REPORT

NO. 90

Geotechnical Engineering Exploration and Analysis

Proposed Retail Site "G", "H", Wishkah and Heron Block Aberdeen, Washington

Prepared for:

Rite Aid Corporation
c/o Mulvanny Partnership Architects
Seattle, Washington

July 31, 1997

Project No. 6G-9706008

GILES
ENGINEERING ASSOCIATES, INC.

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GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

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July 31, 1997

Rite Aid Corporation
c/o Mulvanny Partnership Architects
112 Fifth Avenue
Seattle, WA 98109

Attention: Mr. Wayne Miller

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Retail Site
"G", "H", Wishkah and Heron Block
Aberdeen, Washington
Project No. 6G-9706008

Dear Mr. Miller:

In accordance with your request and authorization, Giles Engineering Associates, Inc. (*Giles*) has conducted a *Geotechnical Engineering Exploration and Analysis* for the above-referenced project. The conclusions and recommendations developed from the analysis are discussed in detail in the accompanying report.

We appreciate the opportunity to have been of service on this project. If there are any questions or if we may be of further service, please do not hesitate to call at any time.

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED RETAIL SITE
"G", "H", WISHKAH AND HERON BLOCK
ABERDEEN, WASHINGTON
PROJECT NO. 6G-9706008

1.0 EXECUTIVE SUMMARY

The following is a brief summary outline of the geotechnical conclusions and recommendations. This summary should be read in complete context with the accompanying report for proper interpretation.

Subsurface Conditions

- The subsurface exploration program performed for this study consisted of five hollow-stem auger borings advanced on June 23, 1997. Two borings extended to depths of about 60 feet and three extended to about 15 feet below the existing ground surface.
- A discontinuous and variable fill cap mantles the upper 4 to 5 feet of the site in the majority of the borings. Below the fill, soils consisted of soft organic clayey silt to about 12 feet. Soils to the maximum depth explored, 60 feet, consisted of soft clayey silt. At boring B-2, a dense gravel layer was encountered at 52 to 59 feet.
- Saturated soil conditions were encountered at depths of 4 to 5 feet.

Site Preparation and Development Recommendations

- The asphalt surface could be left in place, outside building areas, for a temporary construction platform.
- Existing pile foundations should be cut off at an angle, at least 3 feet below grade.
- Site soils are moisture and disturbance sensitive. Site earthwork would ideally be planned for drier months. If wet weather construction is planned, subgrade stabilization contingencies should be developed.
- The site should be proofrolled with heavy, rubber-tired equipment to assist in evaluation of the subgrade for pavements.
- Settlement of the site will be ongoing and may result in an uneven pavement surface and utility settlement over time.
- Existing asphalt could be reused for fill outside of building areas if pulverized to meet fill gradation specifications.
- Caving of the site soils should be anticipated in excavations, particularly below the water table. Cuts should meet the requirements of federal, state, and local ordinances, such as OSHA and WISHA. Use of temporary bracing, "trench boxes" and/or sloped cuts should be anticipated.
- Sump pumps placed in temporary excavations for utilities are anticipated to be adequate for dewatering if the excavation extends only 1 to 2 feet below the groundwater level. Otherwise, specialized dewatering techniques could be required.

Building Foundations and Floors

- Driven timber piles could be utilized to support foundation and floors to reduce potential settlements. Piles should be a minimum of 40 feet in length. Piles with an 8-inch tip diameter could be designed with a 15-ton working capacity. Piles with 10-inch tips could be designed with a 20-ton working capacity. Alternately, 12-inch diameter auger-cast piles of the same length could be used.
- At least two pile load tests should be performed prior to installation of production piles.

Pavements

- Subgrade should be compacted to 95 percent of the modified Proctor maximum dry density in the upper 2 feet. Subgrade soils should have an equivalent CBR value of at least 10 to 15.
- Heavy and light pavement sections consisting of 2½ or 3 inches of asphaltic concrete surface over a 6-inch thick aggregate base course, respectively, are considered adequate if the subgrade is prepared as recommended. Truck lanes should utilize a 4-inch asphalt section.
- Poor subgrade conditions where existing fills are thin, such as at boring B-2, may be encountered requiring overexcavation and replacement. Extensive overexcavation and subgrade stabilization is typically required beneath the footprint of demolished pile-supported buildings.

2.0 SCOPE OF SERVICES

The authorized scope of services performed by Giles Engineering Associates, Inc. (*Giles*) for this phase of the project included performing a visual site reconnaissance, subsurface exploration program consisting of hollow-stem auger borings, geotechnical engineering analyses, and preparation of this report. Work was performed as described in *Giles*' proposal addressed to Rite Aid Corporation c/o Mulvanny Partnership Architects dated April 24, 1997 and amended June 13, 1997. The purpose of this study was to establish general subsurface conditions from which conclusions and recommendations regarding foundation design and site development could be formulated. *Giles*' scope of services did not include the collection or testing of soil or water samples for contamination by hazardous or dangerous materials. A *Phase I Environmental Site Assessment* has been completed and is included under a separate report cover.

This report has been prepared in accordance with generally accepted geotechnical design practice and has been prepared for the exclusive use of the Rite Aid Corporation, Mulvanny Partnership Architects and their agents for specific application to this project. This report has not been prepared to serve as the plans and specifications for actual construction without the appropriate

interpretation by the project architect, structural engineer, and/or civil engineer. This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. Available project information at the time of this report preparation included a preliminary building layout only and topographic site survey. It is recommended that the architect, civil engineer and structural engineer, along with any other design professionals involved in this project, carefully review the assumptions noted in this report regarding the proposed development to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. We recommend that we be allowed to review the project plans and specifications to verify that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted and are incorporated into the design. Reproduction and distribution of this report must be authorized by the client and *Giles.*

3.0 SITE AND PROJECT DESCRIPTION

The subject property is located in the block surrounded by "G", "H", Wishkah and Heron Streets in Aberdeen, Washington as shown on the *Site Vicinity Map* (Figure 1). The subject property currently supports three structures, one to two stories high, which are situated on the northeast, northwest, and southeast corners of the site. The remaining area of the site is currently paved for parking except for one abandoned building lot on the south side of the site which is occupied by a concrete building floor slab. It appears the majority of the site once supported structures. Old pile foundations for these buildings are evident as regularly spaced mounds in the pavement caused by settlement of the site around the piles. In some areas, the edges of demolished buildings are visible where pavement has settled around remaining concrete grade beams. A utility corridor crosses through the center of the site in an east-west direction and includes both overhead and buried utilities. In general, the site is level; however, undulations on the order of several inches to a foot have resulted from settlement of the site around fixed structures. The topographic site survey indicates an average site elevation of about 14.5 feet. The subject site and proposed layout, and locations of our borings are shown on the *Site and Exploration Plan*, Figure 2, located in Appendix B.

The project as proposed consists of constructing an approximately 16,500-square foot retail store which will be located primarily on the western half of the site. The remaining portion of the site will be paved for parking and drives except for an existing restaurant which will remain on the lot at the southeast property corner. It appears that a below-grade truck-loading dock will be located on the southwest corner of the building. Proposed grading plans were not available. However, we understand from conversations with the City Engineer that a minimum finished floor elevation of 15.3 feet will be required due to flood plain concerns. As such, additional fill depths of less than 1 foot have been assumed. For purposes of this report, it was necessary to make several assumptions regarding the structural loadings of the proposed retail store. It was assumed that individual column loads would not exceed 100 kips, and continuous wall loads would not exceed 3 kips per lineal foot. Floor slab loading was assumed to be in the 150 psf range, or less.

4.0 SUBSURFACE CONDITIONS

The site conditions were evaluated for this study on June 23, 1997. Surface, subsurface, and groundwater conditions are discussed below. The subsurface explorations performed for this study consisted of five hollow-stem auger borings which included three borings to approximately a 15-foot depth and two borings extending to approximately 60 feet. The approximate boring locations are indicated on the *Site and Exploration Plan* (Figure 2) enclosed in Appendix B. The subsurface conditions described below have been simplified for ease of report interpretation. Copies of the Boring Logs, and a description of field procedures, are enclosed in Appendix C and should be consulted for a more detailed description of the conditions encountered. Appendix D contains a summary of laboratory testing and soil classification.

