Appendix C:

Geotechnical Engineering Study, Wood Trails, June 2004

GEOTECHNICAL ENGINEERING STUDY
WOOD TRAILS
RESIDENTIAL DEVELOPMENT
WOODINVILLE, WASHINGTON

E-10683

June 9, 2004

PREPARED FOR
PHOENIX DEVELOPMENT, INC

RAYMOND A. COGLAS
STATE OF WASHINGTON
36018
REGISTERED
PROFESSIONAL ENGINEER

6/9/04

EXPIRES 7/29/2004

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IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should not be used*:

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership, or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their geotechnical consultants through the construction stage*, to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems. *No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. *These logs should not under any circumstances be redrawn* for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, *give contractors ready access to the complete geotechnical engineering report* prepared or authorized for their use. Those who do not provide such access may proceed under the *mistaken* impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are *not* exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

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0788/3M

Earth Consultants, Inc.

Geotechnical Engineers, Geologists & Environmental Scientists
Construction Testing & ICBO / WABO Inspection Services

Established 1975

June 9, 2004

E-10683

Phoenix Development, Inc.
P.O. Box 3167
Lynnwood Washington 98046

Attention: Ms. Loree Quade

Dear Ms. Quade:

Earth Consultants, Inc. (ECI) is pleased to submit this report titled "Geotechnical Engineering Study, Wood Trails Residential Development, Woodinville, Washington". This report presents the results our field exploration, selective laboratory tests, and engineering analyses. The purpose and scope of this study was outlined in our February 12, 2004 proposal.

In our opinion, the planned residential development is feasible from a geotechnical standpoint.

Based on the results of our study, there are two primary geotechnical considerations relevant to the proposed residential development. Due to the presence of steep slope areas, appropriate building setback criteria will need to be established for the development. The City of Woodinville Development Standards, (Section 21.24.300 Steep Slope Hazard Areas) provide criteria for establishing setbacks from slopes 40 percent or steeper. Additionally, the planned detention pond construction will require excavations on the order of twenty (20) feet at some locations. A compacted fill berm and tiered rockeries at some locations will also be necessary, according to preliminary plans, to complete the detention pond construction. These geotechnical considerations and other pertinent geotechnical recommendations are discussed in greater detail in this report.

We appreciate this opportunity to be of continued service to you. If you have any questions, or need further assistance, please call.

Respectfully submitted,

EARTH CONSULTANTS, INC.

Raymond A. Coglas, P.E.
Manager of Geotechnical Services

RAC/csm

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WOODINVILLE, WASHINGTON

E-10683

INTRODUCTION

General

This report presents the results of the geotechnical engineering study completed by Earth Consultants, Inc. (ECI) for the proposed Wood Trails residential development, Woodinville, Washington. The general location of the site is shown on the Vicinity Map, Plate 1.

The purpose of this study was to explore the subsurface conditions at the site, and based on the conditions encountered, to develop geotechnical recommendations for the proposed site development. The scope of services included subsurface explorations to characterize soil conditions at the site, engineering analysis, and preparation of this study with geotechnical recommendations for the proposed site development. As part of this study, we also reviewed the City of Woodinville Development Standards for Environmentally Sensitive Areas (Section 21.24). As such, erosion and steep slope hazards with respect to the proposed development are also addressed. ECI also reviewed a Preliminary Site Plan prepared by Triad Associates, dated May 13, 2004.

Project Description

We understand it is planned to develop the site with sixty-six (66) single-family residences. The development areas will largely be confined to the east and central portions of the property. The proposed development area including roads and building lots occupy approximately 12.9 acres of the overall site, with the remainder of the site consisting of Native Growth Protection Area (NGPA)/Open Space. The proposed residences will consist of relatively lightly-loaded wood-frame construction utilizing crawl space and slab-on-grade floor areas. Preliminary plans indicate the building lots will be terraced at some locations, and may utilize partial daylight basements. At the time this study was performed, the site, proposed site layout and our exploratory locations were approximately as shown on the Test Pit Location Plan, Plate 2.

At the time this report was prepared, a final grading plan had not been completed. However, we anticipate cuts and fills throughout the proposed roadway and building lot areas could be on the order of approximately 8 feet to 14 feet. We understand rockery construction is also planned as part of the grading for the roadway and building lot areas. At this time we understand the maximum rockery height will be on the order of approximately 12 feet.

Stormwater generated from the site will be conveyed to a stormwater detention pond that will be located near the west property line. At the location of the proposed stormwater detention pond, cuts on the order of twenty (20) feet below existing grade will be necessary at some locations. A compacted fill berm will be constructed along the westerly side of the detention pond, and a series of rockeries will be utilized along the excavation for the pond. Fill depths necessary to complete the compacted fill berm will typically be on the order of five feet or less. The maximum rockery height along the pond excavation will be on the order of eight feet. A select number of the building lots that cannot be serviced by the stormwater detention pond will utilize dispersion systems. We understand the building sites that may require dispersion systems will be identified during the design phase of the project.

If the above project description is incorrect or the project information changes, ECI should be consulted to review the recommendations contained in this study and make modifications to our geotechnical recommendations, if necessary.

Site History

As part of our study, ECI reviewed a series of aerial photographs of the site dating back to 1936. These photographs are included in Appendix D of this report. Based on our review of the aerial photographs, it appears the site was previously developed with a series of roadways, dwellings, and outbuildings, located primarily throughout the westerly portions of the site. These features are visible on the 1968 and 1936 photographs. A 1980 photograph indicates a more extensive network of roadways throughout the central and northerly sections of the property. A relatively extensive area of clearing throughout the southerly sections of the property is visible on the 1936 aerial photograph. This clearing was likely associated with logging operations.

SITE CONDITIONS

Surface

The undeveloped property is located west of 148th Avenue Northeast in Woodinville, Washington. The approximate location of the site is illustrated on the attached Vicinity Map (Plate 1), and the approximate site layout is illustrated on the Test Pit Location Plan (Plate 2). The gross area of the property is on the order of fifty (50) acres. The site topography is varied, with the overall trend consisting of west descending slopes. The most distinguishing site features include a series of east-west trending ravine areas with relatively moderate to steep side slopes. Relatively gently sloping areas are located along the margins of the ravines, and throughout the easterly portions of the site.

The ravine areas of the site generally contain slopes of 40 percent or steeper. The slopes are well vegetated with an established understory of bushes and groundcover. Mature evergreen trees are established throughout the site. The bottom of the ravine areas were observed for signs of severe erosion or deep rutting. Based on our observations, it does not appear the slopes have experienced any recent episodes of severe erosion. An assessment of the steep slope areas is discussed in greater detail in the *Steep Slope Assessment* section of this report.

The development areas of the site are bordered to the east by 148th Avenue Northeast and existing residential development. The undeveloped portions of the subject property will border the development areas to the north and west. To the south, the proposed development area is bordered by 195th Street Northeast and existing residential development.

Subsurface

A total of twenty-eight (28) test pits were excavated throughout the proposed development areas of the site. The approximate test pit locations are illustrated on the attached Test Pit Location Plan (Plate 2). The test pit logs are attached as Plates A2 through A30. Medium dense to very dense weathered and unweathered glacial till (Qvt) deposits were observed at the test sites. The glacial till consisted primarily of silty sand with gravel that graded from a brown weathered condition to a gray unweathered condition at a depth of approximately three to four feet. Localized deposits of medium dense to very dense sandy silt and sand associated with the glacial till deposit were also occasionally encountered at the test pit locations.

The geologic map of the Bothell Quadrangle identifies Advance Outwash (Qva) deposits throughout the site and surrounding area. Glacial Till (Qvt) deposits are mapped immediately to the east of the property, and to the west of the property, west of 144th Avenue Northeast. Landslide deposits (Qls) are not identified on the site, or on surrounding properties. Based on our observations, the soils encountered at the test pit locations are consistent with glacial till deposits which typically overlie the Advance Outwash deposits that are mapped throughout the site.

The King County Soil Conservation Survey identifies Alderwood (AgC and AgD) soil deposits throughout the majority of the site. These deposits are described as gravelly sandy loam, and are associated with glacial deposits. Based on our observations and subsurface exploration, the soils encountered at the site are consistent with Alderwood deposits. Based on the 1997 King County Surface Water Design Manual, the Alderwood soils identified at the site are classified as Hydrologic Soil Group C.

Groundwater

Groundwater seepage was encountered at two test pit locations located within the footprint of the proposed stormwater detention pond. The test pits located in the vicinity of the proposed pond were excavated in February and April 2004. The groundwater seepage was observed at test pit locations Test Pits TP-104 and TP-201. The seepage is associated with a seasonal perched condition that develops along the contact between the upper weathered glacial till and the underlying dense to very dense unweathered glacial till. The groundwater seepage was observed at a depth of approximately four to six feet below the existing ground surface elevation. Typically, the groundwater seepage level is higher and the rate of seepage is greater in the wetter winter months (typically October through May).

Groundwater seepage was not observed at the test pit locations located throughout the proposed residential development areas. However, perched zones of groundwater seepage may be encountered in the site excavations. Based on the conditions observed at the test pit locations, we anticipate groundwater seepage rates will generally be light. Moderate groundwater seepage, however, could be encountered in the deeper excavations, such as utility trench excavations.

Steep Slope Assessment

As part of our study, the steep slope areas of the site were observed for signs of instability. The King County Sensitive Areas Map Folio identifies the majority of the site as an Erosion Hazard Area. However, the King County Sensitive Areas Map does not identify the site as a Landslide Hazard Area.

Based on the subsurface conditions observed at the test pit locations, the slopes adjacent to the planned development areas consist primarily of medium dense to very dense native glacial till deposits. Based on the site survey, the grades along the native slopes outside the development area are on the order of 40 percent or greater. The overall height of the steep slopes is on the order of sixty (60) to one hundred (100) feet. Vegetation along the slopes is well established with an understory of bushes and groundcover. Mature evergreen trees are also established along the slope areas.

Based on our observations, the steep slope areas located adjacent to the proposed development areas of the site appear to be in a stable condition. There were no indications of severe erosion or debris flow activity. Evidence of past slope instability or severe erosion such as tension cracks, slide scarps, or deep rutting were not observed. Review of aerial photographs of the site also did not reveal any evidence of significant past instability.

In our opinion, establishing appropriate building setback criteria for the planned development will adequately mitigate the potential for slope related impacts to the planned development. The planned installation of storm drains and controlling surface water runoff above the slopes will also help improve the overall stability of the adjacent slopes, and help to reduce erosion hazards. Setback recommendations are discussed in the *General* and *Foundations* section of this report.

Laboratory Testing

Laboratory tests were conducted on several representative soil samples to verify or modify the field soil classification and to evaluate the general physical properties and engineering characteristics of the soil encountered. Visual field classifications were supplemented by grain size analyses on representative soil samples. Moisture content tests were performed on all samples. The results of laboratory tests performed on specific samples are provided either at the appropriate sample depth on the individual test pit logs or on a separate data sheet contained in Appendix B. It is important to note that these test results may not accurately represent the overall in-situ soil conditions. Geotechnical recommendations provided in this study are based on interpretation of these test results. ECI cannot be responsible for the interpretation of these data by others.

In accordance with the Standard Fee Schedule and General Conditions, the soil samples for this project will be discarded after a period of fifteen (15) days following completion of this report unless directed otherwise in writing.

DISCUSSION AND RECOMMENDATIONS

General

Based on the results of our study, it is our opinion the planned residential development is feasible from a geotechnical standpoint. The proposed building structures can be supported on conventional spread and continuous footing foundation systems bearing on competent native soil or on structural fill used to modify existing site grades. Based on the generally dense to very dense condition of the native soils observed at our exploration sites, and considering there are no observable signs of past instability or severe erosion, we recommend for design purposes a minimum building setback of twenty-five (25) feet from the top of the steep slope areas. Based on preliminary plans, we understand the lot lines will be setback a minimum distance of ten (10) feet from the top of the steep slopes, and the building foundations an additional fifteen (15) feet, for a total minimum setback of twenty-five (25) feet.

We understand there are a limited number of building lots that may require side yard setbacks that are less than twenty-five (25) feet. In our opinion, a limited reduction in the setback at some side yard locations is likely feasible. Where a reduced side yard setback is utilized, the lot line will be setback a minimum distance of ten feet from the top of the steep slopes, and the building foundations an additional distance of at least five feet. ECI should assess the feasibility of a reduced side yard setback on a case by case basis, once the building lots have been identified.

In our opinion, construction of the proposed storm water detention pond is feasible from a geotechnical standpoint. The presence of groundwater seepage should be expected in the excavation for the detention pond. We understand the excavation for the pond will require cuts on the order of twenty (20) feet below existing grade at some locations. In our opinion, due to the dense to very dense soils conditions observed throughout the pond area, the use of rockeries around the margins of the pond excavation is feasible from a geotechnical standpoint.

With respect to stormwater dispersion, it is our opinion the use of dispersal trenches is feasible for a limited number of the proposed building lots. We understand, however, that the majority of the stormwater generated on-site will be conveyed to the stormwater detention pond. In our opinion, storm water infiltration for individual building lots at the site is generally not feasible.

This report has been prepared for specific application to this project only and in a manner consistent with that level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area for the exclusive use of Phoenix Development, Inc. and their representatives. No warranty, expressed or implied, is made. This report, in its entirety, should be included in the project contract documents for the information of the contractor.

Site Preparation and General Earthwork

The proposed development areas of the site should be stripped and cleared of existing surface vegetation, topsoil, and other deleterious materials. Existing utility pipes that will be abandoned should be plugged or removed. Based on the conditions observed at the test pit locations, the thickness of the topsoil layer ranges between approximately 4 inches to 12 inches. The thickness of the topsoil layer will vary throughout the site, and could exceed twelve (12) inches in some areas.

The ground surface where foundations or structural fill are to be placed should be observed by a representative of ECI. An ECI representative should also observe the excavation for the proposed storm water detention pond. Due to the moderately high fines content of the native soils, moisture sensitivity of the soils should be expected. Building and pavement subgrade areas that are exposed to extended periods of precipitation could degrade. If the subgrade soil in the proposed foundation and pavement areas becomes saturated and degrades, aeration and moisture conditioning of the soils, or overexcavation and replacement with structural fill may be necessary.

