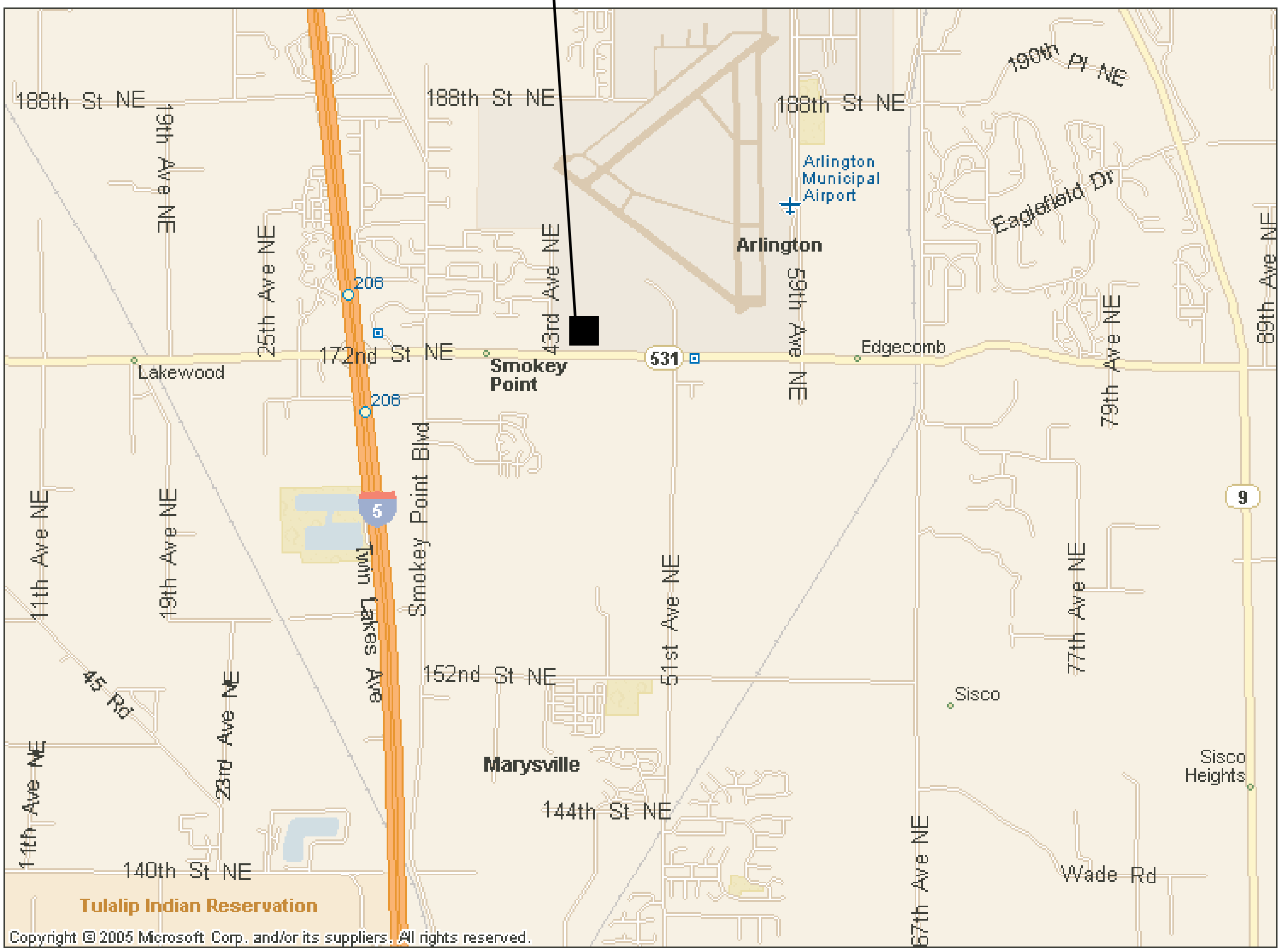
Dong-Soo Lee, P.E.
Geotechnical Engineer

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Figure 3	Typical Footing Drain Section
	Figure 4	Typical Utility Trench Section
	Appendix A	Field Explorations and Laboratory Testing

REFERENCES

Pessl Jr., F, Dethier, D.P., Booth, D.B., Minard, J.P. 1989. *Surficial Geologic Map of Port Townsend 30- by 60-Minute Quadrangle, Puget Sound Region, Washington.* United States Geological Survey. Map I-1198-F.

