Washington State University

Agricultural Research and Technology Building

GEOTECHNICAL INVESTIGATION REPORT

Design Development Outline Specification

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DRAFT

GEOTECHNICAL INVESTIGATION

PROPOSED WSU MOUNT VERNON EXPANSION

MOUNT VERNON, WASHINGTON

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1.0 PROJECT AND SITE DESCRIPTION

Golder Associates Inc. (GAI) is pleased to present the results of our geotechnical engineering study for the proposed WSU Mount Vernon project. This project was completed in accordance with our proposal dated December 2, 2003.

The project site is located at the south side of State Route 536 in Mount Vernon, Washington (Figure 1). The project will consist of the construction of several additional structures at the Washington State University Mount Vernon Branch. The locations for the three proposed borings were provided to us by KPFF Consulting Engineers.

The purpose of the study was to investigate the subsurface conditions at the site and develop geotechnical design recommendations for the proposed structures. Specific design issues addressed in this report include:

- Subsurface soil and groundwater conditions at the site;
- Foundation support options and design recommendations;
- Retaining wall design criteria;
- Liquefaction assessment;
- Pavement design recommendations;
- Earthworks and construction related considerations.

2.0 INVESTIGATION PROCEDURES

A field investigation was completed between January 8th and 9th, 2004. The field investigation consisted of drilling one borehole to a depth of 99 feet below the existing ground surface and completing two cone penetration tests (CPT) to depths of 83.66 and 87.60 feet below the existing ground surface, in order to characterize the subsurface conditions. The boring and CPT locations are shown on Figure 2. In addition, laboratory classification testing was performed on selected borehole samples.

The drilling investigation was conducted using a truck-mounted Mobile B-59 drill turning 4.25-inch diameter (inside), hollow-stem, continuous flight augers. Water was maintained in the borehole during drilling to help prevent sample disturbance when sampling loose cohesionless deposits. Representative Standard Penetration Test (SPT) disturbed soil samples were obtained at 5 foot to 10 foot intervals. The samples were obtained in accordance with ASTM D-1586 "Penetration Test and Split Barrel Sampling of Soils". The test and sampling methods consist of driving a split spoon sampler a distance of 18 inches into undisturbed soils. The number of blows required to drive the sampler the final 12 inches is considered the Standard Penetration Resistance (N), which provides an indication of the relative density of granular soils and the relative consistency of cohesive soils. Field judgment is required when assigning density descriptions to soils containing a high percentage of coarse gravel and cobbles. In addition, Shelby samples were attempted in areas where cohesive soils were observed. All soil samples were placed in sealed sample jars to prevent moisture loss during transport to our Redmond, Washington laboratory.

The boring was monitored by an engineering geologist from our firm who located specific exploration locations, examined and classified the materials encountered, obtained representative samples, and recorded pertinent information including soil depths, stratigraphy, soil engineering characteristics, and groundwater levels. Representative soil samples were classified in accordance with Golder Associates Technical Procedures and the Unified Soil Classification System as described in Appendix A.

The results of twelve moisture content tests and two Atterberg Limits tests are provided on the borehole record in Appendix A. The ground surface elevation shown on the Borehole Record was estimated.

The cone penetration test results are presented in Appendix B. The measured data recorded in the field (i.e. tip resistance corrected for end area effects (Qt), porewater pressure (PWP), and sleeve friction resistance) and two interpreted parameters of classification index (Ic) and SPT blow counts corrected for energy and depth (N_{160}) are included on the sheets in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 Geologic Setting

The recent geologic history of the Puget Sound Lowland region has been dominated by several glacial episodes. The most recent, the Vashon stage of the Fraser glaciation is responsible for most of the present day geologic and topographic conditions. The Puget lobe of the Cordilleran ice sheet deposited a heterogeneous assemblage of proglacial lacustrine deposits, advance outwash, lodgment till, and recessional outwash upon either bedrock or older pre-Vashon sediments and bedrock. As the glacier retreated northward, it uncovered a sculpted landscape of elongate uplands and intervening valleys. Post glacial deposits include: alluvium deposited within active stream channels, modern lacustrine deposits, organic silt and local peat deposits within kettle depressions, drainages, and outwash channels; volcanic mudflow deposits and landslide deposits. The project is located upon a relatively flat floodplain containing primarily fine-grained alluvium overlying coarser grained alluvium.

3.2 Soil Conditions

The detailed subsurface conditions encountered in the borehole advanced during this investigation, together with the results of laboratory tests carried out on selected soil samples are presented on the Record of Borehole and CPT Sheets, see Appendix A and B. It should be noted that the stratagraphic boundaries shown on the borehole record is inferred from non-continuous sampling, observations of drilling progress, SPT results and laboratory test data. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

Based on the results of our recent subsurface explorations, the soils generally consist of very loose to compact sand alluvial deposits and very soft to firm silty clay and clayey silt alluvium/flood deposits overlying dense sand alluvial deposits. Descriptions for these soil units are provided below.

- **Alluvium:** These deposits were encountered near the ground surface to the bottom of each boring. The alluvium deposits consist of very loose to dense, nonstratified to weakly stratified sand with trace to some gravel, trace to little silt and trace organics. These alluvial deposits are, at times, interbedded with finer grained alluvium/flood deposits. The alluvium becomes dense below a depth of about 82 feet below ground surface.
- **Alluvium/Flood Deposits:** These deposits were encountered from the ground surface to a maximum depth of 74.0 feet below the existing ground surface. The alluvium/flood deposits consisted of very soft, nonstratified to weakly stratified, clayey silt and silty clay with trace fine sand, trace organics and trace peat layers. The alluvium/flood deposits were interbedded with the coarser grained alluvium deposits.

The following represents a simplified design site stratigraphy based on the borehole and CPT data.

Layer Number	Soil Designation (i)	Depth (ft)	Average N_{160} (ii)
1	Upper Clay	0 to 7	3
2	Upper Sand	7 to 15	15
3	Intermediate Clay (a)	15 to 18	3 to 4
4	Intermediate Sand (a)	18 to 21	10
5	Intermediate Clay (b)	21 to 34	4
6	Intermediate Sand (b)	34 to 38	15
7	Lower Clay	38 to 70	3 to 4
8	Interlayered Silts	70 to 82	4
9	Lower Sand	82 to 100	20 to 50 increasing with depth

Notes:

(i) A simplified soil designation has been utilized for soil types. Clay represents a clayey silt to silty clay (MH) and sand represents a range of sand trace silt to silty sand (SP-SM).

(ii) N160 value represents SPT blow counts corrected for depth and hammer energy.

3.3 Groundwater Conditions

Groundwater was encountered at a depth eight feet below the existing ground surface at the time our field explorations. A 1-inch piezometer was installed in the borehole to allow further groundwater monitoring. It is reasonable to assume that the groundwater surface fluctuates seasonally and that groundwater levels may be higher during wetter portions of the year. The measured groundwater level is shown on the borehole record in Appendix A. The groundwater level was measured at a depth of 8.4 feet below ground surface on January 21, 2004.

4.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained during the current investigation. The recommendations provided are intended for the guidance of the design engineers and are intended for this project only. The data may not be sufficient for construction and where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling, safety and the like.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous site activities or uses of the site and/or resulting from the introduction onto the site of materials from off site sources are outside the terms of reference for this report and have not been investigated or addressed.

This report addresses aspects of the foundation design for the proposed structures. We understand that the configuration of buildings proposed for the site has not been finalized. The project information utilized for the preparation of this report was provided by ARC Architects and KPFF.

For design purposes a groundwater level at Elevation 8.5 feet was utilized.

4.1 Seismic Criteria

We understand that the project will be designed using the 1997 UBC seismic design criteria. The project is located in seismic zone 3 and the seismic zone factor (z) is therefore 0.3. Based on our boring and CPT data, a site coefficient for soil characteristics designation of S_E can be assumed for the calculation of shear and lateral load on the building. However, the upper 100 feet of soil contains soil zones that are susceptible to liquefaction which would imply that a site specific dynamic soil response is required and a soil characteristics designation of S_F would be appropriate. We have carried out a liquefaction assessment which is discussed below and we consider that the settlements calculated from that assessment combined with the S_E site designation for calculation of structure loads will be conservative. A site specific dynamic response where the ground surface response is calculated, when taking into account liquefaction occurring in the soil, is beyond the current scope of our work.

We consider that a Magnitude 7.5 earthquake is appropriate for design and assuming the source zone is over 10 km away.

4.1.1 Liquefaction Assessment

The potential for soil liquefaction at the site was evaluated. Liquefaction refers to the temporary loss of soil shear strength due to increased pore water pressure, and a corresponding decrease in effective stress. This condition can develop in cohesionless soils subjected to cyclic loading. The main consequences of liquefaction are settlement, loss of foundation support and lateral spreading. The settlement which occurs can induce downdrag loads and lateral spreading can impose horizontal loading on a deep foundation system (piles, drilled shafts).

Based on USGS seismic hazard maps and assuming a Magnitude 7.5 earthquake with a 10 % probability of occurring in 50 years the design seismic event would have a peak acceleration of about 0.25 g. Note that the peak acceleration obtained from the USGS is bedrock acceleration and does not include any amplification or damping effect of the soil column. For assessment purposes and based on experience this value was not amplified through the soil column and a ground surface value of 0.25 g was assumed. As noted above a full site dynamic response assessment would be required if the liquefaction occurring in the soil were to be taken into account.

Several methods exist for the evaluation of liquefaction potential; however the most frequently used involve empirical correlations developed by Seed et al (1971, 1983). This method involves calculation of the earthquake induced cyclic shear stress and comparison with the liquefaction or cyclic resistance derived from in-situ testing. If the induced cyclic shear stress is greater than the cyclic resistance then the analysis indicates that liquefaction is likely to occur.

The induced cyclic shear stress was calculated using the simplified procedure of Seed and Idriss (1971) using the design peak surface acceleration.

The liquefaction or cyclic resistance was calculated using the NCEER procedure described in "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils" held in Salt Lake City, January 1996. The procedure utilizes several corrections to the recorded CPT tip resistance (q_c), including vertical effective stress and fines contents, to develop a corrected tip resistance ($q_{c1N(cs)}$) which is then correlated with liquefaction resistance. The liquefaction resistance or cyclic resistance ratio can then be compared with the induced cyclic stress ratio.

