FILE COPY

LIMITED GEOTECHNICAL REPORT
COSON RESIDENCE
7709 W. MERCER WAY
MERCER ISLAND, WASHINGTON

Submitted to:

Mid-Mountain Construction, Inc.
P.O. Box 2909
Kirkland, Washington 98083-2909

Submitted by:

AMEC Earth & Environmental, Inc.
11335 N.E. 122nd Way, Suite 100
Kirkland, Washington 98034-6918

January 14, 2002

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Mid-Mountain Construction, Inc.
P.O. Box 2909
Kirkland, Washington 98083-2909

Attention: Mr. Jeff Levere, P.E.

Subject: Limited Geotechnical Report
Coson Residence
7709 W. Mercer Way
Mercer Island, Washington

Dear Jeff:

AMEC Earth & Environmental, Inc. (AMEC) is pleased to submit this report describing our limited geotechnical evaluation for the above-referenced project. The purpose of our evaluation was to derive conclusions and recommendations concerning slope stability and retaining wall options. We previously provided a *Geotechnical Reconnaissance Report* dated June 22, 2001.

As outlined in our proposal letter dated June 27, 2001, our scope of work consisted of limited field explorations, limited laboratory testing, geotechnical research, stability analyses, and report preparation. We received your authorization for our evaluation on July 5, 2001. This report has been prepared for the exclusive use of Mid-Mountain Construction, Inc., the homeowner, and their consultants, for specific application to this project, in accordance with generally accepted geotechnical engineering practice.

1.0 SITE AND PROJECT DESCRIPTION

The project site is an existing waterfront residence on the southwest end of Mercer Island, located at 7709 W. Mercer Way as shown on the enclosed *Location Map* (Figure 1). The residence is situated on an irregularly shaped parcel that measures about 200 feet by 90 feet overall. The site topography is steeply sloping down to the west, with maximum topographic relief between the driveway on the east and Lake Washington on the west of about 75 feet. The enclosed *Site & Exploration Plan* (Figure 2) illustrates the site boundaries and adjacent existing features.

We understand that sometime during the spring of 2001, the water service to the residence broke, resulting in a flow of muddy water down to Lake Washington. During the same time, a landslide occurred on the west side of the residence. We understand the owners would like to repair the area using multiple retaining walls, to create a terraced yard.

The conclusions and recommendations contained in this report are based on our understanding of the currently proposed utilization of the project site, as derived from written and verbal information supplied to us. Consequently, if any changes are made in the currently proposed project, we may need to modify our conclusions and recommendations contained herein to reflect those changes.

2.0 EXPLORATORY METHODS

We explored surface and subsurface conditions at the project site during July 2001. Our exploration and testing program comprised the following elements:

- A visual surface reconnaissance of the site;
- Three borings (designated B-1 through B-3), advanced at strategic locations across the site;
- One test hole (designated TH-1), excavated at the west side of the patio, to establish soil conditions and depth of footing;
- Grain size analyses and moisture content determinations, performed on selected soil samples obtained from our explorations;
- A review of the logs of previous subsurface explorations made as part of geotechnical engineering reports for the initial site development; and
- A review of published geologic and seismologic maps and literature.

Figure 2 depicts the approximate relative locations of our explorations. The following sections describe the procedures used for auger borings and test holes.

The specific number, locations, and depths of our explorations were selected in relation to the existing and proposed site features, under the constraints of surface access, underground utility conflicts, and budget considerations. We estimated the relative location of each exploration by measuring from existing features and scaling these measurements onto a layout plan supplied to us, then we estimated their elevations by interpolating between contour lines shown on this same plan. Consequently, the locations depicted on Figure 2 should be considered accurate only to the degree permitted by our data sources and implied by our measuring methods.

It should be realized that the explorations performed and utilized for this evaluation reveal subsurface conditions only at discrete locations across the project site and that actual conditions in other areas could vary. Furthermore, the nature and extent of any such variations would not become evident until additional explorations are performed or until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations contained in this report to reflect the actual site conditions.

2.1 Auger Boring Procedures

Our exploratory borings were advanced through the soil with a hollow-stem auger, using a portable drill rig operated by an independent drilling firm working under subcontract to AMEC. A geologist from our firm continuously observed the borings, logged the subsurface conditions, and collected representative soil samples. All samples were stored in watertight containers and later transported to our laboratory for further visual examination and testing. After each boring was completed, the borehole was backfilled with a mixture of bentonite chips and soil cuttings.

Throughout the drilling operation, soil samples were obtained at 2½- to 5-foot depth intervals by means of the Standard Penetration Test (SPT) per ASTM:D-1586. This testing and sampling procedure consists of driving a standard 2-inch-diameter steel split-spoon sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "SPT blow count." If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed *Boring Logs* describe the vertical sequence of soils and materials encountered in each boring, based primarily on our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the borings, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth is depicted on the boring log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted height on the drilling rods, and the water level measured in the borehole after the auger has been extracted.

2.2 Test Hole Procedures

Our exploratory test hole was advanced with a shovel by an AMEC geologist, who logged the subsurface conditions and obtained representative soil samples. All samples were stored in watertight containers and later transported to our laboratory for further visual examination and testing. After the test hole was completed, we backfilled it with excavated soils and tamped the surface.

3.0 SITE CONDITIONS

The following sections of text present our observations, measurements, findings, and interpretations regarding surface, soil, groundwater, and seismic conditions at the project site.

3.1 Surface Conditions

During our reconnaissance of the project site on June 20, 2001, we observed that the break in the water line service had been located. An about 1-inch long split in the plastic pipe was observed. Workers with Mid-Mountain construction were repairing the break in the line at the time of our visit. We toured the site, and observed the path of mud below the water line break. The path of mud was down the driveway to the garage, then along the north side of the house, and in to the rear yard. A landslide feature was observed in the rear yard, below which it could be seen where the mud path entered Lake Washington.

The headscarp of the landslide was arcuate-shaped, located as close as 3 feet from the patio in the center of the west side of the house, and located about 10 feet west of both the northwest and southwest corners of the house. The slide mass below the headscarp was observed to have a downset of about 3 feet. We estimate the maximum plan dimension of the slide mass was about 60 feet wide (north-south), by about 55 feet (east-west). A short (4-foot high) timber wall bordered the western edge of the observed movement. About 50 feet of the wall had deformed by the slide mass, and a sight bulge was observed for about two feet in front of (west of) the wall. A level grass bench area (presumably the location of a 10-foot wide sanitary sewer easement) did not exhibit any obvious deformation. West of the level bench, the ground sloped down to a rock bulkhead along the waterfront. Horsetail vegetation was noted on the slope between the level bench and the bulkhead, suggesting the location of springs. No other areas of seepage were noted at the time of our reconnaissance.

We did not observe any damage or cracking of the residence that can be attributed to the landslide. We did note soil had pulled away by about 1 inch from the southern about 5 feet of the patio area, but no distress to the patio was observed. We probed with a long steel rod in this opening in the soil, and were able to probe about three feet to dense soils. A slight tension crack with about a ¼-inch opening was noted in the soil at the south west corner of the house, but no distress to the house or supporting soils was noted. Finally, we observed the roof downspout at the northwest corner of the house may have deflected down by about 1 inch, probably due to the buried outlet pipe being pulled down as the slide mass moved.

3.2 Soil Conditions

We reviewed a *Preliminary Soil Investigation, Lots 3,4 and 5, 7600 Block West Mercer Way*, by Earth Consultants, Inc., dated August 26, 1977 (project number E-309). This report pertains to the general site vicinity, and generally described the underlying soils as glacial till. Numerous wet areas were noted in the lower lakeside portion of the site. We also reviewed another report on the project site by Earth Consultants, Inc. entitled *Geotechnical Investigation Report, Lewis Short Plat, Lots A, B, and C,* dated August 3, 1983, project number E-309-6. It appears that one test pit made on the northeast corner of the residence encountered medium dense to dense silt with sand to sandy silt and no groundwater was observed.