Soil Conditions

The explorations performed for this study generally revealed the site to be underlain by a surficial mantle of firm fill approximately 4 to 5 feet thick underlain by soft organic silts of alluvial and marine deltaic origin. Generally, the borings encountered a gravelly sand fill immediately below the asphalt and extending to depths of less than 1 foot. Fill soils below approximately 1 foot in borings B-1 and B-4 primarily consisted of a firm, moist to wet, orange sandy silt with trace gravel interpreted to represent sandstone/siltstone derived fill from local sources. Fill at borings B-3 and B-5, fill consisted of a firm, gravelly sand with traces of wood and brick. Depth of relatively firm fills in borings B-1, B-3, B-4 and B-5 varied between 4 and 5 feet. This layer was not present in boring B-2. At B-2, abundant building debris consisting of glass, porcelain and wood were encountered to approximately 2½ feet and were immediately underlain by soft organic silt. This points to the possibility of a discontinuous or variable-depth fill cap on the site which may not be present below some of the demolished or existing building sites. It is possible that the fill was not placed continuously on the site to cap the soft silts due to the pile-supported nature of the structures. Therefore, fill quality, density, and depth should be expected to vary across the site.

Soils below the fill cap (typically at depths beginning at 4 to 5 feet) consisted of very soft to soft, saturated, greenish-grey silt with variable organic content and trace fine sand and clay content. Organic material retrieved in the samples included what appeared to be sawdust, peat interbeds to 8 inches thick, and wood fragments. A decrease in organic content was noted in samples obtained beginning below 10 to 12 feet. As such, we have interpreted the soils between roughly 5 and 12 feet to have origins as either dredge spoils or original near-surface delta deposits or a combination of both. Below 10 to 12 feet, soils were generally consistent and primarily were a grey silt with trace clay, fine sand and contained a slight amount of fine organics. These soils have their origins in low-energy alluvial or deltaic deposits. The deeper borings, B-1 and B-2, revealed a slight increase in relative density (to medium stiff) between roughly 25 to 35 feet. An increased fine sand fraction in the form of sand stringers was noted in the samples obtained from this depth range. Boring B-1 was

terminated at 59 feet in soft, alluvial silt. Boring B-2 encountered a dense, coarse sand/fine gravel at 52 feet and was terminated at 59 feet in this material. Boring logs obtained from the "F", "G", Wishkah and Market block (one block northeast) indicate the soft, alluvial deposits extend to depths in excess of 100 feet.

Groundwater Conditions

Saturated soil conditions were encountered immediately below the fill cap on the site at an average depth of approximately 5 feet. We anticipate that seepage would be encountered in excavations below this depth. Seepage into the exploration was relatively light given the low permeability of the silt soils, resulting in generally less than 1 foot of accumulated water in the boring at its completion. The phreatic groundwater surface is anticipated to roughly correspond to the level at which saturated conditions were encountered. Groundwater levels would likely have risen to this elevation given sufficient time. Groundwater levels will be influenced by factors including level of adjacent rivers (two to five blocks distant), prevalent tides, precipitation patterns, season, and surrounding site usage.

Soil Vapor Scan

Samples obtained for the *Geotechnical Engineering Exploration and Analysis* were also screened for total volatile organics by means of the headspace method, utilizing a photoionization detector (PID) equipped with an 11.7 eV lamp. This involved placing approximately 4 ounces of sampled soil directly into a Ziplock® plastic bag and allowing the sample to volatilize. Headspace analysis was then performed in each sample with the PID, and the highest value displayed by the instrument was recorded for each sample. This value indicate the total concentration, in arbitrary units, of volatilized organic compounds having ionization potentials less than 11.7 eV. This class of compounds includes numerous constituents of petroleum hydrocarbons. However, the PID is not capable of determining the species of these compounds or their actual concentrations in the soil samples. Consequently, it should be considered merely a rough screening tool that aids in detecting the presence of contaminants. Soil sample readings from the samples which were screened ranged from zero to three, which are not necessarily considered significant concentrations, and generally within the range caused by background sources.

Seismic Conditions

According to the *Seismic Zone Map of the United States* contained in the *1994 Uniform Building Code*, the project site lies within seismic risk zone 3. Based on native soil conditions encountered at the site, the subsurface site conditions are interpreted to correspond to a seismic soil profile type S_4 as defined by Table 16-J of the *1994 Uniform Building Code*. Soil profile type S_4 applies to a profile consisting of more than 40 feet of soft clay. The site soils are not considered to be prone to liquefaction due to the predominant silt and clay fraction of the native soils.

5.0 CONCLUSIONS AND RECOMMENDATIONS

In our opinion, development as proposed is feasible from a geotechnical standpoint, utilizing pile foundation and floor support. In their present condition, the site soils are unsuitable for support of conventional footings and slab-on-grade floor system without resulting in excessive post-construction settlement.

5.1 Site Preparation Recommendations

Preparation of the site subgrades, including fill placement and compaction should precede foundation pile driving. We anticipate that less than 1 foot of fill will be placed above the existing fill soils to achieve a finished floor elevation of 15.3 feet. Based on the uneven and discontinuous nature of the existing asphalt parking areas, we assume that existing pavements will be demolished. We suggest that grinding (pulverizing) of existing asphalt be considered. Pulverized asphalt could be reused for structural fill or new pavement base course material (outside of building areas) provided it meets applicable gradation specifications. We recommend that fill not be placed above intact, existing asphalt as lateral movement of water along the buried asphalt could create construction problems, and saturated near-surface conditions due to groundwater perching above the asphalt. However, it may be prudent to leave existing asphalt in place (outside of building areas) for a construction platform until the final phase of the project.

Several existing buildings will be demolished to accomplish construction of the proposed retail facility. Debris from existing buildings which are demolished should be hauled off-site to an approved dump site. The implications of possible asbestos-containing structures should be considered and are discussed in our *Phase I Environmental Site Assessment* submitted under separate cover. We anticipate that all or some of these structures are founded on timber piles. Once the buildings have been demolished, we recommend that the pile tops be cut off at least 3 feet below finished grade. It is customary to cut the pile butts at an angle rather than flat to reduce the effects of settlement on pavement structures. There is a risk of pavement cracking over cutoff piles due to long-term settlement of the site which will continue to occur. Currently, existing pile foundations for previously demolished buildings are manifested as regularly-spaced hummocks several inches high in the existing paved areas. In some areas, the pavement has been cut out and patched to remove the hummocks. We recommend that these piles be located and cut off per the above recommendations as well. Location of these piles would only be feasible prior to initiation of the site preparation activities. If necessary, a *Giles* representative could assist in the location of these piles.

Utilities should be removed, relocated, or abandoned in-place in accordance with applicable state and local laws. All trench backfilling should be performed in accordance with the recommendations presented in the "Structural Fill" section of this report. The effects of vibration

on adjacent (and particularly on old and/or brittle utilities) due to pile driving activity should not be ignored. We understand that the utilities in the east-west corridor passing through the site will be relocated as they are situated below the proposed building footprint.

Subgrade Preparation

Following site stripping activities, the subgrade within the proposed building areas and new pavement areas should be proofrolled, in the presence of the geotechnical engineer, with appropriate rubber-tire mounted heavy construction equipment or loaded dump truck to detect soft, yielding soils which must be removed to expose a stable subgrade. The subgrades should then be scarified as necessary, moisture conditioned and recompacted to at least 90 percent of the modified Proctor (ASTM:D-1557-91) maximum density. Structural fill may then be placed in lifts with suitable structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

Subgrade Protection

Soils exposed by subgrade preparation operations are anticipated to consist of a highly moisture- and disturbance-sensitive, silty fine sands/sandy silts. These soils could become unstable if disturbed by construction equipment while at elevated moisture contents thereby requiring additional soil removal. We recommend heavy construction equipment be kept off existing soft soils where they are exposed or close to the surface during wet site/weather conditions. Site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface particularly during extended wet weather periods when the presence of excess moisture would render the site soils more prone to disturbance and softening. Ideally, earthwork would be planned for the driest portion of the year, typically July through August or September. During wet site conditions, equipment traffic should not be allowed on exposed subgrade areas. During all phases of construction, equipment traffic should be restricted to limited areas to reduce the extent of disturbance to the subgrade. Traffic areas may require use of rock armoring during wet weather to provide a suitable construction platform and stable subgrade. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations. We recommend that subgrades be "sealed" with a smooth drum roller before wet weather leads to their deterioration. Accumulated water must be removed immediately along with any unstable soil. The severity of construction problems will be dependent, to a large degree, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. The existing asphalt could be left in place outside the building pad, for a temporary construction platform.

Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and or weather conditions. If unsuitable subgrades are encountered or an inefficient existing fill cap is discovered, overexcavation and replacement would be required. The need of overexcavation

should be confirmed through continuous monitoring and testing by a *Giles* representative. Selective drying and recompaction of unsuitable subgrades may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather, generally only occurring during summer months in Western Washington. During winter months, special techniques may be required to achieve stable subgrades and could include 1) overexcavation and replacement with select, granular fill or crushed rock, 2) use of geotextile fabrics, and 3) chemical stabilization with additives such as flyash or Portland cement. We recommend that contingencies for stabilization alternatives be included within the project documents if construction will occur during winter months. We can assist in developing contingency plans once grading plans and construction scheduled have been developed.

Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

5.2 Structural Fill

All structural fill should be placed in accordance with the recommendations presented herein for structural fill. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended.

Structural fill includes any fill material placed under footings, pavements, or other permanent structures or facilities. The use of on-site soils for structural fill should be limited as described in this report. Materials typically used for import structural fill include clean, well-graded sand and gravel ("pit run"); clean sand; various mixtures of gravel; crushed rock; controlled-density fill (CDF); and lean-mix concrete. Recycled concrete derived from crushed parent material is also useful for structural fill provided the material is thoroughly crushed to a size deemed appropriate by the geotechnical engineer (usually less than 2 inches). Structural fill materials should be free of deleterious, organic, or frozen matter and should contain no chemicals that may result in the material being classified as "contaminated".

Structural fill for raising site grades can consist of a combination of "common" and "select granular" material. For this project, however, we recommend that select granular material be specified to provide better performance across a wider range of weather and site conditions. Select granular fill should contain less than 5 percent by weight passing the U.S. No. 200 sieve, based on

the fraction passing the U.S. No. 4 sieve. Select granular fill is recommended for use in wet weather conditions and wet site and subgrade conditions, as discussed in the pertinent sections of this report. Common structural fill should consist of predominantly granular soil, free of organics and deleterious materials, which is compactable to a firm and unyielding condition at the specified compaction levels. Common structural fill should meet the requirements of specification 9-03.14(3), "Common Borrow" in the 1996 WSDOT/APWA *Standard Specifications for Road, Bridge, and Municipal Construction*. Select structural fill should meet the requirements of specification 9-03.14(1) or (2) *Standard Specifications*. However, for wet weather filling or fill exposed to wet site conditions, we recommend that the percent passing the U.S. No. 200 sieve be held below 5.0 percent.

The suitability of soils used for structural fill depends primarily on the gradation and moisture content of the soil when it is placed. As the fines content (that portion passing the U.S. No. 200 sieve) of a soil increases, it becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult or impossible to achieve. Soils containing more than about 5 percent fines by weight (such as the on-site soils) cannot be consistently compacted to the recommended degree when the moisture content is more than about 2 percent above or below optimum. Drying of the soils may only be accomplished during favorable dry weather by methods such as scarifying and windrowing. It should be noted that the placement of structural fill is in many cases weather-dependent and delays due to inclement weather are common even when using select granular fill. When moisture conditioning of the soils is required to increase moisture content of dry-of-optimum soils, it is recommended that the soils be uniformly blended with the added moisture to provide a uniform moisture content throughout the affected soils. Use of tillers or thorough blending using excavators or repeated windrowing with bladed equipment is recommended.

Structural fill should be placed in lifts not exceeding 8 inches in loose thickness. Individual lifts should be compacted such that a minimum density of at least 90 percent of the modified Proctor maximum dry density is achieved. Higher compaction levels should be achieved where called for in the project specifications or the local development standards. The upper 2 feet of subgrade soils below pavement areas, concrete slabs and walkways, and all base course materials should be compacted to 95 percent of the modified Proctor dry density. It is recommended that a geotechnical engineering representative be present during the placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of earthwork may be evaluated as grading progresses.

Reuse of On-Site Soil

Reuse of the native silts is not considered feasible due to moisture and organic content. Near-surface granular fill soils as encountered to depths of about 4 feet in the majority of the borings could be reused for structural fill provided moisture content can be adjusted to within approximately ±2 percent of optimum.

5.3 Underground Utilities, Temporary Excavations and Dewatering

Underground Utilities

We anticipate that excavations for underground utilities may terminate within the soft, compressible organic silts. Manholes, catch basins, and rigid pipes which extend into the soft soils should be underlain by a minimum of 12 inches of imported granular fill soil compacted to a firm and unyielding condition. This granular material could consist of either crushed rock or a clean sand and gravel pit-run. Where groundwater is encountered in the excavations, it should be removed prior to fill placement. A granular fill thickness of 12 inches between pipes and native soils is considered to be the minimum recommended thickness. Additional separation could be required for excessively soft or organic soils. It may be necessary to place a geotextile fabric over the native subgrade soils should they be soft, to provide a separation between the foundation material and subgrade soils. The actual amount of overexcavation will be a function of the pipe burial depth, condition of exposed subgrade soils and type of utility. Utilities should be designed for submerged conditions. Periodic groundwater levels as shallow as 4 feet below existing grades are anticipated.

Utility bedding and cover should be placed in accordance with manufacturer's recommendations and local ordinances. Generally, we recommend that a minimum of 6 inches of bedding material be placed in the trench bottom. Bedding material would be placed above the 12-inch (minimum) utility foundation material recommended above. Bedding material for rigid and thermoplastic pipe should conform with Sections 9-03.15 and 9-03.16, respectively, of the 1996 WSDOT/APWA *Standard Specifications for Road, Bridge, and Municipal Construction.* All excavations should be wide enough to allow for compaction around the haunches of pipes. Otherwise, materials such as controlled density fill or pea gravel could be used to eliminate the compactive effort required around the haunches if allowed in the project specifications.

Backfilling for the remainder of the trenches should be completed utilizing select granular fill. Alternatively, a light-weight fill material such as a flyash/sand mixture may be more appropriate for deep pipe burial to reduce soil loads on utilities and underlying settlement-prone silts. The native silts should not be used for backfill. Compaction of backfill material should be accomplished with soil within $\pm$ 2 percent of their optimum moisture content in order to achieve the minimum compaction levels recommended within this report. In addition, we recommend that a representative of *Giles* be allowed to perform field inspections and density tests on all backfill to verify compliance with the recommendations contained within this report.

Temporary Excavations

Excavation bank stability problems for utility construction should be anticipated due to the presence of near-surface fills, groundwater, and soft site soils. Relatively flat slopes, benching or temporary bracing may be needed. Temporary slope stability is a function of many factors including the presence and abundance of groundwater, the type and density of the various soil strata, the depth

of excavation, surcharge loadings adjacent to the excavation, and the length of time and weather conditions while the excavation remains open. It is exceedingly difficult under these variable circumstances to pre-establish a safe and "maintenance free" temporary excavation slope angle. Therefore, it should be the responsibility of the Contractor to maintain safe slope configurations since the Contractor is continuously at the job site, able to control some of the factors, and able to observe and alter the nature and condition of the cut slopes. For this project, it is anticipated that the majority of shallow excavations can be completed with temporary sloped open cuts or typical "trench box"-type shoring where excavations terminate within approximately the upper 5 feet of the site surface. Deeper excavations will encounter increasing amounts of groundwater seepage which would lead to bank instability problems. All utility excavations should conform to applicable sloping or shoring standards for worker access. It is recommended that utility trenching, installation, and backfilling conform to all applicable federal, state, and local regulations such as WISHA and OSHA for open excavations. Exceptions to typical slope angles should be made for any excavation which extends through the compacted fill into underlying soft soils, any excavations which are open during wet weather, any excavations which encounter groundwater, and excavations encountering variable conditions.

Construction Dewatering

We anticipate that filtered sump pumps combined with a trench box would provide adequate temporary dewatering for relatively shallow excavations. Deeper utility excavations may require more specialized dewatering techniques which should be the responsibility of the contractor. If alternate dewatering techniques or deep utility excavations are contemplated, *Giles* should be consulted for specific design and construction recommendations.

5.4 Settlement Considerations

The presence of thick deposits of compressible alluvial/deltaic silt below the site and near-surface organic silt and peat layers below existing granular fill, introduces a potential for long-term settlement of the site structures and surfacings such as sidewalks and pavement, as well as utilities. Settlement of these deposits is a continuous and ongoing feature which has been documented in the Aberdeen vicinity. Although actual settlement rates were not available, we understand that settlement of the region is measured in inches over decades. Since the most recent paving of the site, 2 to 3 inches of settlement has apparently occurred as evident by the hummocks formed in the pavement by underlying old building foundation piles. This settlement was likely accelerated by the placement of several feet of fill over most of the site as noted in the borings. If grades are raised for this project, the loading due to new fill placement may trigger additional settlements. Our current understanding of the project is that a new finished floor elevation will be less than 1 foot above average existing grades and, therefore, the effect on current rates of settlement should be minor.