The results of the liquefaction assessment are shown in Figure 3. The figure shows the induced stress ratio from the Magnitude 7.5 event for the peak acceleration of 0.25g and the liquefaction resistance for each CPT value. Clay samples are considered to be non-liquefiable and are assigned a CRR of 0.5. The results indicate that liquefaction of seams/layers within the upper 12 feet to 25 feet, 40 to 55 feet and 65 to 75 feet of saturated granular deposits is likely to occur. The dense sand/gravel deposits and clay deposits are not liquefaction susceptible. The total thickness of liquefiable material was calculated to be about 20 to 25 feet.

An assessment of the post liquefaction settlement was carried out according to the method described by Tokimatsu and Seed (1987). The analysis indicated that between 4 inches to 6 inches of total settlement was likely to occur following the design seismic event. The settlement would probably occur relatively uniformly across the site with differential settlements on the order of 2 inches. In general, reduction of the liquefaction risk would involve some method of improving liquefaction resistance by increasing relative density. However, the site would be difficult to improve given the interlayered nature of the deposits and we consider that the settlement effects could either be avoided by use of a deep foundation system or allowed for in the design of a shallow foundation system. In either case we recommend that flexible utility connections are utilized at the building interface.

4.2 Foundation Recommendations

We consider that the use of lightly loaded shallow spread footings and/or a deep foundation system is feasible at this site. The deep foundation system could consist of driven steel piles or auger-cast piles. The main issue with respect to the site is that thick deposits of soft to firm clay are present and any foundation system has to be designed with respect to likely long term settlement response. The use of a preload to increase the allowable bearing pressure is also feasible and is discussed below.

4.2.1 Geotechnical Model

The following describes the geotechnical model for the site based on the geotechnical data collected, engineering correlations and experience. In particular the CPT data was utilized to produce distributions of undrained shear strength and over-consolidation ratio with depth, see Figures 4 and 5. Note these correlations are appropriate only for the cohesive (clay) type deposits.

Layer Number	Soil Designation	Unit Weight (pcf)	Compressibility Parameters (i)	Strength Parameters (ii)
1	Upper Clay	110	$c_c = 0.35$, $c_r = 0.03$, $e_o = 1.3$ OCR= 10 at GS to 3 at 7 feet	Su = 450 psf
2	Upper Sand	120	E= 160 tsf	c = 0 Φ=34 degrees
3	Intermediate Clay (a)	110	$c_c = 0.35$, $c_r = 0.03$, $e_o = 1.3$ OCR= 2	Su = 400 psf
4	Intermediate Sand (a)	120	E = 110 tsf	c = 0 Φ=32 degrees
5	Intermediate Clay (b)	105	$c_c = 0.6$, $c_r = 0.04$, $e_o = 1.5$ OCR= 2 at 21 feet to 1 at 34 feet	Su = 400 psf
6	Intermediate Sand (b)	120	E = 160 tsf	c = 0 Φ=34 degrees
7	Lower Clay	110	$c_c = 0.6$, $c_r = 0.04$, $e_o = 1.5$ OCR= 1	Su = 400 psf at 38 to 55 feet increasing to 1000psf at 70 feet
8	Interlayered Silts	110	E = 40 tsf	Su = 1000psf at 70 feet to 1400 psf at 82 feet
9	Lower Sand	125	E = 50 tsf at 82 feet to 320 tsf at 100 feet	c = 0 Φ=36 degrees

Notes:

(i) c_c = compression index, c_r = recompression index, e_o = initial void ratio, E = Young's Modulus, , OCR = overconsolidation ratio

(ii) c = effective cohesion Φ= effective stress friction angle, Su = undrained shear strength

4.2.2 Spread Footings

Conventional, shallow isolated or continuous spread footings may be used, provided they are founded on a subgrade of undisturbed native. Footings should not be placed on loose uncompacted fills or on topsoil/organic soils. Conventional shallow isolated or continuous spread footing foundations should be designed based on the following parameters:

The following are preliminary recommendations for spread footings.

- Maximum Settlement = 1 inch total, 0.5 inches differential
- Minimum Embedment Depth = 24 inches
- Minimum Footing Size = 24 Inches
- Factor of safety on bearing capacity = 3.0
- Strip footings - width up to 4 feet - Allowable Bearing = 900 psf
- Isolated Footings – up to 6 feet x 6 feet Allowable Bearing = 1,000 psf
  - These values may be increased by 1/3 for short-term wind and seismic loading.

- Assumptions – vertical loading with minimum footing sizes.
- Allowable Base Friction factor (FS = 1.5) = 0.35

The response of a mat foundation to uniform loading was also considered using a configuration based on the conceptual design of the proposed structure. A slab loaded to 500 psf would experience long term settlements of up to 4 inches and differential settlements of about 1.5 inches over a length of 50 feet. The major effect of having a widely loaded area is to transmit the load to deeper depths and in this case the lower clay layer begins to have a major influence on settlement as the loaded area increases. This analysis does not include the stiffness of the slab and therefore the actual differential is likely to be less than 1 inch. Additional combinations of slab configuration and subexcavation can be assessed if required.

For preliminary assessment purposes a modulus of subgrade reaction modulus (k_s) of 40 kcf may be utilized, for deformations under about 0.25 inches. Note the k_s is referenced to a 1 foot square plate or a beam 1 foot wide. Design using subgrade modulus is an iterative procedure between the structural and geotechnical engineers, as the modulus will change with deformation levels.

4.2.3 Deep Foundations – Auger Cast Piles

The following table summarizes our calculation of auger-cast pile capacity. We have determined that the piles would have to be extended to a depth of about 90 feet (or at least 8 feet into the lower sand deposit) in order to transfer the loads to an acceptable bearing layer and avoid pile settlement problems. A factor of safety of 3.0 has been assumed although this factor can be reduced to 2.0 if a full scale load test is carried out on a test pile to confirm the capacity. For short term transient loading the allowable capacities may be increased by 30 percent.

- Pile Diameter 16 inches – Allowable Capacity = 180 kips
- Pile Diameter 18 inches – Allowable Capacity = 230 kips
- Pile Diameter 24 inches – Allowable Capacity = 350 kips.

The above allowable capacities represent pile deformations of about 0.5 inches to 1 inch. The above capacities only consider geotechnical criteria and structural criteria may limit the allowable loads.

When a preliminary pile design layout has been completed the design should be reviewed to assess settlement of the pile group.

The base of the pile caps should be founded a minimum of 18 inches below ground surface for frost protection purposes.

If required we can provide pile capacities for driven pipe or H-piles.

4.2.3.1 Downdrag Loads

The design seismic event would produce ground settlements of up to 6 inches. This level of settlement would produce downdrag loads on the piles. For design purposes the following downdrag loads may be utilized for design;

- Pile Diameter 16 inches – Downdrag load = 140 kips
- Pile Diameter 18 inches – Downdrag load = 190 kips

- Pile Diameter 24 inches – Downdrag load = 200 kips.

The downdrag loads should be used to check the structural capacity of a pile during seismic loading. The downdrag load will produce additional movement of the pile to mobilize the additional resistance required. An estimate of this movement was made using conventional load transfer relationships for drilled shafts and a movement of about 0.5 inches was estimated. This implies that under the design seismic event a pile would settle 0.5 inches.

4.2.3.2 Horizontal Pile Response

The design of a pile subjected to lateral loads should take into account such factors as relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the maximum tolerable deflection at the head of the pile and pile group effects. For design purposes, both the structural and geotechnical capacities should be determined to establish the governing case.

The analysis of a pile under lateral loading is a problem in soil-structure interaction. The deflection of the pile is dependent on the soil response and the soil response is a function of the pile deflection. An iterative solution should be employed because the soil response is a non-linear function of pile deflection. A non-linear analysis typically utilizes curves relating the soil response and the pile deflection (p-y curves).

For the seismic design case the subgrade reaction values should be reduced significantly for the cohesionless deposit which liquefies.

This analysis can be carried out once more details, such as type of pile, pile group arrangement, loadings, pile cap fixity and the like have been determined. This process is typically carried out iteratively between the structural and geotechnical engineers.

4.3 Slab Subgrade

Conventional slab-on-grade floors can be supported on a subgrade of the native bearing soils. Slab-on-grade floors should not be founded on organic soils, very soft soils or uncompacted fills. The slabs should be underlain by a capillary break material, consisting of at least four inches of clean, free draining sand and gravel or crushed rock containing less than 3 percent fines passing the #200 sieve (based on the minus No. 4 sieve fraction) meeting the following specification:

TABLE 4-1

Capillary Break Gradation

Sieve Size or diameter (in)	% Passing
1"	100% Passing
No. 4	0% - 20%
No. 200	0 – 3%

A vapor barrier consisting of reinforced heavy plastic sheeting (6 mil or thicker) can be included between the slab and the capillary break. If desired, an additional a two-inch thick layer of sand may be placed on the vapor barrier to aid in concrete curing. The vapor barrier is recommended for all occupied space, storage area, and any areas to receive floor covering, carpeting, or finishes on the slab.

As an alternative to slab on grade, framed floors can also be used provided adequate subgrade drainage, footing drains, and venting of the crawl space is provided.

4.4 Foundation Drainage

We recommend that all crawl space areas be sloped for drainage and served with a minimum of an interior crawl drain that is connected to an approved storm drainage/outfall system. Additional foundation and slab-on-grade drainage systems may be necessary depending on groundwater presence or foundation design elements. This may include perimeter footing drains or foundation wall treatments for below grade living spaces. A perimeter footing drain is recommended for all external walls. The drainage should consist of a perforated drainpipe placed at the bottom of the footing, enveloped in drain rock and covered with filter fabric. Footing drains should consist of a 4-inch-diameter rigid-walled perforated PVC pipe or equivalent. The pipe should be surrounded by at least 6 inches of clean free-draining sand and gravel, having the gradation specified in Table 4-2. The drain should be tightlined to the storm system or other suitable discharge point.

TABLE 4-2

Drain Rock Gradation

Sieve Size or diameter (in)	% Passing
1 ½"	100% Passing
3/8"	10% - 40%
No. 4	0% - 5%
No. 200	0% – 2%

Roof drains should be collected and conveyed in a tightlined system separate from the footing drain system. The ground surface adjacent to the buildings should be graded to drain away from the building. Cleanouts should be provided on all drain systems.

4.5 Backfilled Walls

Adequate drainage should be provided for basement walls to minimize lateral earth pressures and to prevent buildup of hydrostatic pressures. The wall backfill should be used in conjunction with footing drains and/or other drainage provisions discussed above to provide full wall drainage. The backfill should be compacted firmly in maximum one-foot thick lifts near the optimum moisture content. The optimum moisture content is the water content at which the soil can achieve the highest density, as determined by ASTM D-1557, Proctor maximum dry density laboratory test. The contractor should avoid over-compaction adjacent to the wall in order to prevent an increase in the earth pressure.

