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GEOTECHNICAL ENGINEERING REPORT
Cascade Valley Hospital Expansion
330 S. Stilliguamish Avenue
Arlington, Washington

September 18, 2007

Prepared for

Cascade Valley Hospital and Clinics

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GEOTEST

741 Marine Drive
Bellingham, Washington

GEOTEST 741 Marine Drive Bellingham, WA 98225 PHONE 360 733_7318 TOLL FREE 888 251_5276 FAX 360 733_7418

September 18, 2007
Job No. 07-0556

Cascade Valley Hospital and Clinics
330 S. Stilliguamish Avenue
Arlington, WA 98223

Attn: Connie DiGregorio

Re: Geotechnical Engineering Evaluation
Cascade Hospital Expansion
330 S. Stilliguamish Avenue
Arlington, Washington

Dear Mrs. DiGregorio,

As requested, GeoTest Services, Inc. is pleased to submit this 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 foundation design could be formulated. Specifically, our scope of services included the following tasks:

- Exploration of soil and groundwater conditions underlying the site by drilling five test borings with a truck-mounted drill rig to depths up to approximately 40 feet BGS 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, seismic design, slope stability, foundation recommendations, concrete slab-on-grade construction, foundation and site drainage, infiltration potential, utilities, temporary and permanent slopes, pavement subgrade preparation and geotechnical consultation and construction monitoring.

PROJECT DESCRIPTION

We understand that the Cascade Valley Hospital and Clinics Master Plan Project will consist of the construction of an approximately 58,000 square foot, three story new hospital expansion and remodel of the existing approximately 52,000 square foot 1987 Cascade Valley Hospital building. The new structure will likely be steel framed with conventional concrete foundation and slab-on-grade. The expansion is proposed to be located to the east of the existing three story facility. Three one story structures will need to be demolished to accommodate the new expansion. New paved parking areas and a relocated access road are also proposed for the site. Based on the existing, relatively flat topography within the vicinity of the proposed improvements, site grade changes are anticipated to be minimal. The site is located at 330 S. Stilliguamish Avenue in Arlington, Washington as shown on the Vicinity Map, Figure 1.

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, laboratory testing, and our experience in the project vicinity.

General Geologic Conditions

Geologic information for the project site was obtained from the *Geologic Map of Washington State* (Schuster 2005), published by the Washington Division of Geology and Earth Resources. According to Schuster, surficial soils in the vicinity of the project site consist of Pleistocene continental glacial drift (Qgd). Soils defined as continental glacial drift consist of till and outwash clay, silt, sand, gravel, cobbles, and boulders deposited by or originating from continental glaciers; locally includes peat, non-glacial sediments, modified land, and artificial fill. Native undisturbed soils encountered within our test pit explorations were consistent with the continental glacial drift (till) deposits.

Surface Conditions

The site of the proposed hospital expansion will be located in an improved area to the east of the existing three story hospital building that was constructed in 1987. A one story entrance building, one story MRI building, and one story administration and support services building constructed in 1956 are planned to be demolished to accommodate the new addition. The new expansion will also be located over existing paved parking areas and an access road that leads to the current loading dock and helicopter pad. The existing access road will likely be relocated to the south through the existing daycare facility.

The topography within the area of the proposed expansion is relatively flat and slopes are based on the existing surface stormwater drainage gradients. Immediately west of the proposed building footprint and south of the existing hospital building the topography steps down approximately 10 feet to the loading dock and helicopter pad. This step is faced with a rockery. To the south and west of the existing facility and proposed expansion, the topography slopes down generally to the southwest at an average of approximately 3H:1V (horizontal: vertical) and up to a maximum of approximately 2H:1V. The existing three story facility is currently set back from the top of an approximately 2H:1V slope by approximately 30 to 40 feet. Some apparent fill has been placed over the top of this slope and was evidenced by occasionally buried tree trunks. Fill observed on the slope was likely placed during past site grading activities.

The top of an approximately 3.5H:1V slope will be located approximately 70 feet from the proposed hospital expansion. We have performed a detailed slope stability analysis of this slope relatively to the proposed expansion location, the details of which are provided in the subsequent *Slope Stability* section in this report. Again, some apparent fill may have been placed near the top of the approximately 3.5H:1V portion of the subject slope and is likely attributed to past site grading or road building activities.

Vegetation within the area of the proposed improvements generally includes landscape shrubs, plants, and deciduous trees generally located within islands in the parking areas and along the perimeter of existing buildings. Surface water was not observed within the areas of the proposed improvements or over the face of site slopes at the time of our field investigation in August of 2007.

During our review of the property we observed some vertical cracking in the south and west ground floor exterior walls of the existing three story hospital building. It appeared as though the cracks did not extend into the precast concrete exterior wall panels of the overlying floor. We did not observe cracking at precast concrete panel to panel connections either. We did have the opportunity to observe the structural plans for the existing main hospital building which was constructed in the 1980s. The bearing value prescribed for this project was 8000 pounds per square foot (psf). We did not observe a geotechnical report for the existing building or a reference to such a report with the structural plans. It is possible that the cracking may have been caused by concrete shrinkage after placement or differential settlement. No significant damage to the structure was noted and the actual cause of the concrete cracking cannot be determined at this time.

Subsurface Soil Conditions

Subsurface conditions within the areas of interest at the site were explored by drilling and sampling five exploratory test borings with a truck-mounted drill rig on August 3rd and 4th, 2007. The borings (B-1 through B-5) were advanced to depths between approximately 10 and 39 feet below ground surface (BGS). Our borings were generally located around the perimeter of the new expansion as shown on a site plan provided to us by Taylor Gregory Butterfield Architects. The approximate locations of the borings are shown on the Site and Exploration Map, Figure 2. A discussion of field exploration and laboratory test procedures, together with edited boring logs, is presented in Appendix A.

The subsurface conditions were generally consistent below the project site. The subsurface profile generally consisted of a thin layer of asphalt pavement over fill over native glacial till deposits.

At the surface of all five boring explorations, approximately 2 to 4 inches of asphalt pavement was encountered. Underlying the asphalt pavement at all five boring locations, medium dense to very dense, brown, damp, gravelly, slightly silty to silty sand (possible fill) was encountered to depths ranging between approximately 2½ to 5 feet BGS. It was difficult to determine the exact fill depths because our sampling was limited to 2½ foot intervals. Below the fill at all five boring locations, medium dense to very dense, light brown to gray, damp to occasionally wet, gravelly, clayey to very clayey, fine to coarse sand with occasional cobbles and trace boulders (glacial till) was encountered to the full extent of our explorations.

It should be noted that refusal was met on boulders and/or large cobbles that were encountered at various depths at all five boring locations. At the majority of our boring locations it was necessary to relocate the drill rig in order to miss boulders and achieve our final boring depths. Accordingly, we anticipate that occasional boulders will likely be encountered during site grading activities. In addition, during our site evaluation, we briefly discussed our activities with Connie DiGregorio, Cascade Valley Hospital &

Clinics Assistant Administrator, Resources. Mrs. DiGregorio recalled difficulty encountered by the contractor with the excavation of boulders during the earthwork phase of the construction of the main hospital building.

Groundwater

At the time of our subsurface investigation in August of 2007, groundwater seepage was observed within explorations B-1 through B-3 at depths that ranged between approximately 7½ and 20 feet BGS. In addition, we encountered some relatively loose/soft saturated soils within exploration B-2 between approximately 3 to 8 feet BGS that may require limited overexcavation to reach suitable bearing soils. We were not able to identify the lateral extent and depth of the saturated soils at the time of our subsurface exploration. Therefore, unsuitable and/or saturated soils will need to be identified and removed down to suitable bearing conditions during site grading activities.

The groundwater seepage was encountered within the general vicinity of the proposed basement area during our subsurface investigation in August of 2007. It appeared that the seepage encountered below the site may be locally isolated seepage zones, potentially trapped within sand lenses, or water "perched" on the relatively impermeable dense glacial till deposits and not indicative of a groundwater table. Accordingly, limited dewatering should be anticipated during site grading and excavation activities. Based on the limited seepage encountered during our subsurface investigation, in our opinion, typical sump/trash pumps will likely be sufficient to remove water during excavation activities. If significant seepage is encountered during site grading and excavation activities, we recommend that GeoTest be notified immediately to help determine the appropriate method of dewatering and/or drainage control. In addition, gravel working mats or controlled density fill may be warranted to protect the subgrade during earthwork operations, particularly during extended periods of wet weather.