Due to documented, continued settlement of the site, we recommend that utilities and pavements be designed with sufficient sloping grades to maintain flow gradients to tolerate long-term differential settlement of at least several inches. Utility penetrations through pile-supported sections of the structure, such as the floor slab or pile caps, should be constructed with flexible couplings or other details to accommodate long-term settlement of the estimated magnitudes. Utilities beneath the slab may require structural support by hanging from the structure, if the utilities cannot otherwise be protected from damage due to settlement.

Construction on sites with settlement-prone soils is inherently subject to higher maintenance during the life of the structure, as compared to construction on site with more suitable soil conditions. Although the building will be supported on pile foundations, other site improvements may experience adverse effects as a result of settlement over time. Asphalt and concrete paving, pedestrian sidewalks, luminaries, and other accessory improvements are likely to experience significant total and differential settlements, and to a much larger degree than the building itself. The result can be poor drainage of sidewalks and parking lots, and settlement of sidewalks away from the building perimeter that result in raised edges where pile-supported structures meet structures with shallow foundations. In addition, settlement sensitive utilities can develop diminished or even reversed drainage gradients, and possibly separations near or beneath the building. The recommendations presented in this report are intended to mitigate these possibilities to the extent that it is economically feasible to do so. To remove all possibility of these settlement effects is not considered cost effective and, therefore, generally higher maintenance requirements for the building and site improvements should be expected. Walks immediately adjacent to the building may be pile-supported to eliminate differential settlements at these interfaces, particularly at store entrances. If a below-grade truck loading dock is considered, we recommend that this structure be pile-supported as well. An overexcavation and replacement program would be ineffective at eliminating settlement of this structure and would be difficult due to high groundwater levels.

5.5 Pile Foundations

Due to anticipated long-term settlement of the site surface, shallow foundations cannot be used for support of the proposed structure. For a lightly loaded building as proposed, low-capacity driven timbers piles provide a cost-effective solution. For this reason, timber piles have become the foundation option of choice in Aberdeen, and the majority of existing buildings and new projects utilize a network of driven timber piles for foundation and floor support. Auger-cast piles would provide a feasible alternate pile type. If design information is required for auger-cast piles, we are available to provide this information.

Piles would gain their vertical compressive capacity mainly from side friction between the pile and the native soils. As a result, some settlement of the pile foundations and structure will occur due to settlement of the deposit as a whole, and pile creep. However, pile capacities have been selected which account for downdrag forces and produce tolerable settlement over the predicted

lifespan of the structure. All piles should be driven a minimum of depth of 40 feet below the existing ground surface. We recommend a maximum allowable vertical compressive capacity for an 8-inch tip diameter, timber pile of 15 tons. For a 10-inch tip diameter timber pile, we recommend 20 tons allowable capacity. The above allowable compressive capacities are based on minimum pile spacings of 3D, where D is the average diameter of the piles. If pile groups of more than nine piles per cap are planned, we should be consulted for reductions of group capacity.

For fixed-head timber piles, we recommend designing for an ultimate lateral capacity of 5 tons per pile for a limiting deflection of approximately 1 inch. The depth to zero deflection has been calculated to be approximately 14 feet beneath the pile cap. This lateral capacity assumes that structural stiffness of the pile cap/floor network prevents rotation of the pile top. We recommend the design criteria be utilized with the actual anticipated loads to determine the deflections and maximum moments. These calculated values should be analyzed with respect to the tolerable deflections of the structure and the strength of the pile material (with a suitable safety factor) to resist the moments and shear. It should be noted that the above formulations do not contain a factor of safety. Further, a number of circumstances can alter the actual deflection values obtained in the field as compared to the calculated values as previously mentioned. Pile group action can have an influence. At the minimum pile spacing, the lateral capacity of each pile would be less than that calculated for an individual pile. The effects of group action tend to disappear when piles are spaced eight or more diameters, center-to-center, in the direction of the load and at three or more diameters normal to the direction of the load. Piles subjected to repetitive, cyclic loading may result in deflection magnitudes up to twice that of a pile subjected to a static load.

Obstructions will likely be encountered during pile installation. Nested buried logs are frequently encountered in the Aberdeen area. Where obstructions to pile driving or drilling are encountered close to the surface, typically, a backhoe or trackhoe can excavate and remove the obstruction. Alternatively, spudding or predrilling may be required to advance through debris or buried logs.

We recommend that any fills of more than 1 foot placed to raise building pad grades be completed at least 90 days prior to pile installation. As the upper compressible soils and fill materials consolidate and move downward with respect to the relatively immobile pile, the soils surrounding the pile impart shear stresses acting downward on the surface of the pile. The result is a real additional load, in addition to the structural load, known as "negative skin friction", or "downdrag". The allowable capacities of the piles presented previously in this report have been reduced to account for the effects of long-term "downdrag".

Uplift pile capacity develops as a result of side friction between the pile and adjacent soils. Uplift load resistance can be provided by side friction from that portion of the pile embedded in the medium-dense to dense native sands and soft to stiff cohesive site soils. We recommend utilizing ultimate uplift capacities of 8 to 10 tons for the recommended 8- and 10-inch diameter timber piles.

Uplift capacity for timber piles will likely be limited by structural capacity of the piles and pile cap connection. We recommend using a safety factor of 1.5 or more depending on loading conditions. The structural adequacy of the pile section should be verified to resist the uplift loads.

All timber piles should be pressure-treated conforming to standard specifications outlined in ASTM:D-25. We recommend that timber piles be driven with a diesel, air or steam hammer, having a rated energy of not less than 15,000 foot pounds and not more than 26,000 foot pounds. Due to the low density and consistency of the native soils immediately below existing ground surface, use of a diesel hammer may not be appropriate as continued effective operation of the hammer may be difficult. Use of a dynamic pile driving formula would not be feasible at this site, although "retaps" will be recommended during construction.

We recommend a pile load test program prior to production pile installation. Test pile installation data as well as load test data would be reviewed to develop the most cost-effective combination of pile length and capacity. We recommend consideration of a combination of compressive, lateral, and tensile (uplift) load tests at a minimum of two locations on the site. Pile testing should be completed prior to ordering production piles, so that adjustments to pile length can be made, if necessary. Pile load tests should be performed after a minimum of 24 hours has lapsed following pile driving to allow the piles to "set up".

Compressive, lateral and tensile load testing should be completed in accordance with procedures outlined in ASTM specifications D-1143, D-3966, and D-3689, respectively. We recommend that *Giles* be retained to observe and monitor test pile installation, supervise the load tests, and analyze the data collected. A report would be issued following completion of the testing describing and documenting test pile installation methods and results, load test procedures and results, and other information as appropriate.

Table 1, below, presents our recommended test pile program. Test and reaction piles should be installed in the same manner as the production piles. Provided the test and reaction piles are installed to be to the recommended depths, uplift capacity tests could be performed on the reaction piles.

TABLE 1
Recommended Test Pile Program

Compressive:	Two to evaluate bearing capacity at 40 foot depth
Lateral:	One at any test location if required by structural engineer
Uplift:	One at 40 feet depth if required by structural engineer

5.6 Floor Slab Design Parameters

General Design

The soft, compressible organic silt will experience long-term settlement after development and conventional slab-on-grade floors will be adversely affected. Therefore, we recommend that floor slabs be constructed as reinforced structural slabs connected to pile-supported grade beams. Two-way slabs supported on a pile grid are not recommended as differential settlement, resulting from variable friction pile performance (due to variable subsurface conditions) would be magnified by this system.

Moisture Protection: To minimize moisture infiltration, we recommend that slab-on-grade floors be underlain by the following layers (top to bottom):

- Curing Course - A 2-inch thick layer of clean sand to allow proper curing of the concrete slab and to protect the vapor barrier;
- Vapor Barrier - A layer of plastic sheeting (such as visqueen or Crosstuff) to prevent the upward migration of ground moisture vapors;
- Capillary Break - A 4-inch thick layer of pea gravel or washed rock to retard the upward wicking of groundwater.

5.7 Drainage Considerations

We recommend that the building be provided with a perimeter drain system to collect free water. Drains, with clean outs, should consist of at least 4-inch diameter, perforated, PVC pipe fully enveloped in pea gravel or washed rock and be placed at the footing subgrade elevation around the outside perimeter of the foundation. The pea gravel envelope should be wrapped on the top and sides with a non-woven geotextile fabric to prevent the migration of fine grained soils into the drain rock. All drainage systems should be sloped to drain by gravity to a storm sewer or other suitable discharge location. Water from downspouts and surface water should be independently collected and routed to a storm sewer. This water must not be allowed to enter the footing drain system.