Alternatives to the use of free draining fill behind walls would include continuous geocomposite drain strips placed continuously behind the wall or washed drain rock within 2 feet of the wall for the full height of the wall. Both these alternatives would be used in conjunction with wall footing drainage discussed above.

Backfilled basement walls can be designed for an at-rest earth pressure equivalent to a fluid density of 50 pcf; assuming there is no build up of hydrostatic pressure. The criteria presented section 4.2.2 can be used for preliminary foundation design. Note that the parameters in 4.2.2 are based on a 2 feet embedment below grade and if a basement is required Golder should be given the opportunity to reassess these values.

4.6 Retaining Walls

Retaining walls may be used to accommodate grade changes, and general site grading around buildings. A variety of wall types are feasible, including conventional cast concrete walls, mechanically stabilized earth (MSE) walls, and rockeries. Once the grades are known, and a specific wall type has been decided, detailed design recommendations can be developed. Cast-in-place or gravity walls can be designed with an active earth pressure or equivalent fluid weight equal to 35 pcf assuming a level backslope. In regards to foundations, the recommendations in section 4.2.2 apply to retaining walls as well as building foundations.

Earth Pressures for Retaining Structures:

Restrained Walls	50 pcf
Cantilevered Walls	30 pcf

These values assume a fully drained wall condition and flat backslope.

All retaining walls should be constructed with a permanent, full-face drain system. The wall footing drain should consist of a 4-inch diameter, perforated drainpipe bedded in a clean gravel backfill. The footing drain should convey the water under gravity flow, to the storm water collection system.

4.7 Ground Improvement

The potential use of ground improvement was assessed for the site. The use of shallow stone columns (say about 10 to 15 feet in length) bearing on the upper sand layer were considered. However, long term settlement of the improved ground would still be a problem. The stone columns would have to be extended to a depth of 80 to 90 feet to avoid the settlement problems. The use of deep stone columns would help to mitigate the liquefaction risk, improve the allowable bearing pressure and settlement response of the ground. Based on experience and for preliminary assessment purposes a column spacing of about 7 feet would be required and enable the bearing pressure of a mat foundation to be increased to about 2,000 psf.

As an alternative to in-situ ground improvement a preload could be utilized to induce ground settlement and enable a higher allowable bearing pressure to be used. For example a 10 feet preload placed within the building envelope would enable a mat foundation to be designed for a bearing pressure of at least 1,500 psf. A maximum settlement of about 17 inches would be induced by the preload and any surrounding existing structures and/or utility lines could be affected by the settlement zone that would be created. The estimated time that the preload would have to be applied would be on the order of years, given the thickness of the lower clay deposit. The use of wick drains would increase the settlement rate so that an acceptable level of settlement would be reached in few months.

In summary practical options for ground improvement include the use of deep stone columns or preloading with wick drains.

Any design involving ground improvement would benefit from additional laboratory soil testing. In particular consolidation testing of the lower clay testing would enable improved settlement and time rate of consolidation estimates to be made.

4.8 Pavement Design Criteria

It must be recognized that pavement design is a compromise between high initial cost coupled with low maintenance and low initial cost coupled with the need for periodic repairs. As a result, the owner should take part in the development of an appropriate pavement section. Critical features which determine the durability of the pavement surface include: the stability of the subgrade, presence or absence of moisture, free water, traffic volumes, and the frequency of use by heavy trucks.

The paving requirements will be for an interior access roadway in which the frequency of heavy truck use will be very small over the design life of the pavement. Provided that the pavement subgrade is prepared in accordance with the construction recommendations section of this report and the traffic loading is anticipated to be light, the following pavement sections are recommended for interior access roadways.

Recommended Minimum Pavement Section*:

3-inches AC – Class B
6-inches crushed base

A woven separation geotextile conforming to WSDOT Standard Specification Section 9-33, Table 3 shall be installed over approved, prepared subgrade to ensure proper performance of compacted crushed base.

Over firm and unyielding subgrade compacted to at least 95% of modified Proctor (ASTM D1557).

Alternative Pavement Section*:

2-inches AC – Class B
4-inches ATB
Over firm and unyielding subgrade compacted to at least 95% of modified Proctor (ASTM D1557)

*Paving materials and minimum sections should conform to the local municipal criteria.

If paving is required along the frontage of a city or county street, or a roadway within the development is to be a through roadway, the pavement section should be designed for the anticipated traffic loading.

If the moisture content of the native soils do not allow for compaction of the subgrade to be firm and unyielding, the use of admixes, such as cement and flyash, could be considered to improve the subgrade.

5.0 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

5.1 General

The on-site soils are moisture sensitive and can become soft and unworkable when wet. It is strongly recommended that the major grading and foundation work be performed in the driest summer months to minimize drainage and subgrade degradation problems and to optimize the use of on-site soil for fill.

Wet weather construction will add considerable cost and extend the construction schedule. It will require considerable off-site granular borrow for fills and/or admixtures to aid in the use of wet soils for structural fill, off-site disposal of wet excavated soils, and extensive use of off-site quarry spalls to maintain access roads.

5.2 Site Preparation/Grading

Site preparation should include removal of all existing structures, utilities, vegetation, root mass, organic soils and any other deleterious materials from building and paving areas or those locations where structural fill is to be placed. Such materials should be wasted from the site or utilized as landscaping fill. If clearing and stripping efforts are not accomplished during or after periods of dry weather during the summer months, much greater depths of soil removal can be expected in any wet or saturated areas present across the site. Stripping should also include removal of any uncontrolled fill and underlying organics and topsoil.

Subgrades to receive structural fill, building foundations, or pavement, should be cleared to expose undisturbed native bearing soils. Prior to placing fill and preparing building and pavement subgrades, we recommend proof rolling all exposed areas to determine if any soft areas are present. If any soft areas are observed, these areas should be either removed and replaced with structural fill or dried back and recompacted. However, the native subgrade will generally be soft to firm and therefore we do not recommend over excavating more than about 2 feet unless organic deposits are encountered. If organic deposits are encountered they should be completely removed. All pavement subgrades should be compacted to at least 95% of modified Proctor maximum dry density (ASTM D1557).

5.3 Temporary Excavation Slopes

Safe temporary excavations are the responsibility of the contractor and depend on the actual site conditions at the time of construction. Temporary cuts are the responsibility of the contractor and should comply with applicable OSHA and WISHA standards. We recommend that temporary cut slopes be excavated no steeper than 1H:1V (45 degrees). Cut slopes exposed for any length of time, particularly during wet weather, should be covered with visqueen to maintain stability and minimize erosion.

5.4 Erosion Control

Erosion control for the site will include the Best Management Practices (BMP's) incorporated in the civil design drawings and may incorporate the following recommendations:

- Limit exposed cut slopes;
- Use silt fences, straw, and temporary sedimentation ponds to collect and hold eroded material on the site;

- Seeding or planting vegetation on exposed areas where work is completed and no buildings are proposed; and
- Retaining existing vegetation to the greatest possible extent.

5.5 Construction Drainage

Even during dry weather, we recommend that site drainage measures be incorporated into the project construction. Construction of a detention pond first, either temporary or permanent, is recommended as these can be used for stormwater and silt traps during construction of the up-slope portions of the site.

Surface runoff can be controlled during construction by careful grading practices. Typically, these include the construction of shallow upgrade perimeter ditches or low earthen berms and the use of temporary sumps to collect runoff and prevent water from damaging slopes and exposed subgrades. All collected water should be directed, under control, to a positive and permanent discharge system such as the storm detention pond or vault. The site will need to be graded at all times to facilitate drainage and minimize the ponding of water.

5.6 Use of On-site Soils

The native surface soils encountered in our investigation will not be suitable for structural fill. Imported granular fill materials will be required for utility trench backfill, road subgrade, and any other structural fill.

On site soils are also not suitable for structural fill behind retaining walls. Structural fill behind retaining wall should be well-graded sand and gravel with less than 10 percent fines (% by weight passing the No. 200 sieve).

5.7 Fill Materials and Placement

We recommend using imported granular fill consisting of well graded material free of organic material, with less than 5 percent fines (that portion of the soil that passes the # 200 sieve). Other fill materials may be used with approval of the engineer.

Maximum Lift Thickness:

- Imported Granular fill 12 inches loose

Minimum Compaction Requirements:

- Beneath Building Foundations and Floors - The fill should be compacted to at least 95% of the ASTM D1557 maximum dry density value for the material. The structural fill beneath footings should at a minimum extend laterally at a 1H:1V slope projected down and away from the bottom footing edge.
- Beneath Roadways and Pavements - The fill should generally be compacted to at least 90% of the ASTM D 1557 maximum dry density value for the material, except within three feet of subgrade elevation, where the fill should be compacted to at least 95% of the ASTM D 1557 maximum dry density value for the material.
- Utility Trench Backfill - The fill should generally be compacted to at least 90% of the ASTM D 1557 maximum dry density value for the material, except in paved and

structural areas where the material should be compacted to at least 95% of the ASTM D 1557 maximum dry density value for the material.

- Non-structural/Landscaped Areas - Firmly compacted.

The structural fill should be compacted with equipment suitable to achieve proper compaction. Effective compaction of the granular glacial soils may be achieved with a large steel drum vibrator roller or hoe-pac compactor. A large steel drum, vibratory roller or hoe-pac compactor will be more suitable to compact granular fills. Thin lifts or work in confined areas can also be compacted with a jumping jack compactor. If density tests taken in the fill indicate that compaction is not being achieved, the fill should be scarified, moisture-conditioned, and recompacted. If the required densities cannot be met then the material can be excavated and replaced or a soil admixture used to dry the soil.

5.8 Auger Cast Piles

Auger cast piles are sensitive to the installation methods and contractor experience. Poor equipment and/or an inexperienced contractor can result in piles that are improperly installed and, in the worst case, piles that are completely "necked" providing essentially no significant resistance. Thus it is essential that auger cast piles are installed by experienced contractors with the full time monitoring of experienced geotechnical field engineers.

General monitoring requirements include the auger down pressure, identification of cuttings, grout pressure, the rate of auger withdrawal, and grout take. It is also recommended that as a minimum one small rebar is placed full depth into the grout after auger removal as a check on hole "necking". If the rebar cannot be installed to full depth then the pile should be rejected. Typical practical limits on auger cast pile depths would be 18 inch diameter to about 110 feet. The 90 feet pile length is achievable but the contractor should have demonstrated experience with such deep installations. Another potential limitation is the installation depth for reinforcing cages which are typically limited to a depth of about 30 feet.