The groundwater conditions reported on the boring logs are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and/or times. Groundwater levels are not static and it is anticipated that groundwater conditions will vary depending on local subsurface conditions, season, precipitation, changes in land use both on and off site, and other factors.

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.

Based on the subsurface conditions encountered at the project site, approximately 2½ to 5 feet of asphalt pavement, fill and/or unsuitable soil may have to be removed prior to structural fill placement or foundation formwork. Within the vicinity of exploration B-2, located to the northeast of the existing three story building, as much as approximately 8 feet of soft/loose, saturated soil may have to be removed and replaced with properly compacted import structural fill. In addition, occasional boulders and/or large cobbles were encountered below the site and should be anticipated during site grading activities.

Conventional shallow isolated and continuous footings are considered feasible for this project provided that all of the encountered pavement, topsoil, fill, foundation debris resulting from demolition activities, and loose, organic upper portions of the native soil are removed below structural areas and the foundation elements bear directly on suitably prepared native soil or on properly compacted structural fill placed directly over suitable native soil.

Site Preparation and Earthwork

The portions of the site to be occupied by proposed foundation(s), pavement, or sidewalk should be prepared by removing any existing pavement, concrete, fill, topsoil 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 and spread or continuous foundations 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 demolition and excavation activities.

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 feasible.

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 construction 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

It is our opinion that the generally granular material underlying the site (glacial till) may be used as structural fill below the proposed improvements once properly moisture conditioned and the specific degree of compaction is attained. Reuse of the native soil underlying the site as structural fill will prove most cost effective compared to importing structural fill to the site. However, due to the relatively high fines content of the native soil, it is considered moisture sensitive and may be difficult to achieve compaction if the soil moisture content is not close to optimum. In our opinion, reuse of site soils as structural fill should be avoided during periods of wet weather and only used during the warmest summer months. If the native soil is to be used as structural fill at the site, we recommend that GeoTest be involved with planning and construction monitoring for fill placement activities.

Based on the results of laboratory tests performed on representative samples, native, undisturbed site soils in the upper 10 feet have "fines" contents (percent passing the U.S. No. 200 sieve) ranging between approximately 28 to 34 percent of the dry weights. Soils containing more than approximately 5 percent fines are considered moisture sensitive, and are 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.

Moisture contents of the soil samples recovered from the test pits in the upper 10 feet of the existing undisturbed native soils ranged from about 9 to 18 percent of the dry weight. The moisture contents of most of the soil samples were generally near to slightly above the estimated ranges of optimum moisture contents for the native glacial till deposits encountered in our boring explorations. Accordingly, native site soils will likely need to scarified to a more optimum moisture condition prior to placement.

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 percent 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 percent 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 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.

Backfill and Compaction

Structural fill should be placed in horizontal lifts 8 to 10 inches in loose thickness and thoroughly compacted. All structural fill placed under load bearing 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 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 onsite soils are moisture sensitive. It is our experience that the native till is 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.

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. The relatively dense condition of the native glacial till and the general absence of saturated conditions effectively preclude seismically induced soil liquefaction. In addition, it is anticipated that the site would not be subject to seismically induced landslides, lateral spreading, or other ground failure.

For structures designed using the seismic design provisions of the 2006 International Building Code, the dense, native, gravelly, silty sand (glacial till), 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 = 48.19, Central Longitude = -122.12

Short Period (0.2 sec) Spectral Acceleration

MCE Value of S_s = 1.044 (g)
Site Response Coefficient, F_a = 1.082 (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{MS} = S_s \times F_a = 1.130$ (g)
Design spectral response acceleration for Site Class D, $S_{DS} = 2/3 \times SM_s = 0.753$ (g)

One Second Period (1 sec) Spectral Acceleration

MCE Value of S_1 = 0.355 (g)
Site Response Coefficient, F_v = 1.689 (Site Class D)
Adjusted spectral response acceleration for Site Class D, $S_{M1} = S_1 \times F_a = 0.660$ (g)
Design spectral response acceleration for Site Class D, $S_{D1} = 2/3 \times SM_1 = 0.400$ (g)

Slope Stability

As mentioned previously, the proposed expansion is planned to be set back approximately 70 feet from an approximately 3.5H:1V soil slope. For our modeling purposes, we assumed that a 3 to 5 foot thick layer of silty sand (possible fill, Density = 115 pcf and Friction Angle = 28 degrees) exists over dense, gravelly, very clayey sand (glacial till, Density = 125 pcf and Friction Angle = 32 degrees).

Global stability analyses were performed on a representative cross-sectional area based on the above mentioned parameters. A computer slope stability program, Slope/W, was used to determine the factors of safety for the global stability of the slopes under both static and seismic conditions. The computer stability program was used to randomly generate and evaluate circular and block failures within the slope area using the simplified Bishop's method of slices. The potential effects of seismic loading on the global stability of the slope were analyzed assuming a peak horizontal ground acceleration of 0.24g for a seismic event with a probability of exceedance of 10 percent in a 50 year period (USGS 2007). The horizontal forces developed during earthquake shaking were represented by a "pseudo-static" seismic coefficient K_h. The horizontal acceleration used in seismic stability analysis for natural soil slopes is typically assumed to be one half of the free-field acceleration. Accordingly, the seismic coefficient used in our stability analysis of the slope was 0.12g.

Based on the results of our stability analysis using the above mentioned parameters and slope geometry, the global stability meets and exceeds the recommended factors of safety of 1.5 and 1.1 for static and seismic loading, respectively. We have attached figures from our computer modeling program that display the resulting static and seismic safety factors for your convenience.

The resulting factors of safety were 2.282 for static condition and 1.570 for seismic condition based on our computer modeling. Therefore, it is our opinion that the subject slope does not present a landslide hazard relative to the proposed expansion location.

Foundation Support and Settlement

As mentioned previously, based on the subsurface conditions encountered at the project site, from the existing ground surface approximately 2½ to 5 feet of unsuitable soil may have to be removed prior to structural fill placement or foundation formwork. Within the vicinity of exploration B-2, located to the northeast of the existing three story building, as much as approximately 8 feet of soft/loose, saturated soil may have to be removed and replaced with properly compacted import structural fill. In addition, occasional boulders and/or large cobbles were encountered below the site and should be anticipated during site grading activities.

Foundation support for the proposed improvements may be provided by continuous or isolated spread footings founded on the proof-rolled, undisturbed, dense to very dense, gravelly, clayey to very clayey sand (glacial till) or.on properly compacted structural fill placed directly over undisturbed native soil. To provide proper support, we recommend that all existing fill or foundation debris beneath the building foundation area(s) be removed and replaced with properly compacted structural fill as described above. Alternatively, localized overexcavations could be backfilled to the design footing elevation with lean concrete or foundations may be extended to bear on dense, undisturbed native soil. In areas requiring overexcavation to competent native soil, the limits of the overexcavation should extend laterally beyond the edge of each side of the footing a distance equal to the depth of the excavation below the base of the footing. If lean concrete is used to backfill the overexcavation, the limits of the overexcavation need only extend a nominal distance beyond the width of the footing.

All continuous and isolated spread footings should be founded a minimum of 18 inches below the lowest adjacent final grade for freeze/thaw protection. The footings should be sized in accordance with the structural engineer's prescribed design criteria and seismic considerations.

Allowable Bearing Capacity

Assuming the above foundation support criteria are satisfied, continuous or isolated spread footings founded directly on the native, light brown to gray, medium dense to very dense, gravelly, clayey to very clayey sand (glacial till) or on compacted structural fill placed directly over undisturbed native soils may be proportioned using a maximum net allowable soil bearing pressure of 3,000 pounds per square ft (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.