To minimize the need for overexcavation resulting from disturbed subgrade conditions, construction traffic should be minimized along subgrade surfaces during periods of rainfall, if possible. Delaying the site stripping and leaving the subgrade high, where possible, will help minimize disturbance to the building and pavement subgrade during periods of extended wet weather conditions. Establishing rock surfaced construction roadways, and restricting construction traffic where possible will also help preserve the subgrade soils.

In our opinion, the majority of the native soils can be considered for use as structural fill, provided the soil is placed during dry weather conditions, and provided the moisture content of the soil is at or near the optimum moisture content at the time of placement. At the time of the subsurface exploration the native silty sand soils were generally in a moist condition and suitable for use as structural fill.

Successful use of the native soils as structural fill will require that soil stockpiles be covered with plastic sheeting. Covering stockpiles will help preserve the natural moisture content of the soil, and will minimize erosion of the stockpiles during wet weather conditions. The entire stockpile down to the toe of the pile should be covered with the plastic sheeting. Excavation and placement of the native soils should only be performed during dry weather conditions. ECI should periodically meet with the contractor during construction to assess the suitability of the on-site soils for use as structural fill.

Imported soil intended for use as structural fill should consist of a suitable, compactible granular soil with a moisture content that is at or near the optimum moisture content, and having a maximum aggregate size of four inches. Imported soil intended for use during wet weather conditions should consist of a granular soil having no more than 5 percent fines passing the No. 200 sieve based on the minus 3/4-inch fraction. Samples of imported soil should be submitted to ECI for approval.

Structural fill is defined as compacted fill placed under foundations, roadways, slabs, pavements, or other load-bearing areas. Structural fill under slabs and footings should be placed in horizontal lifts not exceeding twelve (12) inches in loose thickness and compacted to a minimum of 90 percent of its laboratory maximum dry density. The maximum dry density should be determined in accordance with ASTM Test Designation D-1557-91 (Modified Proctor). The fill materials should be placed at or near the optimum moisture content. Fill under pavements and walks should also be placed in horizontal lifts and compacted to 90 percent of the maximum dry density except for the top twelve (12) inches, which should be compacted to 95 percent of the maximum dry density. If a structural fill berm is necessary to construct the storm water detention pond, the fill should be compacted to at least 95 percent of the maximum dry density.

During site grading, measures to reduce the risk of surface erosion should be implemented. Establishing silt fencing along the margins of the project clearing limits, and mulching of exposed earth surfaces will help reduce erosion and sediment transport. Where possible, native vegetation should be preserved to help minimize exposed earth surfaces.

Foundations

Based on our understanding of the proposed residential development, the foundations for the buildings will be supported primarily on undisturbed native soils that will be exposed in the crawl space excavations. For foundations bearing on the medium dense to dense native soils or granular structural fill, an allowable soil bearing capacity of two thousand five hundred (2,500) pounds per square foot (psf) should be used to design the foundations. This allowable soil bearing capacity has a factor-of-safety in excess of 3.0 against shear failure, provided the foundations are placed on competent native soils or structural fill. A one-third increase in the above allowable soil bearing capacity can be assumed for short-term wind and seismic loading conditions. Continuous and individual spread footings should have minimum widths of eighteen (18) and twenty-four (24) inches, respectively.

If loose or unstable soil conditions are encountered at the footing subgrade elevation, the soil will need to be overexcavated, and replaced with structural fill. The width of the overexcavation should extend a minimum of six inches beyond each edge of the foundation. As previously discussed, care will need to be taken to protect and preserve exposed subgrade surface to limit the amount of disturbance to the subgrade, and to limit the need for overexcavation.

Exterior foundations elements should be placed at a minimum depth of eighteen (18) inches below final exterior grade. Interior spread foundations can be placed at a minimum depth of twelve (12) inches below the top of slab, except in unheated areas, where interior foundation elements should be founded at a minimum depth of eighteen (18) inches.

Provided the foundations are placed in accordance with the recommendations contained in this report, we estimate total settlement of approximately one inch and differential settlement of up to approximately one-half inch. Most of the anticipated settlements should occur during construction as dead loads are applied.

Lateral loads can be resisted by friction between the base of the foundation and the supporting soil, and by passive soil pressure acting on the face of the buried portion of the foundation. Resistance to lateral loads from passive earth pressures can be calculated using an equivalent fluid with a unit weight of three hundred fifty (350) pounds per cubic foot (pcf). To achieve adequate passive resistance, the foundations must be backfilled with structural fill. As an alternative, the foundations can be poured neat against the undisturbed native soil. For frictional capacity, a coefficient of 0.40 can be used for foundations bearing on competent native soils or structural fill. These lateral resistance values are allowable values; a factor-of-safety of 1.5 has been included.

Footing excavations and the foundation subgrade should be observed by a representative of ECI prior to placing the formwork and repar.

Foundation Setbacks

As previously discussed, we recommend a minimum building and foundation setback of twenty-five (25) feet from the top of the existing 40 percent slopes. As previously stated, we understand the lot lines will be setback a minimum distance of ten (10) feet from the top of the steep slopes, and the building foundations an additional fifteen (15) feet, for a total minimum setback of twenty-five (25) feet. In our opinion, where necessary, a minimum side yard setback of fifteen (15) feet can be used. Where a reduced side yard setback is utilized, the lot line will be setback a minimum distance of ten feet from the top of the steep slopes, and the building foundations an additional distance of at least five feet. We understand this reduced side yard setback will only be necessary for a limited number of building lots. The lots requiring a reduced side yard setback should be evaluated on a case by case basis by ECI once the building lots have been identified. The existing 40 percent or greater slopes should be delineated for purposes of establishing the recommended setbacks. ECI should review the final building layout and setbacks.

Permanent Retaining and Foundation Walls

Retaining and foundation walls should be designed to resist lateral earth pressures from the retained soils, and any surcharge loading. Walls that are unrestrained and free to move at the top can be designed using an equivalent fluid with a unit weight of thirty-five (35) pcf. The earth pressure imparted on restrained walls should be calculated using an equivalent fluid with a unit weight of fifty (50) pcf. The above equivalent fluid values assume surcharges due to traffic, adjacent foundations, construction loads, or any other loadings will not apply. If surcharges are to apply, they should be added to the above design lateral pressures.

For traffic surcharge loading, a uniform pressure of seventy (70) psf should be applied in a rectangular distribution along the height of the retaining wall. If sloping backfill conditions are present behind the walls, ECI should review the slope configurations and provide modified equivalent fluid values, as necessary.

Retaining and foundation walls should be provided with a four-inch diameter perforated drainpipe and backfilled with a free-draining granular soil with less than 5 percent fines (percent passing the No. 200 sieve based on the minus 3/4-inch fraction). The zone of free-draining granular soil should extend along the entire height of the wall, and a distance of at least eighteen (18) inches behind the wall. A surface seal consisting of a less permeable silty sand soil can be placed along the upper one foot of the wall backfill, if desired. The remainder of the backfill behind the zone of free draining soil should consist of a suitable granular structural fill.

As an alternate to free draining backfill around the building foundation walls, the use of a sheet drain such as Mira-drain 6000 or equivalent can be considered. If a sheet drain is utilized, the foundation or retaining wall backfill should contain no more than 30 percent fines.

Seismic Design Considerations

The Puget Sound region is classified as Zone 3 by the Uniform Building Code (UBC). The largest earthquakes in the Puget Sound region have been subcrustal (intraplate) events, ranging in depth from fifty (50) to seventy (70) kilometers. Such deep events have exhibited no surface faulting. Weaver and Shedlock (1989) researched the probable or known source areas for the crustal, intraplate, and subduction zone earthquakes in the Washington and Oregon area. Crustal and intraplate earthquakes are the only events in Washington and Oregon in which there is a historical record. Shallow crustal earthquakes occur within the North American Plate, and typically do not exceed focal depths of approximately 20 kilometers. Intraplate earthquakes occur in the subducting Juan de Fuca plate, and typically occur below depths of forty (40) kilometers. The recent February 28, 2001 earthquake that was focused just north of Olympia, Washington was an intraplate earthquake, and had a magnitude of $M_L = 6.8$. The subduction zone earthquake, in which there is no historical record in the Washington and Oregon area, would have its source along the interface between the North American Plate and the subducting Juan de Fuca Plate. Magnitude 8+ earthquakes are thought to be possible along this interface, and would occur at depths of approximately 50 to 60 kilometers (Weaver and Shedlock, 1989).

The UBC Earthquake regulations have established a series of soil profile types that are used as a basis for seismic design of structures. Based on the encountered soil conditions, it is our opinion that soil type S_C from Table 16-J of the 1997 UBC should be used for design.

Liquefaction is a phenomenon in which soils lose all shear strength for short periods of time during an earthquake. The effects of liquefaction may be large total and/or differential settlement for structures with foundations founded in the liquefying soils. Groundshaking of sufficient duration results in the loss of grain-to-grain contact and rapid increase in pore water pressure, causing the soil to behave as a fluid for short periods of time.

To have potential for liquefaction, a soil must be cohesionless with a grain size distribution of a specified range (generally sands and silt); it must be loose to medium-dense; it must be below the groundwater table; and it must be subject to sufficient magnitude and duration of groundshaking.

Based on the soil and groundwater conditions observed at the site, it is our opinion that the site has a low susceptibility to liquefaction. The dense condition of the native soils is the primary basis for this conclusion.

Slab-on-Grade Floors

Slab-on-grade floors should be supported on competent native soils or structural fill. The proposed fill areas of the site will likely be more susceptible to disturbance from construction traffic during wet weather conditions. Loose or unstable subgrade soils should be stabilized prior to construction of the slab. If the construction is performed during the drier summer months, measures to preserve the subgrade soils will likely be minimal. During the wet season, however, a free draining structural fill may need to be utilized throughout the upper twelve (12) inches of the building pads to help preserve the integrity of the subgrade.

A minimum four-inch capillary break consisting of a free draining poorly graded gravel with less than 5 percent fines (percent passing the No. 200 sieve, based on the minus 3/4-inch fraction) should be placed below the slab. A vapor barrier consisting of a minimum 6-mil plastic membrane should be placed above the capillary break. To aid in curing of the concrete slab, two inches of sand can be placed over the plastic membrane. The subgrade soils in slab-on-grade areas of the site should be observed by a representative of ECI prior to placing the capillary break material.

Site Drainage

During construction, surface water runoff must not be allowed to stand in construction areas. Interceptor trenches should be established, as necessary, along the perimeter of the building site before it enters the construction area. During construction, loose surfaces should be compacted to reduce the potential for moisture infiltration into the soils. Finish grades around the buildings must be sloped such that surface water is directed away from the buildings. Measures to accommodate drainage around the building structures, such as "positive drains" or footing drains, should be incorporated into the design. All roof downspouts must be separately tightlined to the site storm water system.

In the deep utility excavations, the presence of groundwater seepage should be expected, particularly if the excavation is performed during the wet season. Temporary construction dewatering of excavations may be necessary depending on the rate of seepage encountered. Light groundwater seepage may also be encountered along the cuts for the proposed building pad areas.

Downspout Dispersion Systems

In our opinion the use of a downspout dispersion system is feasible for a limited number of the proposed building lots. As previously stated, the majority of the stormwater generated on-site will be conveyed to the stormwater detention pond. Where site elevations will prohibit connection to the stormwater conveyance system, a dispersion system can be utilized. The design criteria contained in Section 5.1.2 of the King County Surface Water Design Manual should be used to design the dispersion systems. In our opinion, due to the generally dense and sandy loam soil conditions, infiltration of site stormwater is not feasible.

Excavations and Slopes

The following information is provided solely as a service to our client. Under no circumstances should this information be interpreted to mean that ECI is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

In no case should excavation slopes be greater than the limits specified in local, state, and Federal safety regulations. Based on the information obtained from our field exploration, the upper deposits of weathered glacial till would be classified as Type C soils by Occupational Safety and Health Administration (OSHA). Temporary cuts in Type C soils should be sloped at an inclination no steeper than 1.5H:1V (Horizontal:Vertical). The unweathered glacial till soils encountered below a depth of approximately four feet to six feet would be classified as Type A and Type B soils by OSHA. Temporary cuts in Type A and Type B soils should be sloped at an inclination no steeper than 0.75H:1V and 1H:1V, respectively. ECI should observe the excavations to assess soil and groundwater conditions, and to verify the OSHA soil type.

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. Cut slopes should be observed by ECI during excavation to verify that conditions are as anticipated. Supplementary recommendations can then be developed, if needed, to improve stability, including flattening of slopes or installation of surface or subsurface drains. In any case, water should not be allowed to flow uncontrolled over the top of slopes.

Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

Utility Trench Backfill

Based on the soil conditions encountered at the time of our exploration, the native and existing fill soils should provide adequate support for utilities. If remedial measures are necessary to provide adequate support for utilities, the unsuitable soils can be overexcavated and replaced with crushed rock and a pipe bedding material such as pea gravel. The presence of groundwater seepage should be expected in the deeper utility trench excavations and the proposed detention pond excavations.

In our opinion, the native glacial till soils can be considered for use as backfill for the utility trenches, provided the soil moisture content is at or near its optimum level. Due to the moisture sensitive nature of the native soils, placement and compaction of the soil will need to be performed during dry weather conditions. To protect the native soil and preserve the moisture content, stockpiles should be immediately covered with plastic sheeting. The plastic sheeting should cover the entire stockpile. In our opinion, the native silt soils and the existing fill soils may be difficult to use as structural fill. ECI will work with the contractor to assess the suitability of the soils as structural backfill in utility trenches.

Due to the moisture sensitive nature of the native soils, the upper twelve (12) inches of the trench backfill in the building and pavement areas may become disturbed if exposed to wet weather conditions and construction traffic. Construction traffic should be kept to a minimum along utility trench alignments where backfilling and compaction have been completed. As an alternative, a free draining gravel can be used to backfill the upper twelve (12) inches of the trench excavations to provide a wearing surface, and to help protect the underlying native backfill. Use of a free draining backfill along the upper twelve (12) inches of the trench excavation would likely only be necessary if construction is performed during the wet season.