5.9 Utilities

Maintaining safe utility excavations is the responsibility of the utility contractor. The utility trenches should be backfilled as noted in Fill Placement section (Section 5.7) of this report.

5.10 Construction Monitoring

We recommend that critical site construction elements be observed and documented by a qualified geotechnical consultant. These include: confirming suitable subgrade soils for building foundations and slabs, auger cast pile installation, compaction of structural fills, and utility trench backfill compaction under pavements.

6.0 USE OF THIS REPORT

This report has been prepared for the exclusive use of ARC Architects and their consultants for specific application to this project. Once the proposed project plans have been prepared, we recommend that we be given the opportunity to review the plans and specifications to verify that they are in accordance with our recommendations.

This study has been performed in general accordance with locally accepted geotechnical engineering practice to provide information on the site area. There are possible variations in the subsurface conditions between and adjacent to our exploration area, and in the groundwater conditions with time. We recommend that a contingency for unanticipated conditions be included in the construction contract to either confirm the anticipated conditions and/or provide for remedial action.

7.0 REFERENCES

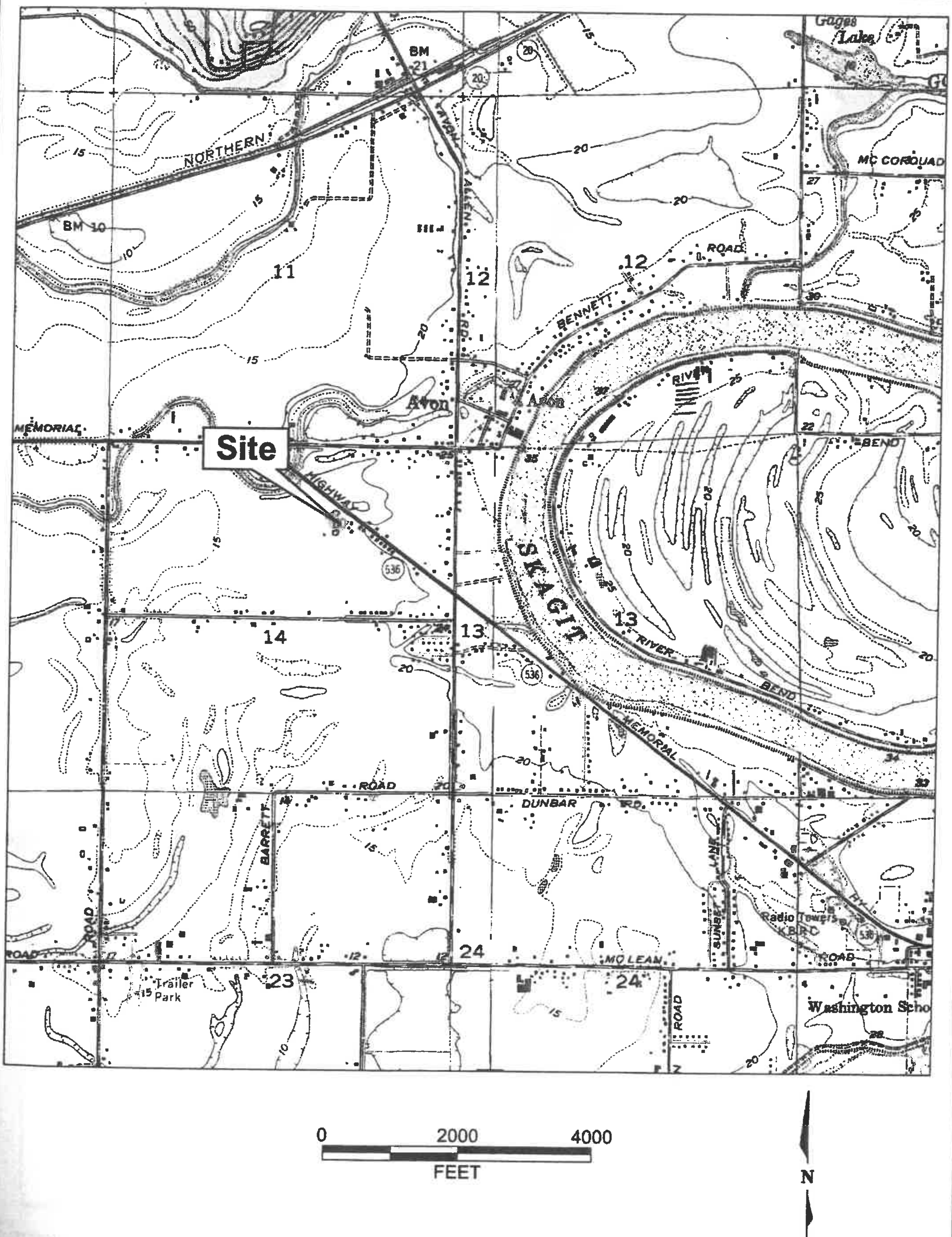
Seed, H.B. and Idriss, I.M., (1971). "Simplified Procedure for Evaluating Soil Liquefaction Potential." Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol.107, No.SM9, pp.1249-1274.

Seed, H.B., Idriss, I.M. and Arango, I. (1983). "Evaluation of Liquefaction Potential using Field Performance Data," Journal of Geotechnical Engineering, ASCE, Vol.109, No.3, pp.458-482.

Tokimatsu, K. and Seed, H.B. (1987). "Evaluation of Settlements in Sand due to Earthquake Shaking," Journal of Geotechnical Engineering, ASCE, Vol.113, No.8, pp.861-878.

Atrim, Ernest R. and Wunder, John M. (1976). "Preliminary Geologic Map of the La Conner Quadrangle, Skagit County, Washington. Department of Natural Resources, Division of Geology and Earth Resources, Open File Map OF-76-1.

FIGURES

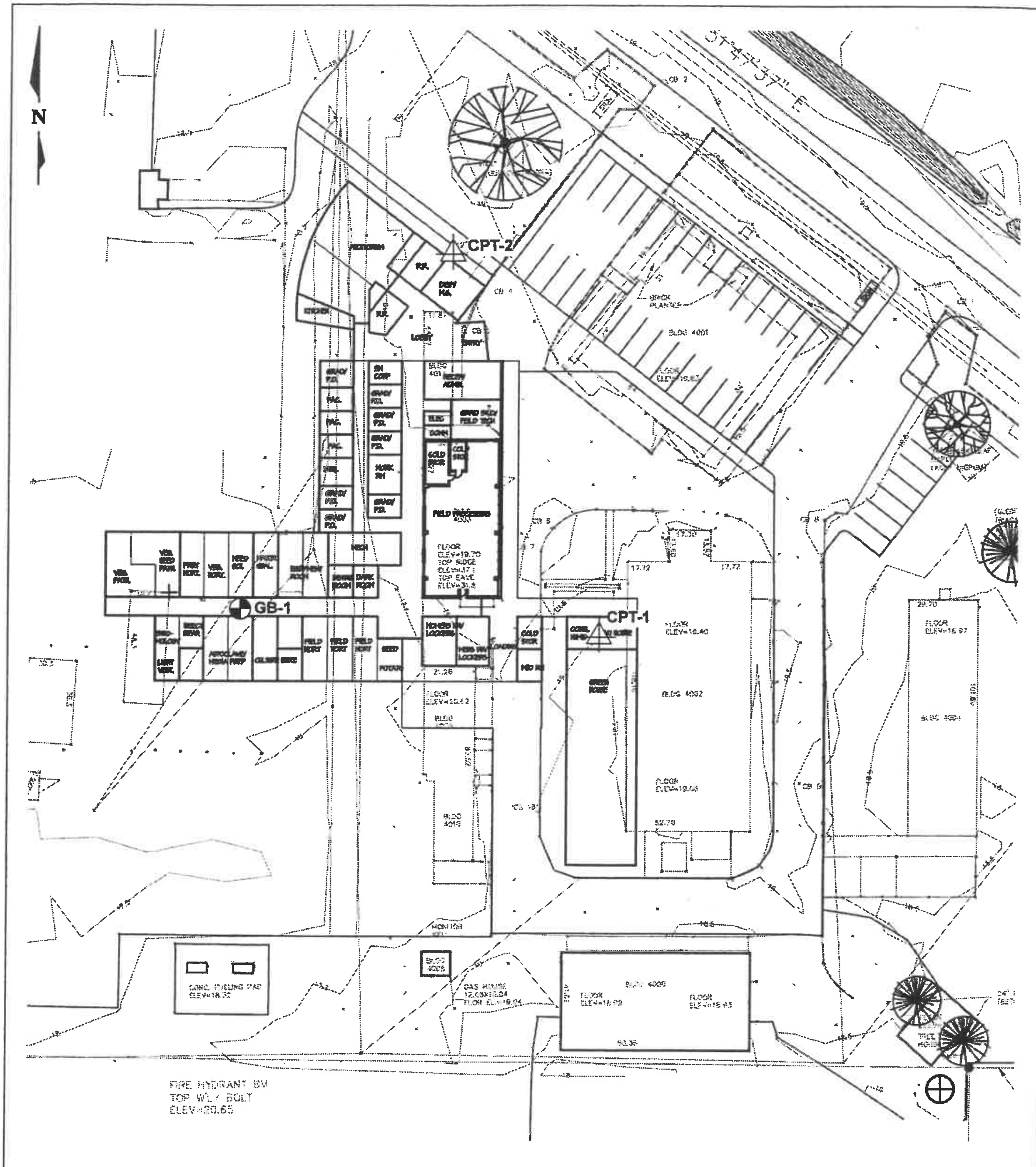


Source: USGS Topographic Map, Mount Vernon (1981), La Conner (1973)