We understand that in 1987 the main building was designed using a net allowable bearing capacity of 8,000 psf. We have limited our bearing capacity to 3,000 psf for the following reasons:

- Based on SPT blow counts, we encountered medium dense soils within the upper 15 feet that would be directly influenced by the proposed building loads. Native, undisturbed medium dense soils generally do not exceed a bearing capacity of 3,000 psf.
- In addition, we encountered several boulders below the proposed building location and some relatively loose/soft saturated soils that will likely need to be removed/overexcavated and replaced with structural fill. The bearing

capacity for well compacted structural fill generally does not exceed 3,000 psf.

We have removed deep foundation design recommendations, such as piles, from our report due to the presence of large boulders encountered at various depths at all five of our boring exploration locations. The drill rig we used during our subsurface investigation suffered from excessive wear on encountered boulders and large cobbles and our subsurface program was terminated after the drill rig blew a transmission. We would anticipate that the presence of boulders and large cobbles at various depths below the site would cause significant difficulties during pile installation.

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 one inch and differential settlement between two adjacent load-bearing components supported on competent soil should be less than about one half the total settlement. The soil response to applied stresses caused by building and other loads is expected to be predominantly 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 0.25 for undisturbed native soil and native soil reused as structural fill and an allowable coefficient of base friction of 0.30 for import structural fill, applied to vertical dead loads only, may be used between the underlying soil and the base of the footing. However, 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.

The lateral earth pressures that develop against subsurface building and retaining walls will depend on the method of backfill placement, degree of compaction, slope of backfill, type of backfill material, provisions for drainage, magnitude and location of any adjacent surcharge loads, and the degree to which the wall can yield laterally during or after placement of backfill. If the wall is allowed to rotate or yield so the top of the wall moves an amount equal to or greater than about 0.001 to 0.002 times its height (a yielding wall), the soil pressure exerted will be the active soil pressure. When a subsurface wall is restrained against lateral movement or tilting (a nonyielding wall), the soil pressure exerted is the at-rest soil pressure. Wall restraint may develop if a rigid structural network is constructed prior to backfilling or the wall is inherently stiff.

We recommend that yielding walls with level backfill under drained conditions be designed for an equivalent fluid density of 35 pounds per cubic ft (pcf) for structural backfill (pit run) in active soil conditions. Nonyielding walls with level backfill under drained conditions should be designed for an equivalent fluid density of 55 pcf for at-rest conditions. Design of subsurface walls should include appropriate lateral pressures caused by surcharge loads located within a horizontal distance equal to or less than the height of the wall. For uniform surcharge pressures, a uniformly distributed lateral pressure equal to 35 percent and 50 percent of the vertical surcharge pressure should be added to the lateral soil pressures for yielding and nonyielding walls, respectively.

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 properly placed and 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 6 inches of compacted, clean, free-draining sand and 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, at a minimum a continuous impermeable membrane of 6- to 10-mil polyethylene sheeting with tape-sealed joints should 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 be carried up the back of wall and 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 the proposed building to direct surface water away from the foundation and toward suitable drainage 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 or paved areas during or after construction. 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.

Infiltration Potential

Native glacial till soils encountered below the site are over-consolidated, generally cemented in nature and generally have a high "fines" content (that which passes the U.S. No. 200 sieve). Therefore, the native glacial till soils underlying the site are relatively impermeable and are generally unsuitable for infiltration purposes.

In our opinion, stormwater runoff at the site should be collected using a subsurface detention vault, detention pond, or directed into the City storm system. We do not recommend considering infiltration for stormwater management at the site.

Utilities

It is important that utility trenches be properly backfilled and compacted to minimize the possibility of cracking or localized loss of foundation, slab, or pavement support. It is anticipated that excavations for new underground utilities will be in medium dense to very dense existing near-surface glacial till soils or compacted fill material.

Trench backfill in improved areas (beneath structures, pavements, sidewalks, etc.) should consist of structural fill as defined earlier in this report. Based on the soil grain-size distributions and moisture conditions encountered in our test pits, it may be difficult to utilize portions of the near-surface site soils for compacted utility trench backfill without proper moisture conditioning. The use of imported, granular soil should be anticipated for backfill in improved areas. Outside of improved areas, trench backfill may consist of onsite soil. Trench backfill should be placed and compacted in accordance with the report section *Fill and Compaction* and as shown on Figure 4, Typical Utility Trench Section.

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. The contractor should implement measures to prevent surface water runoff from entering trenches and excavations. In addition, vibration as a result of construction activities and traffic may cause caving of the trench walls.

Actual trench configurations should be the responsibility of the contractor. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored by the contractor during excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring. If groundwater or groundwater seepage is present, and the trench is not properly dewatered, the soil within the trench zone may be prone to caving, channeling, and running. Trench widths may be substantially wider than under dewatered conditions.

Temporary and Permanent Slopes

Actual construction slope 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.

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. If temporary shoring is implemented during excavation activities, the shoring should be designed based on equivalent fluid densities of 45 pcf for the native glacial till in active soil conditions and 60 pcf for at-rest conditions.

Temporary unsupported excavations in medium dense to very dense, gravelly, silty sand glacial soils encountered at the project site are classified as a Type B soil according to WAC 296-155-657 and may be sloped as steep as 1H:1V. All soils encountered are classified as Type C soil in the presence of groundwater seepage. Flatter slopes or temporary shoring may be required in areas where groundwater flow is present and unstable conditions develop.

We recommend that permanent cut or fill slopes be designed for inclinations of 2H:1V or flatter. All permanent cut slopes should be vegetated or otherwise protected to limit the potential for erosion as soon as practical after construction. Permanent slopes requiring immediate protection from the effects of erosion should be covered with either mulch or erosion control netting/blankets. Areas requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

Occasionally subsurface conditions may result in the concentration of seepage within particular soil zones. The need for additional drainage within specific seepage zones can best be determined during or following construction.

Pavement Subgrade Preparation

New pavement sections must be installed over a dense and unyielding subgrade. Structural fill placed to establish subgrade elevation should be compacted to at least 92 percent, except the upper 24 inches of the subgrade, which should be compacted to a minimum of 95 percent of its maximum dry density, as determined using test method ASTM D 1557. Prior to the placement of base-course material and paving, the exposed subgrade under all areas to be occupied by asphalt pavement should be proof rolled. Proof rolling should be accomplished with a loaded dump truck, large self-propelled vibrating roller, or equivalent piece of equipment. The purpose of this effort is to identify possible loose or soft soil and recompact disturbed areas of subgrade.

Proof rolling should be carefully observed by geotechnical personnel. Areas exhibiting significant deflection, pumping, or saturated soils that cannot be readily compacted should be overexcavated to firm soil. Overexcavated areas should be backfilled with compacted granular 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 feasible.

Base-course material should be compacted to a minimum 95 percent of maximum dry density, as determined by test method ASTM D 1557. Prevention of road-base saturation is essential for pavement durability; thus, efforts should be made to limit the amount of water entering the base course.

Geotechnical Consultation and Construction Monitoring

GeoTest Services recommends that a geotechnical engineer familiar with the project design review the earthwork and foundation portions of the design drawings and specifications. The purpose of the review is to verify that the recommendations presented in this report have been properly interpreted and incorporated in the design and specifications.

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. 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 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 Cascade Valley Hospital and Clinics and their design consultants for specific application to the design of the proposed Cascade Valley Hospital Expansion in Arlington, Washington. 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 or comments regarding the information contained in this report, or if we may be of further service, please call.

Respectfully Submitted,
GeoTest Services, Inc.