Our on-site explorations revealed somewhat variable near-surface soil conditions but confirmed the mapped stratigraphy. Soils underlying the site generally consist of 2½ to 12½ feet of loose to medium dense silty sand (fill) mantling 2½ to 4 feet of very loose to loose silty sand with some organics (colluvium). Underlying these shallow surficial soils, we encountered medium dense to dense glacial till and very dense sand. The very dense sands deposits were encountered to the full depth explored in our borings.

The enclosed exploration logs provide a detailed description of the soil strata encountered in our subsurface explorations. The enclosed *Cross-Section A-A'* (Figure 3) illustrates our stratigraphic interpretations at a selected location across the project site.

Our geotechnical laboratory tests revealed that site soils generally have a fines content (silt and clay) greater than 20 percent. We interpret these soils to be currently above their optimum moisture contents, and to be moderately sensitive to highly sensitive to moisture content variations. The enclosed laboratory testing sheets graphically present our test results.

3.3 Groundwater Conditions

At the time of drilling (July, 2000), groundwater was encountered in all three of our borings at depths ranging from 13½ to 17½ feet below ground surface. Following drilling of boring B-1, the groundwater rose from the original depth of 17½ feet to a stabilized depth of 11 feet. Because our explorations were performed during an extended period of generally dry weather, these observed groundwater conditions may closely represent the yearly low levels; somewhat higher levels probably occur during the winter and spring months. At all times of the year, groundwater levels would likely fluctuate in response to precipitation patterns, off-site construction activities, lake tides and site utilization.

3.4 Seismic Conditions

Based on our analysis of subsurface exploration logs and our review of published geologic maps, we interpret the on-site soil conditions to correspond to seismic soil profile type S_C and C as defined by Table 16-J of the 1997 *Uniform Building Code* and the 2000 *International Building Code*, respectively. Current (1996) *National Seismic Hazard Maps* prepared by the U.S. Geological Survey indicate that a bedrock site acceleration coefficient of about 0.30 is appropriate for an earthquake having a 10-percent probability of exceedance in 50 years (corresponding to a return interval of 475 years). According to Figure 16-2 of the *1997 Uniform Building Code*, the site lies within seismic risk zone 3.

4.0 SLOPE STABILITY ANALYSES

In order to determine the feasibility of constructing the proposed retaining walls, we analyzed the slope stability under selected conditions: more specifically, we analyzed existing slope stability and global stability when the walls are completed. The following sections describe our method of analysis and present our results.

4.1 Method of Analysis

Slope stability analyses typically involve five basic slope parameters: (1) location and shape of the potential failure surface, (2) internal friction angle of the various soils, (3) cohesion of the various soils, (4) density of the various soils, and (5) location of the piezometric groundwater surface. Unfortunately, few of these parameters are accurately known at the start of an analysis. Instead, these parameters usually must be estimated, interpreted, and/or assumed on the basis of visual observations, field testing, laboratory testing, empirical correlations, and experience with similar soil types.

Once all five parameters have been tentatively established, the critical slip surface and associated safety factor of a given slope can be calculated. A "critical slip surface" is defined as the most likely surface along which a soil mass will slide, and a "safety factor" is defined as the ratio of the sum of all moments resisting slope movement versus the sum of all moments tending to cause slope movement. Consequently, a slope that possesses a safety factor of 1.0 is on the verge of sliding, whereas a slope with a safety factor greater than 1.0 has some resistance to sliding. According to standard geotechnical engineering practice, a static safety factor of 1.50 and a seismic safety factor of 1.10 are considered the desirable minimum values for most slopes, but 1.25 and 1.01, respectively, are often regarded as acceptable values.

Slope stability conditions for the project site were analyzed by means of Bishop's Simplified Method of Slices, which utilizes a limit-equilibrium technique. All calculations were performed by means of the computer program SLOPE-W. This program utilizes topographic, soil, and groundwater information input by the user to determine the most critical slip surface.

Our estimated values of internal friction angle, cohesion, and density for each soil layer are listed in Table 1. By convention, seismic stability conditions are analyzed by applying a horizontal acceleration equal to one-half of the appropriate peak ground acceleration. Based on a peak bedrock acceleration of 0.30g for the site, we utilized a design value of 0.15g.

TABLE 1 ESTIMATED PROPERTIES OF ON-SITE SOILS FOR STABILITY ANALYSIS			
Soil Type	**Density (pcf)**	**Cohesion (psf)**	**Internal Friction Angle (degrees)**
Fill and Colluvium	120	90	17
Colluvium	120	200	32
Stronger Existing Fill	125	150	33
Reinforced Fill	125	300	34
Advance Sands	138	50	45

4.2 Results of Analysis

Utilizing the aforementioned values of internal friction angle, cohesion, and density, we calculated the safety factors associated with numerous slip surfaces. We calculated the safety factors for the existing slope with a shallow and deep seated failure for different possible groundwater conditions. Our analysis indicates that sliding is occurring within the relatively loose fill and colluvium atop the denser soils at depth. The recent movement was apparently initiated by the elevated groundwater conditions stemming from the water leak. The toe of the slide mass appears to coincide with the timber bulkhead at the base of the slope. During our reconnaissance, we did not find any evidence of deep-seated movement. We also calculated the global stability for a finished configuration with the proposed reinforced soil walls. Table 2 summarizes these analytical results.

In summary, the static safety factor of the repaired slope with the retaining walls in place is essentially slightly improved over the current condition. Seismic factors of safety in all cases are less than 1.0. While a higher static and seismic safety factor would be desirable, the construction cost to achieve this is very high in relation to the value of the property and end use of the backyard area.

If the backyard area shifted during an earthquake event, our analysis indicates this would not adversely affect the stability of the residence. If a scarp in the soil reappeared as a result of moderate to high earthquake loading, the slope would need to be regraded so that such a scarp would not regress and undermine the residential foundation.

TABLE 2 CALCULATED SAFETY FACTORS FOR SELECTED CONDITIONS		
Condition	**Sliding Mode**	**Static Safety Factor**
Existing slope (shallow)	Circular	1.04
Existing slope (deep)	Circular	1.11
Existing slope (field measured groundwater table)	Circular	1.17
Constructed walls (high groundwater table)	Circular	1.10
Constructed walls (field measured groundwater table)	Circular	1.17
Note: The seismic safety factor for the backyard area is less than one. However, the seismic safety factors for soils beneath the residence are greater than one (see discussion, Section 4.2).		

5.0 CONCLUSIONS AND RECOMMENDATIONS

Improvement plans call for constructing three new retaining walls in the backyard of the residence in the area that previously experienced slope movement. We offer the following general geotechnical conclusions and recommendations concerning this project.

- Feasibility: Based on our field explorations, research, and analyses, the proposed retaining walls appear feasible from a geotechnical standpoint, contingent on proper design and construction.

- Cause of Landsliding: In our opinion, based on our surface observations and geotechnical research, the primary cause of the recent landslide was the infiltration of large quantities of water into the ground, stemming from the break in the water line above the residence. A contributing cause to the water line break may have been the February 28, 2001 earthquake. It appears that the soils involved in earth movement are the fill soils, which were emplaced atop the relatively dense native soils. Our analysis indicates that sliding is occurring within the relatively loose fill and colluvium atop the denser soils at depth. The toe of the slide mass appears to coincide with the timber bulkhead at the base of the slope. We did not find any evidence of deep-seated movement. In our opinion, the house is not in imminent danger of damage due to landsliding.