FIGURE 1
SITE VICINITY
ARC/WSU GEOTECH INVESTIGATION/WA

DRAWING NO. 0331040000fg01.fh11 DATE 02/27/04 DRAWN BY AMP

Golder Associates



LEGEND

GB-1 Borehole with Monitoring Well

CPT-1 Core Penetration Test Location

Source: Drawing provided by ARC Architects

0 60 120
FEET

FIGURE 2
SITE EXPLORATION PROGRAM
ARC/WSU GEOTECH INVESTIGATION/WA

DRAWING NO. 0331040000fg03.fh11 DATE 02/27/04 DRAWN BY EFS

Golder Associates

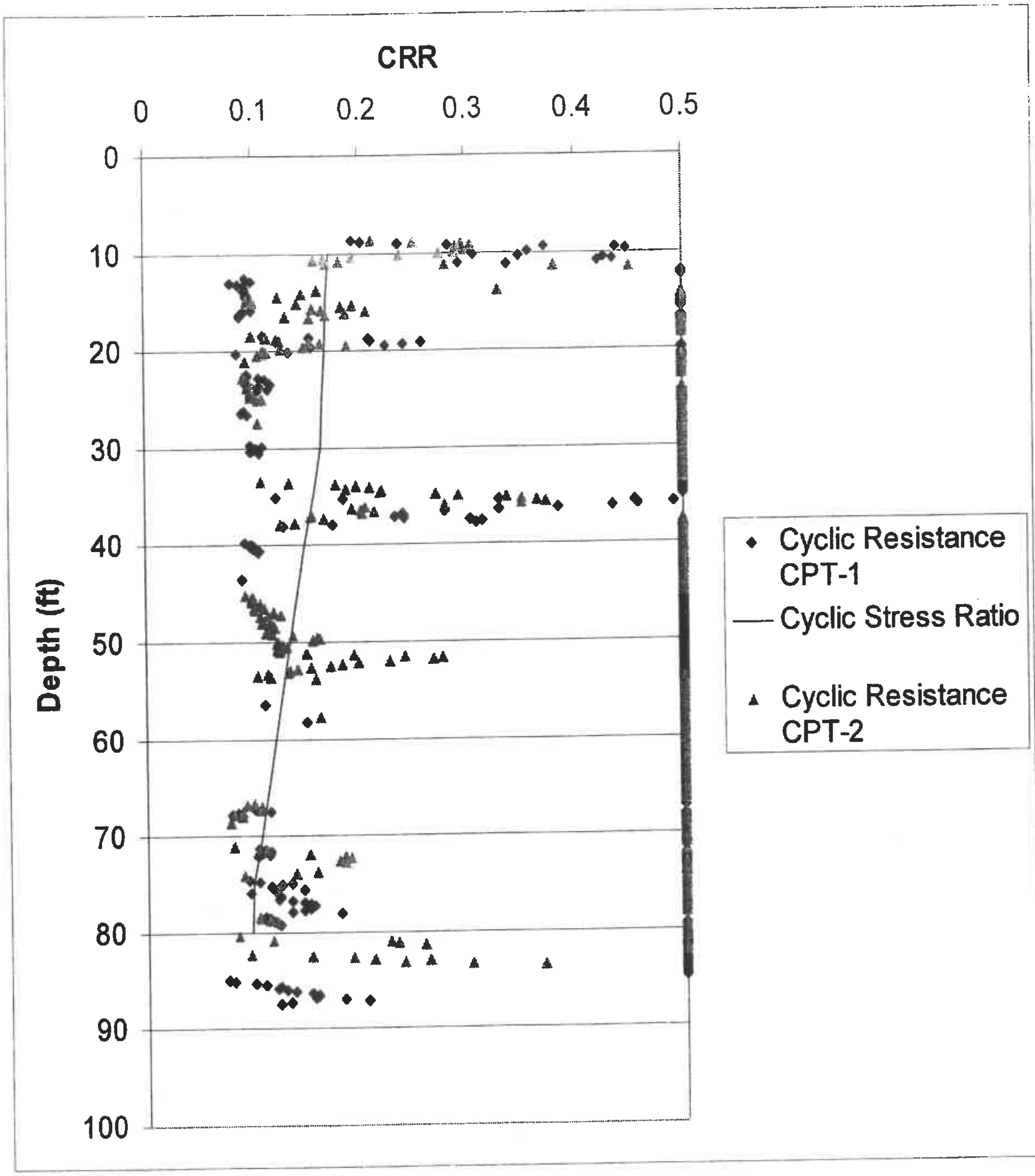


FIGURE 3

Results of Liquefaction Assessment

ARC/Mount Vernon/WA 033-1040

Golder Associates

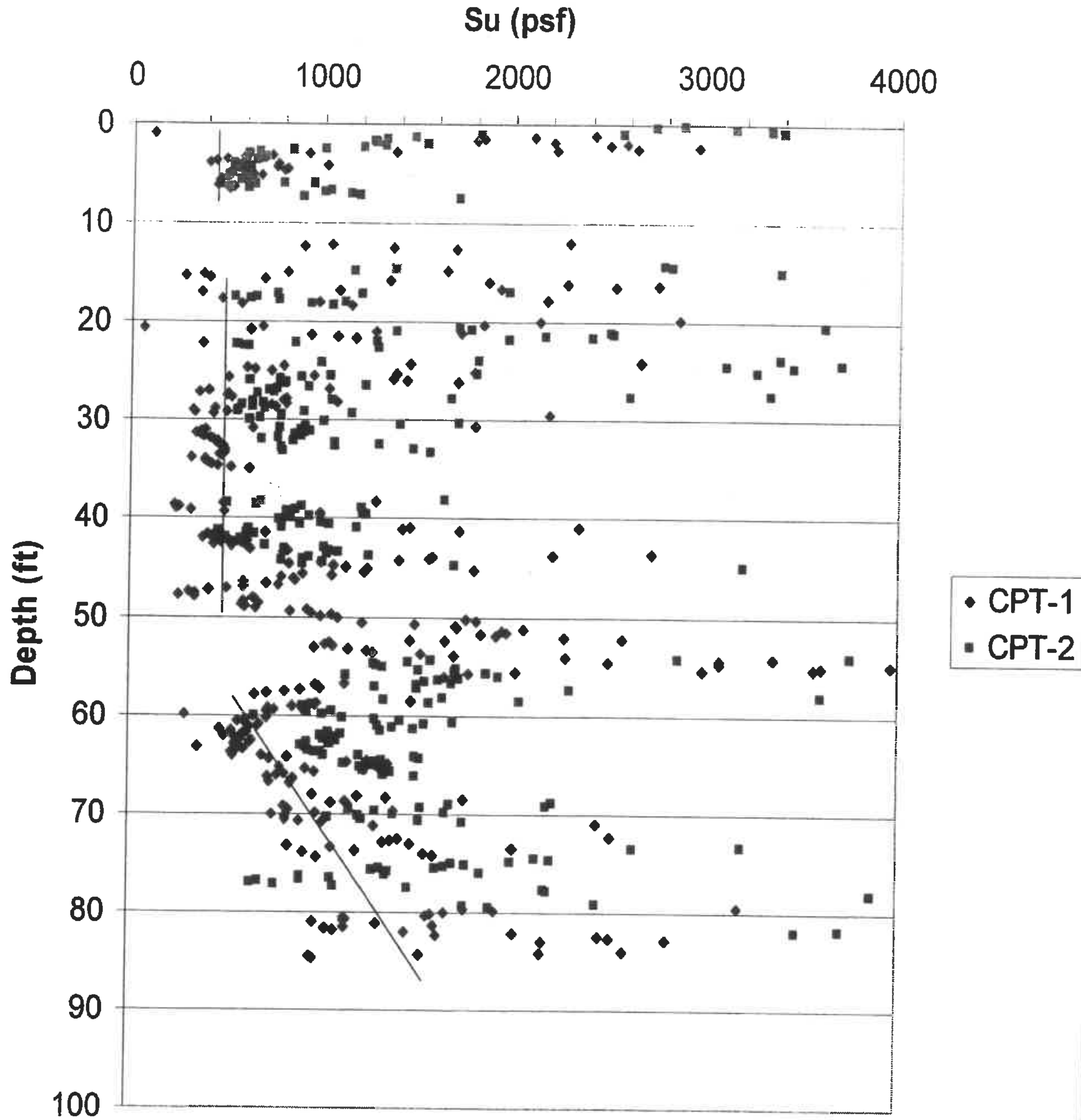


FIGURE 4
Undrained Shear Strength Summary
ARC/Mount Vernon/WA 033-1040

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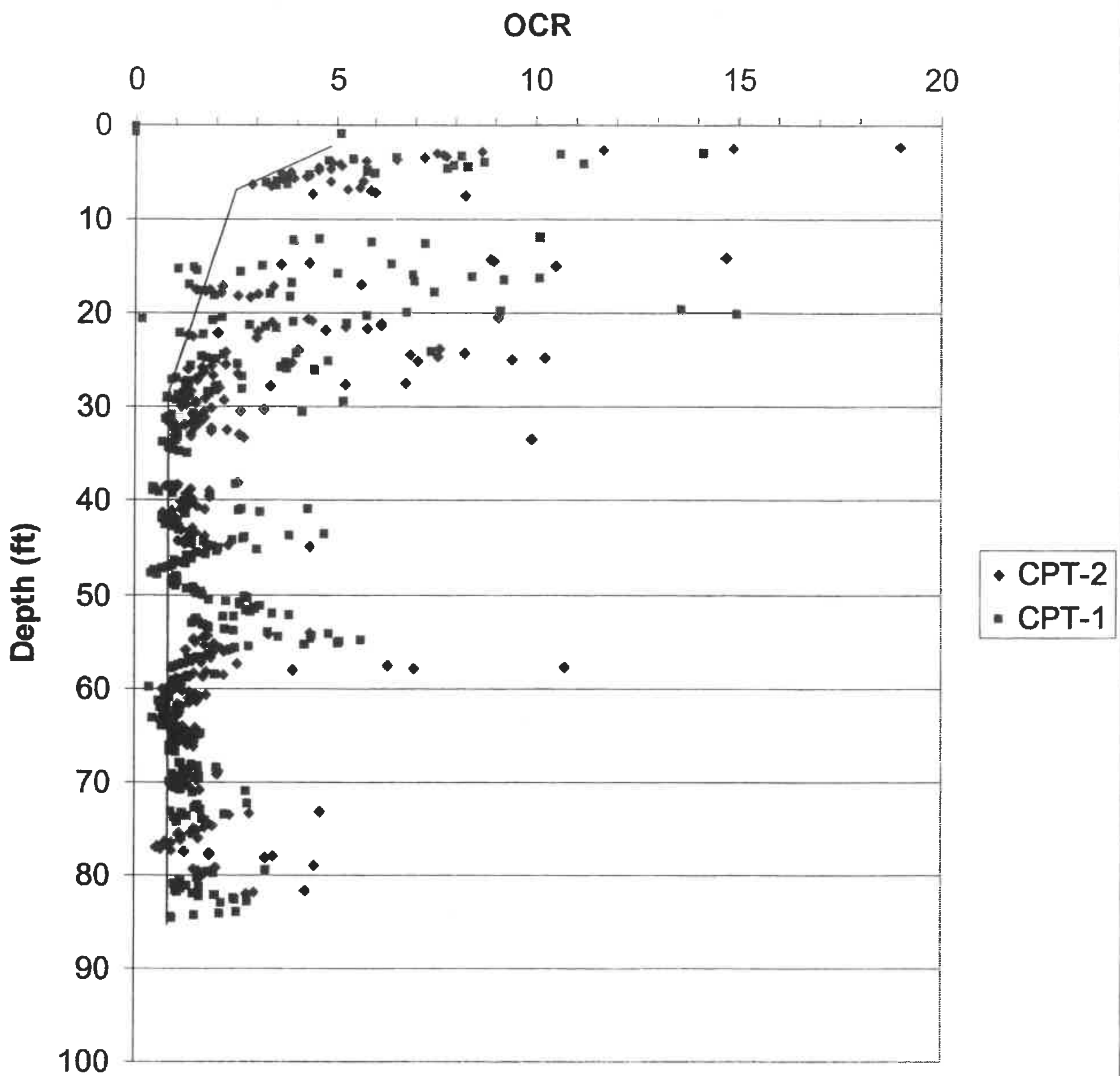