Final exterior grades should promote free and positive drainage from the building areas at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building area. It is recommended that a gradient of at least three percent for a minimum distance of ten feet from the building perimeter be provided. In paved locations, a minimum gradient of one percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structure. A larger gradient slope may be considered desirable to mitigate the possible gradient reversal caused by differential settlements of 1 to 2 inches.

Below-grade truck loading docks will require special drainage considerations. These should be designed assuming periodic groundwater levels as shallow as 3 to 4 feet below existing grades.

5.8 Pavement Design Parameters

It must be recognized that pavement design is a compromise between high initial cost and little maintenance on one side and low initial cost coupled with the need for periodic repairs. As a result, the owner will need to take part in the development of an appropriate pavement section. Critical features which govern the durability of the surface include the stability of the subgrade soils, the traffic volume, and the frequency of use by heavy vehicles.

Subgrade

Pavement subgrades should be adequately prepared per the preceding recommendations of this report to expose a subgrade capable of being compacted to the recommended level. Subgrades should be evaluated in the presence of a qualified geotechnical representative to detect any soft and yielding areas which may need to be overexcavated. Subgrade evaluation should include proofrolling with a fully loaded dump truck in the presence of a qualified geotechnical representative.

Compaction of subgrades and fills below pavement areas should conform to WSDOT *Standard Specification* 2-03.3(1)C Method B except that the reference maximum dry density should be based upon the modified Proctor test (ASTM:D-1557 or AASHTO T-180). This specification requires the upper 2 feet of subgrade soils to be compacted to 95 percent and all soils below to be compacted to 90 percent. Subgrade soils should have an equivalent CBR value of at least 10 to 15. We recommend that a qualified geotechnical representative be on site during site grading so that a representative number of density tests may be performed to verify compliance with the project specifications. We anticipate that poor subgrade conditions could exist at variable locations. Boring B-2 encountered only a thin veneer of fill over building debris with soft organic silt beginning at approximately 3 feet. If a stable subgrade cannot be achieved, overexcavation and replacement with suitable fill may be required.

Asphaltic Concrete Pavement

Specifications for pavements and crushed base/top course should conform to those presented in the 1996 Washington State Department of Transportation, *Standard Specifications for Road, Bridge, and Municipal Construction*. In lieu of crushed gravel base/top course, asphalt treated base (ATB) can be substituted. The ATB would provide a durable wearing surface if the pavement subgrade areas will be exposed to construction traffic prior to final paving with Class B asphalt. Some degradation of the ATB should be expected if it is subjected to concentrated construction traffic. Repair of degraded areas would be required prior to final Class B paving. Pavement section

recommendations are based on an assumed California Bearing Ratio (CBR) value of 10 to 15 for the silty site soils. All fill used for the project should be capable of meeting or exceeding this value in its compacted state. We recommend a pavement section consisting of 3 inches of Class B asphalt for standard traffic lanes, 2½ inches for parking, and 4 inches for truck lanes. We recommend a minimum of 6 inches of granular, crushed base course. Base course should meet the requirements for crushed surfacing in Section 9-03.9(3) of the *Standard Specifications*. Asphalt concrete pavement should be Class B as addressed in Sections 5-04 and 9-03.8 of the Standard Specifications. Asphalt-Treated Base (ATB) may be substituted for the granular base and is recommended during wet site conditions to seal the site surface prior to final paving operations. A 3-inch ATB thickness is recommended. Sections 4-06.3, 5-04, and 9-03.6 of the WSDOT *Standard Specifications* apply to the use of ATB.

Recommended pavement sections are based on assumed traffic loadings. Pavement design recommendations assume proper drainage and construction monitoring and are based on AASHTO Design parameters are for a 20-year design period. However, continual flexible pavement maintenance along with a major rehabilitation after about ±8 to 10 years should be expected to obtain a 20-year service life. Due to the presence of fill, maintenance may be required as soon as five years following construction to repair low areas resulting from settlement.

5.10 Conclusion

The conclusions and recommendations presented in this report are based, in part, on the explorations accomplished for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate the recommendations. Project plans were in the preliminary stage at the time of this report preparation. Therefore, we recommend that we be provided the opportunity for general review of the project plans and specifications when they become available in order to confirm that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design package.

The integrity of earthwork, structural fill placement, and foundation and pavement performance depend greatly on proper site preparation and construction procedures. We recommend that *Giles* be retained to provide geotechnical engineering services during the earthwork and foundation construction phases of the project. If variations in the subsurface conditions are observed at that time, we would be able to provide additional geotechnical engineering recommendations to the contractor and design team in a timely manner as the project construction progresses.

We appreciate the opportunity to have been of service on this project and would be pleased to discuss the contents of this report or other aspects of this project with you at your convenience. If you have any questions, please do not hesitate to call.

APPENDIX A

GENERAL COMMENTS

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practice in the field of geotechnical engineering. No other warranty is either expressed or implied.

APPENDIX B

FIGURES

The Site and Exploration Plan contained herein was prepared based upon information supplied by Giles' client, or others, along with Giles' field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation. Other figures are presented to aid in interpretation of the report. Sources of the figures are as indicated.

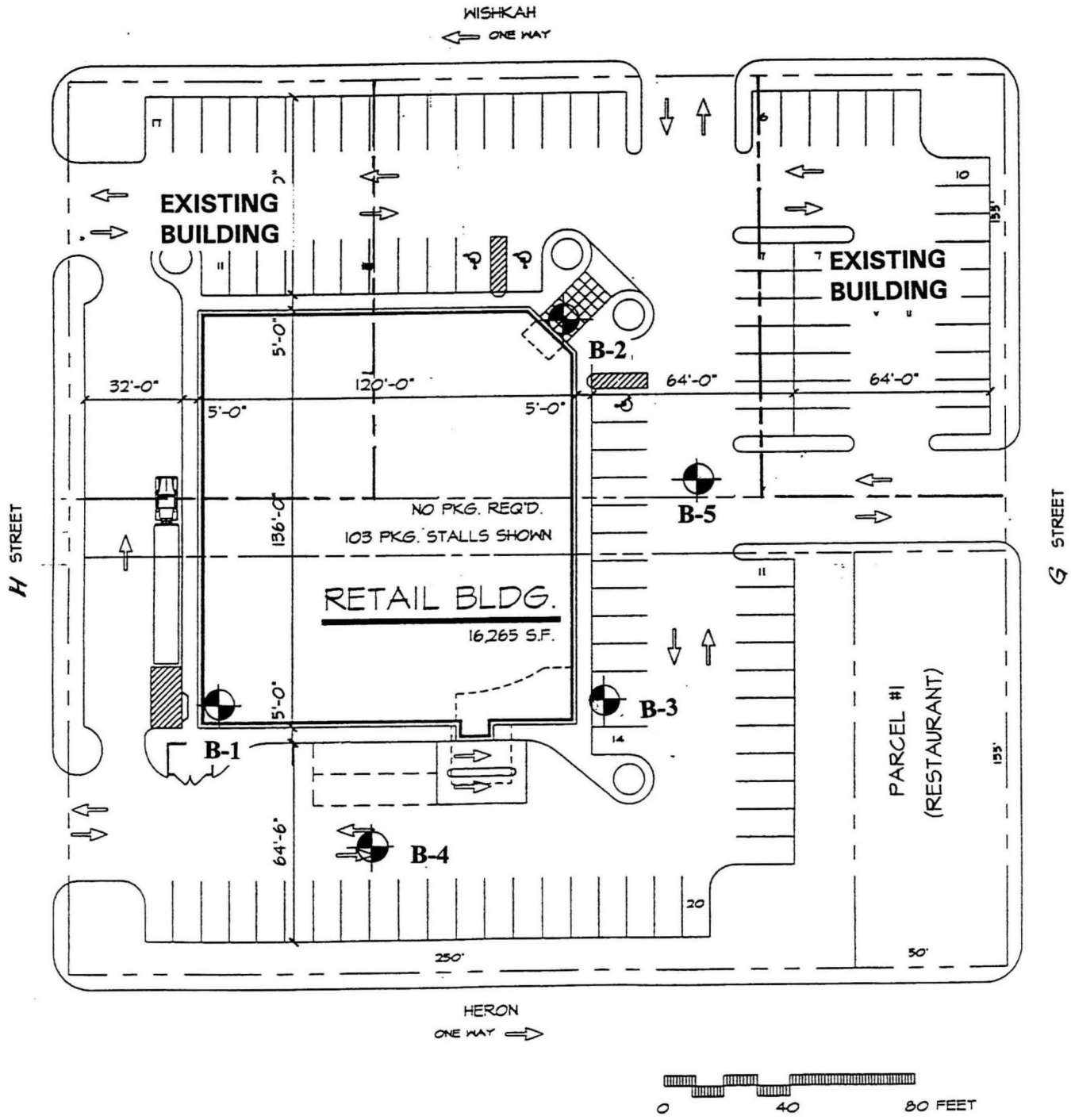
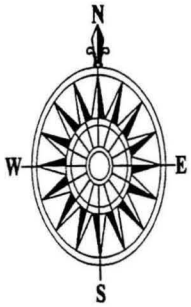