- Existing Residence Foundations: The west wall foundation does not appear affected by the recent slope movement. The result of our test hole indicated that the west footing is underlain by native soil, and there was no indication of settling of the footing or soil loss under the footing. Additionally, the footing drain appeared to be installed correctly and functioning.

- Utility Conditions: We understand that Mid-Mountain Construction has performed a video survey of the pipes surrounding the property. We understand that no bowing or breaks were noted in these pipes. This is consistent with our geologic mapping of the site, which indicates that the earth movement occurred as shallow surficial sliding. No evidence of deep-seated movement affecting the lower bench area or surrounding slopes to the north and south was observed in either the pipeline video surveys, in the test borings, or in our surficial mapping.

- Retaining Wall Options: In our opinion, a reinforced soil wall would be suitable for repairing the landscape area in the backyard. However, due to the significant thickness of looser native soils, we recommend that some form of ground improvement or deep foundation system be used for wall support. In our opinion, the most cost-effective approach would be to underpin the facing of each retaining wall using needle piles and a grade beam. Our stability analyses indicate that construction of multiple walls will improve the current stability of the backyard.

- On-Site Soil Reuse: Our visual soil classifications and laboratory testing indicate that the on-site soils are not suitable for reuse as structural fill due to high fines content and over optimum moisture content.

- Subgrade Protection: Due to the moisture-sensitive nature of the on-site soils, the contractor should install appropriate temporary drainage systems to keep water out of the construction areas, and should minimize traffic over any prepared subgrades formed within these soils.

The following text sections of this report present our specific geotechnical conclusions and recommendations concerning site preparation, reinforced soil walls, and needle piles. WSDOT Standard Specifications and Standard Plans cited herein refer to WSDOT publications M41-10, 2000 *Standard Specifications for Road, Bridge, and Municipal Construction*, and M21-01, *Standard Plans for Road, Bridge, and Municipal Construction*, respectively.

5.1 Site Preparation

Preparation of the project site might or should involve temporary drainage, clearing, stripping, cutting, filling, erosion control, and subgrade compaction. The paragraphs below discuss our geotechnical comments and recommendations concerning site preparation.

Temporary Drainage: We recommend intercepting and diverting any potential sources of surface or near-surface water within the construction zones before stripping begins. Because the selection of an appropriate drainage system will depend on the water quantity, season, weather conditions, construction sequence, and contractor's methods, final decisions regarding drainage systems are best made in the field at the time of construction. Nonetheless, we anticipate that curbs, berms, or ditches placed along the uphill side of the work areas will adequately intercept surface water runoff.

Clearing and Stripping: After surface and near-surface water sources have been controlled, the construction areas should be cleared and stripped of all trees, bushes, sod, topsoil, and debris. Our explorations indicate that an average thickness of about 6 inches of sod and topsoil may be encountered across the site, but significant variations could exist. Furthermore, it should be realized that if the stripping operation proceeds during wet weather, a generally greater stripping depth might be necessary to remove disturbed moisture-sensitive soils; therefore, stripping is best performed during a period of dry weather.

Erosion Control Measures: Because stripped surfaces and soil stockpiles are typically a source of runoff sediments, they should be given particular attention. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion. Similarly, soil stockpiles and cut slopes should be covered with plastic sheeting for erosion protection. We also recommend that a staked silt fence be installed around the area to be disturbed. The base of the silt fence should be buried so that sediment cannot pass beneath it, and the silt fence should be inspected and maintained during the time that the site soils are exposed, on a periodic basis, and after any major rainstorm event. It may be prudent to maintain a berm and swale around the downslope side of stripped areas and stockpiles in order to capture runoff water and thereby reduce the downslope sediment transport. In addition, the stripped areas should be revegetated as soon as possible, also reducing the potential for erosion.

Dewatering: Our explorations encountered groundwater at depths from 11 to 17½ feet below grade at the time of drilling, but we expect that groundwater levels could rise several feet during the winter and spring. If groundwater is encountered, we anticipate that an internal system of ditches, sumpholes, and pumps will be adequate to temporarily dewater the excavation.

Temporary Cut Slopes: All temporary cut slopes associated with site regrading should be adequately inclined to prevent sloughing and collapse. For the various soil layers that will likely be exposed in on-site cuts, we tentatively recommend a maximum cut slope inclinations of 1½H:1V (Horizontal:Vertical). However, appropriate inclinations will ultimately depend on the actual soil conditions exposed during earthwork.

On-Site Soils: Because moderate cuts are planned for the project, we expect that moderate quantities of on-site soils will be generated during earthwork activities. As such, we offer the following evaluation of these on-site soils in relation to potential use as structural fill.

- *Surficial Organic Soils*: The sod, topsoil, and organic-rich soils mantling most of the site are *not* suitable for use as structural fill under any circumstances, due to their long-term compressibility. Consequently, these materials can be used only for non-structural purposes, such as in landscaping areas.

- *Upper Silty Sands and Sandy Silts*: The silty sands and sandy silts underlying the surficial organic soils do not appear suitable for reuse as structural fill.

Permanent Slopes: All permanent cut slopes and fill slopes should be adequately inclined to minimize long-term raveling, sloughing, and erosion. We generally recommend that no slopes be steeper than 2H:1V. For all soil types, the use of flatter slopes (such as 3H:1V) would further reduce long-term erosion and facilitate revegetation.

Slope Protection: We recommend that a permanent berm, swale, or curb be constructed along the top edge of all permanent slopes to intercept surface flow. Also, a hardy vegetative groundcover should be established as soon as feasible, to further protect the slopes from runoff water erosion. Alternatively, permanent slopes could be armored with quarry spalls or a geosynthetic erosion mat.

5.2 Reinforced Soil Walls

In our opinion, multiple reinforced soil walls would be suitable for repair of the landscape area in the backyard. The paragraphs below present our design and construction recommendations, and the attached Figure 4 presents our wall design.

Wall Types: Reinforced soil walls consist of structural fill lifts interlayered with reinforcing grids or strips and supported at the face by a reinforcing material or segmental (modular) concrete facade. Suitable options include the proprietary systems produced by Allan Block, Keystone, Pisa, Stonewall, and VSL, all of which are available with decorative segmental concrete facades. We understand that the Keystone system will be used for this project.

Design Values: Reinforced soil walls with proprietary facades are typically designed by the wall supplier or a specialty consultant, using design values provided by the geotechnical engineer. These design values include soil density, internal friction angle, cohesion, and allowable bearing capacities, as well as seismic acceleration. Table 1 summarizes the various soil parameters we used for wall design, based on our explorations and subsequent interpretations.

TABLE 1
SOIL PARAMETERS USED FOR REINFORCED SOIL WALL DESIGN

Soil Type	Density (pcf)	Internal Friction Angle (degrees)	Cohesion (psf)	Allowable Bearing Capacity (psf)
Reinforced Soil (imported granular fill)	125	32	0	N/A
Retained Soil (native soil)	120	30	0	N/A
Subgrade Soil (native)	120	30	100	1,500

Subgrade Preparation: The entire area beneath the new reinforced soil zone should be stripped of all vegetation and organic soils, as per the *Site Preparation* section of this report. All subgrade soils should then be compacted to a firm, unyielding condition.

Wall Embedment: For frost protection, erosion protection, and sliding resistance, we recommend that the face of all walls be embedded at least 8 inches below future grades in front of the wall, measured from the bottom of any topsoil or landscaping bark layer. As such, the lowest row of blocks will be completely embedded.