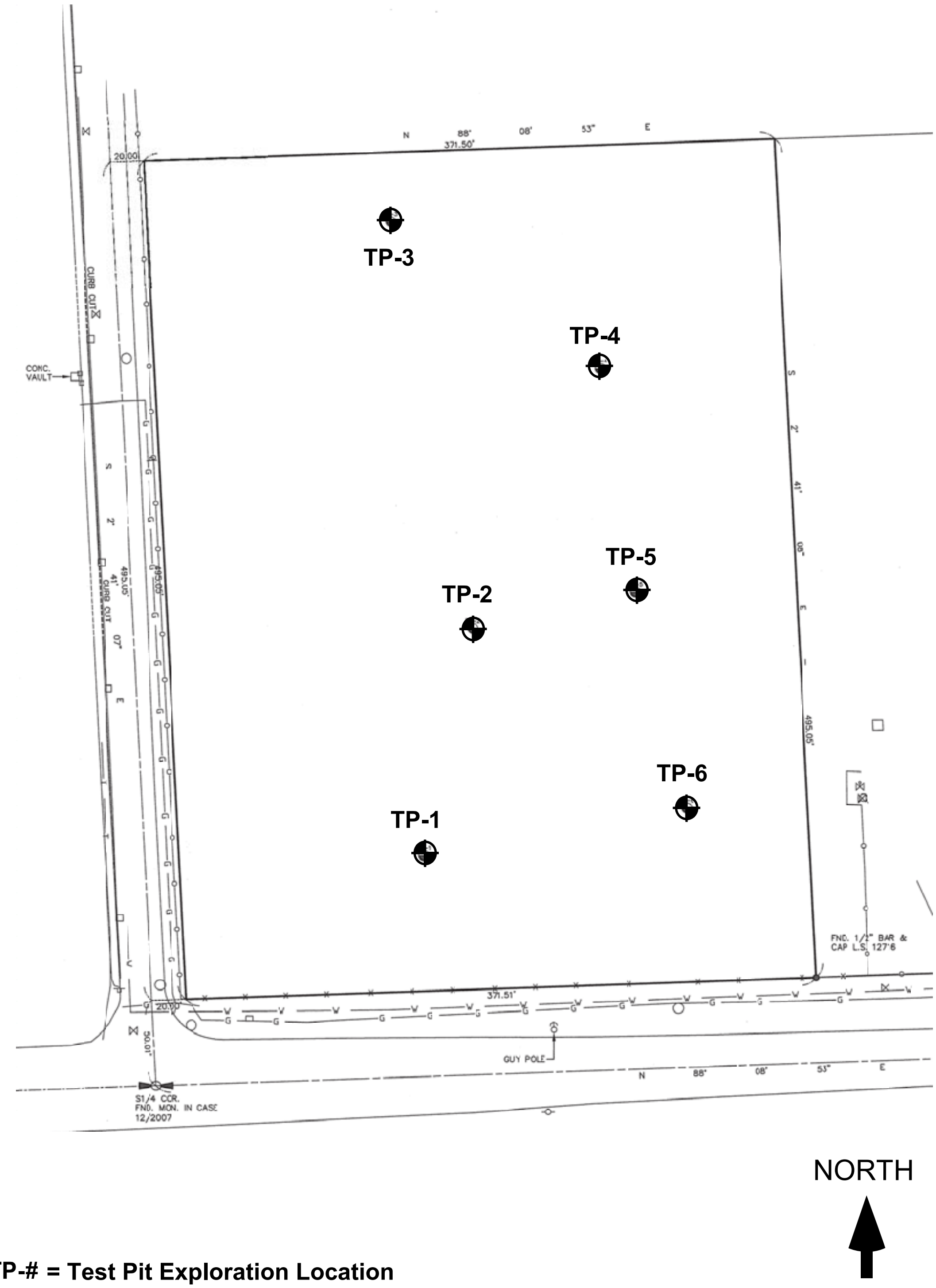
PROJECT LOCATION



NORTH

Reference Map Provided By
Microsoft Streets and Trips 2006

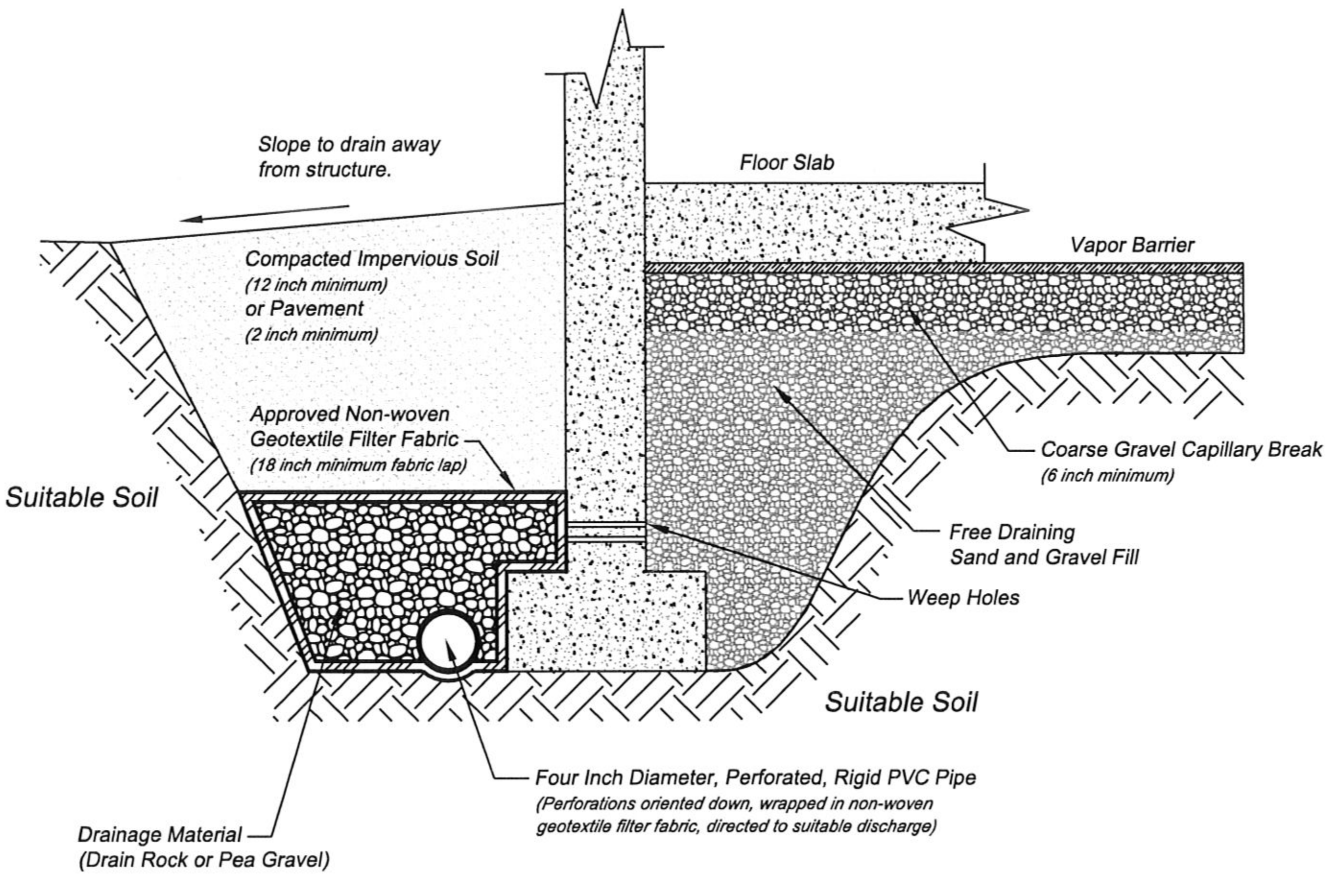
GEOTEST SERVICES, INC. 741 Marine Drive Bellingham, WA 98225 phone: (360) 733-7318 fax: (360) 733-7418	Date: 12-28-07	By: DPB	Scale: NONE	Project **07-0935**
	SITE VICINITY MAP **ARLINGTON FIRE STATION** **172 STREET NE AND 43 AVENUE NE** **ARLINGTON, WASHINGTON**			Figure **1**



TP-# = Test Pit Exploration Location

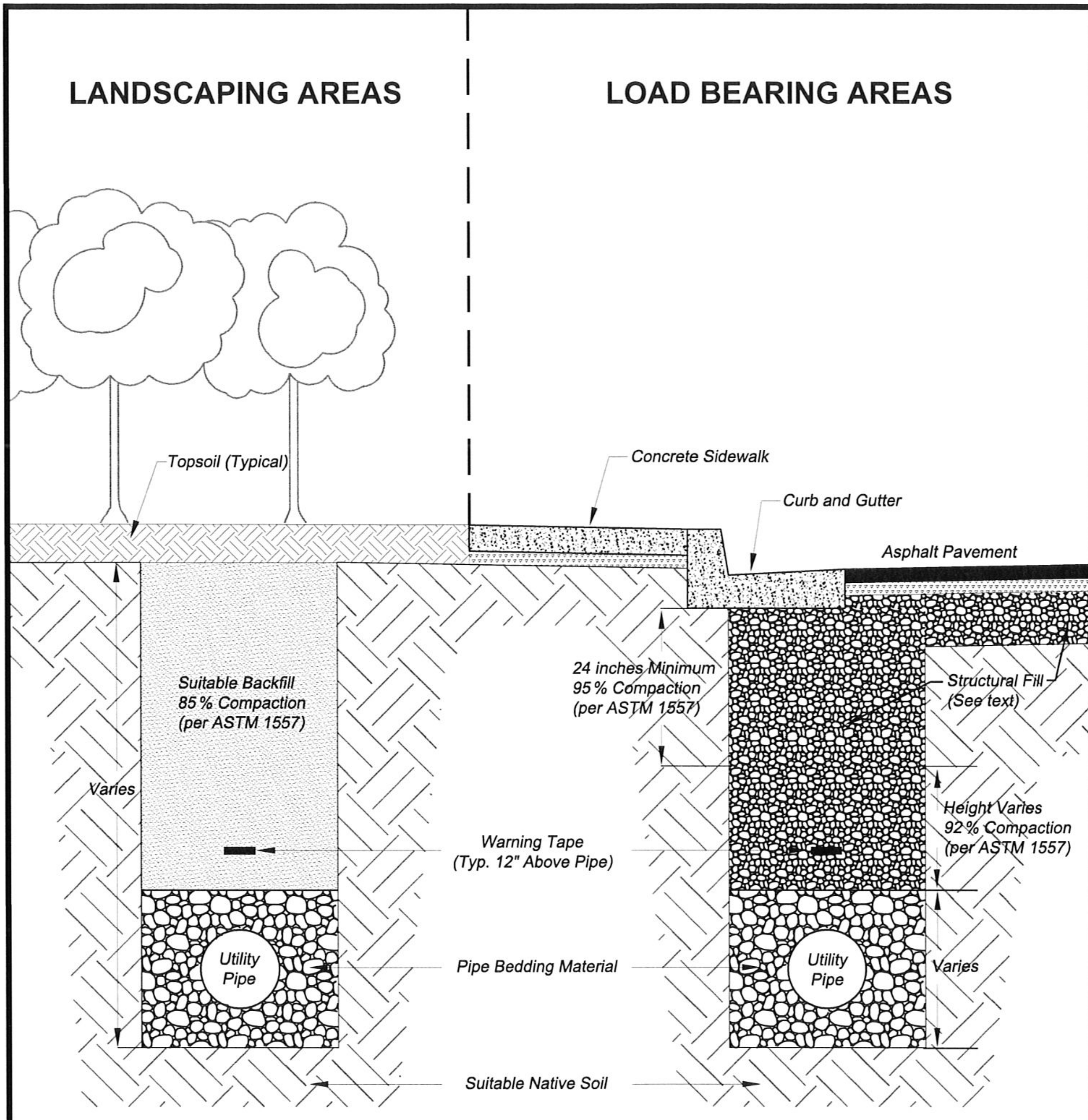
GEOTEST SERVICES, INC. 741 Marine Drive Bellingham, WA 98225 phone: (360) 733-7318 fax: (360) 733-7418	Date: 1-3-08	By: DPB	Scale: As shown	Project **07-0935**
	SITE AND EXPLORATION PLAN **ARLINGTON FIRE STATION** **172 STREET NE AND 43 AVENUE NE** **ARLINGTON, WASHINGTON**			Figure **2**

SHALLOW FOOTINGS WITH INTERIOR SLAB-ON-GRADE



Notes:
Footings should be properly buried for frost protection in accordance with International Building Code or local building codes.
(Typically 18 inches below exterior finished grades)

GEOTEST SERVICES, INC. 741 Marine Drive Bellingham, WA 98225 Phone: (360) 733-7318 Fax: (360) 733-7418	Date: 12-28-07	By: DPB	Scale: NONE	Project **07-0935**
	TYPICAL FOOTING & WALL DRAIN SECTION **ARLINGTON FIRE STATION** **172ND STREET NE AND 42ND AVENUE NE** **ARLINGTON, WASHINGTON**			Figure **3**



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Date: 12-28-07	By: DPB	Scale: NONE	Project 07-0935

TYPICAL UTILITY TRENCH SECTION
ARLINGTON FIRE STATION
172ND STREET NE AND 42ND AVENUE NE
ARLINGTON, WASHINGTON

Figure **4**

APPENDIX A

FIELD EXPLORATIONS AND

LABORATORY TESTING

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Subsurface conditions at the site were explored on December 13, 2007. The exploration program consisted of excavating and sampling six test pits (TP-1 through TP-6) at the approximate locations illustrated on the Site and Exploration Plan (Figure 2 of this report). The test pits were excavated with a tracked excavator to depths between approximately 9 and 10 feet BGS. Excavation services were provided by our client. Our exploration program was laid out based on the proposed site improvements on a map provided by the client. The explorations were located in the field by Cascade Surveying and Engineering. Ground surface elevations at the exploration locations were not determined during the field exploration program.

The field explorations were monitored by geologists from our staff who obtained representative soil samples, maintained a detailed record of observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed was described using the soil classification system shown on Figure A-1, in general accordance with ASTM D 2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Logs of the test pit explorations are presented on Figures A-2 through A-4. These logs represent our interpretation of subsurface conditions identified during the field explorations. The stratigraphic contacts shown on the individual test pits logs represent the approximate boundaries between soil types; actual transitions may be more gradual. Also, the soil and groundwater conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

Representative soil samples encountered in boring and test pit explorations were obtained at selected intervals, placed in sealed plastic bags, and transported to our laboratory for further classification and testing. Laboratory tests were performed on representative soil samples to characterize certain physical properties of the site soil. The laboratory testing program was limited to visual inspection to confirm field soil descriptions, determination of natural moisture content and soil grain size distribution.