FIGURE 2 - SITE & EXPLORATION PLAN
PROPOSED ABERDEEN RETAIL PROJECT
"G", "H", WISHKA & HERON BLOCK
ABERDEEN, WASHINGTON
PROJECT NO. 6G-9706008



LEGEND

B-1 APPROXIMATE BORING LOCATION & NUMBER

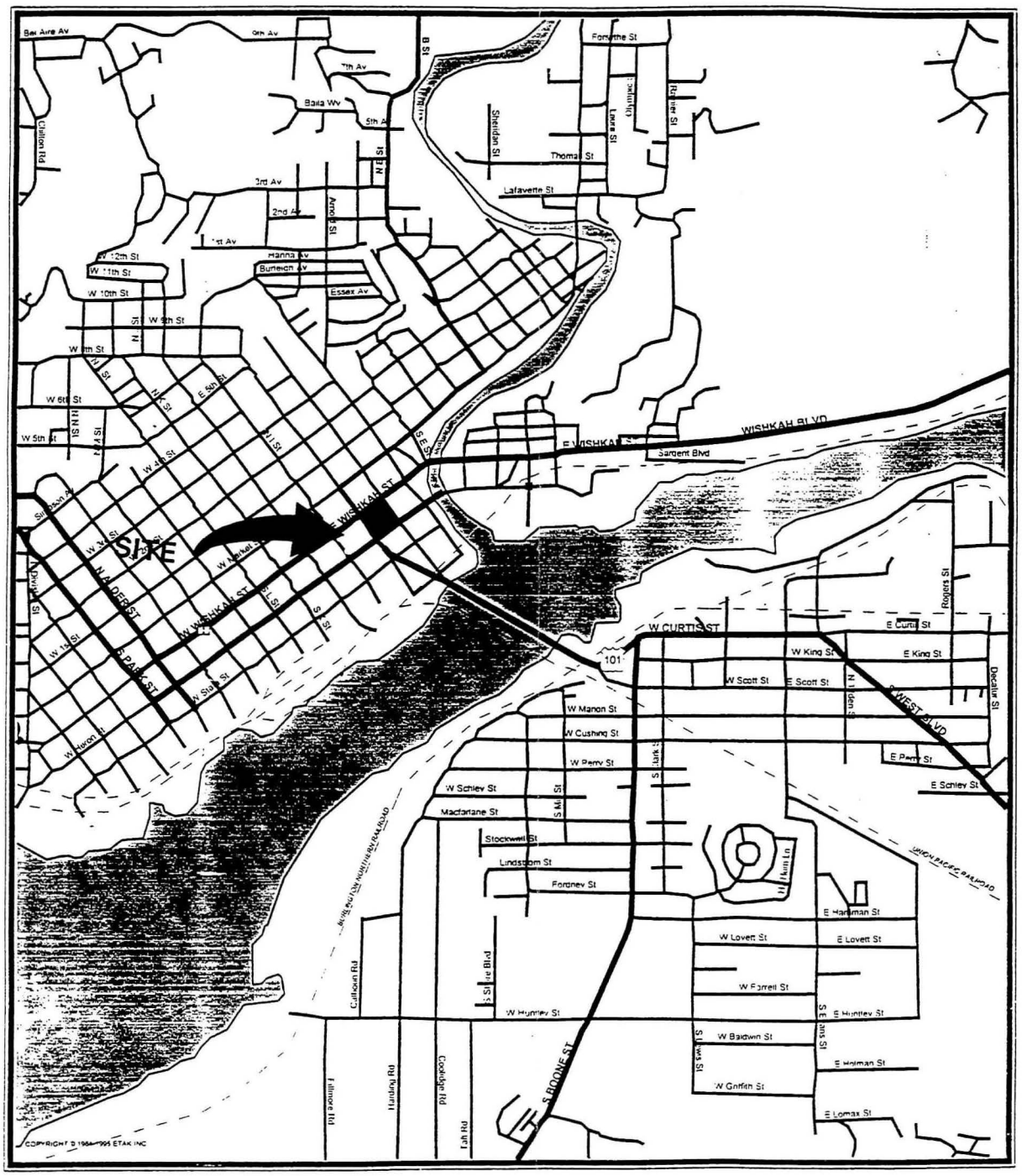


FIGURE 1 - SITE VICINITY MAP
PROPOSED ABERDEEN RETAIL PROJECT
"G", "H", WISHKA & HERON BLOCK
ABERDEEN, WASHINGTON
PROJECT NO. 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, GEOENVIRONMENTAL
AND CONSTRUCTION MATERIALS CONSULTANTS

APPENDIX C

FIELD PROCEDURES AND EXPLORATION LOGS

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D 420 entitled "Standard Guide for Sampling Soil and Rock" and/or other relevant specifications. Soil samples were preserved and transported to Giles' laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "standard practice for preserving and transporting soil samples." Brief descriptions of the sampling, testing and field procedures commonly performed by Giles are provided herein.

The Exploration Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific exploration locations on the date that the exploration was performed. Subsurface conditions may differ between exploration locations and within areas of the site that were not explored. The subsurface conditions may also change at the exploration locations over the passage of time.

GENERAL FIELD PROCEDURES

Test Boring Methods

Soil borings were advanced with hollow-stem, continuous-flight augers using a truck-mounted drill rig operated by an independent firm working under subcontract to Giles. An experienced geotechnical engineer or engineering geologist from Giles continuously observed the boring, logged the subsurface conditions, obtained representative soil samples, and transported the samples to Giles laboratory for further classification and testing as necessary.

Test Boring Locations/Elevations

The test borings were located on-site based on the existing site features and/or apparent property lines. Borings were located in the field by measuring from existing site features by using a handheld tape and/or pacing distances. Elevations were estimated by use of a hand level to existing site features or by interpolating between elevation contours on site plans provided for our use. The locations and elevations should only be considered as accurate as the methods used for field measurements.

Test Boring Logs

The Boring Logs describe the various types of soils encountered in the boring, based primarily on visual interpretations made in the field and supported by subsequent laboratory testing where appropriate. The logs indicate the approximate depth of the contacts between different soil types, although these contacts may be gradational or undulating. Where a change in soil type occurred between sampling intervals, the depth of the contact was inferred. In addition, the log indicates the Standard Penetration Test (SPT) resistance at each sample location and any laboratory tests performed on the soil samples.

Water Level Measurement

The water levels reported on the Boring Logs represent the depth of water encountered during drilling and/or after the drilling tools were removed from the borehole. Groundwater conditions are typically estimated based on measured water levels, wetted height of drilling sample rods when removed from the hole and/or on sample or auger-cutting soil moisture. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation within cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must, therefore, be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfill with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by Giles's client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) - (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12 inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or "N-Value." The N-value is representative of the soils' resistance to penetration. The N-value is, therefore, an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Bulk Sample (BS)

A relatively large volume of soil is collected with a shovel or other manually-operated tool. The sample is typically transported to Giles' materials laboratory in a sealed bag or bucket.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-Values) are reported on the Test Boring Logs. Explanation of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes."