FIGURE 5
Over Consolidation Ratio Summary
ARC/Mount Vernon/WA 033-1040

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APPENDIX A

RECORD OF BOREHOLES

Unified Soil Classification System

Criteria for Assigning Group Symbols and Names			Soil Classification	Generalized Group Descriptions
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 Sieve	CLEAN GRAVELS Less than 5% fines	GW	Well-graded Gravels
			GP	Poorly-graded gravels
		GRAVELS WITH FINES More than 12% fines	GM	Gravel and Silt Mixtures
			GC	Gravel and Clay Mixtures
	SANDS 50% or more of coarse fraction passes No. 4 Sieve	CLEAN SANDS Less than 5% fines	SW	Well-graded Sands
			SP	Poorly-graded Sands
		SANDS WITH FINES More than 12% fines	SM	Sand and Silt Mixtures
			SC	Sand and Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50	INORGANIC	CL	Low-plasticity Clays
			ML	Non-plastic and Low-Plasticity Silts
		ORGANIC	OL	Non-plastic and Low-Plasticity Organic Clays; Non-plastic and Low-Plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50	INORGANIC	CH	High-plasticity Clays
			MH	High-plasticity Silts
		ORGANIC	OH	High-plasticity Organic Clays; High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

Component Definitions by Gradation

Component	Size Range
Boulders	Above 12 in.
Cobbles	3 in. to 12 in.
Gravel	3 in. to No. 4 (4.76mm)
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 (4.76mm)
Sand	No. 4 (4.76mm) to No. 200 (0.074mm)
Coarse sand	No. 4 (4.76mm) to No. 10 (2.0mm)
Medium sand	No. 10 (2.0mm) to No. 40 (0.42mm)
Fine sand	No. 40 (0.42mm) to No. 200 (0.074mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

Samples

SS	SPT Sampler (2.0" OD)
HD	Heavy Duty Split Spoon
SH	Shelby Tube
P	Pitcher Sampler
B	Bulk
C	Cored

Unless otherwise noted, drive samples advanced with 140 lb. hammer with 30 in. drop.

Relative Density or Consistency Utilizing Standard Penetration Test Values

Cohesionless Soils (a)			Cohesive Soils (b)		
Density (c)	N, blows/ft. (c)	Relative Density (%)	Consistency	N, blows/ft. (c)	Undrained Shear Strength (psf) (d)
Very loose	0 to 4	0 – 15	Very soft	0 to 2	<250
Loose	4 to 10	15 – 35	Soft	2 to 4	250–500
Compact	10 to 30	35 – 65	Firm	4 to 8	500–1000
Dense	30 to 50	65 – 85	Stiff	8 to 15	1000–2000
Very Dense	over 50	>85	Very Stiff	15 to 30	2000–4000
			Hard	over 30	>4000

(a) Soils consisting of gravel, sand, and silt, either separately or in combination, possessing no characteristics of plasticity, and exhibiting drained behavior.

(b) Soils possessing the characteristics of plasticity, and exhibiting undrained behavior.

(c) Refer to text of ASTM D 1586-84 for a definition of N; in normally consolidated cohesionless soils Relative Density terms are based on N values corrected for overburden pressures.

(d) Undrained shear strength = 1/2 unconfined compression strength.

Laboratory Tests

Test	Designation
Moisture	(1)
Density	D
Grain Size	G
Hydrometer	H
Atterberg Limits	(1)
Consolidation	C
Unconfined	U
UU Triax	UU
CU Triax	CU
CD Triax	CD
Permeability	P

(1) Moisture and Atterberg Limits plotted on log.

Descriptive Terminology Denoting Component Proportions

Descriptive Terms	Range of Proportion
Trace	0-5%
Little	5-12%
Some or Adjective (a)	12-30%
And	30-50%

(a) Use Gravelly, Sandy or Silty as appropriate.

Silt and Clay Descriptions

Description	Typical Unified Designation
Silt	ML (non-plastic)
Clayey Silt	CL-ML (low plasticity)
Silty Clay	CL
Clay	CH
Plastic Silt	MH
Organic Soils	OL, OH, Pt

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Figure

SOIL CLASSIFICATION/LEGEND

773-1064/FORM 573

RECORD OF BOREHOLE GB-1

PROJECT: WSU/Mount Vernon/WA
PROJECT NUMBER: 033-1040.000
LOCATION: Mount Vernon, WA

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 1/8/04 to 1/9/04
DRILL RIG: Mobile B-59

DATUM: MSL
AZIMUTH: N/A
COORDINATES: not surveyed

ELEVATION: 18' (Estimated)
INCLINATION: -90

BORING METHOD: 4.25 in. I.D. HSA w/ 140 lb. autohammer

DEPTH (ft)	DESCRIPTION	USCS	ELEV. / DEPTH (ft)	NUMBER	TYPE	BLOWS per 6 in (140 lb hammer, 30 inch drop)	N	REC / ATT
0.0 - 5.0	Very soft, olive gray to reddish brown, mottled, nonstratified, CLAYEY SILT, trace organics, moist to wet. (ALLUVIUM/FLOOD DEPOSITS) Sample 1: LL - 52; PL - 37; PI - 15	MH		1	SS	0-0-0	0	1.5/1.5
5.0 - 24.0	Very loose to compact, medium gray, nonstratified to weakly stratified, fine SAND, trace to little silt, trace organics, trace 1-in. silt interbeds @ 23ft. depth, trace organics, wet. (ALLUVIUM)	SP-SM	5.0	2	SH	NA	NA	0.0/2.0
				3	SS	0-2-4	6	1.5/1.5
				4	SS	4-8-9	17	1.5/1.5
				5	SS	2-3-1	4	1.5/1.5

PENETRATION RESISTANCE BLOWS / ft ■ (10, 20, 30, 40)
WATER CONTENT (PERCENT) W_p — W — W_l

NOTES / WATER LEVELS / GRAPHIC:
Concrete and flush mount monument
ATD 1/21/04
Bentonite Gel
1-in. PVC riser

Log continued on next page

1 in to 3 ft
DRILLING CONTRACTOR: Holt Drilling
DRILLER: Mike Reynolds

LOGGED: T. Sager
CHECKED: AJW
DATE: 2/26/2004

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RECORD OF BOREHOLE GB-1

PROJECT: WSU/Mount Vernon/WA
PROJECT NUMBER: 033-1040.000
LOCATION: Mount Vernon, WA

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 1/8/04 to 1/9/04
DRILL RIG: Mobile B-59

DATUM: MSL
AZIMUTH: N/A
COORDINATES: not surveyed

ELEVATION: 18' (Estimated)
INCLINATION: -90

DEPTH (ft)	BORING METHOD	SOIL PROFILE: DESCRIPTION	USCS	GRAPHIC LOG	ELEV. / DEPTH (ft)	SAMPLES: NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	PENETRATION RESISTANCE BLOWS / ft ■ (10 20 30 40); WATER CONTENT (PERCENT) W_p — W — W_l	NOTES WATER LEVELS GRAPHIC
20	4.25 in. I.D. HSA w/ 140 lb. autohammer	5.0 - 24.0 Very loose to compact, medium gray, nonstratified to weakly stratified, fine SAND, trace to little silt, trace organics, trace 1-in. silt interbeds @ 23ft. depth, trace organics, wet. (ALLUVIUM) *(Continued)*	SP-SM									
						6	SS	1-1-1	2	1.5/1.5	■	
		24.0 - 74.0 Very soft to firm, medium gray, nonstratified to weakly stratified, SILT to CLAYEY SILT to SILTY CLAY, trace fine sand, trace organics, trace layers of peat, wet. (ALLUVIUM/FLOOD DEPOSITS)			24.0							
25												
						7	SS	1-0-1	1	1.5/1.5	■	
30												1-in. PVC riser
			MH			8	SS	0-2-3	5	0.8/1.5	■ ○	
35												Bentonite Gel
						9	SS	7-4-2	6	1.5/1.5	■ ○	
40												

Log continued on next page

DRAFT

1 in to 3 ft
DRILLING CONTRACTOR: Holt Drilling
DRILLER: Mike Reynolds

LOGGED: T. Sager
CHECKED: AJW
DATE: 2/26/2004

Golder Associates

RECORD OF BOREHOLE GB-1

PROJECT: WSU/Mount Vernon/WA
PROJECT NUMBER: 033-1040.000
LOCATION: Mount Vernon, WA

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 1/8/04 to 1/9/04
DRILL RIG: Mobile B-59

DATUM: MSL
AZIMUTH: N/A
COORDINATES: not surveyed

ELEVATION: 18' (Estimated)
INCLINATION: -90

DEPTH (ft)	BORING METHOD	SOIL PROFILE: DESCRIPTION	USCS	GRAPHIC LOG	ELEV. / DEPTH (ft)	SAMPLES: NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	PENETRATION RESISTANCE BLOWS / ft ■ (10 20 30 40) / WATER CONTENT (PERCENT) W_p —O^W— W_l	NOTES WATER LEVELS GRAPHIC
40	4.25 in. I.D. HSA w/ 140 lb. autohammer	24.0 - 74.0 Very soft to firm, medium gray, nonstratified to weakly stratified, SILT to CLAYEY SILT to SILTY CLAY, trace fine sand, trace organics, trace layers of peat, wet. (ALLUVIUM/FLOOD DEPOSITS) *(Continued)*										
						10	SS	2-3-5	8	1.5/1.5		
45												
						11	SS	0-0-2	2	1.5/1.5		
50			MH									1-in. PVC riser
		Sample 12: LL - 81; PL - 64; PI - 17				12	SS	0-2-1	3	1.5/1.5		Bentonite Gel
55												
						13	SS	0-0-0	0	1.5/1.5		
60												