The natural moisture contents of selected soil samples were determined in general accordance with ASTM D 2216 test procedures. The results from the moisture determinations are indicated on the summary logs, adjacent to the corresponding samples. Grain size analyses of selected soil samples were conducted in general accordance with ASTM D 422 test procedures. The results are presented in the form of grain size distribution curves on Figures A-5 and A-6.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL	TYPICAL DESCRIPTIONS[(1)(2)]
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

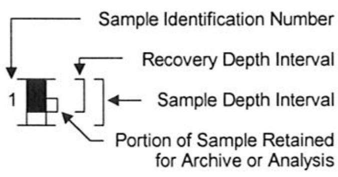
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.

2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50%	"GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50%	"very gravelly," "very sandy," "very silty," etc.
	> 12% and ≤ 30%	"gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 12%	"slightly gravelly," "slightly sandy," "slightly silty," etc.
	≤ 5%	"trace gravel," "trace sand," "trace silt," etc., or not noted.

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Other - See text if applicable

Groundwater

▽ ATD — Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

GEOTEST | Arlington Fire Station, Arlington, WA | Soil Classification System and Key | Figure A-1

1/4/08 X:\0-PROJECTS GEO\ARLINGTON FIRE STATION - 07-0935\ARLINGTON FIRE STATION.GPJ SOIL CLASS SHEET

TP-1

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0	1	d	W = 36		OL	Soft, dark brown, wet, organic, sandy SILT (topsoil).	
	2	d	W = 18 GS		SP/SM	Medium dense, brown, moist to wet, silty, fine to medium SAND (weathered glacial outwash).	
2					SM	Medium dense, light brown, moist to wet, silty, fine to medium SAND (glacial outwash).	
4	3	d	W = 4		SP	Loose to medium dense, light brown, dry to moist, medium to coarse SAND (glacial outwash). occasional gravel	
8	4	d	W = 5		SP	Medium dense, gray, moist to wet, gravelly, coarse SAND (glacial outwash).	
	5	d	W = 12				
10							Rapid groundwater seepage encountered at 10.0 ft.

Test Pit Completed 12/13/07
Total Depth of Test Pit = 10.0 ft.

TP-2

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0					OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
	1	d	W = 17		SM	Medium dense, brown, moist to wet, silty, fine to medium SAND (weathered glacial outwash).	
2	2	d	W = 14		SP	Medium dense, light brown, moist to wet, medium SAND (glacial outwash). occasional gravel	
7	3	d	W = 5 GS		SP	Medium dense, gray, dry to moist, slightly gravelly, medium to coarse SAND (glacial outwash). increase in gravel content	
9							Moderate groundwater seepage encountered at 9.0 ft.

Test Pit Completed 12/13/07
Total Depth of Test Pit = 9.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

TP-3

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0					OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
					SP-SM	Medium dense, brown, moist to wet, slightly silty, fine to medium SAND (weathered glacial outwash).	
2	1	d	W = 13 GS		SP	Medium dense, light brown, dry to moist, gravelly, medium SAND (glacial outwash). decrease in density for 2-5 feet BGS occasional gravel	
4	2	d	W = 4 GS				
8	3	d	W = 6		SP	Medium dense, gray, moist, gravelly, coarse SAND (glacial outwash).	Moderate groundwater seepage encountered at 9.5 ft.

Test Pit Completed 12/13/07
Total Depth of Test Pit = 9.5 ft.

TP-4

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0					OL	Soft, dark brown, wet, organic, sandy SILT (topsoil).	
					SM	Medium dense, brown, moist to wet, silty, fine to medium SAND (weathered glacial outwash).	
2	1	d	W = 4 GS		SP	Medium dense, light brown to gray, dry to moist, gravelly, medium to coarse SAND (glacial outwash). occasional gravel increase in gravel increase in grain size and gravel	
	2	d	W = 4 GS				
8	3	d	W = 10				Moderate groundwater seepage encountered at 9.0 ft.

Test Pit Completed 12/13/07
Total Depth of Test Pit = 9.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

-352276.00 1/4/08 X:\0-PROJECTS GEO\ARLINGTON FIRE STATION - 07-0935\ARLINGTON FIRE STATION.GPJ TEST PIT LOG

TP-5

SAMPLE DATA				SOIL PROFILE		GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0				OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
	1	d	W = 22	SM	Medium dense, brown, moist to wet, silty, fine to medium SAND (weathered glacial outwash).	
2	2	d	W = 10 GS	SP/ SM	Medium dense, light brown, moist, silty, fine to medium SAND (glacial outwash).	
4	3	d	W = 5	SP	Medium dense, gray, dry to moist, medium SAND (glacial outwash).	
8				GP	Dense, gray, wet, slightly sandy, medium to coarse GRAVEL (glacial outwash).	Moderate groundwater seepage encountered at 9.0 ft.

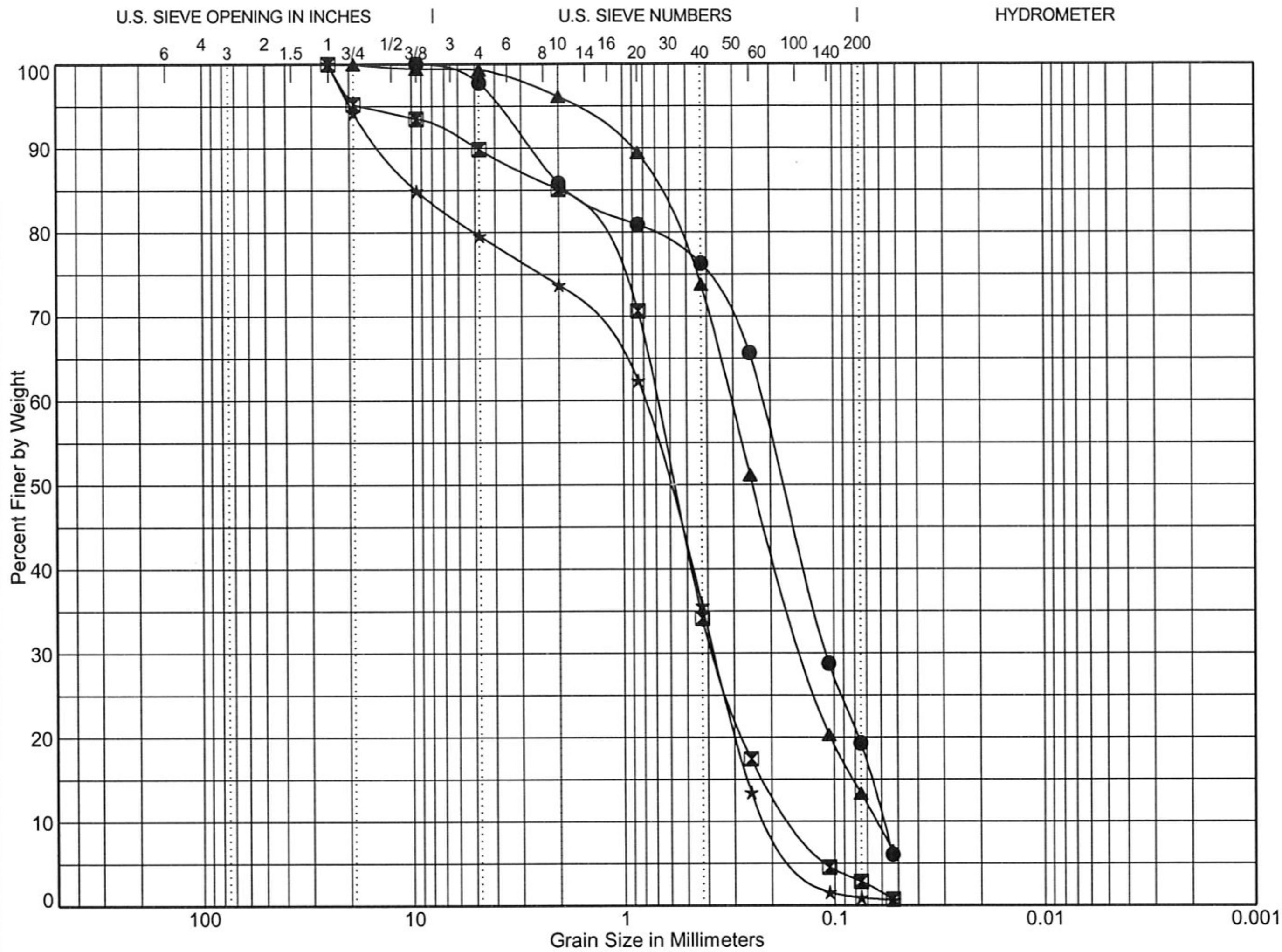
Test Pit Completed 12/13/07
Total Depth of Test Pit = 9.0 ft.

TP-6

SAMPLE DATA				SOIL PROFILE		GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Tracked Excavator Ground Elevation (ft): not determined	
0				OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
	1	d	W = 20	SM	Medium dense, brown, wet, silty, fine to medium SAND (weathered glacial outwash).	
2	2	d	W = 8	SM	Medium dense, light brown, moist, silty, fine to medium SAND (glacial outwash).	
4				SP	Medium dense, gray, dry to moist, medium SAND (glacial outwash).	
6	3	d	W = 2	SP	Dense, gray, wet, gravelly, medium to coarse SAND (glacial outwash).	
8	4	d	W = 21 GS			Moderate groundwater seepage encountered at 9.0 ft.