David Jellum
Staff Geologist

DONG-SOO LEE
STATE OF WASHINGTON
42610
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 12/22/08

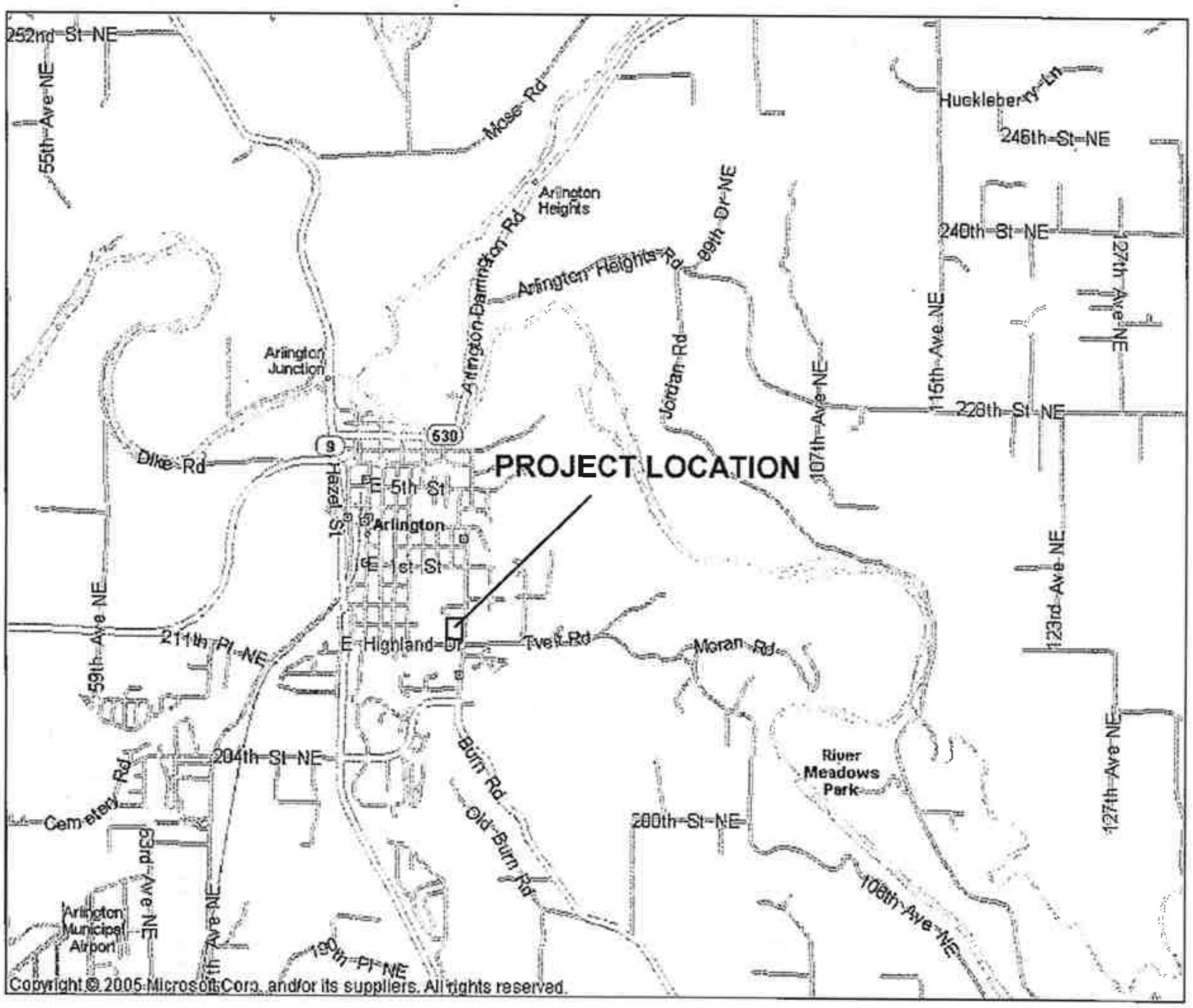
9/18/07

Dong-Soo Lee, P.E.
Geotechnical Engineer

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Figure 3	Typical Footing and Wall Drain Section
	Figure 4	Typical Utility Trench Section
	Appendix A	Field Explorations and Laboratory Testing
	Attached	Cascade Valley Hospital Slope Stability Analysis Figures (2 Pages)

REFERENCES

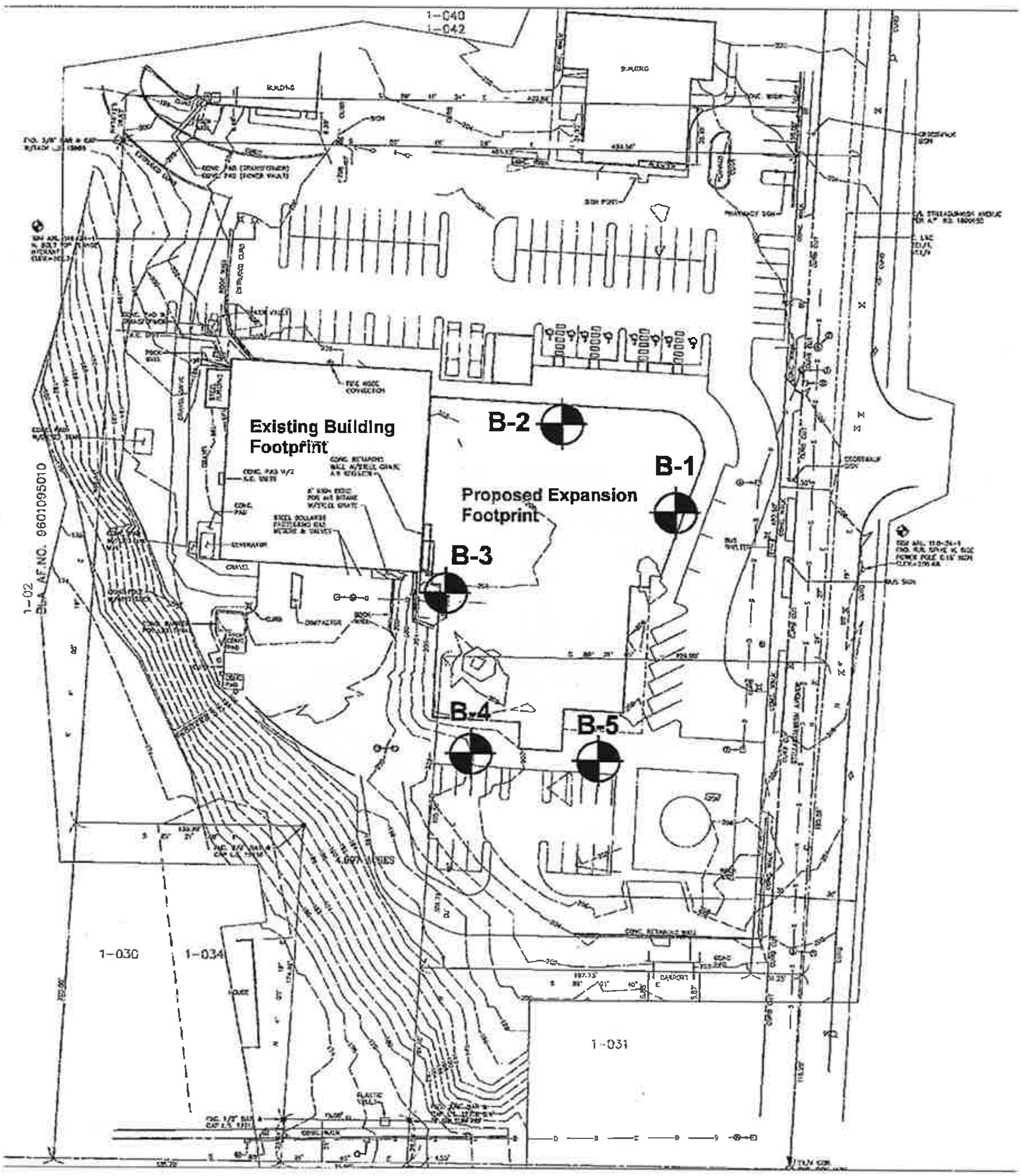
Schuster, J.E. 2005. *Geologic Map of Washington State.* Washington Division of Geology and Earth Resources. Geologic Map GM-53.



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: 9-5-07	By: DJ	Scale: NONE	Project **07-0556**
	SITE VICINITY MAP **CASCADE VALLEY HOSPITAL EXPANSION** **330 S STILLIGUAMISH AVENUE** **ARLINGTON, WASHINGTON**			Figure **1**



B-# = Approximate Exploration Location

*Site plan provided by taylor gregory butterfield architects
Dated - 7/27/07.