Log continued on next page

1 in to 3 ft
DRILLING CONTRACTOR: Holt Drilling
DRILLER: Mike Reynolds

LOGGED: T. Sager
CHECKED: AJW
DATE: 2/26/2004

Golder Associates

RECORD OF BOREHOLE GB-1

PROJECT: WSU/Mount Vernon/WA
PROJECT NUMBER: 033-1040.000
LOCATION: Mount Vernon, WA

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 1/8/04 to 1/9/04
DRILL RIG: Mobile B-59

DATUM: MSL
AZIMUTH: N/A
COORDINATES: not surveyed

ELEVATION: 18' (Estimated)
INCLINATION: -90

Boring method: 4.25 in. I.D. HSA w/ 140 lb. autohammer

SOIL PROFILE

DEPTH (ft)	DESCRIPTION	USCS	ELEV. / DEPTH (ft)
60	24.0 - 74.0 Very soft to firm, medium gray, nonstratified to weakly stratified, SILT to CLAYEY SILT to SILTY CLAY, trace fine sand, trace organics, trace layers of peal, wet. (ALLUVIUM/FLOOD DEPOSITS) *(Continued)*	MH	
74	74.0 - 82.0 Very loose, medium gray, nonstratified, fine SAND, trace gravel trace silt, wet. (ALLUVIUM)	SP-SW	74.0

SAMPLES

NUMBER	TYPE	BLOWS per 6 in (140 lb hammer, 30 inch drop)	N	REC / ATT
14	SS	0-3-3	6	1.5/1.5
15	SS	0-0-0	0	1.5/1.5
16	SS	0-0-0	0	1.5/1.5

PENETRATION RESISTANCE BLOWS / ft ■ (10, 20, 30, 40)
WATER CONTENT (PERCENT) W_p — W — W_l

NOTES / WATER LEVELS / GRAPHIC:
- 1-in. PVC riser
- Bentonite Gel
- Silica Sand
- 1-in. #20 slot PVC screen

Log continued on next page

1 in to 3 ft
DRILLING CONTRACTOR: Holt Drilling
DRILLER: Mike Reynolds

LOGGED: T. Sager
CHECKED: AJW
DATE: 2/26/2004

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RECORD OF BOREHOLE GB-1

PROJECT: WSU/Mount Vernon/WA
PROJECT NUMBER: 033-1040.000
LOCATION: Mount Vernon, WA

DRILLING METHOD: Hollow Stem Auger
DRILLING DATE: 1/8/04 to 1/9/04
DRILL RIG: Mobile B-59

DATUM: MSL
AZIMUTH: N/A
COORDINATES: not surveyed

ELEVATION: 18' (Estimated)
INCLINATION: -90

DEPTH (ft)	BORING METHOD	DESCRIPTION	USCS	ELEV. / DEPTH (ft)	NUMBER	TYPE	BLOWS per 6 in (140 lb hammer 30 inch drop)	N	REC / ATT	NOTES / WATER LEVELS / GRAPHIC
80	4.25 in. I.D. HSA w/ 140 lb. autohammer	74.0 - 82.0 Very loose, medium gray, nonstratified, fine SAND, trace gravel trace silt, wet. (ALLUVIUM) *(Continued)*	SP-SW							1-in. #20 slot PVC screen
		82.0 - 99.0 Dense to very dense, dark gray, weakly stratified, fine to coarse SAND, little to some gravel, trace silt, wet. (ALLUVIUM)	SW	82.0						Silica Sand
85										
					17	SS	13-19-19	38	1.5 / 1.5	
90										Hole slough
95										
					18	SS	17-34-50/0.3	50/0.3	0.0 / 1.3	
		Boring completed at 99.0 ft.		99.0						
100										

PENETRATION RESISTANCE BLOWS / ft ■ (10, 20, 30, 40); WATER CONTENT (PERCENT) W_p — W — W_l

1 in to 3 ft
DRILLING CONTRACTOR: Holt Drilling
DRILLER: Mike Reynolds

LOGGED: T. Sager
CHECKED: AJW
DATE: 2/26/2004

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APPENDIX B

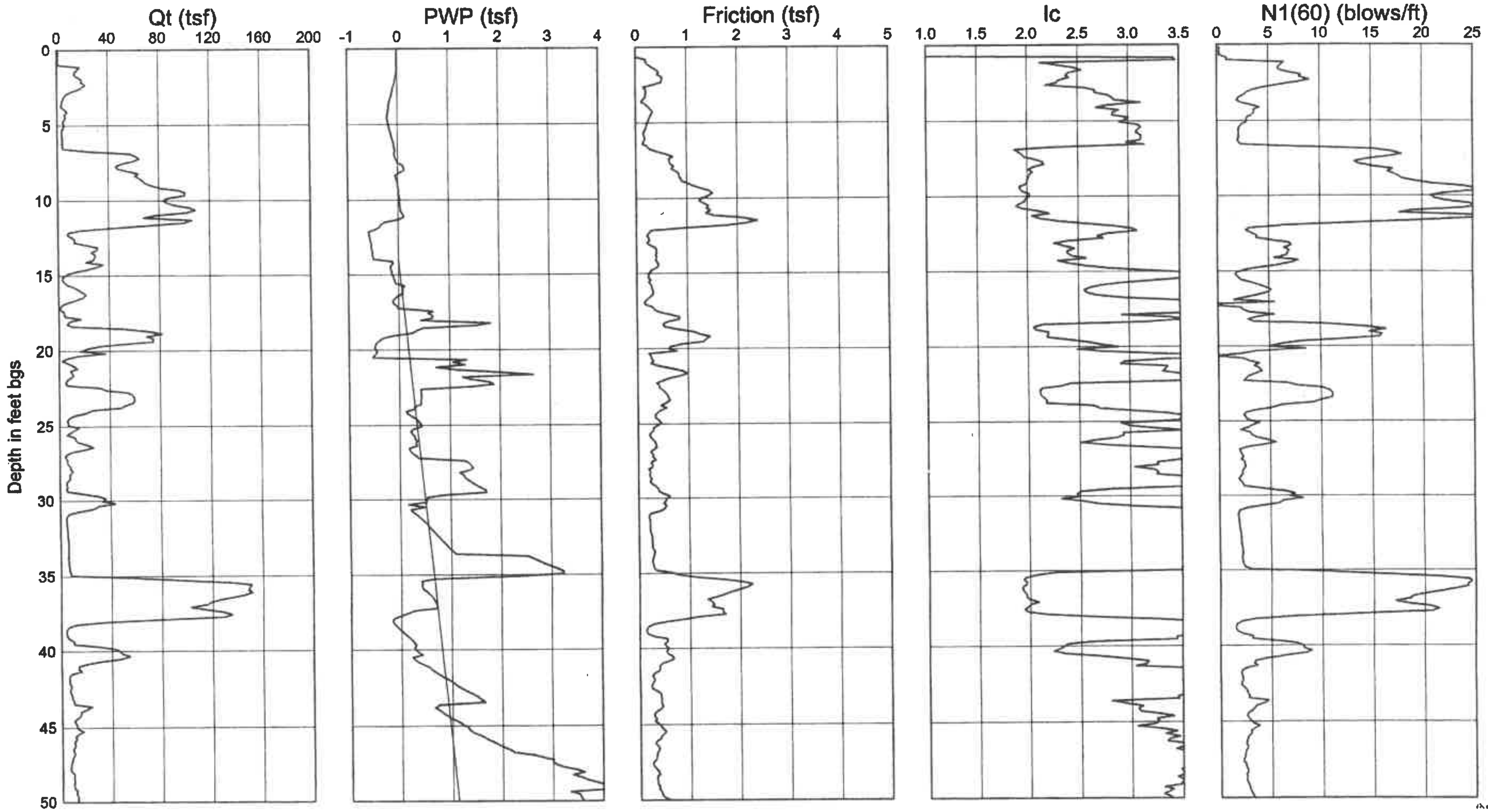
RECORD OF CPT'S

Cone Penetration Test - CPT-01

Test Date : 1/8/2004
Location : Mt Vernon WSU

Operator : NW Cone

Ground Surf. Elev. : 0.00
Water Table Depth : 14.00



Qt normalized for unequal end area effects

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

After Jefferies and Davies (1993)