Test Pit Completed 12/13/07
Total Depth of Test Pit = 9.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point	Depth	Classification	LL	PL	PI	C_c	C_u
● TP-1	1.5	silty, fine to coarse SAND (SM)				0.92	3.73
⊠ TP-2	7.0	slightly gravelly, fine to coarse SAND (SP)				1.31	4.55
▲ TP-3	1.5	silty, fine to medium SAND (SP-SM)				0.99	4.85
★ TP-3	4.0	gravelly, fine to coarse SAND (SP)				0.88	4.10

Point	Depth	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
● TP-1	1.5	9.5	0.219	0.174	0.109	0.059	0.0	2.2	11.9	9.5	57.0	19.3
⊠ TP-2	7.0	25	0.694	0.574	0.373	0.153	4.8	5.3	4.7	51.0	31.3	2.8
▲ TP-3	1.5	19	0.308	0.242	0.139	0.063	0.0	0.7	3.2	22.3	60.5	13.3
★ TP-3	4.0	25	0.799	0.617	0.371	0.195	5.8	14.7	5.9	38.1	34.7	0.9

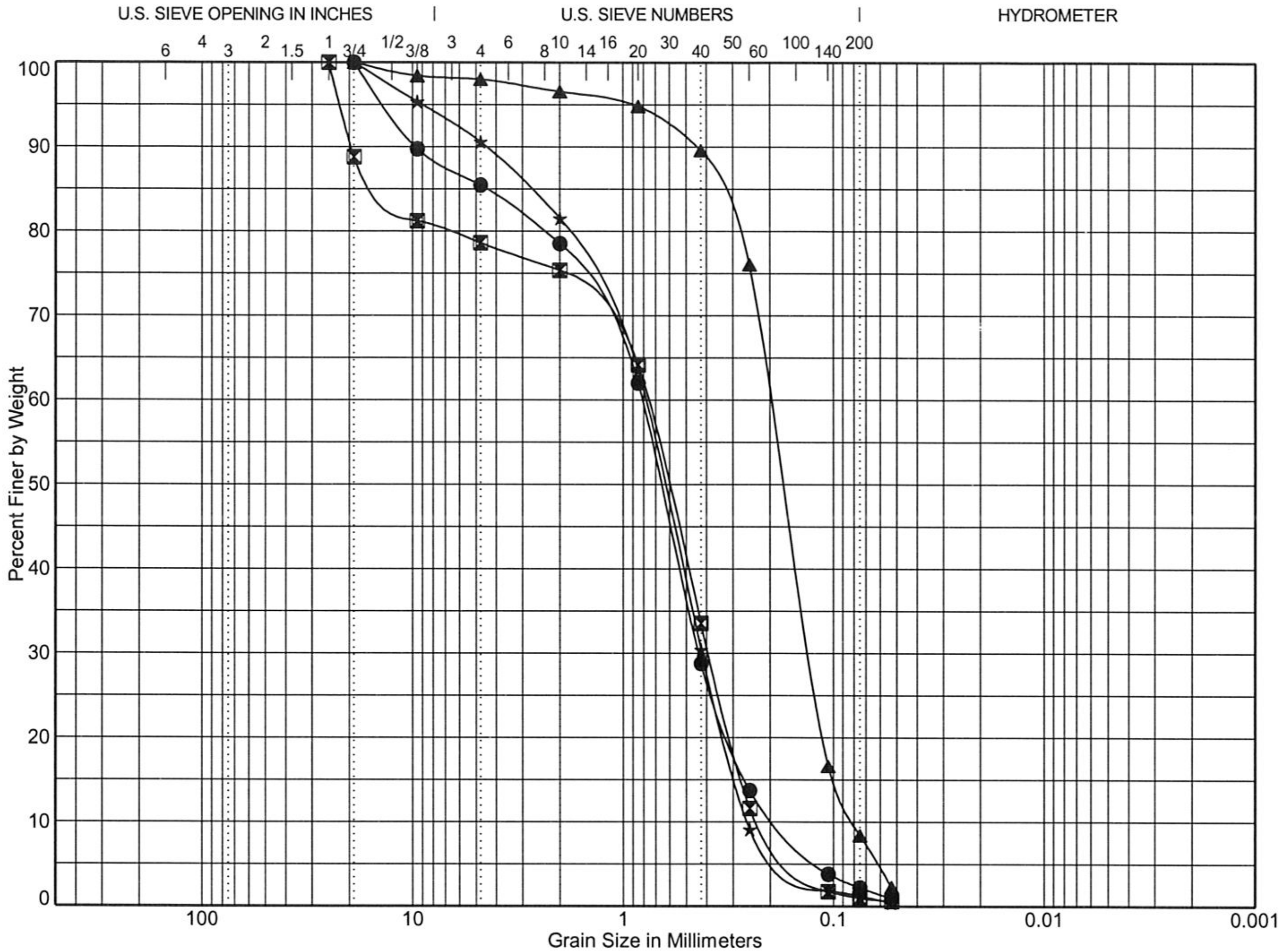
$C_c = D_{30}^2/(D_{60} * D_{10})$

$C_u = D_{60}/D_{10}$

To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

-352276.00 1/4/08 X:\0-PROJECTS GEO\ARLINGTON FIRE STATION - 07-0935\ARLINGTON FIRE STATION.GPJ GRAIN SIZE W/STATS

GEOTEST	Arlington Fire Station Arlington, WA	Grain Size Test Data	Figure A-5



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point		Depth	Classification	LL	PL	PI	C_c	C_u
●	TP-4	2.0	gravelly, fine to coarse SAND (SP)				1.29	4.50
⊠	TP-4	3.0	gravelly, fine to medium SAND (SP)				0.90	3.55
▲	TP-5	2.5	slightly silty, fine to medium SAND (SP-SM)				1.04	2.47
★	TP-6	8.0	slightly gravelly, fine to coarse SAND (SP)				0.88	3.09

Point		Depth	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	TP-4	2.0	19	0.815	0.662	0.436	0.181	0.0	14.5	6.9	49.7	26.6	2.2
⊠	TP-4	3.0	25	0.773	0.617	0.39	0.218	11.2	10.2	3.3	41.8	32.6	1.0
▲	TP-5	2.5	19	0.198	0.172	0.129	0.08	0.0	2.0	1.4	7.0	81.2	8.4
★	TP-6	8.0	19	0.789	0.64	0.421	0.256	0.0	9.4	9.1	51.1	29.2	1.2

$C_c = D_{30}^2/(D_{60} * D_{10})$
$C_u = D_{60}/D_{10}$

To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

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GEOTEST | Arlington Fire Station, Arlington, WA | Grain Size Test Data | Figure A-6

GEOTECHNICAL ENGINEERING REPORT
LIQUEFACTION ANALYSIS
Arlington Fire Station
172nd Street NE and 43rd Avenue NE
Arlington, Washington

February 13, 2008

Prepared for

City of Arlington

GEOTEST

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Bellingham, Washington

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February 13, 2008
Job No. 07-0935

City of Arlington
238 North Olympic Avenue
Arlington, WA 98223

Attn.: Paul Ellis

Re: Addendum Report
Liquefaction Analysis
Proposed Arlington Fire Station
172nd Street NE and 43rd Avenue NE
Arlington, Washington

Dear Mr. Ellis,

As requested, GeoTest Services, Inc. is pleased to submit this addendum report summarizing the results of our liquefaction analysis for the referenced project. The purpose of this evaluation was to establish the potential for earthquake related liquefaction beneath the site from which conclusions and recommendations for the proposed development could be formulated. Specifically, our scope of services included the following tasks:

- Exploration of soil and groundwater conditions underlying the site by advancing two test borings to evaluate subsurface conditions.
- Advance two dynamic cone penetration tests in order to collect continuous soil consistency under the job site.
- Laboratory testing on representative samples in order to classify and evaluate the engineering characteristics of the soils encountered.
- Provide this addendum report which includes observations and recommendations with respect to seismic design considerations including liquefaction hazard potential.

PROJECT DESCRIPTION

We understand that the site is approximately 3.0 acres in size and is proposed to be entirely redeveloped into the new Arlington Fire Station. At this time the property is heavily forested with a mix of evergreen and deciduous trees. The proposed new building will most likely consist of a mix of concrete, masonry and steel construction with a concrete slab-on-grade floor. Final design of the building has not been decided as of the time of this report. The remaining property is planned to be fully developed with paved parking areas and heavy traffic lanes for truck traffic. We understand that some of the planned paved parking areas may incorporate permeable pavement.

GENERAL GEOLOGIC CONDITIONS

Geologic information for the project site was obtained from the *Surficial Geologic Map of Port Townsend 30- by 60-Minute Quadrangle, Puget Sound Region, Washington* (Pessl, et. al 1989), published by the U.S. Geological Survey. According to Pessl, near-surface soils in the vicinity of the project site consist of glacial recessional-marine deposits of the Vashon Stade of the Fraser Glaciation. Recessional-marine deposits at the site are described by Pessl as a complex assemblage medium to well-sorted, massive to laminated sand, silt, and clay. Thicknesses of the unit typically range from 1 to 10 meters, with exceptional thicknesses observed to be up to 18 meters. Recessional-marine glacial outwash was deposited by meltwater flowing south from the stagnating and receding Vashon glacier. Site soils were relatively consistent with the mapped geology.