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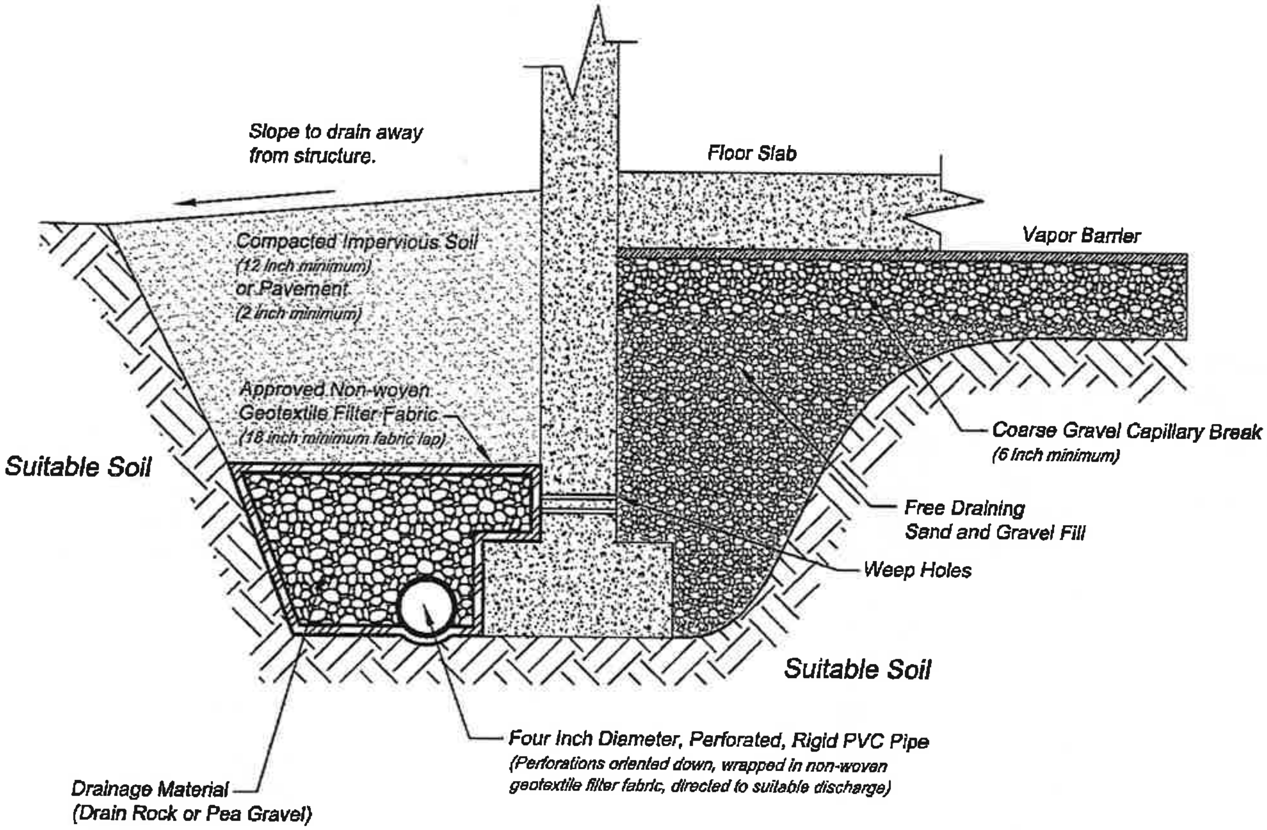
741 Marine Drive
Bellingham, WA 98225
phone: (360) 733-7318
fax: (360) 733-7418

Date: 8-31-07	By: DJ	Scale: As shown	Project 07-0556
SITE AND EXPLORATION PLAN CASCADE VALLEY HOSPITAL EXPANSION 330 S STILLIGUAMISH AVENUE 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)

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741 Marine Drive
Bellingham, WA 98225
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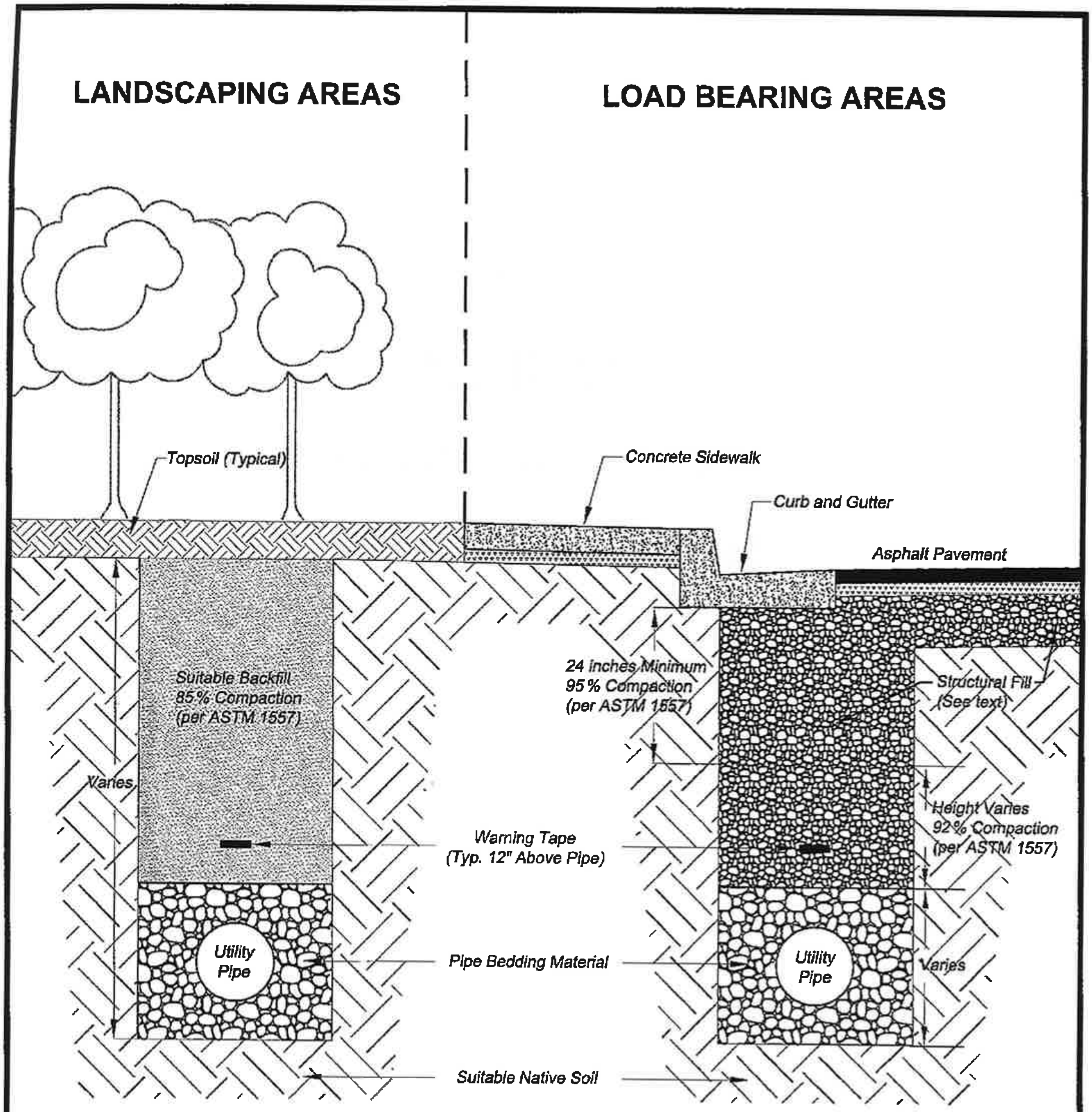
Fax: (360) 733-7418

Date: 9-5-07	By: DJ	Scale: NONE	Project 07-0556
TYPICAL FOOTING & WALL DRAIN SECTION			
CASCADE VALLEY HOSPITAL EXPANSION			Figure 3
330 S STILLIGUAMISH AVENUE			
ARLINGTON, WASHINGTON			



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

Date: 9-5-07 | By: DJ | Scale: NONE

TYPICAL UTILITY TRENCH SECTION
CASCADE VALLEY HOSPITAL EXPANSION
330 S STILLIGUAMISH AVENUE
ARLINGTON, WASHINGTON

Project **07-0556**

Figure **4**

APPENDIX A

FIELD EXPLORATIONS AND

LABORATORY TESTING

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Subsurface conditions at the site were explored on August 3rd and 4th, 2007. The exploration program consisted of drilling and sampling five test borings (B-1 through B-5) at the approximate locations illustrated on the Site and Exploration Plan (Figure 2 of this report). The Borings were explored with a truck mounted drill rig to depths ranging from 10 to 39 feet below ground surface by Environmental Drilling, Inc. of Snohomish, Washington. Our exploration program was laid out based on the location of the proposed hospital expansion as shown on a site plan drawn by Taylor Gregory Butterfield Architects. The explorations were located in the field by taping and pacing from existing property corners and other features shown the referenced plan. Exploration locations should be considered accurate to the degree implied by the methods used.