PROJECT NO. 033-1040 DATE: March 5, 2004 DRAWN BY: AJW

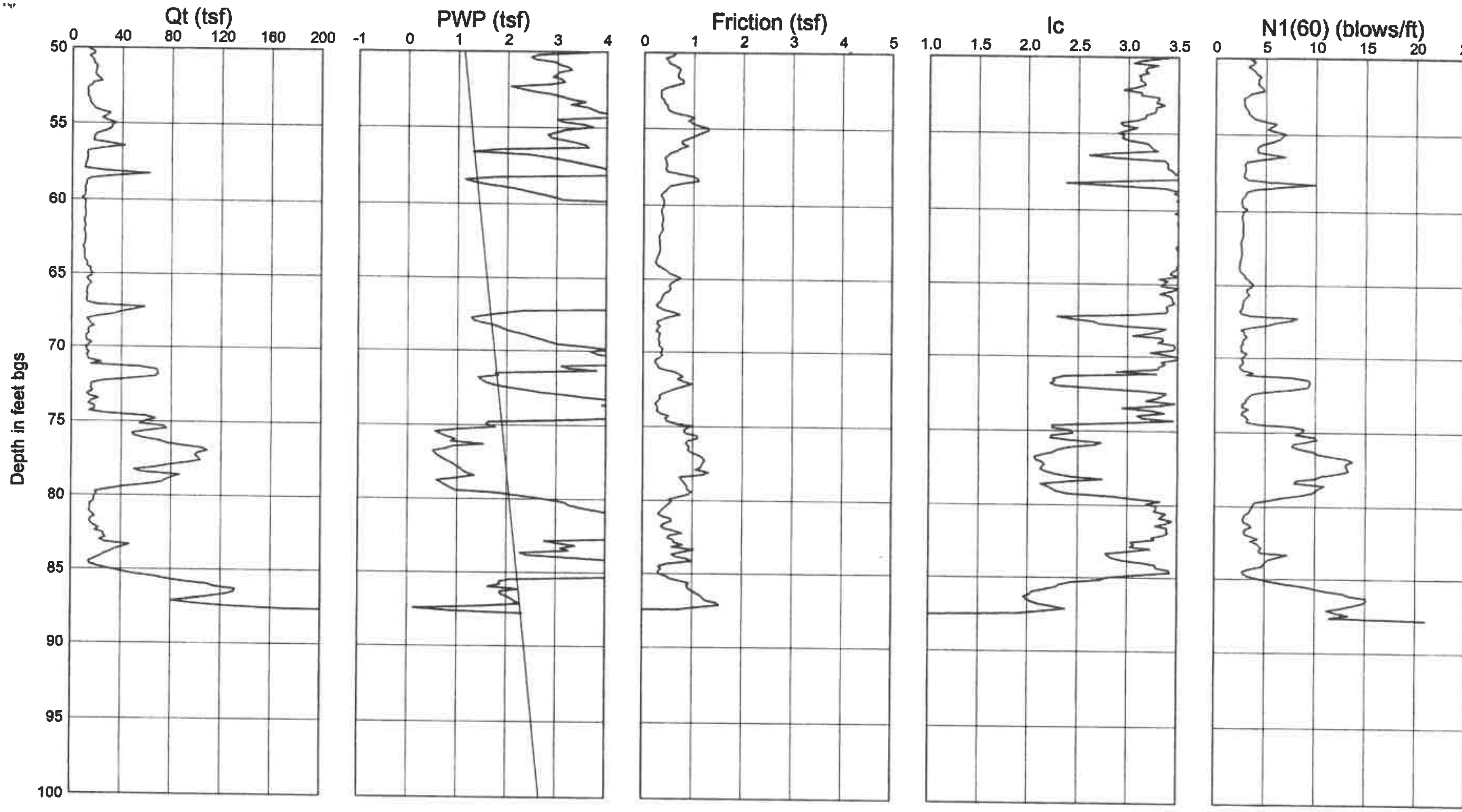
ARC/WSU Geotechnical Investigation

Cone Penetration Test - CPT-01

Test Date : 1/8/2004
Location : Mt Vernon WSU

Operator : NW Cone

Ground Surf. Elev. : 0.00
Water Table Depth : 14.00



Qt normalized for
unequal end area effects

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

After Jefferies and Davies (1993)

PROJECT NO. 033-1040   DATE: March 5, 2004   DRAWN BY: AJW

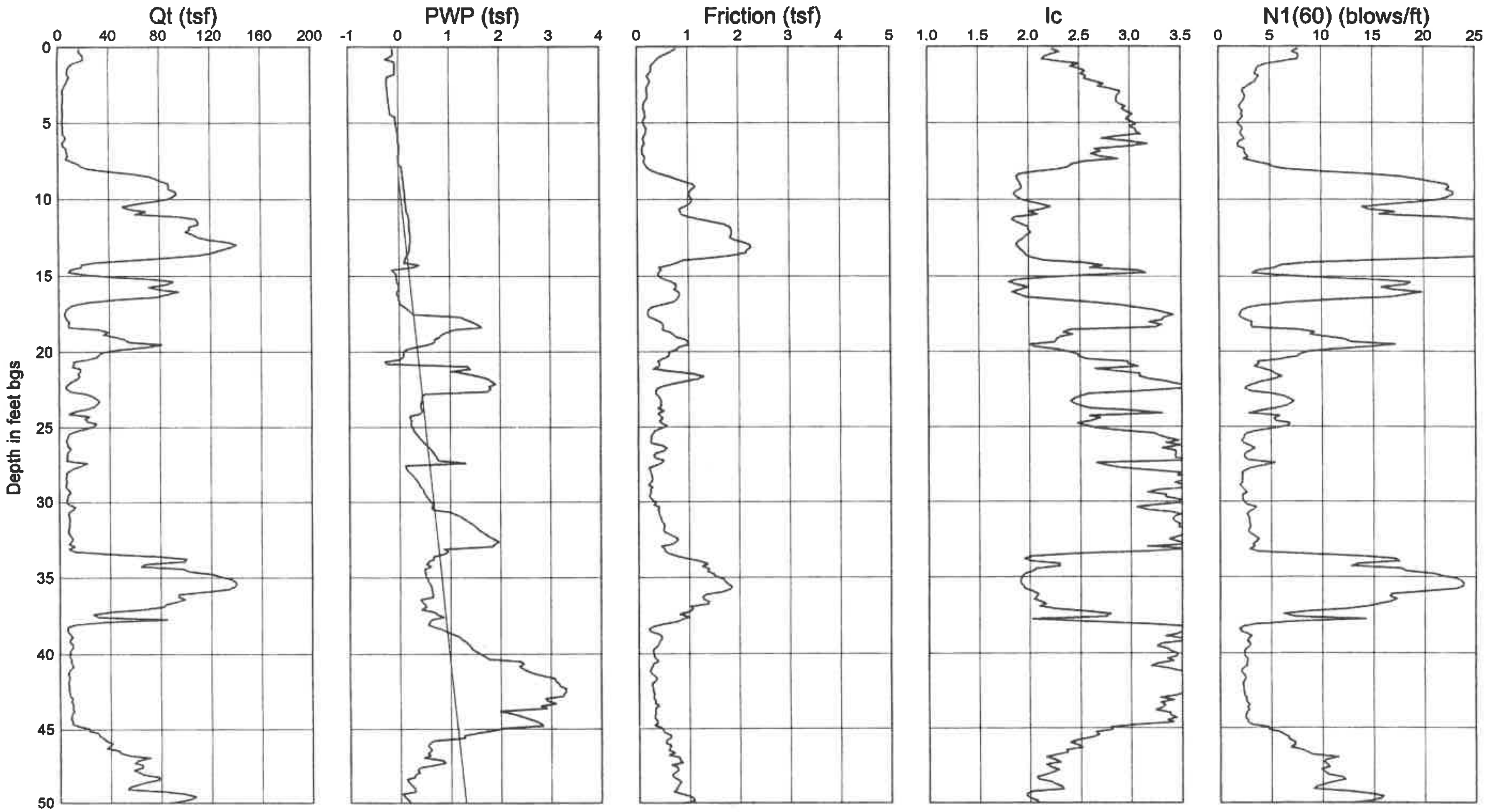
ARC/WSU Geotechnical Investigation

Cone Penetration Test - CPT-02

Test Date : 1/8/2004
Location : Mt Vernon WSU

Operator : NW Cone

Ground Surf. Elev. : 0.00
Water Table Depth : 8.50



Qt normalized for
unequal end area effects

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

After Jefferies and Davies (1993)

PROJECT NO. 033-1040 DATE: March 5, 2004 DRAWN BY: AJW

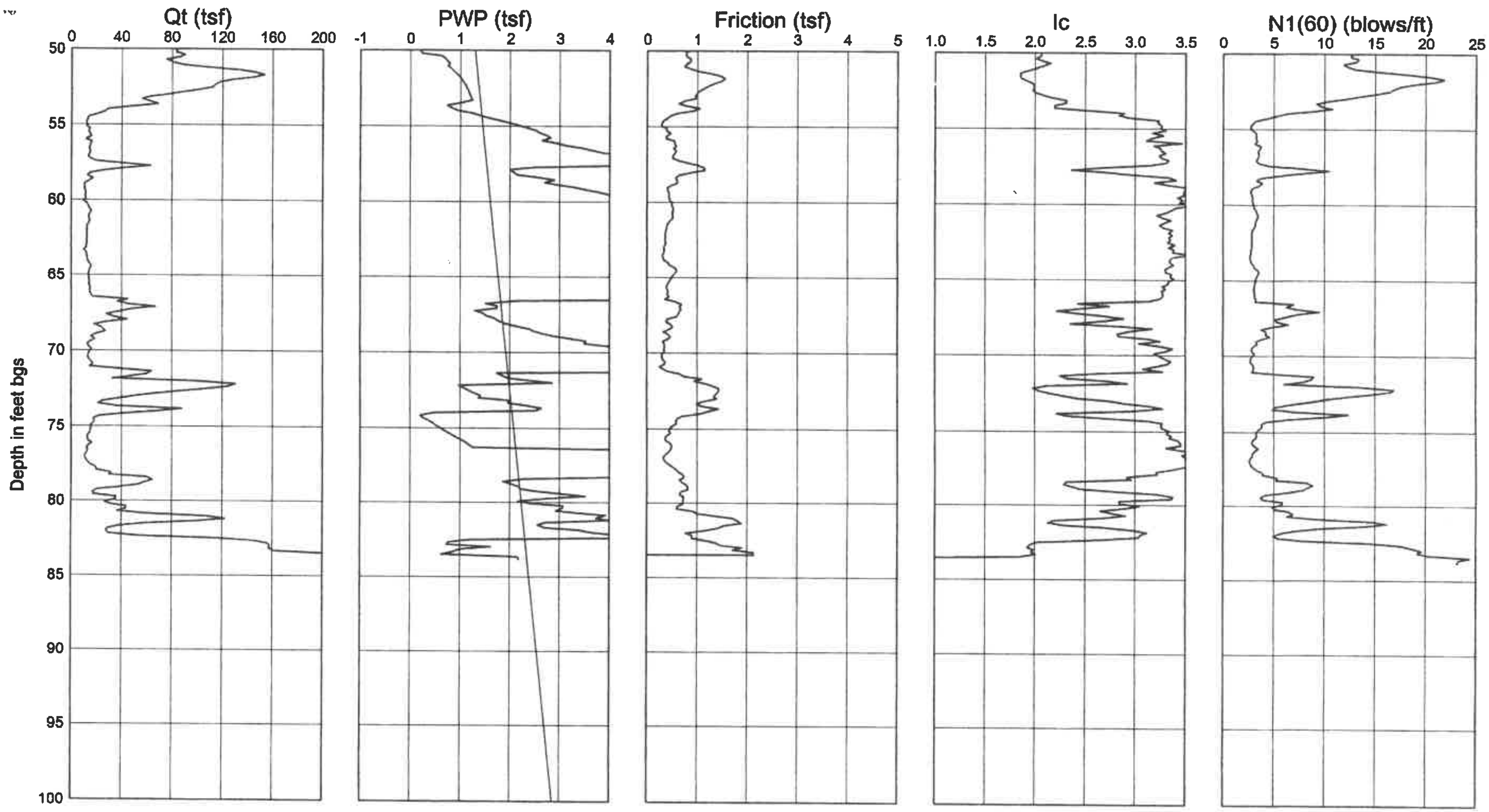
ARC/WSU Geotechnical Investigation

Cone Penetration Test - CPT-02

Test Date : 1/8/2004
Location : Mt Vernon WSU

Operator : NW Cone

Ground Surf. Elev. : 0.00
Water Table Depth : 8.50



Qt normalized for
unequal end area effects

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

After Jefferies and Davies (1993)

PROJECT NO. 033-1040 DATE: March 5, 2004 DRAWN BY: AJW

ARC/WSU Geotechnical Investigation