SUBSURFACE SOIL CONDITIONS

Subsurface conditions within the areas of interest at the site were explored by advancing and sampling two exploratory test borings and two dynamic cone penetration (DCP) tests to evaluate subsurface conditions between January and February, 2008. The test borings (B-1 and B-2) were advanced to depth of 16½ feet below the existing ground surface (BGS) and dynamic cone penetration tests (DCP-1 and DCP-2) were advanced to depths ranging 19¾ to 26½ feet below BGS. The test borings were advanced with a hollow stem auger drill provided by Boretec, Inc and dynamic cone penetration tests were conducted with a DCP probe which includes recording the number of blows necessary to advance a pointed steel rod into the ground with a 35-pound slide hammer within the general vicinity of the proposed improvements. The blows necessary to advance the rod into the soil have been correlated with the density of granular soil deposits and the consistency of cohesive soils. The approximate locations of the test borings and DCP tests are shown on the Site and Exploration Plan, Figure 2. A discussion of field exploration and laboratory test procedures, together with edited logs of the test borings and DCP tests, are presented in Appendix A.

The subsurface soil profile generally consisted of topsoil overlying native glacial outwash deposits. At the surface of all explorations soft, dark brown, moist, organic, sandy silt (topsoil) was encountered to depth of 6 inches BGS. Below the topsoil generally a medium dense, brown to light brown silty sand (SM), weathered glacial outwash, was encountered to depths ranging approximately 1½ to 3½ feet BGS. Generally below approximately 1½ to 3½ feet BGS we encountered a medium dense, brown to gray, moist to wet, poorly graded, fine to coarse sand (SM to SP) to the full depths of exploration. At B-2 between 10 and 15 feet BGS, we encountered medium dense, gray, wet, slightly sandy, fine to medium gravel (GP). Please refer to the individual test boring logs, attached with this report (A-2 and A-3), for more detail.

Based on the results of our DCP exploration program, loose to dense material (possible glacial outwash) was encountered at two explorations to the full depths ranging 19¾ to 26½ feet BGS.

GROUNDWATER

At the time of our subsurface investigation on January 22, 2008, groundwater seepage was observed in borings B-1 and B-2 at approximate depth of 11 feet BGS.

Groundwater elevations were inferred based on the wetted interval indicated on the sampling tube, moisture conditions of the soil samples obtained at depth, and correlation to test pit investigations in our original report. Groundwater levels are not static and vary with respect to surface runoff, precipitation, season and other factors. In general, groundwater levels are higher during the wetter winter months, October through June.

LIQUEFACTION HAZARD POTENTIAL

Based on *Liquefaction Susceptibility Map of Snohomish County, Washington* (September, 2004) by Washington Department of Natural Resources, the subject site is high liquefaction susceptibility area (flood plain). However, this map only provides an estimate of the likelihood that soil will liquefy as a result of earthquake shaking and is meant as a general guide to delineate areas prone to liquefaction. It is required that detailed geotechnical studies to determine relative seismic risk for the subject site.

Near-surface conditions at the site typically consist of loose to dense, sandy glacial soils. At time of explorations (January 2008) groundwater was encountered at 11 feet below ground surface (BGS) in our test boring explorations. And relatively loose, assumed sandy saturated granular soil layers were encountered during DCP exploration at location DCP-1 (12 to 18 feet BGS) and DCP-2 (15 to 18 feet BGS) that would be considered low to moderately susceptible to earthquake induced soil liquefaction. We utilized both test boring and DCP exploration logs to evaluate the liquefaction potential.

Liquefaction is defined as a significant rise in pore water pressure within a soil mass caused by earthquake-induced cyclic shaking. The shear strength of liquefiable soil is reduced during large and/or long-duration earthquakes as the soil consistency approaches that of semi-solid slurry. Liquefaction can result in significant and widespread structural damage if not properly mitigated. Deposits of loose, granular soil below the water table are most susceptible to liquefaction. Damage caused by foundation rotation, lateral spreading, and other ground movements could result from soil liquefaction.

The geotechnical data collected during our subsurface exploration and laboratory testing program was analyzed to estimate the factor of safety against liquefaction and settlement induced by earthquakes occurring at the site. The method of analyses was a simplified procedure originally proposed by Seed and Idriss (1971) that has been modified as discussed by Youd and Idriss (2001). Liquefaction potential was evaluated for a large design-level earthquake having a 10 percent probability of exceedance in a 50-year period, which corresponds to a mean recurrence interval of about 475 years.

The liquefaction analyses assumed a peak horizontal ground acceleration of 0.24g and an earthquake magnitude of 7.5. The analysis indicates the potential for liquefaction occurring at depths 12 to 18 feet BGS at DCP-1 and 15 to 18 feet BGS at DCP-2. The actual magnitude and extent of liquefaction will depend on many factors, including the duration and intensity of the ground shaking during the seismic event, and local soil and groundwater conditions. Therefore, the extent of liquefaction may vary from that estimated above.

The maximum amount of post-liquefaction ground subsidence, assuming no mitigating measures to improve the soil susceptibility to liquefaction and/or seismically induced ground settlement are implemented, was estimated using an empirical method developed by Tokimatsu and Seed (Tokimatsu and Seed 1987) based on field studies of areas that had undergone liquefaction. The magnitude of post-liquefaction ground subsidence under the most unfavorable conditions (e.g., maximum groundwater levels, long duration of ground shaking, etc.) was estimated to be ½ to 1 inch. This settlement is expected to be non-uniform with potential differential settlements equaling the total settlement. Post-liquefaction ground settlements could result in distortion of the existing structure. Again, actual ground subsidence will depend on many factors, including the intensity and duration of ground shaking during the seismic event, and local soil and groundwater conditions.

Based on the results of the liquefaction analysis, and within the limitations of the accuracy of the analyses and limiting assumptions, it is GeoTest's opinion that there is a low to moderate possibility of liquefaction occurring in some areas of the subject site under the design level earthquake. The risk of liquefaction occurring is remote under lower level earthquake loadings. Other earthquake hazards such as ground rupture and lateral spreading are considered to be unlikely at the site.

SEISMIC DESIGN CONSIDERATIONS

The Pacific Northwest is seismically active and the site could be subject to ground shaking from a moderate to major earthquake. Consequently, moderate levels of earthquake shaking should be anticipated during the design life of the project, and the proposed structures should be designed to resist earthquake loading using appropriate design methodology.

For structures designed using the seismic design provisions of the 2006 International Building Code, the medium dense, native, slightly gravelly to gravelly, silty sand and medium dense, native, slightly sandy gravel (glacial outwash), interpreted to underlie site in the upper 100 feet, classifies as Site Class D, stiff soil profile, according to Site Class Definitions, Table 1613.5.2. The corresponding values for calculating a design response spectrum for the assumed soil profile type is considered appropriate for the site.

Please use the following values for seismic structural design purposes:

Conterminous 48 States – 2003 NEHRP Seismic Design Provisions
Zip Code 98223
Central Latitude = 48.33, Central Longitude = -122.01

<u>Short Period (0.2 sec) Spectral Acceleration</u>

Maximum Considered Earthquake (MCE) Value of S_s = 1.129 (g)
Site Response Coefficient, F_a = 1.084 (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{MS} = S_s \times F_a$ = 1.224 (g)
Design spectral response acceleration for Site Class D, $S_{DS} = 2/3 \times SM_s$ = 0.816 (g)

<u>One Second Period (1 sec) Spectral Acceleration</u>

Maximum Considered Earthquake (MCE) Value of S_1 = 0.592 (g)
Site Response Coefficient, F_v = 1.705 (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{M1} = S_1 \times F_v$ = 1.009 (g)
Design spectral response acceleration for Site Class D, $S_{D1} = 2/3 \times SM_1$ = 0.673 (g)

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of the City of Arlington and their design consultants for specific application to the design of the proposed Arlington Fire Station project. Use of this report by others or for another project is at the user's sole risk. Within the limitations of scope, schedule, and budget, our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, either expressed or implied, is made as to the professional advice included in this report.

Our site explorations indicate subsurface conditions at the dates and locations indicated. It is not warranted that they are representative of subsurface conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth of our explorations at the time of our exploration program, a brief geological reconnaissance of the area, and review of published geological information for the site. We assume that the explorations are representative of the subsurface conditions throughout the site during the preparation of our recommendations. If variations in subsurface conditions are encountered during construction, we should be notified for review of the recommendations of this report, and revision of such if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

The earthwork contractor is responsible to perform all work in conformance with all applicable WISHA/OSHA regulations. GeoTest Services, Inc. should not be assumed to be responsible for job site safety on this project, and this responsibility is specifically disclaimed.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the construction phase. If you have any questions regarding the information contained in this report, or if we may be of further service, please contact the undersigned.

Respectfully Submitted,
GeoTest Services, Inc.