The field explorations were coordinated and monitored by a geologist 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 boring explorations are presented on Figures A-2 through A-6. These logs represent our interpretation of subsurface conditions identified during the field explorations. The stratigraphic contacts shown on the individual boring 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, soil grain size distribution, and Atterberg Limits.

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-7 and A-8. The liquid limit (LL), plastic limit (PL), and plasticity index (PI) of selected soil samples obtained from the exploratory test pits were determined in general accordance with ASTM D 4318 test procedures. The test results are presented on Figure A-9.

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

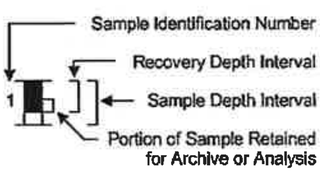
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test 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	Cascade Valley Hospital 330 S Stilliguamish Ave. Arlington, Washington	Soil Classification System and Key	Figure A-1

9/5/07 X:\10-PROJECTS GEO\CASCADE VALLEY HOSPITAL - 07-0558 - HOSPITAL EXPANSION\GINT.GPJ SOIL CLASS SHEET

B-1

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater: Water Level
0					AC	Approximately 2 inches of asphalt pavement encountered.	
	1	b2	64	W = 12	SP-SM	Medium dense to very dense, brown, damp, gravelly, slightly silty to silty SAND (possible fill).	
5	2	b2	34	W = 10 GS	SC	Medium dense to dense, light brown to gray, damp to moist, gravelly, very clayey, fine to coarse SAND, scattered cobbles and occasional boulders (glacial till).	
	3	b2	62	W = 10		Boulder/obstruction encountered at ~7 feet BGS. Relocated approximately 2 feet away and drilled to 7.5 feet BGS.	
10							
	4	b2	26	W = 14			
15							
	5	b2	45			Soil becomes gray below approximately 17.5 feet BGS.	∇ ATD
20							
	6	b2	50/6"	W = 13			
25							
	7	b2	29	W = 10 GS AL			
30							
	8	b2	29	W = 11			
35							
	9	b2	39	W = 10 GS			
40							
45							

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Not Determined

Boring Completed 08/03/07
Total Depth of Boring = 39.0 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.

GEOTEST

Cascade Valley Hospital
330 S Stilliguamish Ave.
Arlington, Washington

Log of Boring B-1

Figure A-2

B-2

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Water Level
0					AC	Approximately 2 inches of asphalt pavement encountered.	
					SM	Loose to medium dense, brown, damp, gravelly, silty SAND (possible fill).	
	1	b2	8	W = 14 GS			
5	2	b2	34	W = 9	SC	Medium dense to dense, light brown to gray, moist to wet, gravelly, very clayey, fine to coarse SAND, scattered cobbles and occasional boulders (glacial till).	
	3	b2	39	W = 13			∇ ATD
10							
	4	b2	38	W = 12			
15						Boulder/obstruction encountered at ~14 feet BGS. Relocated approximately 4 feet away and drilled to 17.5 feet BGS.	
	5	b2	55	W = 8			
20							
	6	b2	40	W = 11 GS		Soil becomes gray below approximately 22.5 feet BGS.	
25							
	7	b2	31	W = 13			
30							
	8	b2	39	W = 14			
35							
	9	b2	50/ 6"	W = 14			
40							
45							

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Not Determined

Boring Completed 08/03/07
Total Depth of Boring = 39.0 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.

-490702.00 9/9/07 X:\0-PROJECTS GEO\CASCADE VALLEY HOSPITAL - 07-0556 - HOSPITAL EXPANSION\GINT.GPJ SOIL BORING LOG

GEOTEST

Cascade Valley Hospital
330 S Stilliguamish Ave.
Arlington, Washington

Log of Boring B-2

Figure A-3

B-3

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
~3	1	b2	21	W = 12 GS
~6	2	b2	29	W = 18
~8	3	b2	37	W = 14
~13	4	b2	42	W = 11 GS
~18	5	b2	44	W = 10
~23	6	b2	61	W = 11
~28	7	b2	50/ 5"	
~33	8	b2	50/ 6"	W = 12 GS
~38	9	b2	42	W = 9

SOIL PROFILE

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Not Determined

USCS Symbol	Description
AC	Approximately 4 inches of asphalt pavement encountered.
SM	Medium dense, brown, damp, gravelly, silty SAND (possible fill).
SC	Medium dense to dense, light brown to brown, damp to moist, gravelly, clayey to very clayey, fine to coarse SAND, scattered cobbles and trace boulders (glacial till). Boulder/obstruction encountered at ~4 feet BGS. Relocated approximately 3 feet away and drilled to 5 feet BGS.

GROUNDWATER

Water Level: Slight (at approximately 20 ft)

Boring Completed 08/03/07
Total Depth of Boring = 39.0 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.

GEOTEST

Cascade Valley Hospital
330 S Stilliguamish Ave.
Arlington, Washington

Log of Boring B-3

Figure A-4

B-4

SAMPLE DATA					SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Hollow-stem Auger Ground Elevation (ft): Not Determined	
0						AC	Approximately 2 inches of asphalt pavement encountered.	Groundwater not encountered.
						SM	Medium dense, brown, damp, gravelly, silty SAND (possible fill).	
	1	b2	41	W = 11		SC	Dense to very dense, light brown to brown, damp, gravelly, clayey to very clayey, fine to coarse SAND, scattered cobbles and occasional boulders (glacial till).	
5	2	b2	53	W = 9				
	3	b2	52	W = 10				
10							Boulder/obstruction encountered at ~10 feet BGS. Relocated approximately 4 feet away and drilled to 12.5 feet BGS.	
	4	b2	54	W = 10 GS				
15							Boulder/obstruction encountered at ~15 feet BGS. Exploration terminated due to second refusal on object.	

Boring Completed 08/04/07
Total Depth of Boring = 15.0 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.

B-5

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater
						Drilling Method: Hollow-stem Auger Ground Elevation (ft): Not Determined	
0					AC	Approximately 2 inches of asphalt pavement encountered.	Groundwater not encountered.
					SM	Medium dense, brown, damp, gravelly, silty SAND (possible fill).	
	1	b2	22	W = 12	SC	Medium dense, light brown, damp, gravelly, clayey to very clayey, fine to coarse SAND, scattered cobbles and occasional boulders (glacial till).	
5	2	b2	27	W = 10			
	3	b2	24	W = 16			
10						Boulder/obstruction encountered at ~10 feet BGS. Exploration terminated due broken transmission on drill rig.	

Boring Completed 08/04/07
Total Depth of Boring = 10.0 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.