Block Placement: Each course of segmental concrete blocks should be placed and interlocked per the manufacturer's recommendations. For aesthetic reasons, it may be desirable to place each course with a setback from the course below, thereby achieving a slight batter, but a near-vertical orientation can be used where lateral space is limited. The interior void in every block should be infilled with crushed rock as each course is completed.

Soil Drainage: Because seasonal groundwater seepage could occur within the retained soil mass of the retaining wall, we recommend that a curtain drain be placed directly behind the concrete blocks. This drain should consist of a 12-inch-wide curtain of clean, uniform crushed rock, such as "Crushed Surfacing Top Course" per WSDOT Standard Specification 9-03.9(3), with a 4-inch-diameter perforated drainpipe at the bottom, as shown on Figure 4. The drainpipe should discharge to a catch basin or other suitable location.

Geogrid Layout: The appropriate number (N) and length (L) of geogrid layers, the height of the initial geogrid layer (S_O), and the vertical spacing between subsequent geogrid layers (S) depend on the wall height (H). Details are shown on Figure 4.

Fill Soils: Ideally, all fill soils located within the reinforced backfill and retained backfill zones would consist of clean, well-graded sand and gravel, such as "Gravel Borrow" or "Ballast" per WSDOT Standard Specifications 9-03.14(1) and 9-03.9(1), respectively. Existing organic matter, sod, or topsoil stripped from the wall subgrade would *not* be suitable for this purpose under any circumstances.

Fill Placement and Compaction: All soils located within the reinforced backfill and retained backfill zones should be placed and compacted in accordance with our recommendations given in the *Structural Fill* section of this report. Specifically, we recommend that all fill be compacted to a uniform density of at least 90 percent (based on ASTM:D-1557).

Toe Prism: The overexcavated zone at the toe of the walls, above the crushed rock bearing pads, should be backfilled with additional crushed rock, as shown on Figure 4. We recommend that these toe prisms be firmly compacted by means of at least two passes with a small vibratory roller or percussion compactor ("jumping jack").

5.3 Needle Piles

In our opinion, needle piles can be used to support a new grade beam beneath the wall facing. The following recommendations and comments are offered for needle pile design and installation purposes. Figure 4 presents our needle pile and grade beam design.

Materials: For relatively low loads, needle piles typically consist of 2-inch-diameter Schedule-80 (2.375-inch O.D.) steel pipe. We infer that such a pipe size will be adequate for the subject house. Individual pipe segments typically range from about 3 to 5 feet long and are successively joined with external threaded couplings, internal slip couplings, or butt welds as pile driving progresses.

Driving Procedures: All 2-inch-diameter needle piles should be driven into the subgrade to a point of refusal by means of a pneumatic hammer, with refusal being defined as 2 inches or less of penetration during 1 minute of sustained driving under body weight. The pneumatic hammer should weigh at least 90 pounds and should have foot stirrups on which the operator can stand to apply downward pressure.

Driving Conditions: Refusal depths are difficult to predict, and soil conditions could vary significantly across the site. Therefore, the contractor should be prepared for variable pile lengths. We recommend tip embedment of at least 5 feet into dense, native soils. Also, it may be necessary to modify pile layouts if rocks or other obstructions are encountered during pile-driving.

Pile Butt Treatment: When refusal has been achieved, the pile butts can be cut off to a predetermined height or elevation. To provide a good bond between the piles and the new footing or pile cap (if used), reinforcing bars with 90-degree bends can be welded to the top of the pile or the top of the pile can be splayed apart.

Axial Load Capacities: In our opinion, a properly installed needle pile driven to refusal (as defined above) will provide the following allowable compressive capacities for a minimum pile spacing (center to center) of six diameters. If desired, an AMEC representative could be retained to verify the bearing capacity of all needle piles during or after installation. The stated uplift capacity would be applicable only to needle piles that are installed with tension-resisting couplings.

Design Parameter	Allowable Value
Static Compressive Capacity	4000 pounds
Seismic Compressive Capacity	5333 pounds

6.0 RECOMMENDED ADDITIONAL SERVICES

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. Consequently, we recommend that AMEC be retained to provide the following post-report services:

- Observe all exposed subgrades after completion of stripping and overexcavation to confirm that suitable soil conditions have been reached for placement of the leveling pad;

- Monitor the installation of all needle piles to verify that adequate embedment has been achieved and to document the installation procedures;

- Verify geogrid type, length, spacing and installation procedures;

- Observe the installation of the wall drainage;

- Monitor the placement and test the compaction of structural fill soils; and

- Prepare a post-construction letter summarizing all field observations, inspections, and test results (as required by the City of Mercer Island).

In addition to the aforementioned services, AMEC can provide inspection and testing of concrete, steel, and other structural materials. Upon request, we could submit a proposal for providing some or all of these construction monitoring, inspection, and testing services. Such a proposal is best prepared after the project plans and specifications have been approved for construction.

7.0 CLOSURE

The conclusions and recommendations presented in this report are based, in part, on the explorations that we performed for this study; therefore, if variations in the subgrade conditions are observed at a later time, we may need to modify this report to reflect those changes. Also, because the future performance and integrity of the project elements depend largely on proper initial site preparation, drainage, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. AMEC is available to provide geotechnical monitoring, soils and concrete testing, steel inspection, and other services throughout construction.

We appreciate the opportunity to be of service on this project. If you have any questions regarding this report or any aspects of the project, please feel free to contact our office.

Sincerely,

AMEC Earth & Environmental, Inc.

JAMES S. DRANSFIELD
STATE OF WASHINGTON
23153
REGISTERED
PROFESSIONAL ENGINEER
1/16/02
EXPIRES 12/19/03

Stephen A. Siebert, P.E.
Senior Project Engineer

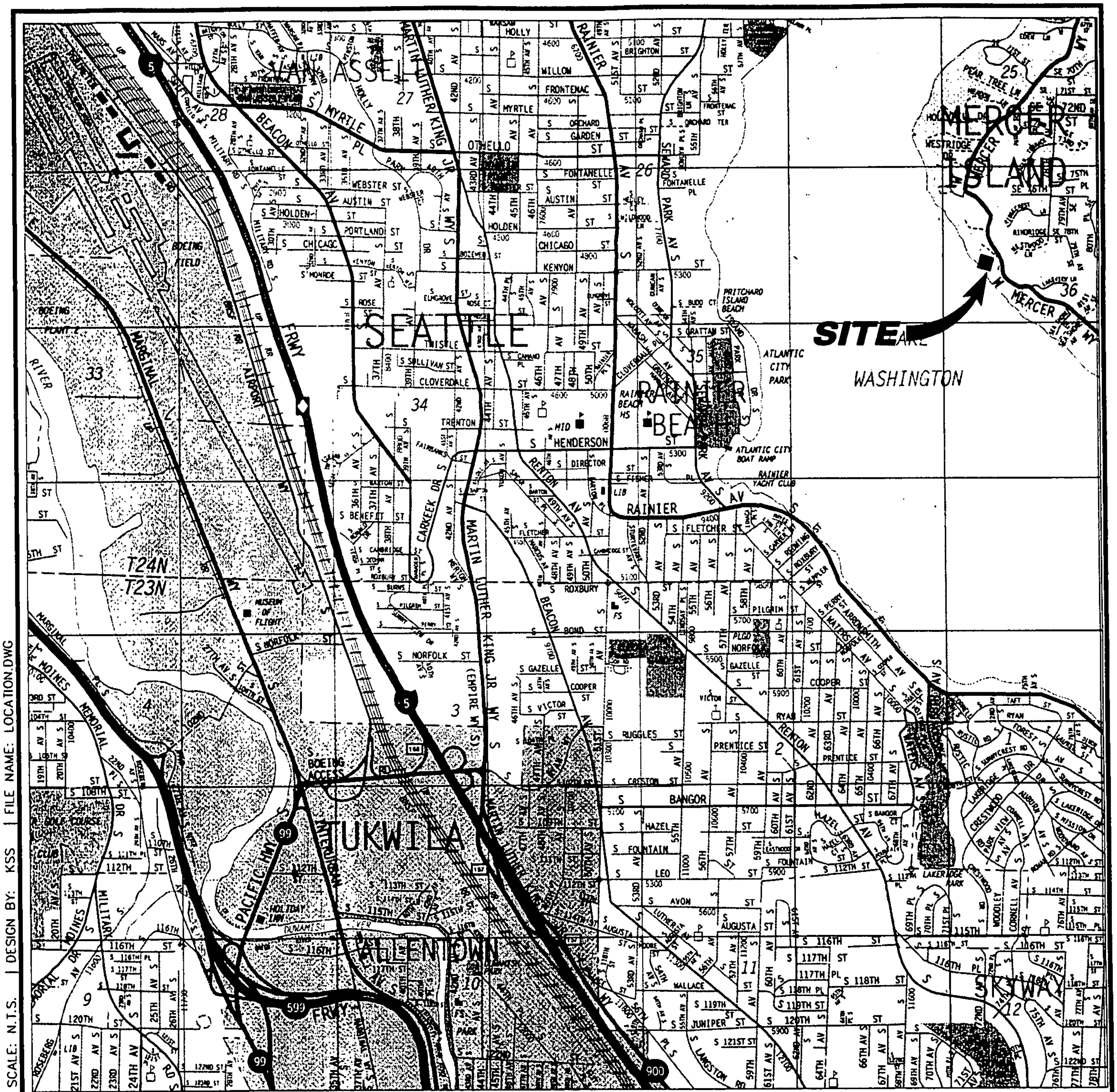
James S. Dransfield, P.E.
Principal Geotechnical Engineer