State of Washington
Engineering Geologist
950
Licensed Geologist
David P. Bufalini

2-13-08

David Bufalini, L.E.G.
Engineering Geologist

DONG-SOO LEE
STATE OF WASHINGTON
42610
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 12/20/08

2/13/08

Dong-Soo Lee, P.E.
Geotechnical Engineer

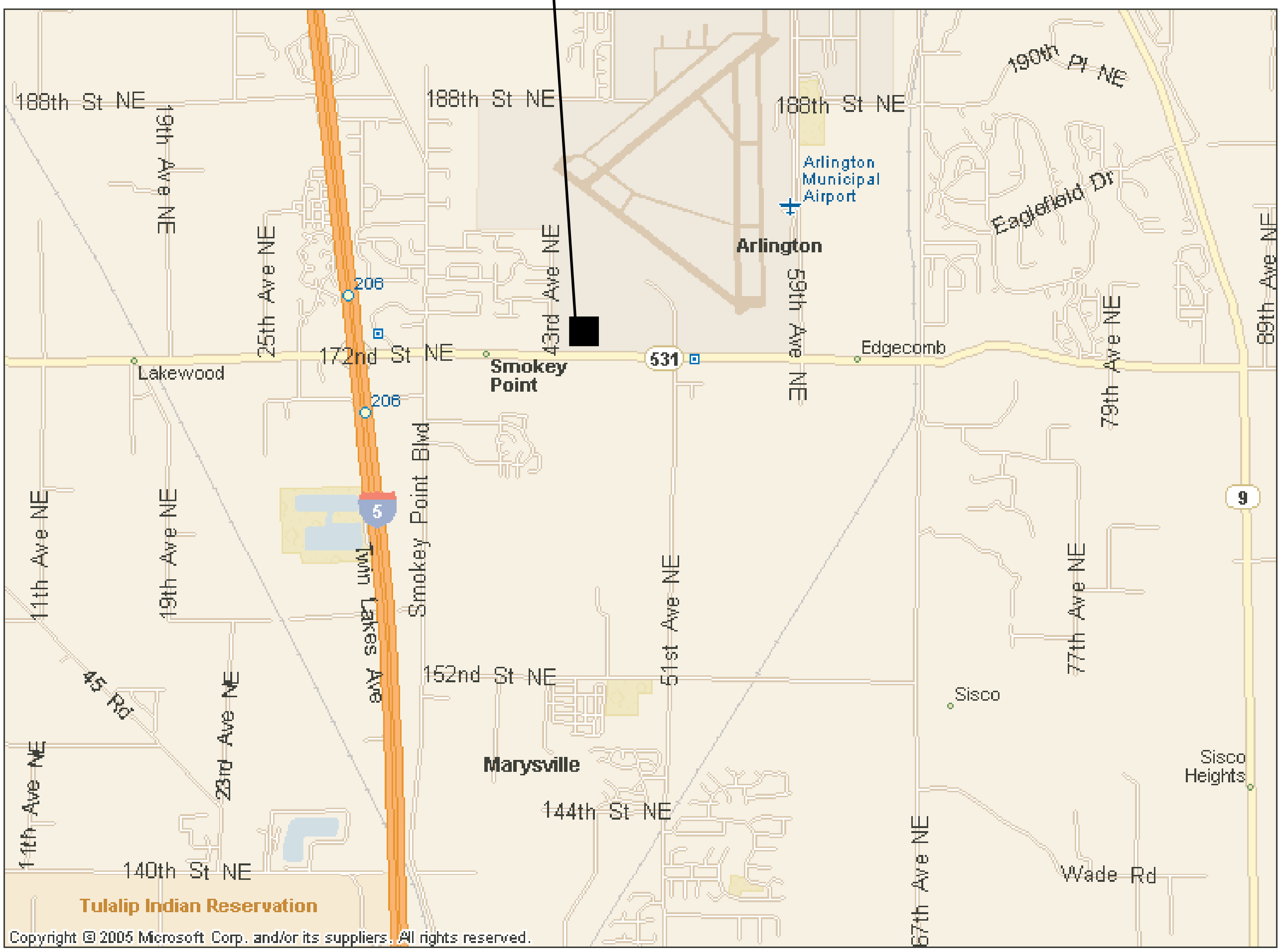
Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Appendix A	Field Explorations and Laboratory Testing
	Attached	Wildcat Dynamic Cone Logs (4 pages)

REFERENCES

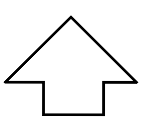
Pessl Jr., F, Dethier, D.P., Booth, D.B., Minard, J.P. 1989. *Surficial Geologic Map of Port Townsend 30- by 60-Minute Quadrangle, Puget Sound Region, Washington.* United States Geological Survey. Map I-1198-F.

Palmer, S.P., Magsino, S.L., Bilderback, E.L., Poelstra, J.L., Folger, D.S., Niggemann, R.A. September, 2004. *Liquefaction Susceptibility Map of Snohomish County, Washington.* Washington Department of Natural Resources. Map 31A.

PROJECT LOCATION



NORTH



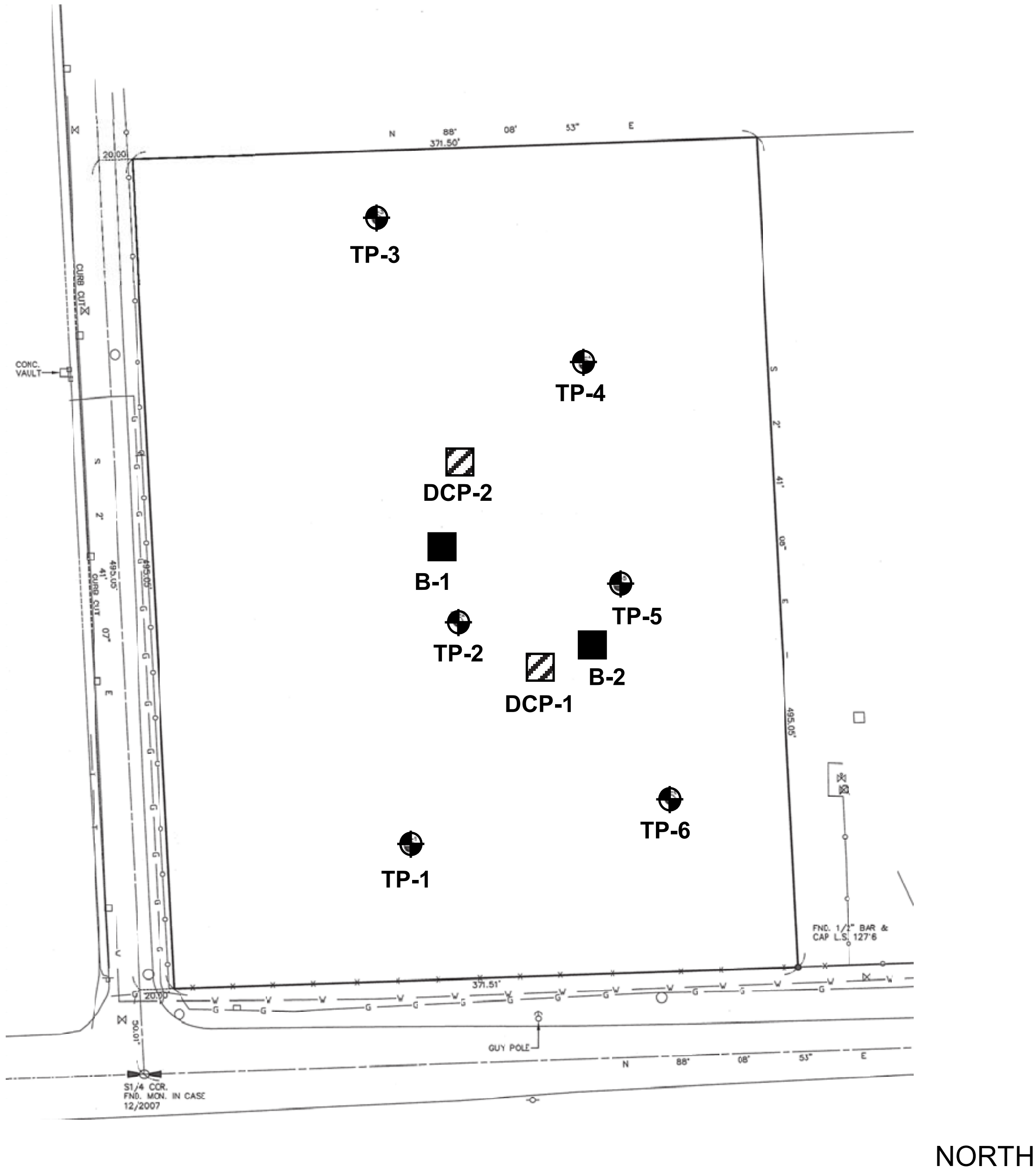
Reference Map Provided By
Microsoft Streets and Trips 2006

GEOTEST SERVICES, INC.
741 Marine Drive
Bellingham, WA 98225
phone: (360) 733-7318
fax: (360) 733-7418

Date: 12-28-07	By: DPB	Scale: NONE	Project 07-0935

SITE VICINITY MAP
ARLINGTON FIRE STATION
172 STREET NE AND 43 AVENUE NE
ARLINGTON, WASHINGTON

Figure
1



B-# = Boring Exploration Location

DCP#- = Boring Exploration Location

TP-# = Test Pit Exploration Location

GEOTEST SERVICES, INC. 741 Marine Drive Bellingham, WA 98225 phone: (360) 733-7318 fax: (360) 733-7418	Date: 2-13-08	By: DPB	Scale: As shown	Project **07-0935**
	SITE AND EXPLORATION PLAN **ARLINGTON FIRE STATION** **172 STREET NE AND 43 AVENUE NE** **ARLINGTON, WASHINGTON**			Figure **2**

APPENDIX A

FIELD EXPLORATIONS AND

LABORATORY TESTING

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Subsurface conditions at the site were explored on January 22, February 5, and February 12, 2008. The exploration program consisted of advancing and sampling two exploratory borings (B-1 and B-2) and two dynamic cone penetration tests (DCP-1 and DCP-2) at the approximate locations illustrated on the Site and Exploration Plan (Figure 2 of this report). The borings were advanced to a depth of 16½ feet below ground surface (BGS). Bortec, Inc. of Bellevue, Washington advanced the borings using a track-mounted drill rig and hollow-stem auger drilling techniques under subcontract to GeoTest Services. Dynamic cone penetration (DCP) tests were advanced to depths ranging between 19¾ and 26½ feet BGS. DCP tests were conducted with a DCP probe which includes recording the number of blows necessary to advance a pointed steel rod into the ground with a 35-pound slide hammer. Our exploration program was laid out based on the proposed site improvements on a map provided by the client. Ground surface elevations at the exploration locations were not determined during the field exploration program.