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-1	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.8 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
2" AC over brown gravelly SAND little silt									
Firm, moist, tan, orange and gray, fine sandy SILT (Weathered SANDSTONE Fill)		1-SS	13				38	0.4	
	▽ 5								
Very soft, saturated, gray SILT with little clay; abundant fine organics and possible sawdust (Organic SILT)		2-SS	2		0.0		119	0.0	
	10								
Very soft, wet to saturated, gray SILT with trace fine sand and sand stringers, clay and fine organics		3-SS	1		0.0		78	0.0	
	15								
		4-SS	8		.25		41	0.0	
	20								
		5-SS	2				68	0.2	
	25								
									Firmer drilling
Grades to medium stiff, wet, gray SILT with little fine sand, sand stringers, trace clay and fine organics		6-SS	5		0.3-0.5		28	0.2	

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING 5.0 ft*	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-1	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.8 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Medium stiff, SILT (as above)									
		7-SS	7		.75-1.25		50	0.4	
	35								
		8-SS	5		.25-0.3		60	1.5	
Grades to soft, wet to saturated, gray SILT with trace fine sand, clay, shell fragments	40								
		9-SS	4		0.4-0.6		46	0.0	
	45								
Grades to very soft		10-SS	2		0.4-0.7		54	3.0	
	50								
		11-SS	3		0.5-0.7		60		
	55								
		12-SS	3		0.0		71	0.3	
Boring terminated at 59.0 feet									

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING5.0 ft*	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-2	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.2 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
2" AC over brown, gravelly SAND with silt									
Building debris (glass, porcelain, wood)									
1 foot sawdust over SILT with sawdust	▽	1-SS	2				138	0.3	
Silty drill cuttings with abundant wood	5								
Grades to very soft, saturated, gray SILT with trace fine sand, clay, organics (Organic silt)		2-SS	2				138	1.0	
	10	3-ST							LL = 75 PI = 32
Grades to very soft, wet to saturated, gray SILT with trace clay, fine sand stringers, trace organics		4-SS	1		0.0		138	0.4	
	15								
		5-SS	2		0.0		247	1.6	
	20								
		6-SS	4		0.2		68	0.8	
	25								
Grades to medium stiff to stiff, wet, gray SILT with little fine sand		7-SS	10		0.75		56	1.0	

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING 3.0 ft*	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-2	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.2 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Medium stiff SILT (as above)									
		8-SS							
	35								
		9-SS	2		0.2		59	1.3	
Grades to very soft / soft, saturated, SILT with trace clay, fine sand, fine organics	40								
		10-SS	3		0.4		12	1.0	
	45								
		11-SS	3		0.75-1.0		50	2.0	
	50								
Dense, saturated, coarse SAND / fine GRAVEL		12-SS	46				140	1.4	
	55								
		13-SS	39				8	1.8	

Boring terminated at 59.0 feet

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING3.0 ft*	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-3	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.1 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
2" AC over crushed rock									
Firm, wet, Gravel with little wood and brick (Fill)		1-SS	12				19	1.6	
	5								
Soft, wet/saturated, brown organic SILT									
		2-SS	1				138	1.0	
	10								
		3-ST	15/5						Consolidation test LL=84 PI=38
Soft/very soft, wet/saturated, gray SILT with scattered organics	15	4-SS	2				55 96	1.2	

Boring terminated at 16.5 feet

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING: N/A	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-4	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 13.9 ft	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
1.5" AC									
Brown gravelly SAND		1-SS	22				25	1.8	
Orange, sandy SILT/silty SAND (Sandstone Fill)									
Cutting generally moist		2-SS	9				21	1.4	
Moist, organic, fine sandy SILT trace fine gravel (Sandstone Fill)									
	5								
Saturated, gray, organic SILT									
		3-SS	1				59	1.0	
Highly organic SILT/silty PEAT									
Gray-green, SILT, little clay, little organics									
	10								
		4-SS	1				152	1.0	
Gray SILT, trace organics									

Boring terminated at 14.0 ft

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING: N/A	* Assumed based on soil moisture; Seepage not evident
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-5	PROJECT: Aberdeen Retail Project
SURFACE ELEVATION: 14.2	PROJECT LOCATION: G & H ; Heron & Wishka Streets
COMPLETION DATE: 6/23/97	Aberdeen,
FIELD REPRESENTATIVE: MMR	PROJECT NUMBER: 6G-9706008

GILES ENGINEERING ASSOCIATES, INC.
Dallas Madison Seattle
Milwaukee Los Angeles
Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
3.5" AC									
Moist, brown, silty gravelly SAND, some silt, trace wood, brick (Fill)		1-SS	43				7	2.8	
		2-SS	12				62	1.8	
Saturated, silty dark brown organic layer (Relic topsoil)	5								
Cuttings grade to green/gray SILT									
Very soft, saturated, gray SILT with little clay trace organics		3-SS	1				238	1.5	
8 inches PEAT interbed (8.2-9.0 ft.) (Fibrous)	10								
Very soft, saturated, gray, SILT trace/little clay, scattered organic throughout		4-SS	0				93	1.6	
Boring terminated at 14.0 ft									

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING:	* Assumed based on soil moisture; Seepage not evident.
▽	WATER LEVEL AFTER REMOVAL: N/A	
	CAVE DEPTH AFTER REMOVAL: N/A	
▼	WATER LEVEL AFTER HOURS: N/A	
	CAVE DEPTH AFTER HOURS: N/A	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

GENERAL NOTES

GILES ENGINEERING ASSOCIATES, INC.

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 in and larger	
Cobbles:	3 in to 8 in	
Gravel:	coarse-	3/4 to 3 in
	fine-	No. 4 (4.76mm) to 3/4 in
Sand:	coarse-	No. 4 (4.76mm) to No. 10 (2.0mm)
	medium-	No.10 (2.0mm) to No. 40 (0.42mm)
	fine-	No. 40 (0.42mm) to No. 200 (0.074mm)
Silt:	No. 200 (0.074mm) and smaller (Non-plastic)	
Clay:	No. 200 (0.074mm) and smaller (Plastic)	

SOIL PROPERTY SYMBOLS

- Dd: Dry Density (pcf)
- LL: Liquid Limit, percent
- PL: Plastic Limit, percent
- PI: Plasticity Index (LL-PL)
- LOI: Loss on Ignition, percent
- Gs: Specific Gravity
- K: Coefficient of Permeability
- w: Moisture content, percent
- qp: Calibrated Penetrometer Resistance, tsf
- qs: Vane-Shear Strength, tsf
- qu: Unconfined Compressive Strength, tsf
- qc: Static Cone Penetrometer Resistance. Correlated to Unconfined Compressive Strength, tsf
- PID: Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-units (BDL=Below Detection Limits)
- N': Penetration Resistance per 6 inch interval, or fraction thereof, for a standard 2 inch O.D. (1-3/8 inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in genral accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N' values where plus sign is shown.
- Nc: Penetration Resistance per 1-3/4 inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
- Nr': Penetration Resistance per 6 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches perASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

- SS: Split-Spoon
- ST: Shelby Tube - 3" O.D. (except where noted)
- CS: 3" O.D. California Ring Sampler
- DC: Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
- AU: Auger Sample
- DB: Diamond Bit
- CB: Carbide Bit
- WS: Wash Sample
- RB: Rock-Roller Bit
- BS: Bulk Sample

Note: Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated.

OIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

OMPARATIVE ONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
ery Soft	0-2	0 - 0.25
oft	3-4	0.25 - 0.50
edium Stiff	5-8	0.50 - 1.00
iff	9-15	1.00 - 2.00
ery Stiff	16-30	2.00 - 4.00
ard	31+	4.00+

EGREE OF ASTICITY	PI
one to Slight	0-4
ght	5-10
edium	11-30
gh to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	PI
Low	0-15
Medium	15-25
High	25+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0-4
Loose	5-10
Firm	11-30
Dense	31-50
Very Dense	51+

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APPENDIX D

LABORATORY TESTING AND SOIL CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory procedures, testing results, and soil classifications are included herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in GEA's analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of certain Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNU (Manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM:D-2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Particle Size Distribution (ASTM:D-421, D-422, and D-1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis", which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis", which is based on the sedimentation of particles suspended in water.

Atterberg Limits (ASTM:D-423 and D-424)

Atterberg limits are used primarily for classification and indexing of cohesive soils. The liquid and plastic limits are two of the five Atterberg limits and are defined as the moisture content of a cohesive soil at arbitrarily established limits for liquid and plastic behavior, respectively. The results of Atterberg limits are presented on a plasticity chart in this appendix where the plastic index (liquid limit minus plastic limit) is related to the liquid limit.