Utility trench backfill is a primary concern in reducing the potential for settlement in pavement areas. It is important that the utilities be adequately supported in the bedding material. The material should be hand tamped to ensure support is provided around the haunches of these structures. Fill should be carefully placed and tamped to about 12 inches above the crown of the pipe before heavy compaction equipment is brought into use. The remainder of the backfill should be placed in lifts having a loose thickness of less than twelve (12) inches and compacted to structural fill requirements.

Rockeries

We understand the use of rockeries is being considered along the cuts for the proposed storm water detention pond, and throughout portions of the proposed residential development. At this time, a tiered rockery with maximum rockery heights of eight feet is being considered along the excavation for the storm water detention pond. Rockeries of up to approximately 12 feet are planned throughout portions of the residential development. In our opinion, construction of the proposed rockeries is feasible from a geotechnical standpoint. Plate 4 illustrates a cross section and general guidelines for rockery construction throughout areas of competent native cut. The rockery construction should be completed in general accordance with the Associated Rockery Contractors (ARC) Standard Rockery Construction Guidelines. A copy of the ARC Guidelines is included in Appendix C of this report. An ECI representative should observe the excavation for the rockeries, and periodically observe the rockery construction. The rockery construction should follow closely behind the excavation to help minimize the period of time the excavation is exposed.

Fill rockeries may also be utilized throughout the new roadway areas. Where fill rockeries will exceed four feet in height, placement of geogrid reinforcement will be necessary throughout the fill zone. ECI can provide a design for construction of reinforced fill rockeries, if requested.

Pavement Areas

The adequacy of site pavements is related in part to the condition of the underlying subgrade. To provide a properly prepared subgrade for pavements, the subgrade should be in a firm and unyielding condition when subjected to proofrolling with a loaded dump truck. Structural fill in pavement areas should be prepared as described in the *Site Preparation and General Earthwork* section of this report. This means the pavement subgrade should be compacted to at least 95 percent of the maximum dry density. It is possible that some localized areas of soft, wet or unstable subgrade may exist after the pavement subgrade is prepared. Overexcavation and a greater thickness of structural fill or crushed rock may be needed to stabilize these localized areas. As previously discussed, the contractor should prepare a strategy/process for limiting disturbance to the subgrade areas, thereby minimizing the amount of overexcavation in the pavement areas.

Assuming a properly prepared subgrade that is in a firm and unyielding condition when subjected to proofrolling, the following pavement section for lightly-loaded areas can be used:

- Two inches of asphalt concrete (AC) over four inches of crushed rock base (CRB) material, or

- Two inches of AC over three inches of asphalt treated base (ATB) material.

Heavier truck-traffic areas will require thicker pavement sections depending upon site usage, pavement life, and site traffic. As a general rule, the following sections can be considered for truck-trafficked areas:

- Three inches of AC over six inches of CRB, or

- Three inches of AC over four and one-half inches of ATB.

These pavement thicknesses may be modified based on anticipated traffic loads and frequency.

Asphalt concrete (AC), asphalt treated base (ATB), and crushed rock base (CRB) materials should conform to WSDOT specifications. All rock bases should be compacted to at least 95 percent of the maximum dry density.

LIMITATIONS

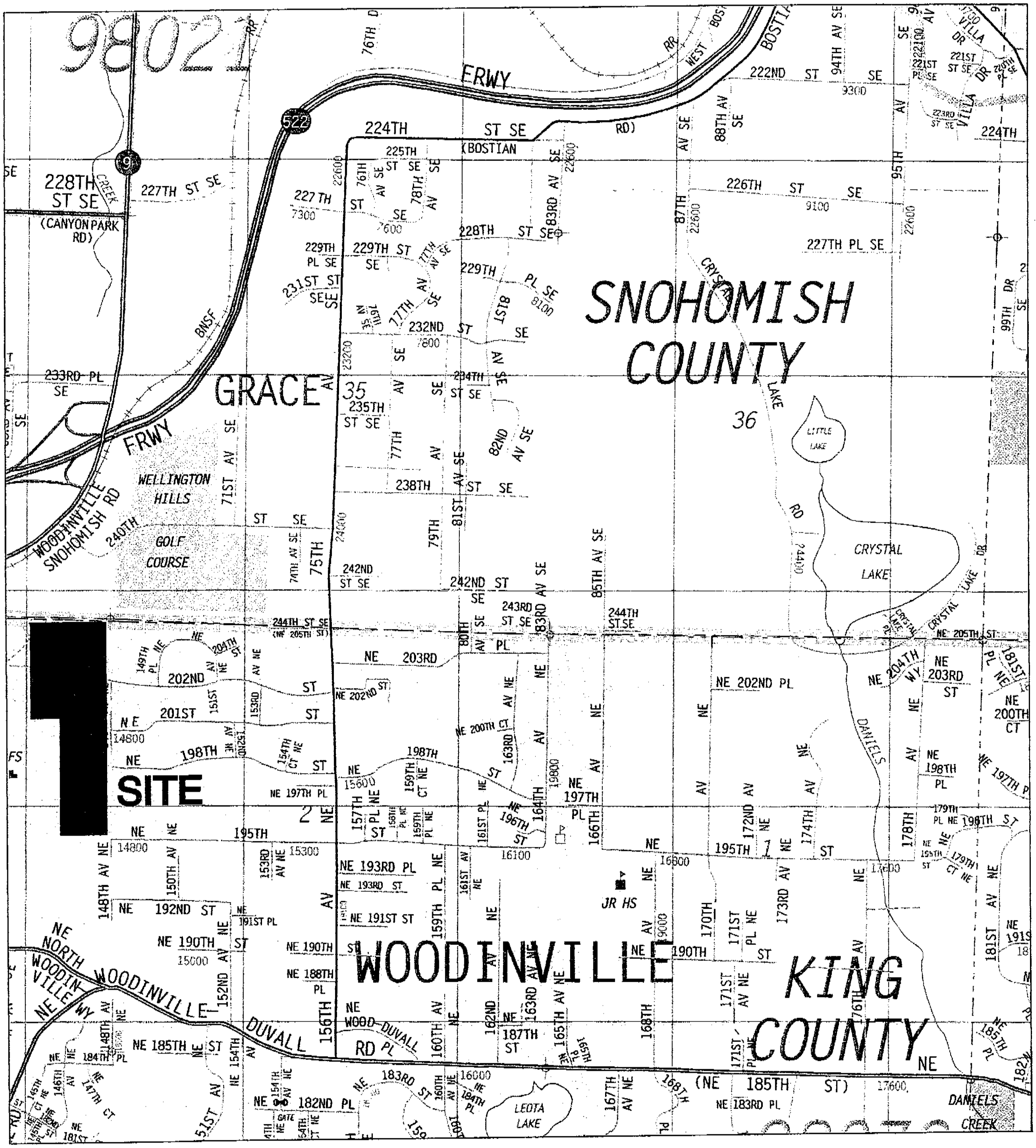
Our recommendations and conclusions are based on the site materials observed, selective laboratory testing and engineering analyses, the design information provided to us, and our experience and engineering judgement. The conclusions and recommendations are professional opinions derived in a manner consistent with that level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area. No warranty is expressed or implied.

The recommendations submitted in this report are based upon the data obtained from the test pits. Soil and groundwater conditions between exploration sites may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, ECI should be requested to reevaluate the recommendations of this report and allowed to modify or verify our recommendations in writing prior to proceeding with the construction.

Additional Services

We recommend that ECI be retained to perform a general review of the final design and specifications to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design and in the construction specifications.

We also recommend that ECI be retained to provide geotechnical services during construction. This is to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. ECI should be retained to review the construction drawings and specifications, and to provide construction observation and testing.



Reference:
King County
Map 477
By Thomas Brothers Maps
Dated 2004

NOTE: This plate may contain areas of color. ECI cannot be responsible for any subsequent misinterpretation of the information resulting from black & white reproductions of this plate.

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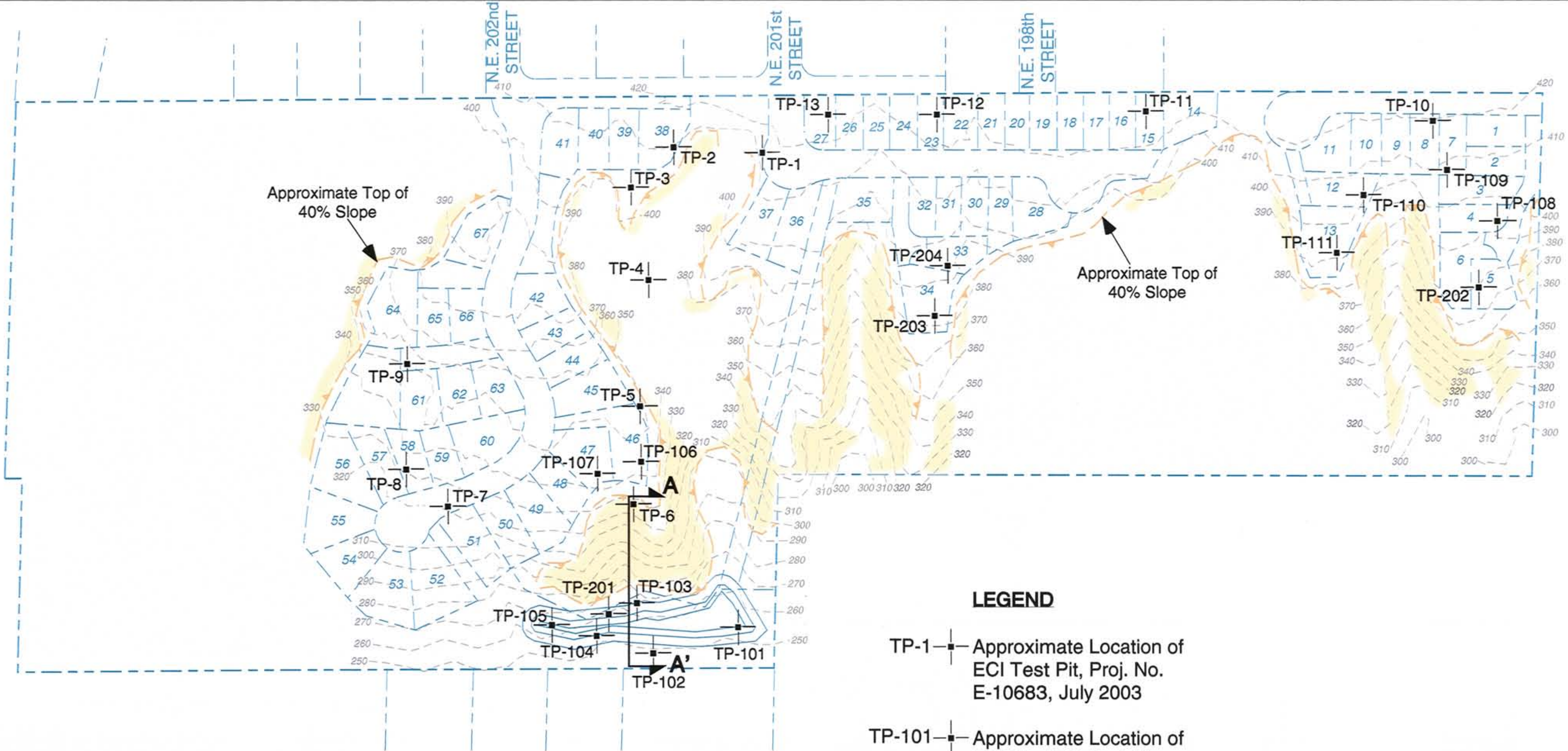
Vicinity Map
Wood Trails
Woodinville, Washington

Drwn. GLS	Date April 2004	Proj. No. 10683
Checked RAC	Date 4/13/04	Plate 1



N

Approximate Scale

0 100 200 400ft.

NOTE: This plate may contain areas of color. ECI cannot be responsible for any subsequent misinterpretation of the information resulting from black & white reproductions of this plate.

LEGEND

TP-1 — Approximate Location of ECI Test Pit, Proj. No. E-10683, July 2003

TP-101 — Approximate Location of ECI Test Pit, Proj. No. E-10683, Feb. 2004

TP-201 — Approximate Location of ECI Test Pit, Proj. No. E-10683, April 2004

Subject Site

23 Proposed Lot Number

Approximate Area of Slope ≥ 40%

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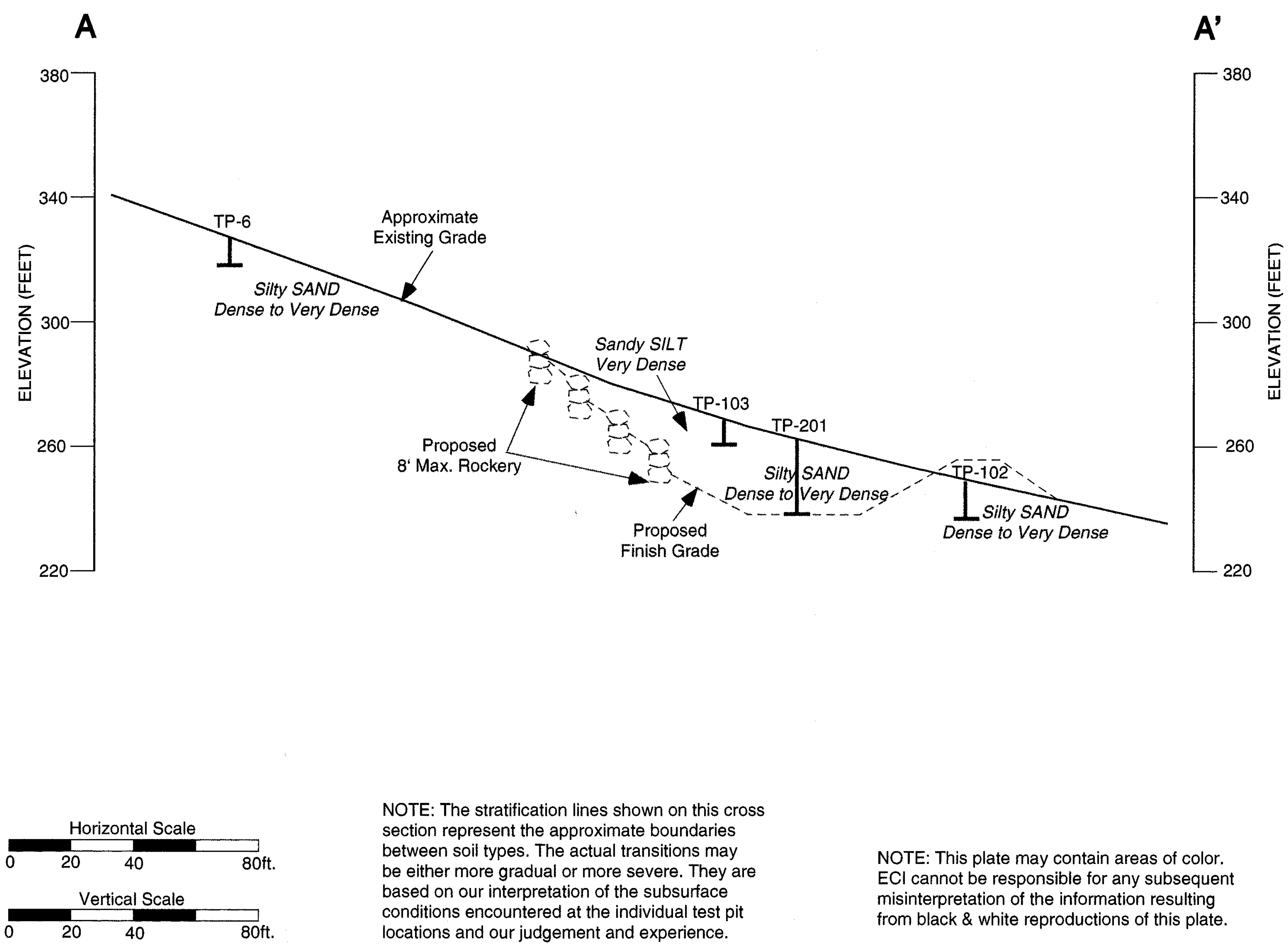
Test Pit Location Plan
Wood Trails
Woodinville, Washington