The field explorations were monitored by geologists from our staff who obtained representative soil samples, maintained a detailed record of observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed was described using the soil classification system shown on Figure A-1, in general accordance with ASTM D 2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Logs of the boring explorations are presented on Figures A-2 through A-3. These logs represent our interpretation of subsurface conditions identified during the field explorations. The stratigraphic contacts shown on the individual test pits logs represent the approximate boundaries between soil types; actual transitions may be more gradual. Also, the soil and groundwater conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

Representative soil samples encountered in boring explorations were obtained at selected intervals, placed in sealed plastic bags, and transported to our laboratory for further classification and testing. Laboratory tests were performed on representative soil samples to characterize certain physical properties of the site soil. The laboratory testing program was limited to visual inspection to confirm field soil descriptions, determination of natural moisture content and soil grain size distribution.

The natural moisture contents of selected soil samples were determined in general accordance with ASTM D 2216 test procedures. The results from the moisture determinations are indicated on the summary logs, adjacent to the corresponding samples. Grain size analyses of selected soil samples were conducted in general accordance with ASTM D 422 test procedures. The results are presented in the form of grain size distribution curves on Figures A-4.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL	TYPICAL DESCRIPTIONS[(1)(2)]
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

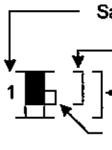
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.

2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50%	- "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50%	- "very gravelly," "very sandy," "very silty," etc.
	> 12% and ≤ 30%	- "gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 12%	- "slightly gravelly," "slightly sandy," "slightly silty," etc.
	≤ 5%	- "trace gravel," "trace sand," "trace silt," etc., or not noted.

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



- Sample Identification Number
- Recovery Depth Interval
- Sample Depth Interval
- Portion of Sample Retained for Archive or Analysis

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Other - See text if applicable

Groundwater

▽ ATD — Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

GEOTEST	Arlington Fire Station - Liquefaction Analysis Arlington, WA	Soil Classification System and Key	Figure A-1

2/4/08 X:\0-PROJECTS GEOCITY OF ARLINGTON - 07-0935 - ARLINGTON FIRE STATION - 07-0935\LIQUIFACTION ANALYSIS\ARLINGTON FIRE STATION\ARLINGTON FIRE STATION - LIQUIFACTION ANALYSIS.GPJ SOIL CLASS SHEET

B-1

Drilling Method: Hollow-stem Auger

Ground Elevation (ft):

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Water Level
0					OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
					SM	Medium dense, brown, moist to wet, silty, SAND (weathered glacial outwash).	
					SP	Medium dense, brown, moist, gravelly, fine to coarse SAND (glacial outwash)	
	1	b2	13	W = 16			
	2	b2	20	W = 7			
					SP	Medium dense, gray, moist to wet, slightly gravelly, fine to medium SAND (glacial outwash).	
	3	b2	23	W = 26 GS			∇ ATD
	4	b2	22	W = 23 GS			

Boring Completed 01/22/08
Total Depth of Boring = 16.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Arlington Fire Station - Liquefaction Analysis
Arlington, WA

Log of Boring B-1

Figure A-2

-352276.00 2/4/08 X:\0-PROJECTS GEO\CITY OF ARLINGTON - 07-0935 - ARLINGTON FIRE STATION - 07-0935\LIQUIFACTION ANALYSIS\ARLINGTON FIRE STATION\ARLINGTON FIRE STATION - LIQUIFACTION ANALYSIS.GPJ SOIL BORING LO

B-2

Drilling Method: Hollow-stem Auger

Ground Elevation (ft):

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Water Level
0					OL	Soft, dark brown, moist, organic, sandy SILT (topsoil).	
					SM	Medium dense, light brown, moist, silty, fine to medium SAND (weathered glacial outwash).	
	1	b2	16	W = 10			
					SP	Medium dense, brown to gray, moist, slightly gravelly, fine to coarse SAND (glacial outwash).	
	2	b2	23	W = 5 GS			
10					GP	Medium dense, gray, wet, slightly sandy, fine to medium GRAVEL (glacial outwash).	
	3	b2	23	W = 14			∇ ATD
					SP	Medium dense, gray, wet, slightly gravelly, medium SAND (glacial outwash).	
	4	b2	22	W = 17			

Boring Completed 01/22/08
Total Depth of Boring = 16.5 ft.

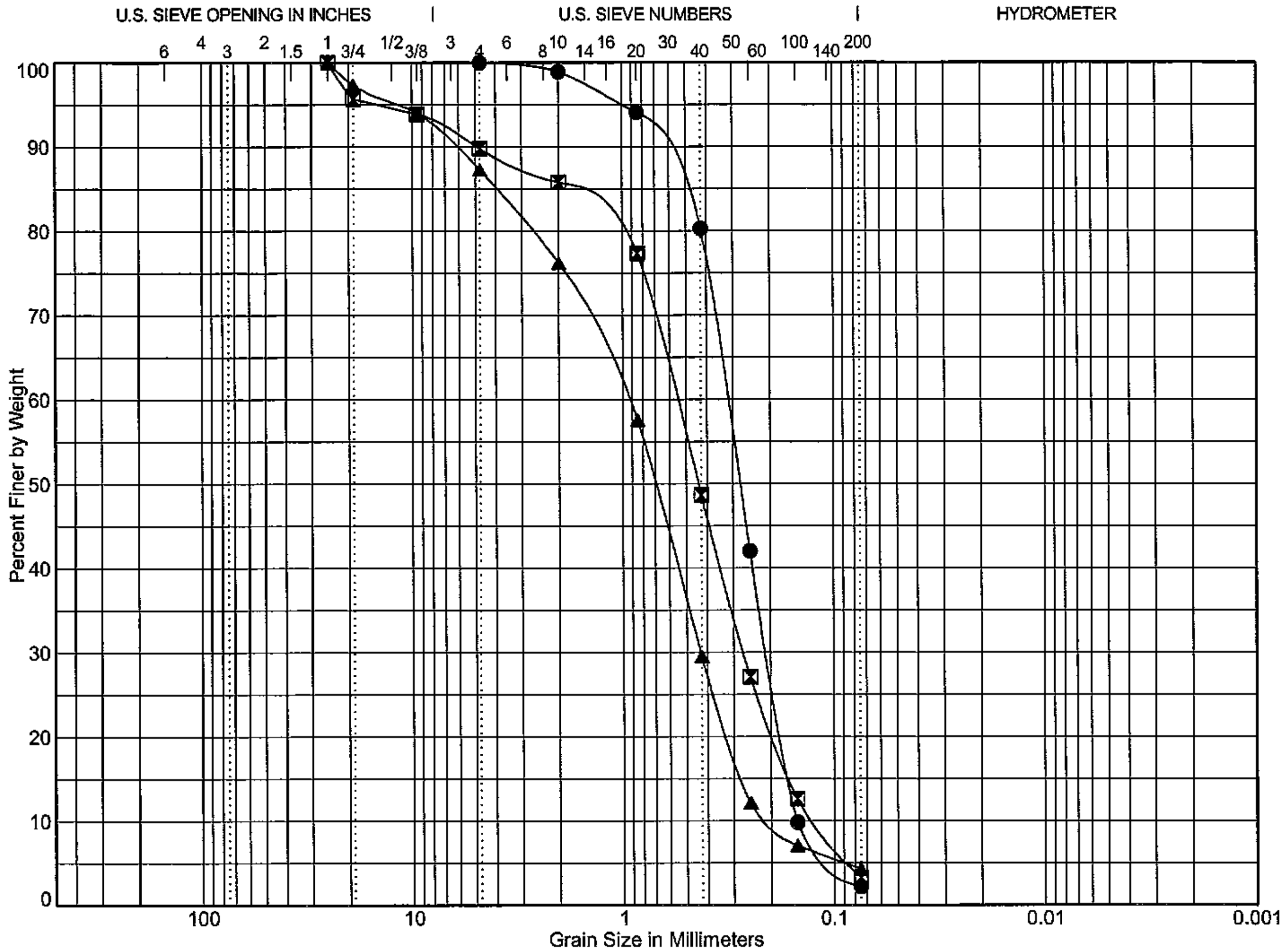
Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Arlington Fire Station - Liquefaction Analysis Arlington, WA

Log of Boring B-2

Figure A-3



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point		Depth	Classification	LL	PL	PI	C_c	C_u
●	B-1	10.0	Fine to medium SAND (SP)				0.88	2.13
⊠	B-1	15.0	Slightly gravelly, fine to medium SAND (SP)				1.05	4.53
▲	B-2	5.0	Gravelly, fine to coarse SAND (SP)				0.96	4.72

Point		Depth	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	B-1	10.0	4.75	0.321	0.279	0.207	0.15	0.0	0.0	1.1	18.6	78.1	2.2
⊠	B-1	15.0	25	0.559	0.439	0.269	0.124	4.3	5.9	4.0	37.1	45.4	3.3
▲	B-2	5.0	25	0.951	0.705	0.43	0.202	2.6	10.1	11.1	46.7	25.2	4.3

$C_c = D_{30}^2/(D_{60} * D_{10})$ To be well graded: $1 < C_c < 3$ and

$C_u = D_{60}/D_{10}$ $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Arlington Fire Station - Liquefaction Analysis
Arlington, WA