Consolidation Test (ASTM: D-2435)

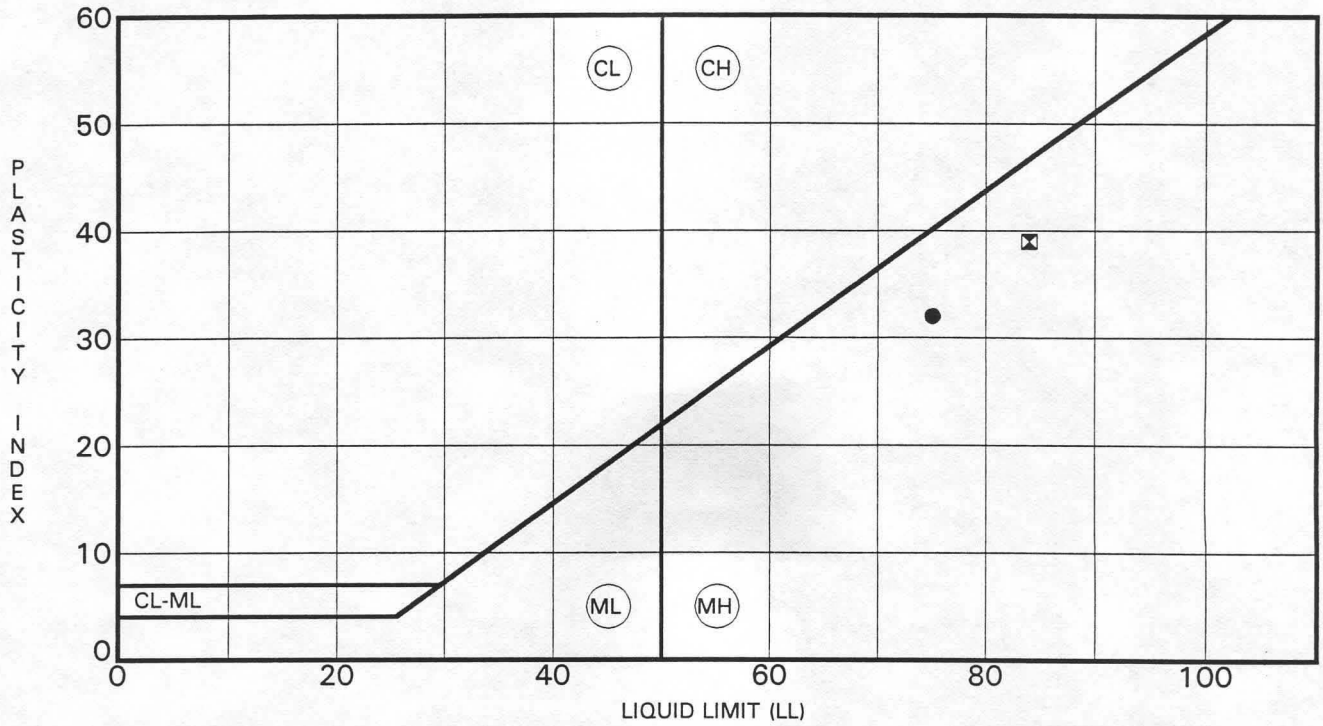
A one-dimensional consolidation test was performed in general accordance with ASTM:D-2435 on a selected sample of the site soils to provide data for developing settlement estimates. The undisturbed soil sample was carefully trimmed and fit into a rigid ring. Porous stones were placed on both the top and bottom of the sample to allow drainage. Vertical loads were then applied to the sample incrementally in such a way that the sample was allowed to consolidate under each load increment. The rebound of the sample during unloading was also measured. The results of the consolidation test are presented in this appendix as a plot of percent consolidation versus applied load (stress).

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM:D-2488-75). The classifications are reported on the Boring Logs.

Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Pit Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs are provided on the appendix enclosure entitled "General Notes".



Specimen Identification			LL	PL	PI	Fines	Classification
●	B-2	11.0	75	43	32		Gray, organic SILT w/ clay
⊠	B-3	14.4	84	45	39		Gray, organic SILT w/ clay

PROJECT Aberdeen Retail Project - G & H ; Heron & Wishka Streets Aberdeen,

JOB NO. 6G-9706008

DATE 7/23/97

ATTERBERG LIMITS' RESULTS

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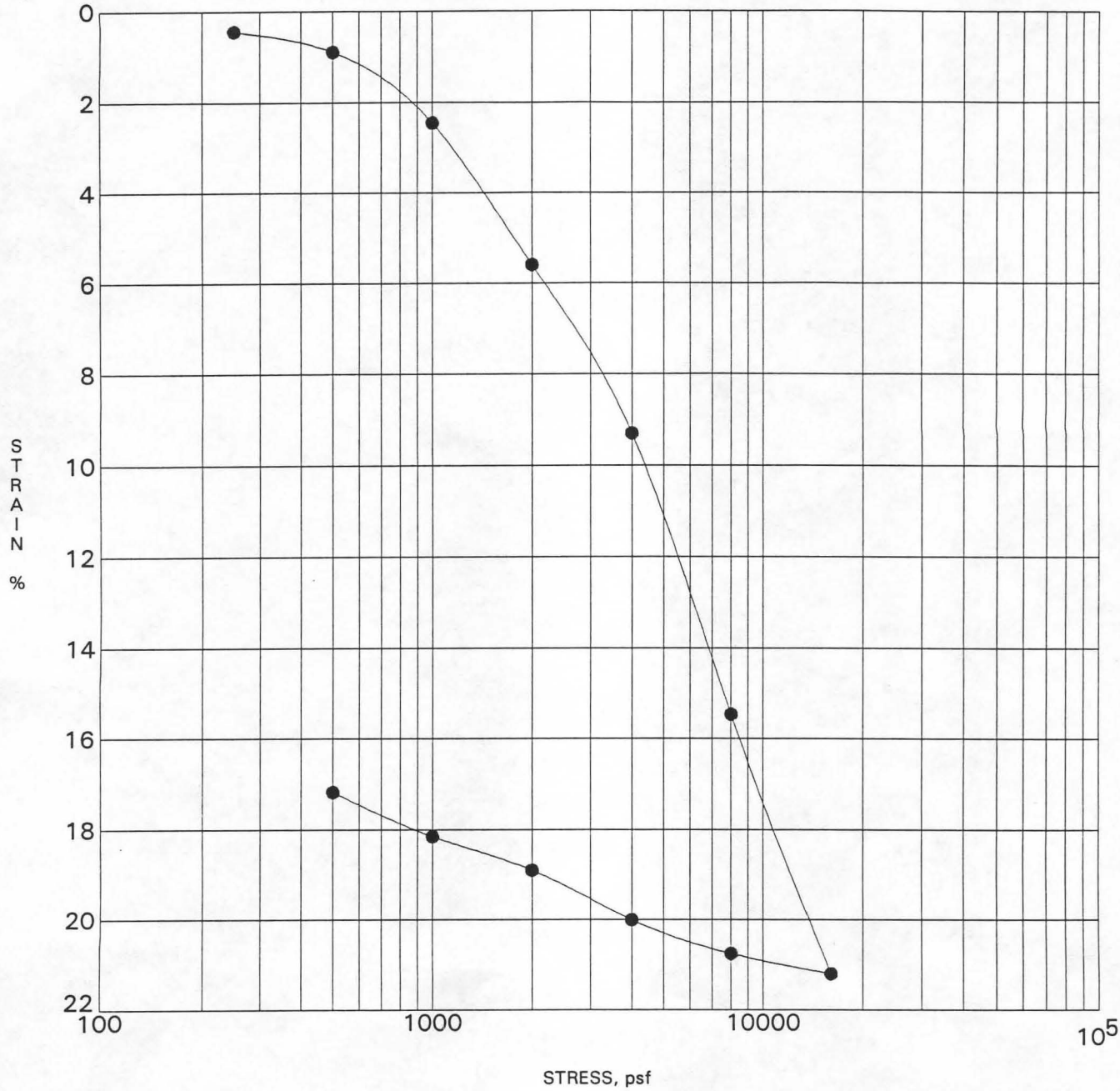


Specimen Identification			Classification	DD	MC%
●	B-3	14.5	Gray, organic SILT	51	55

PROJECT Aberdeen Retail Project - G & H ; Heron & Wishka Streets Aberdeen,

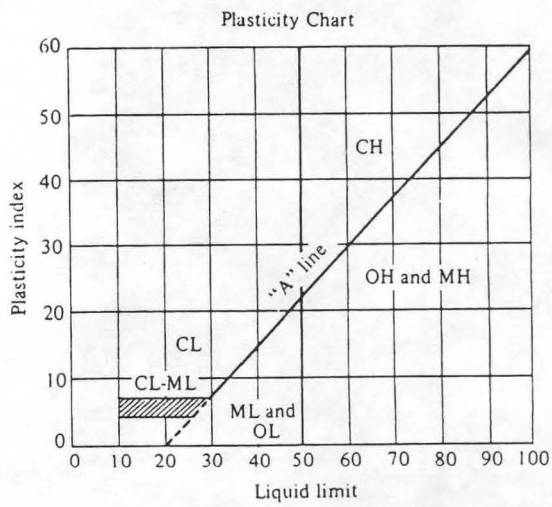
JOB NO. 6G-9706008
DATE 7/17/97

CONSOLIDATION TEST

giles engineering assoc., inc
lynnwood, wa

UNIFIED SOIL CLASSIFICATION SYSTEM. (ASTM D-2487)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent GW, GP, SW, SP; More than 12 per cent GM, GC, SM, SC; 5 to 12 per cent *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a] d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d / u	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			



[a]Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

[b]Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

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