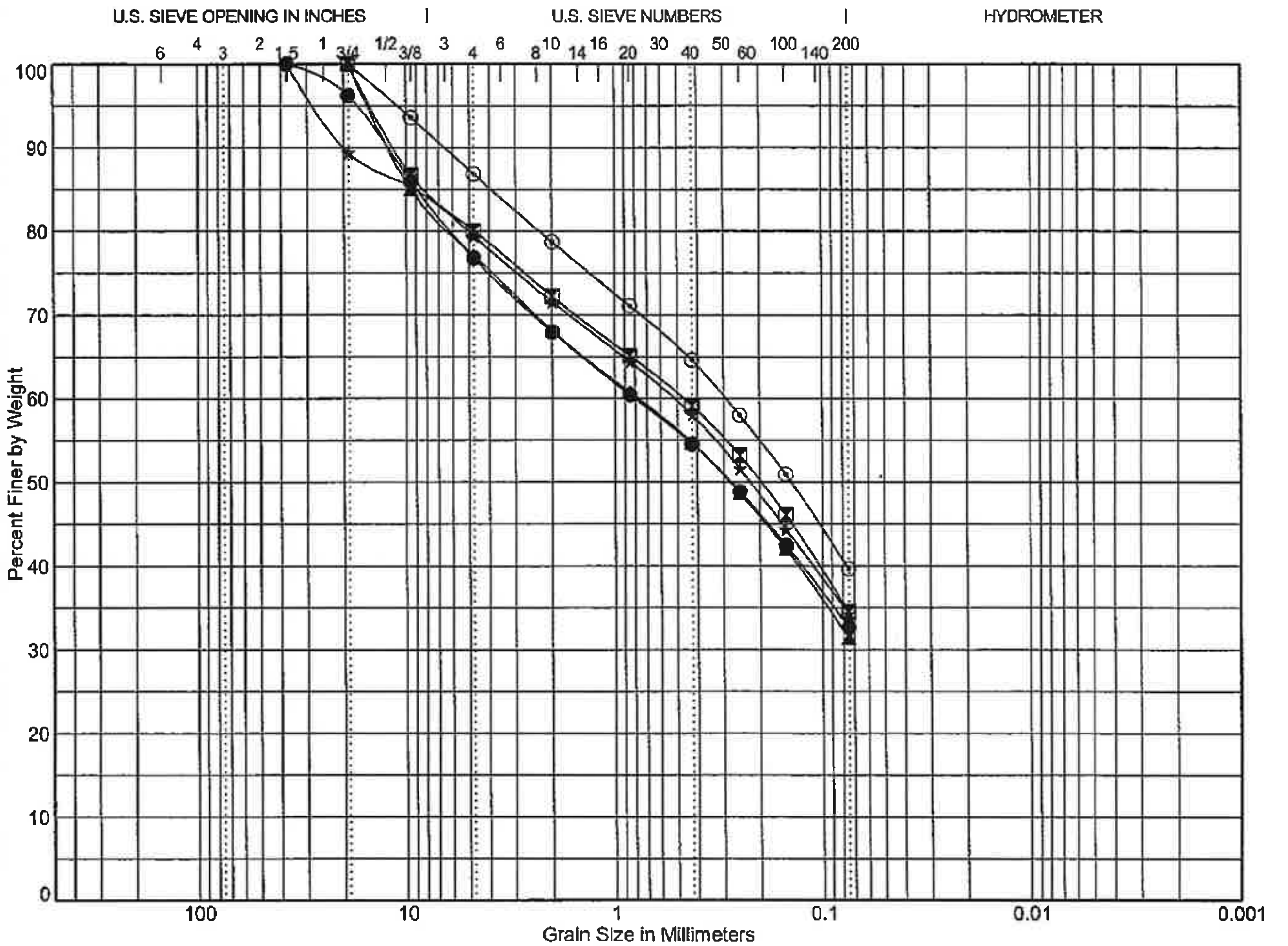
-490702.00 9/9/07 X:\0-PROJECTS GEO\CASCADE VALLEY HOSPITAL - 07-0556 - HOSPITAL EXPANSION\GINT.GPJ SOIL BORING LOG

GEOTEST

Cascade Valley Hospital
330 S Stilliguamish Ave.
Arlington, Washington

Log of Boring B-5

Figure A-6



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

	Point	Depth	Classification	LL	PL	PI	C_c	C_u
●	B-1	5.0	Gravelly, very clayey, fine to coarse SAND (SC)					
⊠	B-1	27.5	Gravelly, very clayey, fine to coarse SAND (SC)	22	12	10		
▲	B-1	37.5	Gravelly, very clayey, fine to coarse SAND (SC)					
★	B-2	2.5	Gravelly, very clayey, fine to coarse SAND (SC)					
⊙	B-2	22.5	Gravelly, very clayey, fine to coarse SAND (SC)					

	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	5.0	37.5	0.805	0.277			3.8	19.4	8.8	13.4	21.9	32.7
⊠	B-1	27.5	19	0.47	0.199			0.0	19.9	7.9	13.1	24.6	34.5
▲	B-1	37.5	19	0.781	0.281			0.0	22.9	9.0	13.4	23.3	31.4
★	B-2	2.5	37.5	0.526	0.223			10.6	10.0	7.9	13.5	23.7	34.3
⊙	B-2	22.5	19	0.294	0.142			0.0	13.1	8.2	14.1	24.9	39.7

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

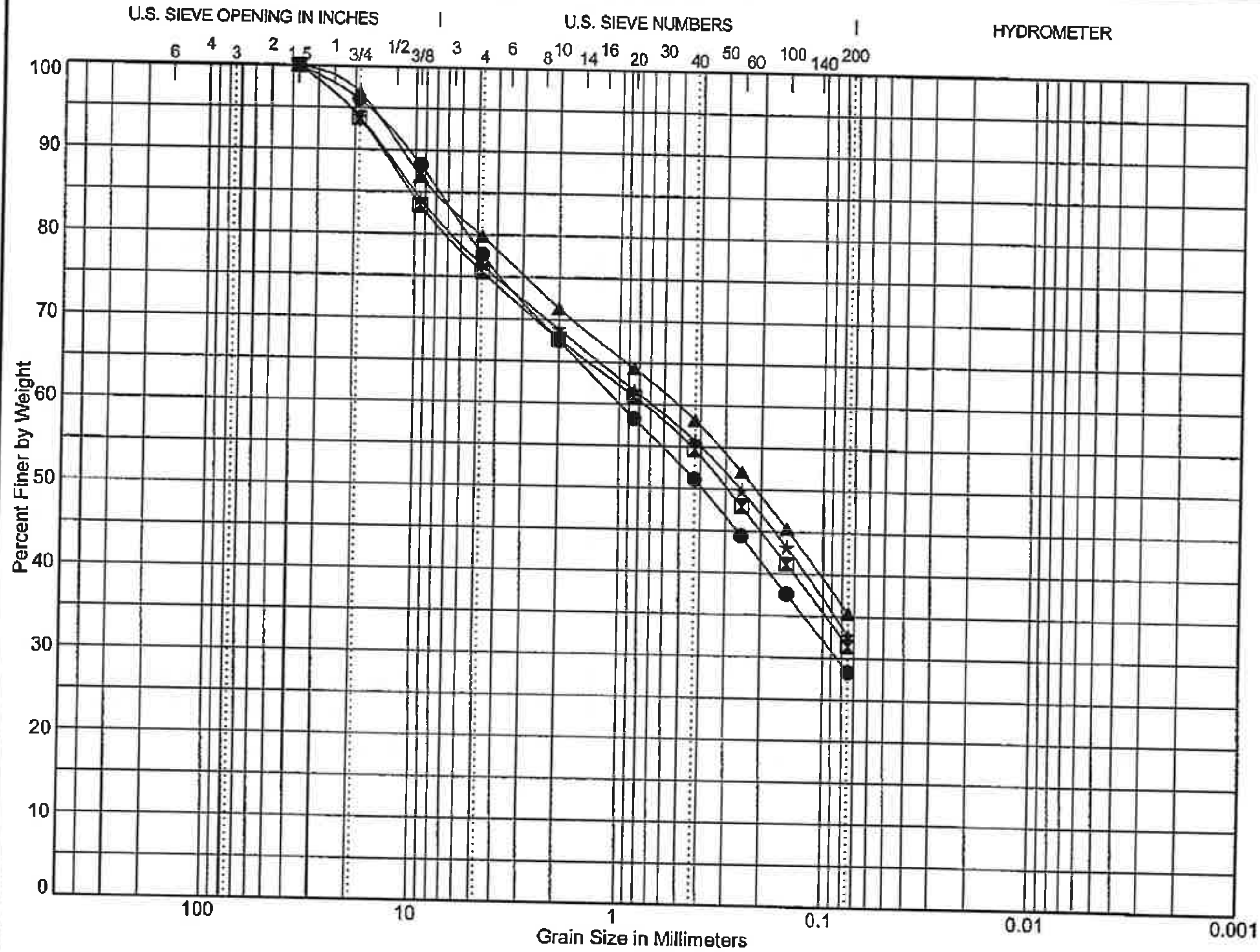
To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