Grain Size Test Data

Figure A-4

WILDCAT DYNAMIC CONE LOG

GeoTest Services, Inc.
741 Marine Drive
Bellingham, WA 98225

PROJECT NUMBER: 07-0935
DATE STARTED: 02-05-2008
DATE COMPLETED: 02-05-2008

HOLE #: DCP-1
CREW: DPB/AH
PROJECT: Arlington Fire Station
ADDRESS: 172nd Street NE and 43rd Ave NE
LOCATION: Arlington, WA

SURFACE ELEVATION: Unknown
WATER ON COMPLETION: 11'
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	4	17.8		5	LOOSE	MEDIUM STIFF
-	4	17.8		5	LOOSE	MEDIUM STIFF
- 1 ft	4	17.8		5	LOOSE	MEDIUM STIFF
-	6	26.6		7	LOOSE	MEDIUM STIFF
-	8	35.5		10	LOOSE	STIFF
- 2 ft	10	44.4		12	MEDIUM DENSE	STIFF
-	12	53.3		15	MEDIUM DENSE	STIFF
-	16	71.0		20	MEDIUM DENSE	VERY STIFF
- 3 ft	27	119.9		-	DENSE	HARD
- 1 m	29	128.8		-	DENSE	HARD
-	26	100.4		-	MEDIUM DENSE	VERY STIFF
- 4 ft	25	96.5		-	MEDIUM DENSE	VERY STIFF
-	31	119.7		-	DENSE	HARD
-	35	135.1		-	DENSE	HARD
- 5 ft	37	142.8		-	DENSE	HARD
-	40	154.4		-	DENSE	HARD
-	45	173.7		-	DENSE	HARD
- 6 ft	50	193.0		-	VERY DENSE	HARD
-	50	193.0		-	VERY DENSE	HARD
- 2 m	37	142.8		-	DENSE	HARD
- 7 ft	40	136.8		-	DENSE	HARD
-	47	160.7		-	DENSE	HARD
-	32	109.4		-	DENSE	HARD
- 8 ft	25	85.5		24	MEDIUM DENSE	VERY STIFF
-	25	85.5		24	MEDIUM DENSE	VERY STIFF
-	25	85.5		24	MEDIUM DENSE	VERY STIFF
- 9 ft	25	85.5		24	MEDIUM DENSE	VERY STIFF
-	30	102.6		-	MEDIUM DENSE	VERY STIFF
-	27	92.3		-	MEDIUM DENSE	VERY STIFF
- 3 m 10 ft	19	65.0		18	MEDIUM DENSE	VERY STIFF
-	20	61.2		17	MEDIUM DENSE	VERY STIFF
-	17	52.0		14	MEDIUM DENSE	STIFF
-	13	39.8		11	MEDIUM DENSE	STIFF
- 11 ft	17	52.0		14	MEDIUM DENSE	STIFF
-	13	39.8		11	MEDIUM DENSE	STIFF
-	12	36.7		10	LOOSE	STIFF
- 12 ft	11	33.7		9	LOOSE	STIFF
-	9	27.5		7	LOOSE	MEDIUM STIFF
-	15	45.9		13	MEDIUM DENSE	STIFF
- 4 m 13 ft	17	52.0		14	MEDIUM DENSE	STIFF

WILDCAT DYNAMIC CONE LOG

PROJECT: Arlington Fire Station PROJECT NUMBER: 07-0935

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	20	55.4	•••••••••••••••	15	MEDIUM DENSE	STIFF
-	25	69.3	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
- 14 ft	20	55.4	•••••••••••••••	15	MEDIUM DENSE	STIFF
-	18	49.9	••••••••••••••	14	MEDIUM DENSE	STIFF
-	15	41.6	•••••••••••	11	MEDIUM DENSE	STIFF
- 15 ft	15	41.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	21	58.2	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	30	83.1	•••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
- 16 ft	25	69.3	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
- 5 m	14	38.8	•••••••••••	11	MEDIUM DENSE	STIFF
-	13	33.0	•••••••••	9	LOOSE	STIFF
- 17 ft	16	40.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	20	50.8	••••••••••••••	14	MEDIUM DENSE	STIFF
-	14	35.6	••••••••••	10	LOOSE	STIFF
- 18 ft	13	33.0	•••••••••	9	LOOSE	STIFF
-	14	35.6	••••••••••	10	LOOSE	STIFF
-	16	40.6	•••••••••••	11	MEDIUM DENSE	STIFF
- 19 ft	18	45.7	•••••••••••••	13	MEDIUM DENSE	STIFF
-	19	48.3	•••••••••••••	13	MEDIUM DENSE	STIFF
- 6 m	18	45.7	•••••••••••••	13	MEDIUM DENSE	STIFF
- 20 ft						
-						
-						
- 21 ft						
-						
-						
- 22 ft						
-						
-						
- 7 m 23 ft						
-						
-						
- 24 ft						
-						
-						
- 25 ft						
-						
-						
- 26 ft						
- 8 m						
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						

WILDCAT DYNAMIC CONE LOG

GeoTest Services, Inc.
741 Marine Drive
Bellingham, WA 98225

PROJECT NUMBER: 07-0935
DATE STARTED: 02-12-2008
DATE COMPLETED: 02-12-2008

HOLE #: DCP-2
CREW: DPB/AH
PROJECT: Arlington Fire Station
ADDRESS: 172nd Street NE and 43rd Ave NE
LOCATION: Arlington, WA

SURFACE ELEVATION: Unknown
WATER ON COMPLETION: 12'
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	3	13.3	•••	3	VERY LOOSE	SOFT
-	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
- 1 ft	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
-	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
-	3	13.3	•••	3	VERY LOOSE	SOFT
- 2 ft	3	13.3	•••	3	VERY LOOSE	SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
-	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 3 ft	10	44.4	••••••••••••	12	MEDIUM DENSE	STIFF
- 1 m	19	84.4	••••••••••••••••••••••••	24	MEDIUM DENSE	VERY STIFF
-	23	88.8	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
- 4 ft	30	115.8	•••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	31	119.7	••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	32	123.5	•••••••••••••••••••••••••••••••••••	-	DENSE	HARD
- 5 ft	46	177.6	••	-	DENSE	HARD
-	44	169.8	••	-	DENSE	HARD
-	50	193.0	••	-	VERY DENSE	HARD
- 6 ft	42	162.1	••	-	DENSE	HARD
-	37	142.8	•••	-	DENSE	HARD
- 2 m	34	131.2	•••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
- 7 ft	37	126.5	••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	29	99.2	••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	32	109.4	•••••••••••••••••••••••••••••••	-	DENSE	HARD
- 8 ft	36	123.1	•••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	25	85.5	••••••••••••••••••••••••	24	MEDIUM DENSE	VERY STIFF
-	25	85.5	••••••••••••••••••••••••	24	MEDIUM DENSE	VERY STIFF
- 9 ft	33	112.9	••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	41	140.2	••	-	DENSE	HARD
-	48	164.2	••	-	DENSE	HARD
- 3 m 10 ft	40	136.8	•••••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	31	94.9	•••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	33	101.0	•••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	37	113.2	••••••••••••••••••••••••••••••••	-	DENSE	HARD
- 11 ft	31	94.9	•••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	29	88.7	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
-	19	58.1	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 12 ft	16	49.0	•••••••••••••	13	MEDIUM DENSE	STIFF
-	16	49.0	•••••••••••••	13	MEDIUM DENSE	STIFF
-	19	58.1	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 4 m 13 ft	21	64.3	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF

HOLE #: DCP-2

WILDCAT DYNAMIC CONE LOG

PROJECT: Arlington Fire Station

PROJECT NUMBER: 07-0935

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	18	49.9	••••••••••••••	14	MEDIUM DENSE	STIFF
-	18	49.9	••••••••••••••	14	MEDIUM DENSE	STIFF
- 14 ft	15	41.6	••••••••••••	11	MEDIUM DENSE	STIFF
-	21	58.2	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	16	44.3	••••••••••••	12	MEDIUM DENSE	STIFF
- 15 ft	11	30.5	••••••••	8	LOOSE	MEDIUM STIFF
-	17	47.1	•••••••••••••	13	MEDIUM DENSE	STIFF
-	20	55.4	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 16 ft	20	55.4	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 5 m	27	74.8	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	25	63.5	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
- 17 ft	25	63.5	••••••••••••••••••	18	MEDIUM DENSE	VERY STIFF
-	24	61.0	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	20	50.8	••••••••••••••	14	MEDIUM DENSE	STIFF
- 18 ft	13	33.0	•••••••••	9	LOOSE	STIFF
-	15	38.1	•••••••••••	10	LOOSE	STIFF
-	17	43.2	••••••••••••	12	MEDIUM DENSE	STIFF
- 19 ft	28	71.1	••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
-	28	71.1	••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 6 m	30	76.2	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
- 20 ft	32	74.6	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	27	62.9	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	31	72.2	••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 21 ft	36	83.9	•••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
-	32	74.6	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
-	30	69.9	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
- 22 ft	53	123.5	••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	50	116.5	••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	46	107.2	••••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
- 7 m 23 ft	45	104.9	•••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	52	112.3	•••••••••••••••••••••••••••••••	-	DENSE	HARD
-	41	88.6	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
- 24 ft	25	54.0	•••••••••••••••	15	MEDIUM DENSE	STIFF
-	17	36.7	••••••••••	10	LOOSE	STIFF
-	24	51.8	•••••••••••••••	14	MEDIUM DENSE	STIFF
- 25 ft	37	79.9	••••••••••••••••••••••	22	MEDIUM DENSE	VERY STIFF
-	37	79.9	••••••••••••••••••••••	22	MEDIUM DENSE	VERY STIFF
-	34	73.4	•••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
- 26 ft	44	95.0	••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
- 8 m	62	133.9	•••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						