Drwn. GLS	Date April 2004	Proj. No. 10683
Checked RAC	Date 4/13/04	Plate 2



NOTE: The stratification lines shown on this cross section represent the approximate boundaries between soil types. The actual transitions may be either more gradual or more severe. They are based on our interpretation of the subsurface conditions encountered at the individual test pit locations and our judgement and experience. ECI cannot be responsible for the interpretation of the data by others.

NOTE: This plate may contain areas of color. ECI cannot be responsible for any subsequent misinterpretation of the information resulting from black & white reproductions of this plate.

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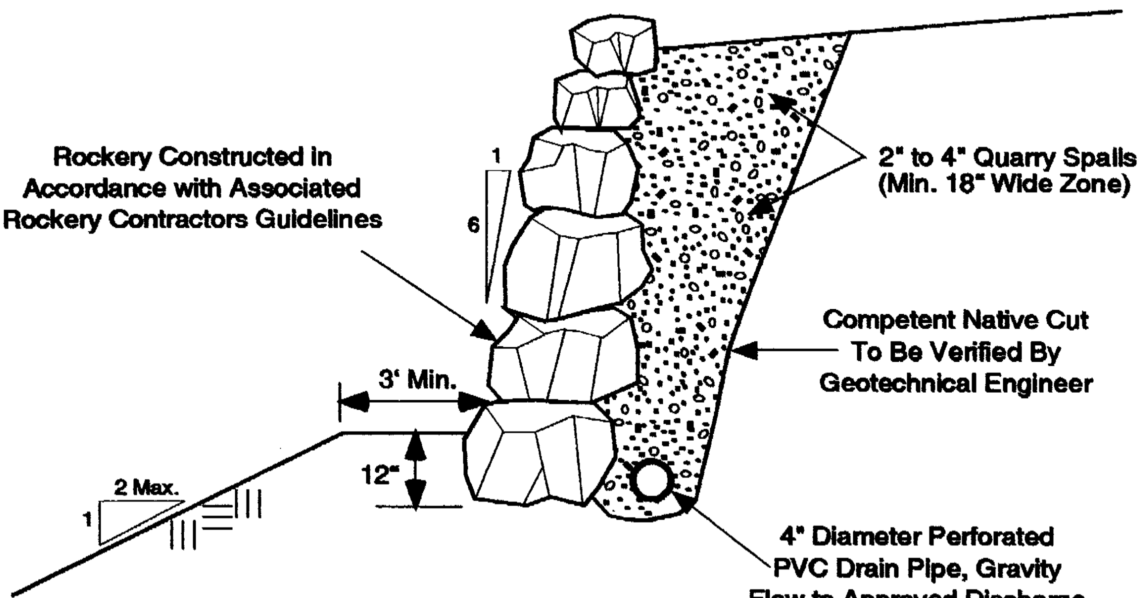
Cross Section A-A'
Wood Trails
Woodinville, Washington

Drwn. GLS	Date April 2004	Proj. No. 10683
Checked RAC	Date 4/21/04	Plate 3



NOTE: Geotechnical Engineer to observe rockery construction on Full-Time basis.

ROCKERY AT COMPETENT NATIVE CUT

HORIZONTAL & VERTICAL SCALE 1" = 5'

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Rockery at Competent Native Cut
Wood Trails
Woodinville, Washington

Drwn. GLS	Date April 2004	Proj. No. 10683
Checked RAC	Date 4/26/04	Plate 4

APPENDIX A

FIELD EXPLORATION

E-10683

Our field exploration was performed on three separate visits during July 2003 through April 2004. Subsurface conditions at the site were explored by excavating twenty-eight (28) test pits to a maximum depth of twenty-two (22) feet below grades. Approximate test pit locations and elevations were determined by pacing from the existing lot boundaries and referencing plans provided by the client. The locations and elevations of the test pits should be considered accurate only to the degree implied by the method used. These approximate locations are shown on the Test Pit Location Plan, Plate 2.

The field exploration was continuously monitored by a geologist from ECI, who classified the soils encountered, maintained a log of each test pit and boring, obtained representative samples, measured groundwater levels, and observed pertinent site features. The samples were visually classified in accordance with the Unified Soil Classification System, which is presented on Plate A1, Legend. Representative soil samples were placed in closed containers and returned to the laboratory for further examination and testing.

Logs of the test pits are presented on Plates A2 through A30. The final logs represent the interpretations of the field logs and the results of laboratory examination and testing. The stratification lines on the logs represent the approximate boundaries between soil types. In actuality, the transitions may be more gradual.

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTION
Coarse Grained Soils More Than 50% Material Larger Than No. 200 Sieve Size	Gravel And Gravelly Soils More Than 50% Coarse Fraction Retained On No. 4 Sieve	Clean Gravels (little or no fines)		GW gw	Well-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
				GP gp	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
		Gravels With Fines (appreciable amount of fines)		GM gm	Silty Gravels, Gravel-Sand-Silt Mixtures
				GC gc	Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sand And Sandy Soils More Than 50% Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)		SW sw	Well-Graded Sands, Gravelly Sands, Little Or No Fines
				SP sp	Poorly-Graded Sands, Gravelly Sands, Little Or No Fines
		Sands With Fines (appreciable amount of fines)		SM sm	Silty Sands, Sand - Silt Mixtures
				SC sc	Clayey Sands, Sand - Clay Mixtures
Fine Grained Soils More Than 50% Material Smaller Than No. 200 Sieve Size	Silts And Clays	Liquid Limit Less Than 50		ML ml	Inorganic Silts & Very Fine Sands, Rock Flour, Silty-Clayey Fine Sands; Clayey Silts w/ Slight Plasticity
				CL cl	Inorganic Clays Of Low To Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
				OL ol	Organic Silts And Organic Silty Clays Of Low Plasticity
	Silts And Clays	Liquid Limit Greater Than 50		MH mh	Inorganic Silts, Micaceous Or Diatomaceous Fine Sand Or Silty Soils
				CH ch	Inorganic Clays Of High Plasticity, Fat Clays.
				OH oh	Organic Clays Of Medium To High Plasticity, Organic Silts
Highly Organic Soils				PT pt	Peat, Humus, Swamp Soils With High Organic Contents

Topsoil		Humus And Duff Layer
Fill		Highly Variable Constituents

The discussion in the text of this report is necessary for a proper understanding of the nature of the material presented in the attached logs.

DUAL SYMBOLS are used to indicate borderline soil classification.

C	TORVANE READING, tsf
qu	PENETROMETER READING, tsf
W	MOISTURE, % dry weight
P	SAMPLER PUSHED
*	SAMPLE NOT RECOVERED
pcf	DRY DENSITY, lbs. per cubic ft.
LL	LIQUID LIMIT, %
PI	PLASTIC INDEX

- 2" O.D. SPLIT SPOON SAMPLER
- 24" I.D. RING OR SHELBY TUBE SAMPLER
- WATER OBSERVATION WELL
- DEPTH OF ENCOUNTERED GROUNDWATER DURING EXCAVATION
- SUBSEQUENT GROUNDWATER LEVEL W/ DATE

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LEGEND

Proj. No. 10683 | Date Apr. 2004 | Plate A1

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-1
Excavation Contactor: NW Excavating			Ground Surface Elevation: 400'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Red brown silty fine to medium SAND, loose, moist
			1			
	13.2		2			
			3			-brown -medium dense
	16.3		4			
			5			
			6			-with gravel -dense
			7			
	16.4		8			-silt / clay nodules -43.5% fines
	18.9		9			Test pit terminated at 9.0 feet below existing grade. No groundwater encountered during excavation. NOTES: Test pits excavated using a CASE 9010B track-hoe by NW Excavating. Test pit elevations based on topographic data on a Preliminary Site Plan provided by the Client.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A2

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-2
Excavation Contactor: NW Excavating			Ground Surface Elevation: 405'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Forest Duff 6": recently graded
					SM	Brown silty fine to medium SAND, loose, moist
			1			
	8.8		2			-medium dense
			3			
	12.9		4			-dense, slightly cemented
			5			-very dense -refusal on very dense soil at 6'
	10.8		6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A3

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-3
Excavation Contactor: NW Excavating			Ground Surface Elevation: 390'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND, loose, moist
			1			
			2			-medium dense
	8.8		3			-with gravel
			4			
			5			-very dense
			6			
			7			-refusal on very dense soil at 7'
	10.1					Test pit terminated at 7.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A4

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-4
Excavation Contactor: NW Excavating			Ground Surface Elevation: 340'
Notes:			

Surface Conditions: Depth of Forest Duff 8"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
			0–4		SM	Brown silty fine to medium SAND, loose, moist
	11.6		4–8		SP-SM	Brown poorly graded fine to medium SAND with silt, medium dense, moist -with gravel
	16.6 16.2		8–12		SM	Brown silty SAND, medium dense, moist -15.7% fines -moist to wet
			12			Test pit terminated at 12.0 feet below existing grade. No groundwater encountered during excavation.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A5

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-5
Excavation Contactor: NW Excavating			Ground Surface Elevation: 355'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Forest Duff 4"
					SM	Brown silty fine to medium SAND, medium dense, moist
			1			
			2			
			3			-with gravel
	11.4		4			-dense to very dense
			5			-slightly cemented
	12.1		6			-refusal at 6'
						Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A6

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-6
Excavation Contactor: NW Excavating			Ground Surface Elevation: 330'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Forest Duff 6"
					SM	Brown silty SAND with gravel, medium dense, moist
			1			
			2			-decrease in gravel content -dense
	9.2		3			-slightly cemented -41.4% fines
			4			
			5			-refusal at 5'
						Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A7

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-7
Excavation Contactor: NW Excavating			Ground Surface Elevation: 325'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty SAND with gravel, medium dense, moist
			1			
			2			
			3			
			4			
	7.8					-dense
			5			
	8.0					-refusal at 6'
			6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A8

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-8
Excavation Contactor: NW Excavating			Ground Surface Elevation: 330'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Forest Duff 6"
					SM	Brown silty SAND with gravel, medium dense, moist
			1			
			2			
	8.1		3			-dense
			4			
			5			-refusal at 5'
						Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A9

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-9
Excavation Contactor: NW Excavating			Ground Surface Elevation: 350'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND, loose, moist
			1			
			2			-medium dense
			3			
	8.2		4			
			5			-dense
			6			
	9.1		7			-refusal at 7'
						Test pit terminated at 7.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A10

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-10
Excavation Contactor: NW Excavating			Ground Surface Elevation: 415'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Red brown silty fine to medium SAND, loose, moist
			1			
			2			
			3			-brown
	10.3		4			-medium dense
			5			-36.8% fines
	14.6					-dense
			6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A11

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails				Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-11	
Excavation Contactor: NW Excavating			Ground Surface Elevation: 415'	
Notes:				

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND, loose, moist
			1			
			2			-medium dense
			3			
			4			-dense
	10.3		5			-36.1% fines, refusal at 5'
						Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A12

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-12
Excavation Contactor: NW Excavating			Ground Surface Elevation: 415'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Red brown silty fine to medium SAND, loose, moist
			1			
			2			-medium dense
			3			-brown
	9.9		4			-dense to very dense
	12.5		5			
			6			-refusal at 6'
						Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A13

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No.: 10683	Logged by: SSR	Date: 7/11/03	Test Pit No.: TP-13
Excavation Contactor: NW Excavating			Ground Surface Elevation: 420'
Notes:			

Surface Conditions: Depth of Forest Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND, loose, moist
			1			
			2			-medium dense
			3			
	10.3		4			-44% fines
			5			-very dense
	13.4					-refusal at 6'
			6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A14

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-101
Excavation Contactor: NW Excavating			Ground Surface Elevation: 253'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 9"
					ML	Brown SILT with sand, dense, moist
			1			-iron oxide staining
			2			-contains gravel
			3			-becomes very dense
			4			-74.6% fines
	16.4		5			
			6			
			7			-trace polished cobbles
	14.3		8			-increase in sand
			9			-becomes gray
	12.5		10			-55.6% fines
						Test pit terminated at 10.5 feet below existing grade. No groundwater encountered during excavation. NOTES: Test pits excavated by NW Excavating using a CASE 901B track-hoe. Test pit elevations based on topographic data on Site Plan provided by the Client.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A15