SAS/JSD/kms

Enclosures: Figure 1 — *Location Map*
Figure 2 — *Site & Exploration Plan*
Figure 3 —*Cross Section A-A'*
Figure 4 — *Keystone Wall and Grade Beam Diagram*

Boring Logs B-1 through B-3
Grain Size Distribution Graphs

Distribution: Mr. Jeff Levere, Mid-Mountain Construction (3)
Mr. Don Cole, City of Mercer Island (2)



N.T.S.

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11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918

LOCATION MAP

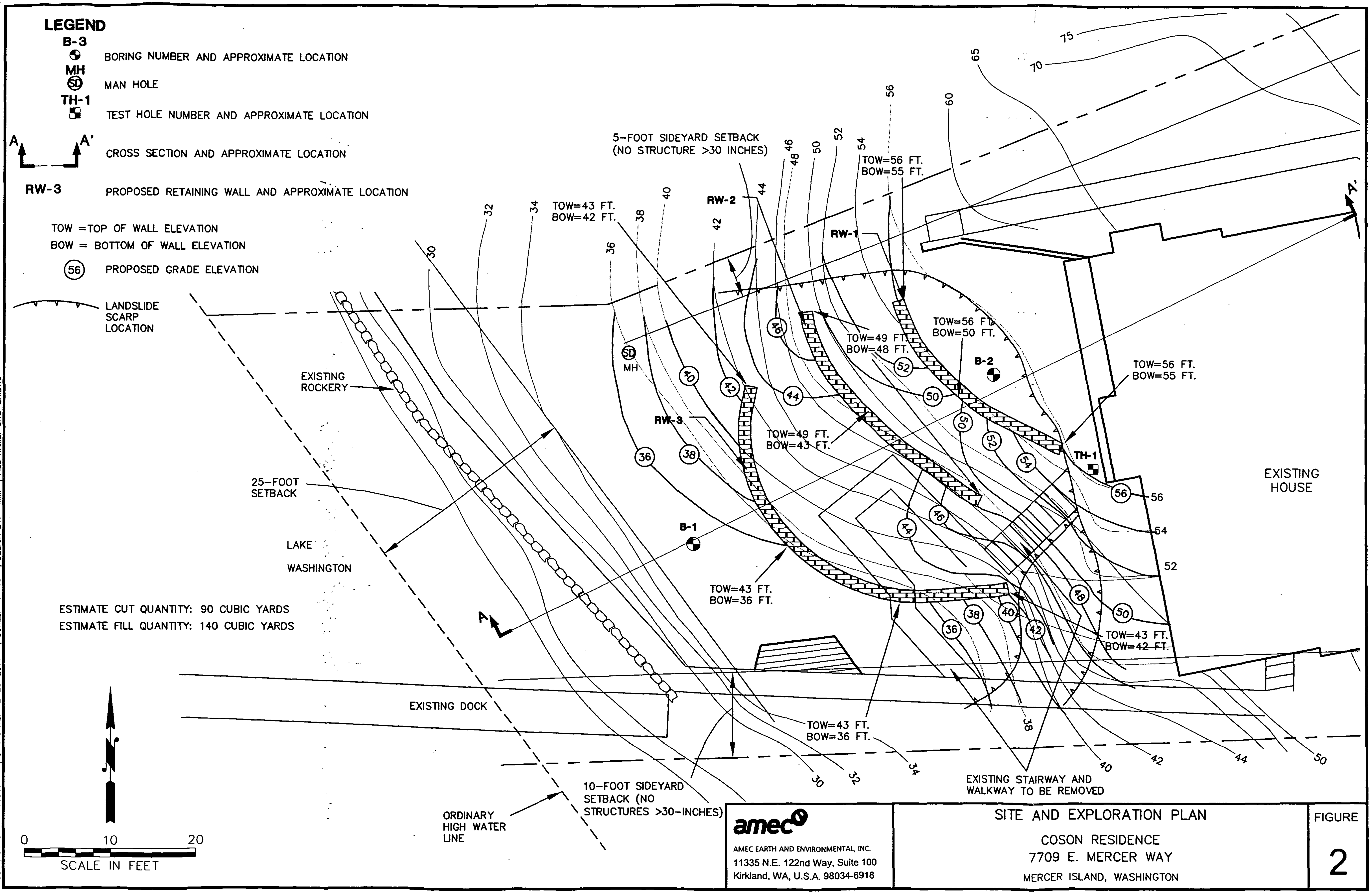
COSON RESIDENCE

MERCER ISLAND, WASHINGTON

FIGURE

1

JOB NO.: 1-91M-14049-0 | DWG DATE: 07-20-2001 | SCALE: N.T.S. | DESIGN BY: KSS | FILE NAME: LOCATION.DWG



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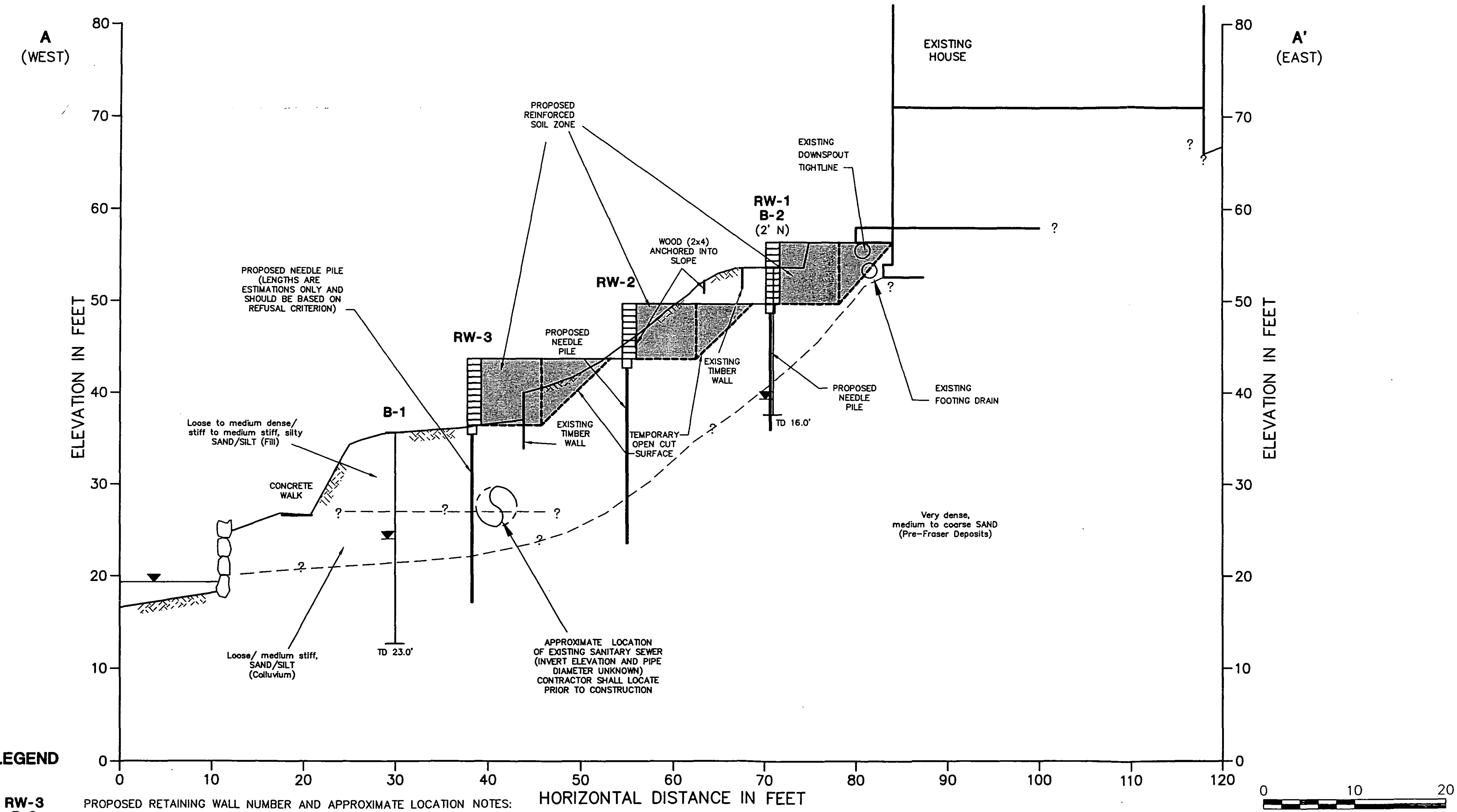