Cascade Valley Hospital
330 S Stilliguamish Ave.
Arlington, Washington

Grain Size Test Data

Figure A-7



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point		Depth	Classification	LL	PL	PI	C_c	C_u
●	B-3	2.5	Gravelly, clayey, fine to coarse SAND (SC)					
⊠	B-3	12.5	Gravelly, very clayey, fine to coarse SAND (SC)					
▲	B-3	32.5	Gravelly, very clayey, fine to coarse SAND (SC)					
★	B-4	12.5	Gravelly, very clayey, fine to coarse SAND (SC)					

Point		Depth	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
●	B-3	2.5	37.5	0.998	0.389	0.085		3.9	18.4	10.1	16.4	22.7	28.4
⊠	B-3	12.5	37.5	0.769	0.293			6.2	18.2	7.9	13.0	23.2	31.5
▲	B-3	32.5	37.5	0.522	0.213			3.1	17.1	8.5	13.1	22.8	35.4
★	B-4	12.5	37.5	0.701	0.252			6.0	17.6	7.5	13.1	23.1	32.7

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

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

GEOTEST

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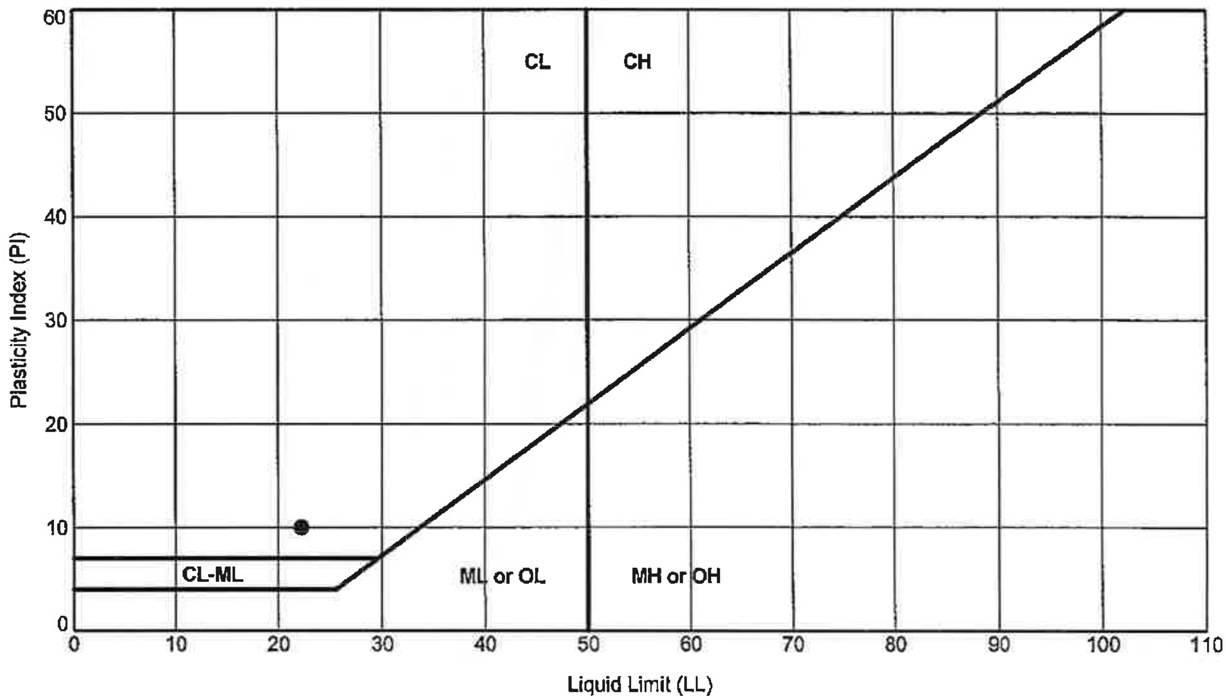
330 S Stilliguamish Ave.
Arlington, Washington

Grain Size Test Data

Figure A-8



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	7	27.5	22	12	10	10	Gravelly, very clayey, fine to coarse SAND	SC
~~ASTM D 4318 Test Method~~									

GEOTEST

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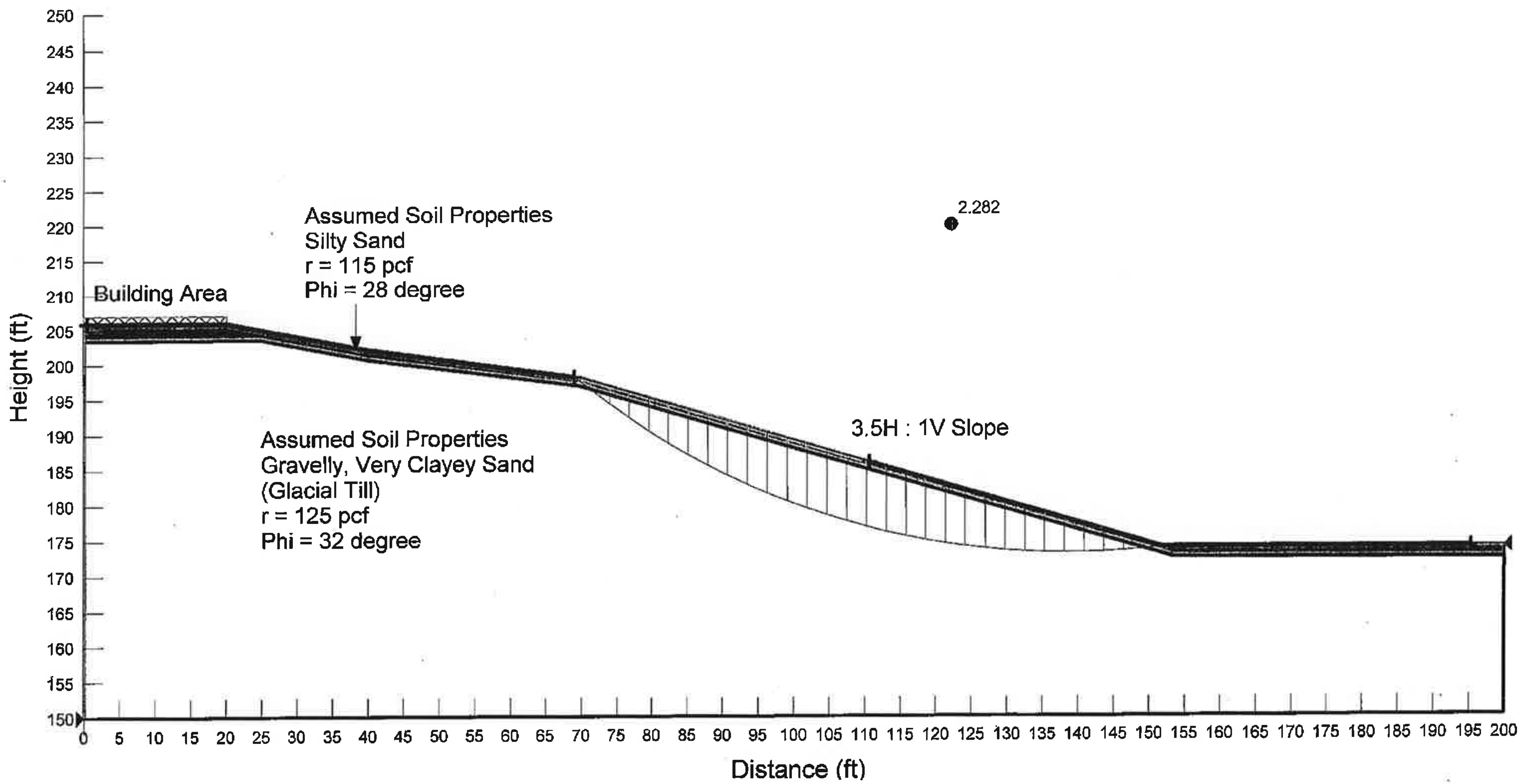
Arlington, Washington

Plasticity Chart

Figure
A-9

Cascade Valley Hospital
(Static Condition)



Cascade Valley Hospital
(Seismic Condition)