Test Pit Log

Project Name: Wood Trails				Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-102	
Excavation Contactor: NW Excavating			Ground Surface Elevation: 248'	
Notes:				

Surface Conditions: Depth of Topsoil & Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
			0		SM	Brown silty SAND, medium dense, moist
			2			-contains gravel and trace cobbles -root mass to 2.5'
			3			-becomes light brown and very dense -iron oxide staining
	14.3		6			
			7		ML	Grades to sandy SILT, medium dense, moist -60.2% fines
	14.5		9			
	17.2		10.5			
			11			Test pit terminated at 11.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A16

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-103
Excavation Contactor: NW Excavating			Ground Surface Elevation: 265'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 4"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					ML	Brown sandy SILT, medium dense, moist
			1			
	19.8		2			-iron oxide staining -contains gravel -52.3% fines
			3			-becomes light brown, very dense
			4			
			5			
	15.3		6			
			7			
	17.2		8			
						Test pit terminated at 8.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A17

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-104
Excavation Contactor: NW Excavating			Ground Surface Elevation: 255'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty SAND, medium dense, moist
			1			
			2			-becomes light brown -iron oxide staining -caving due to seepage -41.6% fines
			3			
	21.1		4			
			5			
			6			
			7			-becomes dense -increase in gravel, trace cobbles and boulders
			8			
	9.9		9			-becomes gray and very dense
						Test pit terminated at 9.5 feet below existing grade. Groundwater seepage encountered at 4.0 feet during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A18

Test Pit Log

Project Name: Wood Trails				Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-105	
Excavation Contactor: NW Excavating			Ground Surface Elevation: 264'	
Notes:				

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"
					ML	Light brown sandy SILT, medium dense, moist
			1			
			2			-iron oxide staining, contains gravel
	16.1		3			
			4			-becomes dense
			5			
	14.4		6			-57.8% fines
			7			
			8			
	15.9		9			
			10			Test pit terminated at 10.0 feet below existing grade. No groundwater encountered during excavating.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A19

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No.: 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-106
Excavation Contactor: NW Excavating			Ground Surface Elevation: 344'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty SAND, medium dense, moist
			1			
			2			-iron oxide staining -contains gravel
	16.1		3			-becomes dense
			4			
	12.8		5			Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A20

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-107
Excavation Contactor: NW Excavating			Ground Surface Elevation: 344'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"
					SM	Brown silty SAND, medium dense, moist
			1			
			2			-iron oxide staining -contains gravel
	16.6		3			-becomes light brown and dense
	15.3		4			
			5			Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A21

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-108
Excavation Contactor: NW Excavating			Ground Surface Elevation: 394'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"
					SM	Brown silty SAND with gravel, medium dense, moist
			1			
			2			-iron oxide staining -trace cobbles
	19.1		3			-becomes dense, 23.5% fines
	23.7		4			-increase in moisture -increase in gravel and cobbles
			5			
						Test pit terminated at 5.5 feet below existing grade. No groundwater encountered during excavation.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A22

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-109
Excavation Contactor: NW Excavating			Ground Surface Elevation: 408'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty SAND, loose to medium dense, moist
			1			
						-iron oxide staining
			2			-contains gravel and trace cobbles
						-becomes light brown and dense
	17.5		3			-24.0% fines
			4			
			5			Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A23

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-110
Excavation Contactor: NW Excavating			Ground Surface Elevation: 403'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 6"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty SAND, loose to medium dense, moist
			1			
			2			-iron oxide staining -contains gravel and trace cobbles
			3			-becomes light brown -becomes dense
	16.6		4			-27.5% fines
			5			Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A24

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: STS	Date: 2/16/04	Test Pit No.: TP-111
Excavation Contactor: NW Excavating			Ground Surface Elevation: 391'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"
					SM	Brown silty SAND, loose to medium dense, moist
			1			-iron oxide staining
			2			-contains gravel and trace cobbles
	14.8		3			-becomes light brown, 23.2% fines
			4			-becomes dense
			5			Test pit terminated at 5.0 feet below existing grade. No groundwater encountered during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A25

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 2
Job No. 10683	Logged by: RAC	Date: 4/6/04	Test Pit No.: TP-201
Excavation Contactor: NW Excavating			Ground Surface Elevation: 257'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"- 8"
					SM	Brown silty fine to medium SAND with gravel, medium dense, wet
			1			
			2			
			3			
			4			-moderately to heavy seepage zone at 4'- 6'
			5			
			6		SM	Becomes unweathered TILL, dense to very dense, moist
			7			
			8			
			9			-very dense, blue gray
			10			
			11			
			12			
			13			
			14			
			15			
			16			
			17			
			18			
			19			-increasing fine sand, possible seepage, medium dense to dense, moist to wet

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A26

Test Pit Log

Project Name: Wood Trails			Sheet 2 of 2
Job No. 10683	Logged by: RAC	Date: 4/6/04	Test Pit No.: TP-201
Excavation Contactor: NW Excavating			Ground Surface Elevation: 257'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	
			21		SM	Gray fine SAND, medium dense to dense, moist to wet
			22			Test pit terminated at 22.0 feet below existing grade. Groundwater seepage encountered at 4.0 - 6.0 feet during excavation.

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A27

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

Test Pit Log

Project Name: Wood Trails				Sheet 1 of 1
Job No. 10683	Logged by: RAC	Date: 4/6/04	Test Pit No.: TP-202	
Excavation Contactor: NW Excavating			Ground Surface Elevation: 376'	
Notes:				

Surface Conditions: Depth of Topsoil & Duff 6"- 8"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND with gravel, loose to medium dense, moist
			1			
			2			
			3			
			4			-becomes dense, cemented till
			5			-very dense
			6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A28

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: RAC	Date: 4/6/04	Test Pit No.: TP-203
Excavation Contactor: NW Excavating			Ground Surface Elevation: 382'
Notes:			

Surface Conditions: Depth of Topsoil & Duff 8"

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	Brown silty fine to medium SAND with gravel, loose, moist to wet
			1			
			2			-becomes medium dense
			3			
			4			-becomes gray, cemented till
			5			
			6			
						Test pit terminated at 6.5 feet below existing grade. No groundwater encountered during excavation.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A29

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Test Pit Log

Project Name: Wood Trails			Sheet 1 of 1
Job No. 10683	Logged by: RAC	Date: 4/6/04	Test Pit No.: TP-204
Excavation Contactor: NW Excavating			Ground Surface Elevation: 382'
Notes:			

General Notes	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Surface Conditions: Depth of Topsoil & Duff 6"- 8"
					SM	Brown silty fine to medium SAND with gravel, loose to medium dense, moist
			1			
			2			
			3			-becomes dense, cemented till, gray
			4			-very dense
			5			
			6			Test pit terminated at 6.0 feet below existing grade. No groundwater encountered during excavation.

TEST PIT LOG 10683.GPJ ECI.GDT 4/14/04

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Test Pit Log
Wood Trails
Woodinville, Washington

Proj. No. 10683	Dwn. GLS	Date April 2004	Checked RAC	Date 4/14/04	Plate A30

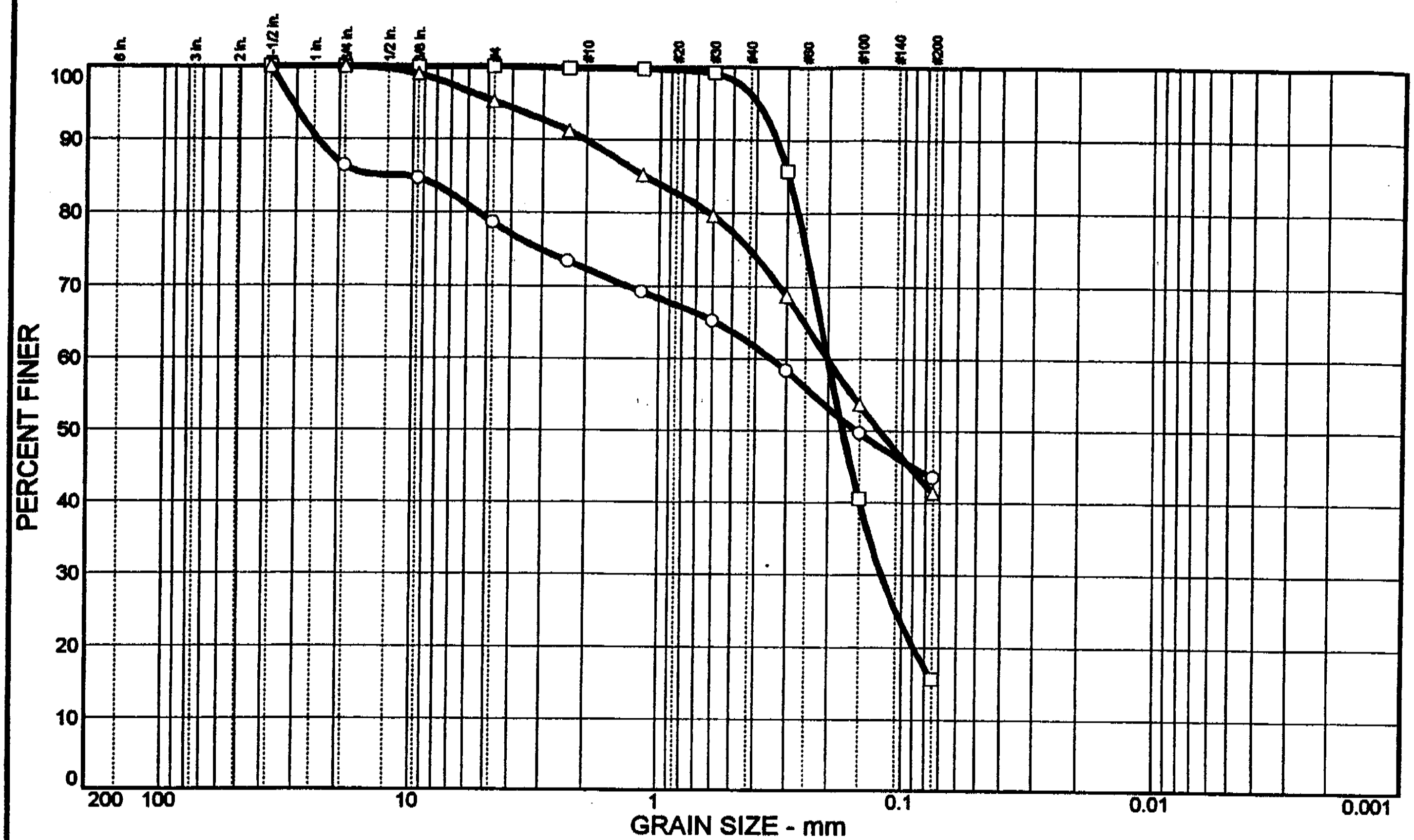
Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

APPENDIX B

LABORATORY TESTING RESULTS

E-10683

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		21.4	35.1	43.5		SM			
□			84.3	15.7		SM			
△		4.8	53.8	41.4		SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	100.0
3/4	86.4	100.0	100.0
3/8	84.7	100.0	99.0

GRAIN SIZE	○	□	△
D_{60}	0.341	0.201	0.200
D_{30}		0.121	
D_{10}			

COEFFICIENTS	○	□	△
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	78.6	100.0	95.2
#8	73.4	99.9	91.2
#16	69.3	99.8	85.2
#30	65.3	99.3	79.6
#50	58.5	85.8	68.8
#100	49.7	40.5	53.7
#200	43.5	15.7	41.4

SOIL DESCRIPTION

○ TP-1: 8' - SM
Silty Sand w/ gravel; 16.4% moisture

□ TP-4: 10' - SM
Silty Sand; 16.6% moisture

△ TP-6: 3' - SM
Silty Sand; 9.2% moisture

REMARKS:

○ tech: SSR/CC

□ tech: SSR/CC

△ tech: SSR/CC

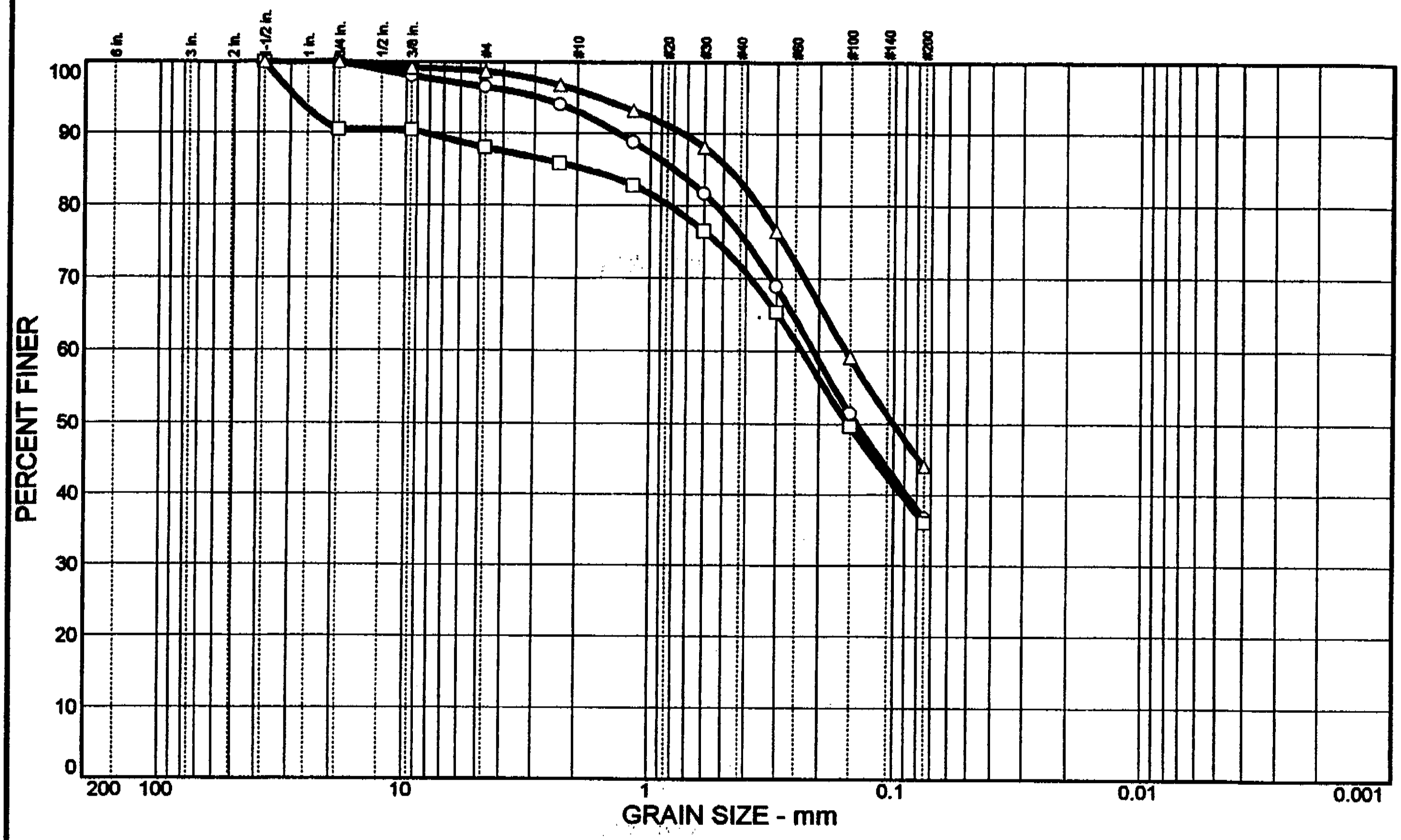
○ Source: Sample No.: TP-1 Elev./Depth: 8'
□ Source: Sample No.: TP-4 Elev./Depth: 10'
△ Source: Sample No.: TP-6 Elev./Depth: 3'

EARTH CONSULTANTS, INC.