SITE AND EXPLORATION PLAN

COSON RESIDENCE
7709 E. MERCER WAY
MERCER ISLAND, WASHINGTON

FIGURE 2

JOB NO.: 0-91M-14049-A | DWG DATE: 02-25-2001 | SCALE: 1"=10' | DESIGN BY: TMM | FILE NAME: SITE-2A.DWG



LEGEND

RW-3	PROPOSED RETAINING WALL NUMBER AND APPROXIMATE LOCATION
B-2	BORING NUMBER AND APPROXIMATE LOCATION
(2' N)	OFFSET FROM SECTION LINE
▼	GROUNDWATER LEVEL AT TIME OF DRILLING
TD 16.0'	TOTAL DEPTH OF BORING

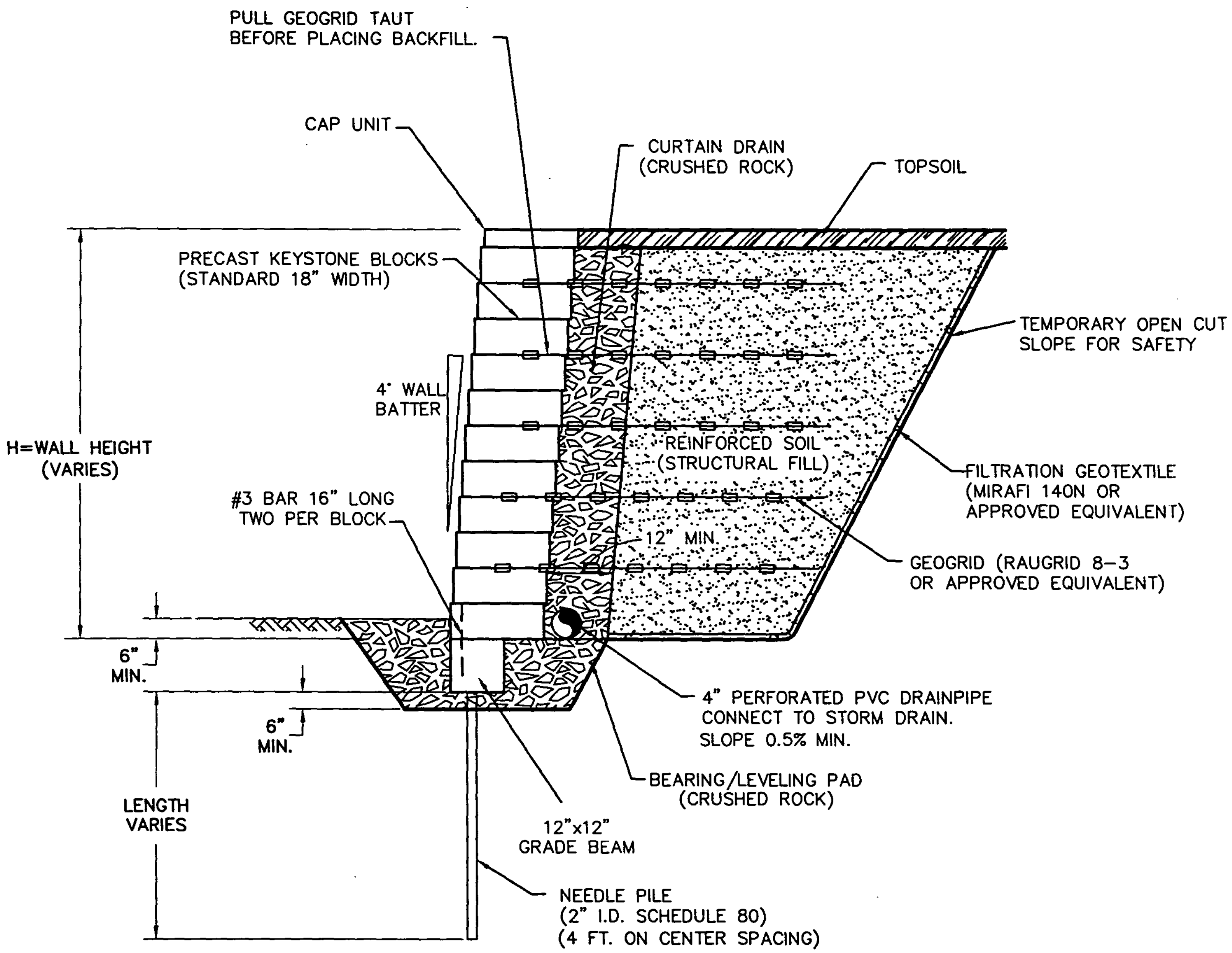
NOTES:

THE STRATA ARE BASED UPON INTERPOLATION BETWEEN EXPLORATIONS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.

SIMPLIFIED NAMES ARE SHOWN FOR SOIL DEPOSITS, BASED ON GENERALIZATIONS OF SOIL DESCRIPTIONS. SEE EXPLORATION LOGS AND REPORT TEXT FOR COMPLETE SOIL DESCRIPTIONS.