Client:
Project: Sundquist Property
Project No.: E-10683

Plate B1

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		3.5	59.7	36.8		SM			
□		12.0	51.9	36.1		SM			
△		1.3	54.7	44.0		SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	100.0
3/4	100.0	90.5	100.0
3/8	98.1	90.5	99.2
	GRAIN SIZE		
D_{60}	0.210	0.234	0.155
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	96.5	88.0	98.7
#8	94.1	85.9	96.8
#16	88.8	82.8	93.2
#30	81.7	76.6	88.1
#50	69.0	65.5	76.6
#100	51.5	49.7	59.2
#200	36.8	36.1	44.0

SOIL DESCRIPTION

○ TP-10: 4' - SM
Brown silty Sand; 10.6% moisture

□ TP-11: 5' - SM
Silty Sand; 10.3% moisture

△ TP-13: 4' - SM
Silty Sand; 10.3% moisture

REMARKS:

○ tech: SSR/CC

□ tech: SSR/CC

△ tech: SSR/CC

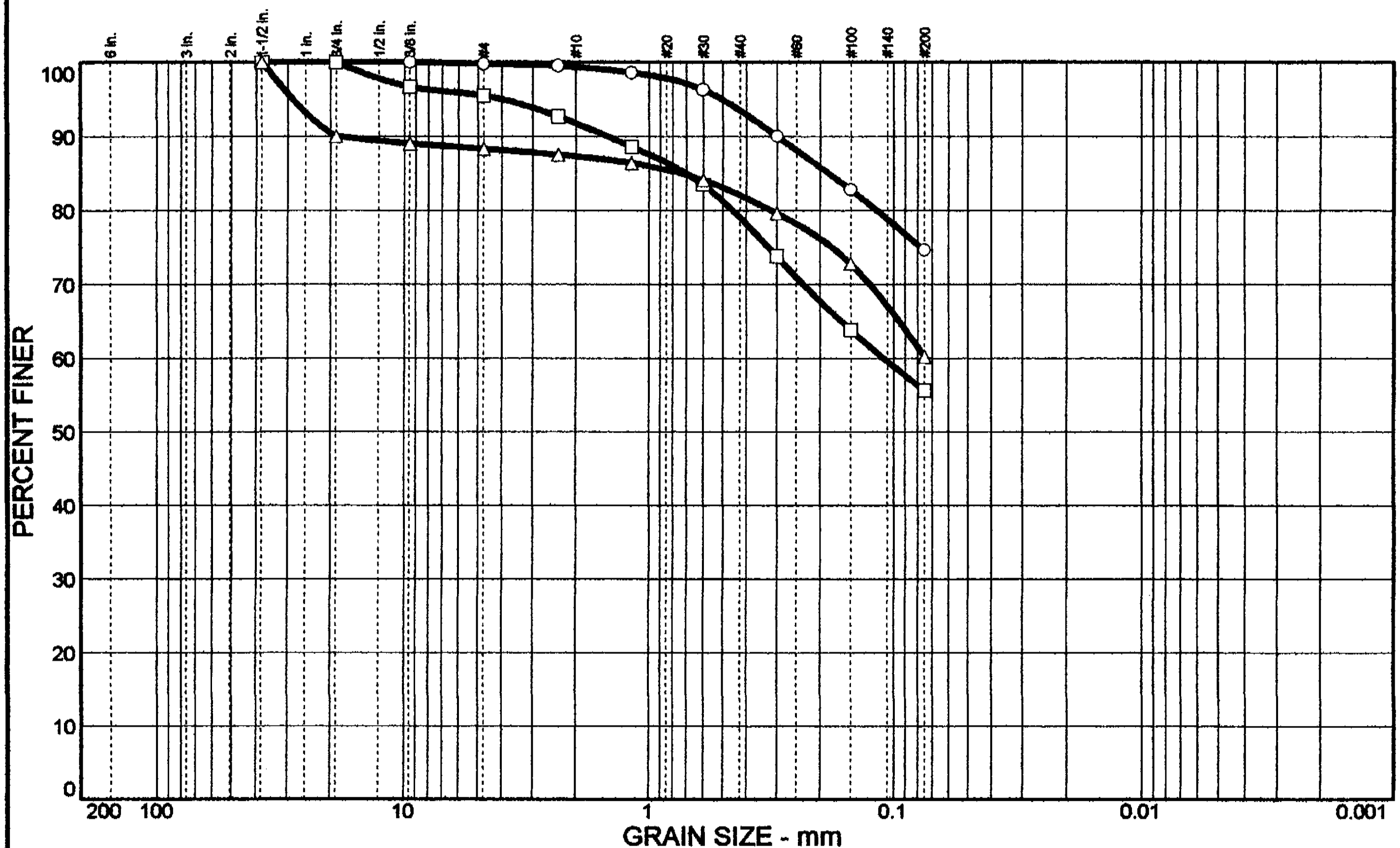
○ Source: Sample No.: TP-10 Elev./Depth: 4'
□ Source: Sample No.: TP-11 Elev./Depth: 5'
△ Source: Sample No.: TP-13 Elev./Depth: 4'

EARTH CONSULTANTS, INC.

Client:
Project: Sundquist Property
Project No.: E-10683

Plate B2

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		0.2	25.2	74.6		ML			
□		4.5	39.9	55.6		ML			
△		11.7	28.1	60.2		ML			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	100.0
3/4	100.0	100.0	90.0
3/8	100.0	96.7	89.0
	GRAIN SIZE		
D_{60}		0.110	
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	99.8	95.5	88.3
#8	99.5	92.7	87.5
#16	98.6	88.6	86.4
#30	96.3	83.5	84.1
#50	90.0	73.8	79.6
#100	82.8	63.8	72.8
#200	74.6	55.6	60.2

SOIL DESCRIPTION

○ TP-101: 5' - ML
Brown Silt w/sand; 16.4% moisture

□ TP-101: 10' - ML
Gray sandy Silt; 12.5% moisture

△ TP-102: 9' - ML
Light brown Silt w/sand; 14.5% moisture

REMARKS:

○ STS

□ STS

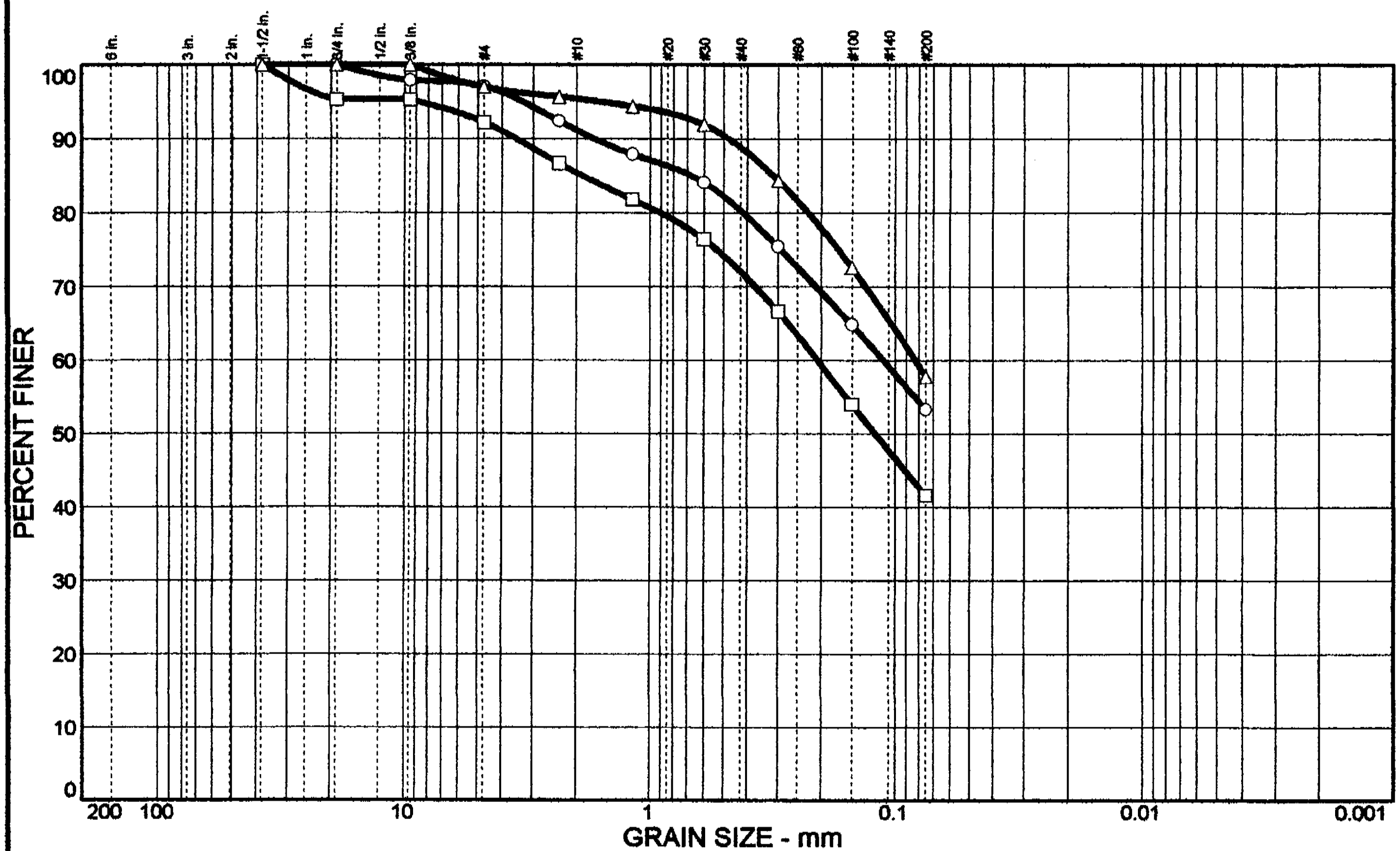
△ STS

○ Source: Sample No.: TP-101 Elev./Depth: 5'
□ Source: Sample No.: TP-101 Elev./Depth: 10'
△ Source: Sample No.: TP-102 Elev./Depth: 9'

EARTH CONSULTANTS, INC.

Client:
Project: Wood Trails
Project No.: E-10683 Plate B3

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		2.9	43.8	53.3		ML			
□		7.8	50.6	41.6		SM			
△		3.0	39.2	57.8		ML			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	100.0
3/4	100.0	95.3	100.0
3/8	97.9	95.3	100.0
	GRAIN SIZE		
D_{60}	0.112	0.207	0.0828
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	97.1	92.2	97.0
#8	92.4	86.7	95.7
#16	87.9	81.8	94.4
#30	84.1	76.4	91.9
#50	75.4	66.6	84.4
#100	64.8	54.0	72.6
#200	53.3	41.6	57.8

SOIL DESCRIPTION

○ TP-103: 2' - ML
Brown sandy Silt; 19.8% moisture

□ TP-104: 4' - SM
Light brown silty Sand; 21.1% moisture

△ TP-105: 6' - ML
Light brown sandy Silt; 14.4% moisture

REMARKS:

○ STS

□ STS

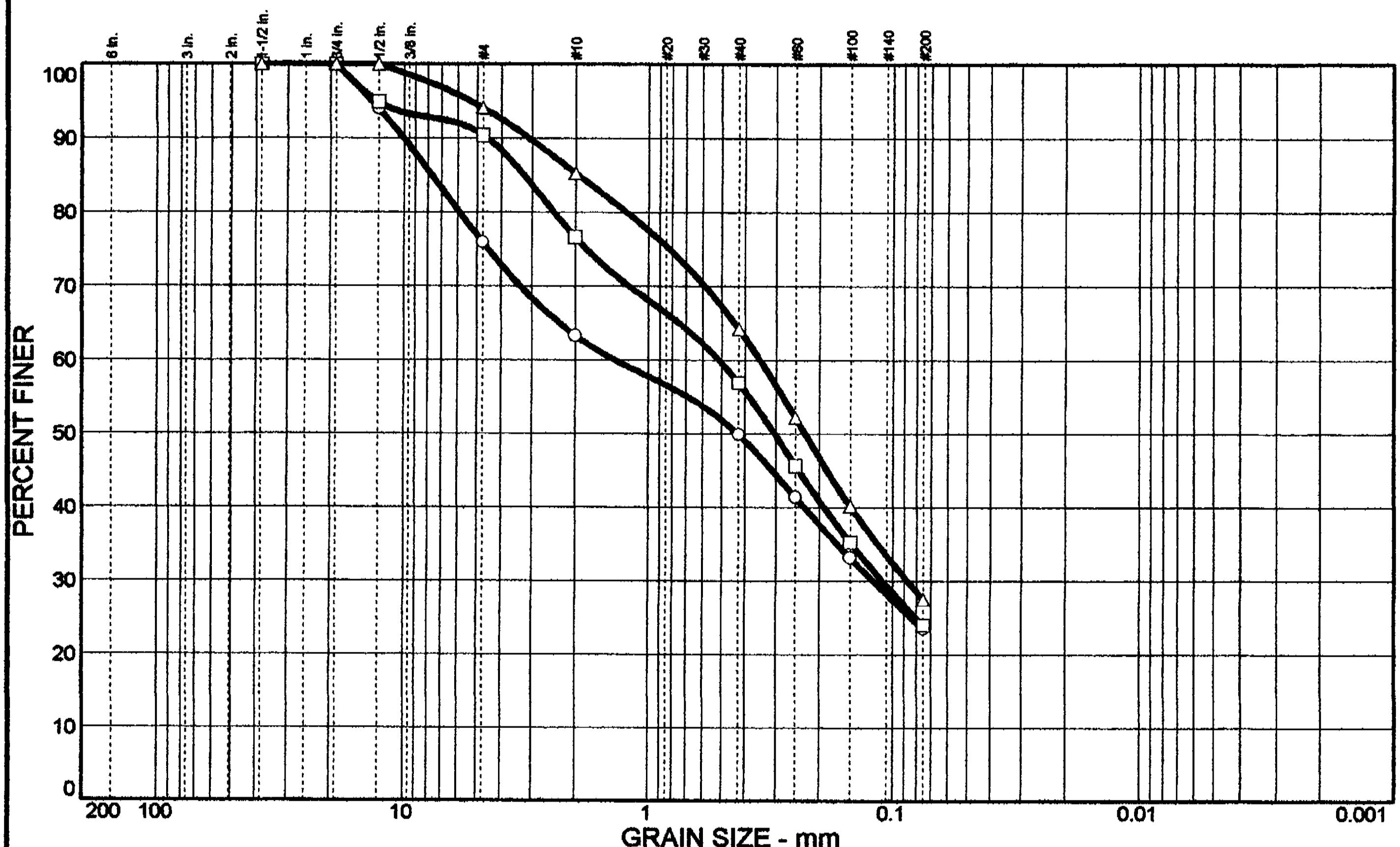
△ STS

○ Source: Sample No.: TP-103 Elev./Depth: 2'
□ Source: Sample No.: TP-104 Elev./Depth: 4'
△ Source: Sample No.: TP-105 Elev./Depth: 6'

EARTH CONSULTANTS, INC.

Client:
Project: Wood Trails
Project No.: E-10683 Plate B4

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		24.1	52.4	23.5		SM			
□		9.6	66.4	24.0		SM			
△		5.9	66.6	27.5		SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	100.0
0.75	100.0	100.0	100.0
0.5	94.0	94.9	100.0

GRAIN SIZE	○	□	△
D_{60}	1.37	0.512	0.349
D_{30}	0.120	0.110	0.0870
D_{10}			

COEFFICIENTS	○	□	△
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	75.9	90.4	94.1
#10	63.3	76.6	85.3
#40	49.9	56.9	64.2
#60	41.4	45.7	52.2
#100	33.2	35.3	40.2
#200	23.5	24.0	27.5

SOIL DESCRIPTION

○ TP-108: 3' - SM
Brown silty Sand w/gravel; 19.1% moisture

□ TP-109: 3' - SM
Brown silty Sand; 17.5% moisture

△ TP-110: 4' - SM
Light brown silty Sand; 16.6% moisture

REMARKS:

○ STS

□ STS

△ STS

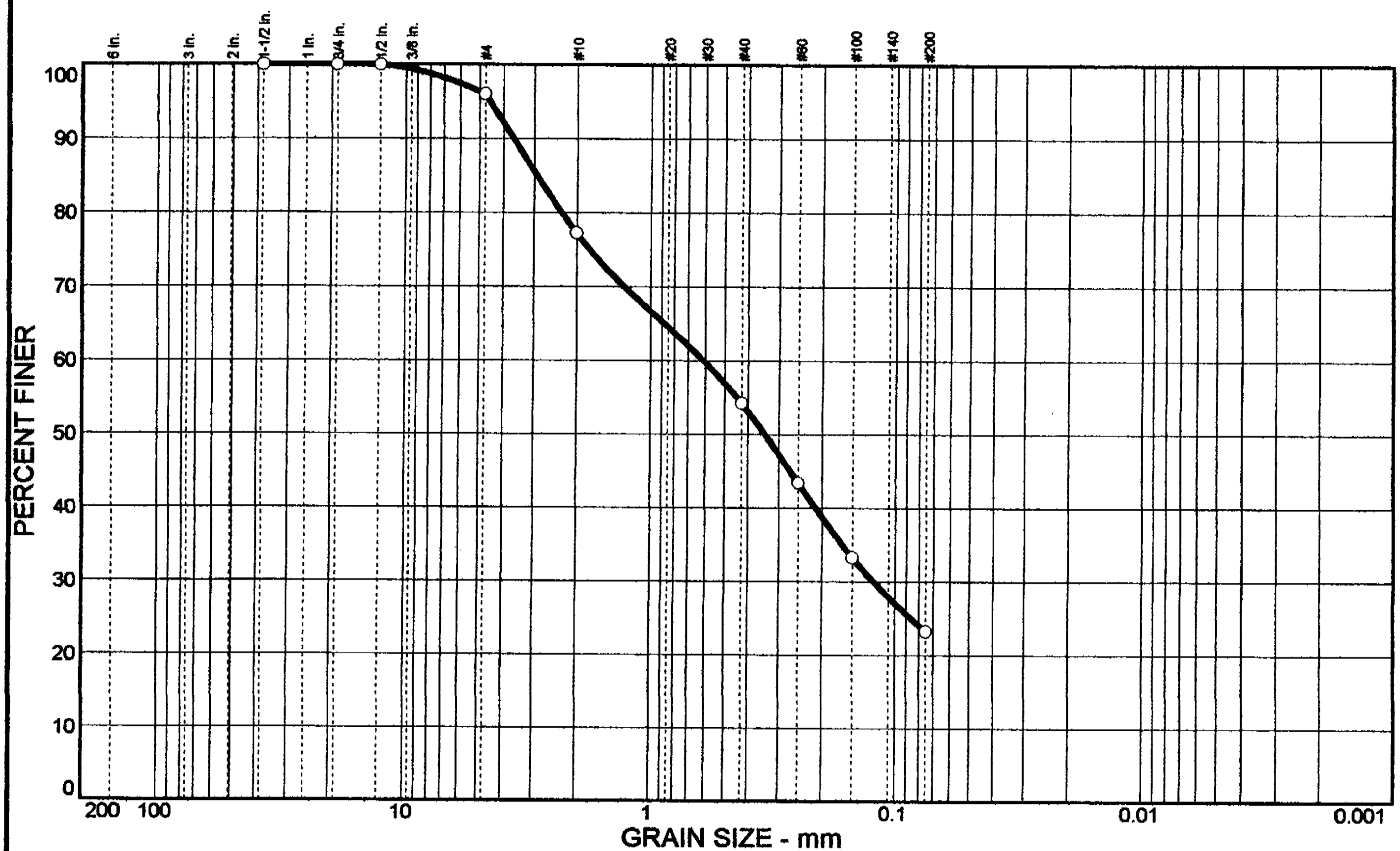
○ Source:	Sample No.: TP-108	Elev./Depth: 3'
□ Source:	Sample No.: TP-109	Elev./Depth: 3'
△ Source:	Sample No.: TP-110	Elev./Depth: 4'

EARTH CONSULTANTS, INC.

Client:
Project: Wood Trails
Project No.: E-10683

Plate B5

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		4.0	72.8	23.2		SM			

SIEVE inches size	PERCENT FINER ○		
1.5	100.0		
0.75	100.0		
0.5	100.0		

	GRAIN SIZE		
D_{60}	0.609		
D_{30}	0.123		
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#4	96.0		
#10	77.2		
#40	54.2		
#60	43.4		
#100	33.3		
#200	23.2		

SOIL DESCRIPTION

○ TP-111: 3' - SM
Light brown silty Sand; 14.8% moisture

REMARKS:

○ STS

○ Source: Sample No.: TP-111 Elev./Depth: 3'

EARTH CONSULTANTS, INC.	Client: Project: Wood Trails Project No.: E-10683 Plate B6

APPENDIX C

ASSOCIATED ROCKERY CONTRACTORS (ARC)

STANDARD ROCKERY CONSTRUCTION GUIDELINES

E-10683

ARC

Associated Rockery Contractors

P.O. Box 1794 Woodinville, Washington 98072
(206) 481-3456 or (206) 481-7222

ASSOCIATED ROCKERY CONTRACTORS
STANDARD ROCKERY CONSTRUCTION GUIDELINES

1.01 Introduction:

1.01.1 Historical Background: These standard rockery construction guidelines have been developed in an effort to provide a more stringent degree of control on rockery materials and construction methodology in the Pacific Northwest. They have been assembled from numerous other standards presently in use in the area, from expertise provided by local geotechnical engineers, and from the wide experience of the members of the Association of Rockery Contractors (ARC).

1.01.2 Goal: The primary goals of this document are to standardize the methods of construction for rockery walls over four feet in height, and to provide a warranty for the materials used in construction and the workmanship employed in construction. This standard has also been developed in a manner that makes it, to the best of ARC's knowledge, more stringent than the other standards presently in use by local municipalities.

2.01 Materials:

2.01.1 Rock Quality: All rock shall be sound, weathering resistant, angular ledge rock. The longest dimension of any individual rock should not exceed three times its shortest dimension. Acceptability of rock will be determined by laboratory tests as hereinafter specified, geologic examination and historical usage records.

All rock delivered to and incorporated in the project shall meet the following minimum specifications:

a. Absorption	*Not more than 2.0% for igneous and metamorphic rock types. Not more than 3.0% for sedimentary rock types.*
*b. Accelerated Expansion (15 days) (CRD-C-148) *1, *2*	*Not more than 15% breakdown*
c. Soundness (MgSO4 at 5 cycles) (CRD-C-137)	*Not greater than 5% loss*
d. Unconfined Compressive Strength ASTM D 2938-79 (reapproved 1979)	*Intact strength of 15,000 psi, or greater for igneous and metamorphic rocks, and 8000 psi or greater for sedimentary rock.*

**1. The test sample will be prepared and tested in accordance with Corps of Engineers Testing procedure CRD-C-148, "Method of Testing Stone for Expansive Breakdown on Soaking in Ethylene Glycol." Test requirements of not more than 15 percent breakdown will be computed by dividing the number of individual pieces of initial sample suffering breakdown (that is, separating into two or more pieces) by the total number of initial pieces in the sample.*

**2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock. Many problems associated with rockery failures are related to the rock fractures and veins found within the rock and not the rock itself.*

2.01.2 Frequency of Testing: Quarry sources for rockery rock shall begin a testing program when either becoming a supplier or when a new area of the source pit is opened. The tests described in Section 2.01.1 shall be performed for every four thousand (4000) tons for the first twelve thousand (12000) tons of material blasted and removed to establish that specific rock source. The tests shall then be performed once a year or at an apparent change in material. If problems with a specific area in a pit or with a particular material are encountered, the initial testing cycle shall be restarted.

2.01.3 Rock Density: Recognizing that numerous sources of rock exist, and that the nature of rock will vary not only between sources but also within each source, the density of the rock shall be greater than one hundred fifty-five (155) pcf. Typically, rocks used for rockery construction shall be sized approximately as follows:

Rock Size	Rock Weight
Small to large one man	50-200 pounds
Small to large two man	200-700 pounds
Small to large three man	700-2000 pounds
Small to large four man	2000-4000 pounds
Five Man	4000-6000 pounds
Six Man	6000-8000 pounds

Two and one-man rock, and sometimes smaller, are often used to fill surface gaps along the top of the completed rockery to create an aesthetically pleasing surface. This is an acceptable practice provided none of the events described in Section 3.01.5 occur, and that the owner prevents people from climbing or walking on the completed rockery.

In rockeries over eight feet in height, it should not be possible to move the large sized rocks (four to six-man size) with a prybar. If these rocks can be moved, the rockery should not be considered capable of restraining any significant lateral load. However, it is both practical and even desirable that smaller rocks, particularly those used for "chinking" purposes, can be moved with a prybar to achieve the "best fit".

2.01.4 Submittals: The rock source shall present current geologic and test data for the testing for the minimum guidelines described in Section 2.01.1 on request by either the rockery contractor, the client, or the applicable municipality.

3.01 Rockery Construction:

3.01.1 General: Rockery construction is a craft and depends largely on the skill and experience of the builder. A rockery is a protective system which helps to retard the weathering and erosion process on an exposed cut or fill soil face. While by its nature (the mass, size and shape of the rocks) it will provide some degree of retention, it is not a designed or engineered system in the sense a reinforced concrete retaining wall would be considered designed or engineered. The degree of retention achieved is dependant on the size of rock used; that is, the mass or weight, and the height of the wall being constructed. The larger the rock, the more competent the wall. To accomplish this, all rockeries in excess of four feet in height should be built on a "mass" basis.

To provide a competent and adequate rockery structure, all rockeries constructed in front of either cuts or fills in excess of eight feet in height should be bid and constructed in accordance with these standard guidelines and the geotechnical engineers supplemental recommendations. Both the standard guidelines and the supplemental geotechnical recommendations should be provided to prospective bidders before bidding and the start of construction.

The same geotechnical engineer should be retained to monitor rockery construction and to verify, in writing, that the rockery was constructed in general accordance with this ARC standard and with his supplemental recommendations, in a professional manner and of competent and suitable materials.

3.01.2 Geotechnical Engineer: The geotechnical engineer retained to provide necessary supplemental rockery construction guidelines shall be a practicing geotechnical/civil engineer licensed as a professional civil engineer in the State of Washington who has at least four years of professional employment as a geotechnical engineer in responsible charge, including experience with fill construction and stability and rockery construction. The geotechnical engineer should be hired either by the rockery contractor or the client.