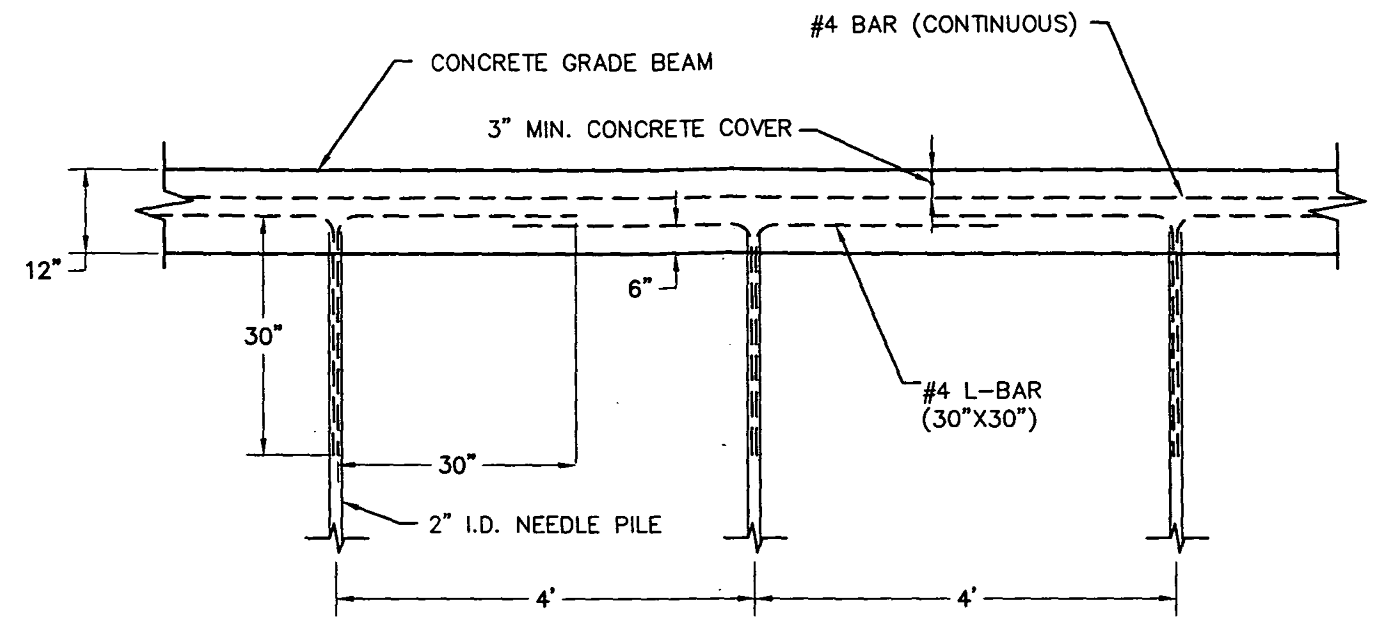
0 10 20
SCALE IN FEET

amec AMEC EARTH AND ENVIRONMENTAL, INC. 11335 N.E. 122nd Way, Suite 100 Kirkland, WA. U.S.A. 98034-6918	CROSS SECTION A–A' COSON RESIDENCE MERCER ISLAND, WASHINGTON	FIGURE 3



TYPICAL KEYSTONE WALL SUPPORTED BY NEEDLE PILES

N.T.S.



GRADE BEAM/ NEEDLE PILE DETAIL

N.T.S.

NOTE:

1. GEOGRID LENGTH SHALL BE MEASURED FROM FACE OF WALL.
2. GEOGRID LENGTH = 8 FEET.
3. ALL CONCRETE SHALL HAVE 3,000 PSI MIN. 28-DAY COMPRESSIVE STRENGTH.
4. NEEDLE PILE INSTALLATION CRITERIA:
   A) ALL NEEDLE PILES SHALL BE DRIVEN TO REFUSAL INTO DENSE NATIVE SOILS.
   B) REFUSAL CRITERIA SHALL BE DEFINED AS 1 INCH OR LESS OF PENETRATION DURING 1 MINUTE OF SUSTAINED DRIVING WITH A 90-POUND PNEUMATIC HAMMER AND UNDER FULL BODY WEIGHT.
5. BACKFILL WITHIN THE REINFORCEMENT ZONE SHALL BE GRANULAR STRUCTURAL FILL WITH MINIMUM UNIT DENSITY OF 125 POUNDS PER CUBIC FOOT. MINIMUM COMPACTION WITHIN FOUNDATION AND BACKFILL ZONE IS 90% OF MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D1557.
6. ENGINEER SHALL BE GIVEN 24 HOURS NOTICE PRIOR TO CONSTRUCTION TO ALLOW FOR SCHEDULING OF INSPECTION.

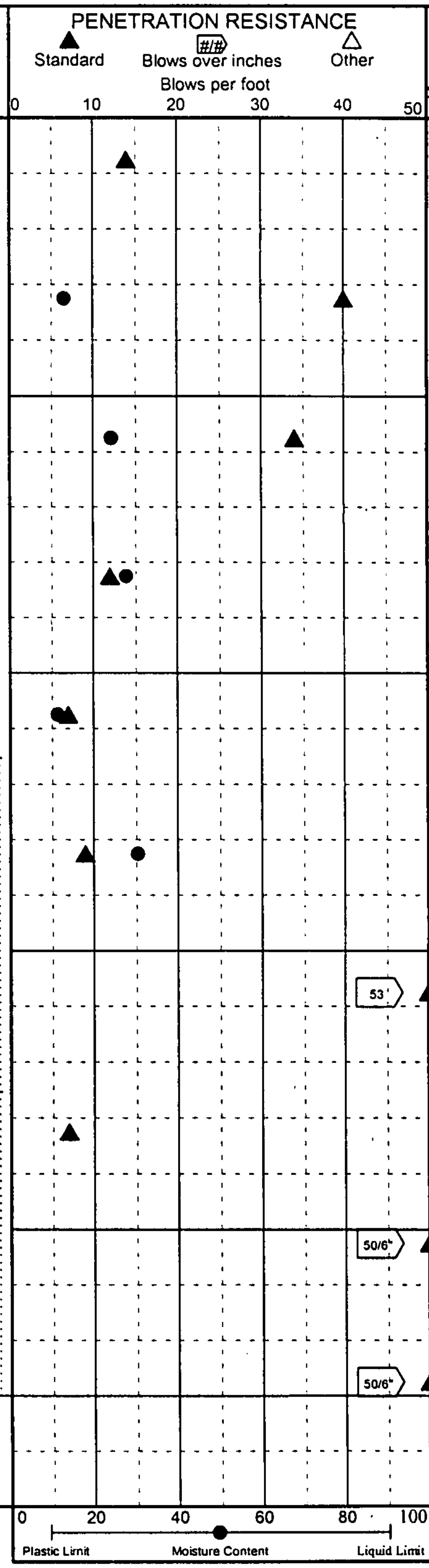
amec AMEC EARTH AND ENVIRONMENTAL, INC. 11335 N.E. 122nd Way, Suite 100 Kirkland, WA, U.S.A. 98034-6918	KEYSTONE WALL AND GRADE BEAM DIAGRAM COSON RESIDENCE MERCER ISLAND, WASHINGTON	FIGURE 4

PROJECT: *Coson Residence* W.O. *1-91M-14049-0* BORING No. *B-1*

Page 1 of 1

Location: Lower grass terrace near dock
Approximate ground surface elevation: 35 feet

DEPTH (feet)	Soil Description	USCS/USGS GRAPHICS	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	TESTING
0	Sod over topsoil					
	Medium dense, damp, tan/light brown, silty fine SAND, trace gravel, rootlets (Fill)		12	S-1		
	Dense, damp, light brown, gravelly, silty, SAND (Fill)		12	S-2		25
5	Wood chips and wood fragments (Fill)					
	Stiff, moist to wet, gray and brown, SILT, trace sand, organics (Fill)		12	S-3		
			14	S-4		
10	Loose, moist, gray, medium to coarse SAND, trace silt (Fill)		18	S-5		
	Medium stiff, wet, gray/brown SILT lens (Colluvium)		18	S-6		
	Loose, moist, gray, medium to coarse SAND, trace silt (Colluvium)					
15	Very dense, moist to wet, gray, silty fine to medium SAND, trace gravel (Pre-Fraser Deposits; Qpf)		18	S-7		
	Loose, wet, gray, fine to medium SAND, some silt (Pre-Fraser Deposits; Qpf)				ATD	
	1-ft of heave		18	S-8		
20	Becomes very dense, wet, gray, fine to medium SAND, some silt (Pre-Fraser Deposits; Qpf)		6	S-9		
	1/2-ft of heave					
	1/2-ft of heave		6	S-10		
	Boring terminated at approximately 23 feet below ground surface					
25						



LEGEND

- 2.00-inch OD split-spoon sampler
- Groundwater Level Fluctuation
- Groundwater level at time of drilling (ATD)
- 200 Wash (percent fines shown)
- Grain Size Analysis

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4IN1 B1TOB3.GPJ WA4IN1.GDT 7/25/01

Drilling Method: HSA Hammer type: Cathead Date drilled: *July 10, 2001* Logged By: *KSS*

PROJECT: *Coson Residence* W.O. *1-91M-14049-0* BORING No. *B-2*

Soil Description

Location: Downset block below tile patio

Approximate ground surface elevation: 54.5 feet

Page 1 of 1

DEPTH (feet)	Soil Description	SAMPLE NUMBER	GROUND WATER	TESTING
0	Sod over topsoil			
	Medium dense, damp, tan, silty SAND, trace gravel (Fill)	S-1		
	Grades to moist, gray/brown, silty SAND, trace gravel (Fill)	S-2		
		S-3		
5	Very loose, wet, gray, silty fine SAND, trace gravel (Fill)	S-4		21
	Medium siff, damp, light gray, fine sandy SILT (Fill)	S-5		
	Poor recovery - Wood chips and woody debrisin sample (Fill)	S-6		
	Very loose, wet, gray, silty fine SAND (Colluvium)	S-7		
10		S-8		
	Very dense, wet to saturated, brown, medium SAND, trace silt (Pre-Fraser Deposits; Qpf)	S-9	ATD	24
15	1-ft of heave	S-10		
	Boring terminated at approximately 15.8 feet below ground surface			
20				
25				

PENETRATION RESISTANCE: ▲ Standard; Blows over inches; △ Other. Blows per foot: 0, 10, 20, 30, 40, 50. S-9: 61"; S-10: 50/4"

0, 20, 40, 60, 80, 100 — Plastic Limit, Moisture Content, Liquid Limit

LEGEND

2.00-inch OD split-spoon sampler

ATD Groundwater level at time of drilling

200 Wash (percent fines shown)

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4IN1 B1TOB3.GPJ WA4IN1.GDT 7/25/01

Drilling Method: HSA Hammer type: Cathead Date drilled: *July 10, 2001* Logged By: *KSS*

PROJECT: *Coson Residence* W.O. *1-91M-14049-0* BORING No. *B-3*

Location: Planter by concrete walkway near entrance

Approximate ground surface elevation: 71 feet

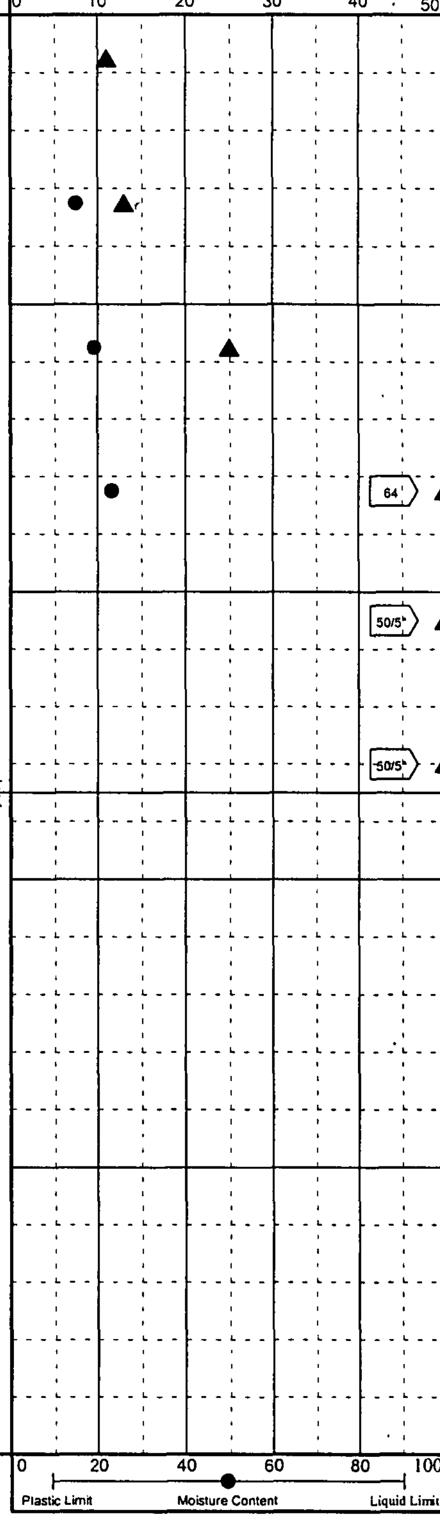
Page 1 of 1

DEPTH (feet)	Soil Description	Sample Type	Sample Number	Ground Water
0	Topsoil			
	Medium dense, moist, tan with orange mottling, silty SAND, some gravel (Fill)	12	S-1	
	Gravelly drilling			
	Medium dense, wet, tan with orange mottling, silty, gravelly, SAND (Glacial Till; Qvt)	18	S-2	
5		18	S-3	
	Very dense, moist, light brown, fine to medium SAND, trace to some silt (Pre-Faser Deposits; Qpf)	18	S-4	
10		12	S-5	
		12	S-6	ATD
	Boring terminated at approximately 13.5 feet below ground surface			
15				
20				
25				

PENETRATION RESISTANCE

▲ Standard   #/# Blows over inches   △ Other

Blows per foot



TESTING

LEGEND

2.00-inch OD split-spoon sampler

ATD Groundwater level at time of drilling

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4IN1 B1TOB3.GPJ WA4IN1.GDT 7/25/01

Drilling Method: HSA   Hammer type: Cathead   Date drilled: *July 10, 2001*   Logged By: *KSS*

Particle Size Distribution Report

% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.4	60.8	37.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50 in.	100.0		
.375 in.	99.2		
.25 in.	99.1		
#4	98.6		
#10	97.4		
#20	92.5		
#30	89.1		
#40	82.7		
#50	74.6		
#60	68.1		
#100	51.6		
#200	37.8		

* (no specification provided)

Soil Description

Dk grey silty sand
MC: 19.1%

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.477 D_{60}= 0.199 D_{50}= 0.141
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Tested by: NB,SS
Reviewed by: ML
ASTM C136,D1140,D2216

Sample No.: 4216.7 **Source of Sample:** **Date:** 7-19-01
Location: B-1 / S-7 **Elev./Depth:**

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

Client:
Project: COSON RESIDENCE
Project No: 191M140450 **Plate**