3.01.3 Responsibility: The ultimate responsibility for rockery "design" and construction should remain with the rockery builder. However, rockeries protecting moderate to thick fills, with steep sloping surfaces above or below them, with multiple steps, with foundation or other loads affecting them, protecting sandy or gravelly soils subject to ravelling, with seepage or wet conditions, or that are more than eight feet in height, all represent special conditions and require consultation and/or advice from qualified experts.

3.01.4 Workmanship: All workmanship is guaranteed by the rockery contractor and all materials are guaranteed by supplying quarry for a period of six years from the date of completion of erection, providing no modification or changes to the conditions existing at the time of completion are made.

3.01.5 Changes to Finished Product: Such changes include, but are not necessarily limited to, excavation of ditches or trenches within a distance of less than 1.5 times the rockery height measured from the toe of the rockery, removal of any material from the subgrade in front of the rockery, excavation and/or removal of material from any location behind the rockery within a distance at least equal to the rockery's height, the addition of any surcharge or other loads within a similar distance of the top of the rockery, or surface or subsurface water forced, directed, or otherwise caused to flow behind the rockery in any quantity.

3.01.6 Slopes: Slopes above rockeries should be kept as flat as possible, but should not exceed 2:1 (Horizontal:Vertical) unless the rockery is designed specifically to provide some restraint to the load imposed by the slope. Any slope existing above a completed rockery should be provided with a vegetative cover by the owner to help reduce the potential for surface water flow induced erosion. It should consist of a deep rooted, rapid growth vegetative mat and typically will be placed by hydroseeding and covered with a mulch. It is often useful to overlay the seed and mulch with either pegged in-place jute matting, or some other form of approved geotechnical fabric, to help maintain the seed in-place until the root mat has an opportunity to germinate and take hold.

3.01.7 Monitoring: All rockeries constructed against cuts or fills in excess of eight feet in height shall be periodically monitored during construction by the geotechnical engineer to verify the nature and quality of the materials being used are appropriate, that the construction procedures are appropriate, and that the wall is being constructed in a generally professional manner and in accordance with this ARC standards and any supplemental recommendations.

On completion of the rockery, the geotechnical engineer shall submit to the client, the rockery contractor, and to the appropriate municipality, copies of his rockery examination reports along with a final report summarizing rockery construction.

3.01.8 Fill Compaction: Where rockeries are constructed in front of a fill, it is imperative that the owner ensure the fill be placed and compacted in a manner that will provide a competent fill mass. To achieve this goal, all fills should consist of relatively clean, organic and debris free, granular materials with a maximum size of four inches. Ideally, but particularly if placement and compaction is to take place during the wet season, they should contain no more than five percent fines (silt and clay size particles passing the number 200 mesh sieve).

All fills should be placed in thin lifts not exceeding ten inches in loose thickness. Each lift should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM Test Method D-1557-78 (Modified Proctor), before any additional fill is placed and compacted. In-place density tests should be performed at random locations within each lift of the fill to verify this degree of compaction is being achieved.

3.01.9 <u>Fill Construction and Reinforcement</u>: There are two methods of constructing a fill against which to build a rockery. The first, which typically applies to rockeries of less than eight feet in height, is to overbuild and then cut back the fill. The second, which applies to all rockeries in excess of eight feet in height, is to construct the fill using a geogrid or geotechnical fabric reinforcement.

Overbuilding the fill allows for satisfactory compaction of the fill mass out beyond the location of the fill face to be protected. Overbuilding also allows the earthwork contractor to use larger and more effective compaction equipment in his compactive efforts, thereby typically achieving a more competent fill mass. Cutting back into the well compacted fill also typically results in construction of a competent near vertical fill face against which to build the rockery.

For the higher rockeries the use of a geogrid or geotechnical fabric to help reinforce the fill results in construction of a more stable fill face against which to construct the rockery. This form of construction leads to a longer lasting and more stable rockery and helps reduce the risk of significant long term maintenance.

This latter form of construction requires a design by the geotechnical engineer for each specific case. The vertical spacing of the reinforcement, the specific type of reinforcement, and the distance to which it must extend back into the fill, and the amount of lapping must be determined on a rockery-by-rockery basis.

3.01.10 <u>Rockery Keyway</u>: The first step in rockery construction, after general site clearing and/or general excavation, is to construct a keyway in which to build the rockery. The keyway shall comprise a shallow trench of between twelve (12) and eighteen (18) inches in depth, extending for the full length of the rockery, and inclined back slightly towards the face being protected. It is typically dug as wide as the rockery (including the width of the rock filter layer).

If the condition of the protected face is of concern, the keyway should be constructed in sections of manageable length, that is of a length that can be constructed in one shift or one days work.

The competency of the keyway subgrade to support the rockery shall be verified by probing with a small diameter steel rod. The rod shall leave a diameter of between three-eights and one-half inch, and shall be pushed into the subgrade in a smooth unaided manner under the body weight of the prober only.

Penetration of up to six inches, with some difficulty, shall indicate a "competent" keyway subgrade unless other factors in the geotechnical engineer's opinion shall indicate otherwise. Penetration in excess of six inches, or of that depth with ease, shall indicate a "soft" subgrade and one that could require treatment. Soft areas of the subgrade can be "firmed up" by tamping a layer of coarse quarry spalls into the subgrade.

3.01.11 <u>Keyway and Rockery Drainage</u>: On completion of keyway excavation, a shallow ditch or trench, approximately twelve (12) inches wide and deep, should be dug along the rear edge of the keyway. A minimum four-inch diameter perforated or slotted ADS drain pipe, or equivalent approved by an engineer, should be placed in this shallow trench and should be bedded on and surrounded by a free-draining crushed rock. Burial of the drain pipe in this shallow trench provides protection to the pipe and helps prevent it from being inadvertently crushed by pieces of the rockery rock. This drain pipe should be installed with sufficient gradient to initiate flow, and should be connected to a positive and permanent discharge.

Positive and permanent drainage should be considered to mean an existing, or to be installed, storm drain system, a swale, ditch or other form of surface water flow collection system, a detention or retention pond, or other stable native site feature or previously installed collection system.

3.01.12 <u>Rockery Thickness</u>: The individual rockery thickness, including the rock filter layer, should be at least 40 percent of the rockery height. Unless otherwise specified in writing, the individual rocks should be arranged in a single course which, when measured to include the filter layer, is equal to the required rockery thickness.

3.01.13 Rock Selection: The contractor should have sufficient space available so that he can select from among a number of stockpiled rocks for each space in the rockery to be filled. Rocks which have shapes which do not match the spaces offered by the previous course of rock should be placed elsewhere to obtain a better fit. Rock should be of a generally cubical, tabular or semi-rectangular shape. Any rocks of basically rounded or tetrahedral form should be rejected or used for filling large void spaces.

Smaller rocks (one to two-man size, or smaller) are often used to create an aesthetically pleasing "top edge" to a rockery. This is acceptable provided none of the events described in Section 3.01.5 occur, and that people are prevented from climbing or walking on the finished rockery. This is the owner's responsibility.

3.01.14 Rock Placement: The first course of rock should be placed on firm unyielding soil. There should be full contact between the rock and soil, which may require shaping of the ground surface or slamming or dropping the rocks into place so that the soil foundation conforms to the rock face bearing on it. As an alternative, it is satisfactory to place and tamp crushed rock into the subgrade to tighten it up. The bottom of the first course of rock should be a minimum of twelve (12) inches below the lowest adjacent site grade.

As the rockery is constructed, the rocks should be placed so that there are no continuous joint planes in either the vertical or lateral direction. Each rock should bear on at least two rocks below it. Rocks should be placed so that there is some bearing between flat rock faces rather than on joints. Joints between courses should slope downward towards the material being protected (away from the face of the rockery).

3.01.15 Face Inclination: The face of the rockery should be inclined at a gradient of about 1:6 (Horizontal:-Vertical) back towards the face being protected. The inclination should not constructed flatter than 1H:4V.

3.01.16 Voids: Because of the nature of the product used to construct a rockery, it is virtually impossible to avoid creating void spaces between individual rocks. However, it should be recognized that voids do not necessarily constitute a problem in rockery construction.

Where voids of greater than six inches in dimension exist in the face of a rockery they should be visually examined to determine if contact between the rocks exists within the thickness of the rockery. If contact does exist, no further action is required. However, if there is no rock contact within the rockery thickness the void should be "chinked" with a smaller piece of rock. If a void of greater than six inches exists in the rear face of the rockery, it should be "chinked" with a smaller rock.

3.01.17 Filter Layer: In order to provide some degree of drainage control behind the rockery, and as a means of helping to prevent loss of soil through the face of the rockery, a drainage filter shall be installed layer between the rear face of the rockery and the soil face being protected. This filter layer should be at least twelve (12) inches thick; and for walls in excess of eight feet in height, it should be at least eighteen (18) inches thick. It should be composed of four inch minus crushed rock, or other material approved by the geotechnical engineer.

If one of the rockery rocks extends back to the exposed soil face, it is not necessary that the filter rock layer extend between it and the soil face.

In the event seepage is encountered emanating from a protected face, we recommend the use of a well-graded filter layer. We do not recommend the use of a geotechnical fabric for other than coverage of relatively small and isolated seepage areas because it has been the industry's experience that the filter fabric tends to clog rapidly. This quickly leads to a buildup of hydrostatic pressure which can subsequently cause failure and collapse of the rockery and is to be avoided.

This clogging is apparently due to the virtual impossibility of achieving full contact between the soil face, fabric and rock filter material. If full surface contact cannot be achieved, there is often a tendency for the soil materials to flush from the protected face into the "pockets" in the fabric which leads to the aforementioned clogging.

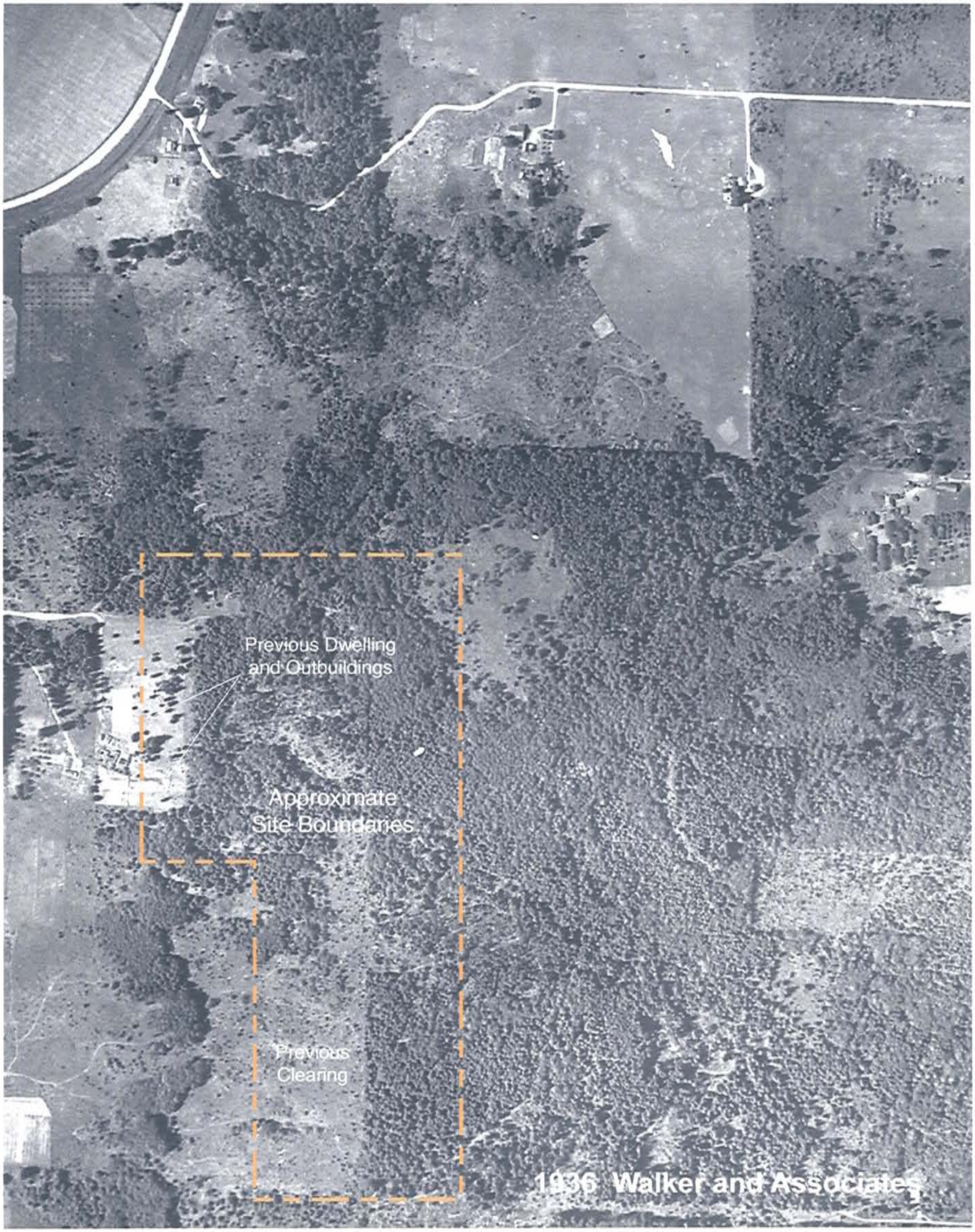
3.01.18 Surface Drainage: It is the owner's responsibility to intercept surface drainage from above the rockery and direct it away from the rockery to a positive and permanent discharge well below and beyond the toe of the wall. Use of other drainage control measures should be determined on a case-by-case basis by the geotechnical engineer prior to bidding on the project.

1/27/89

APPENDIX D

AERIAL PHOTOGRAPHS

E-10683



Walker & Associates Aerial Photo
Dated 1936

Proj. No. 10683
06042004

Walker & Associates Aerial Photo
Dated 1968

Proj. No. 10683
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Walker & Associates Aerial Photo
Dated 1980

Proj. No. 10683
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Walker & Associates Aerial Photo
Dated 2002

Proj. No. 10683
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