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Report of Geotechnical Engineering Services
Proposed Classical Chinese Garden
Portland, Oregon

3029/4

November 25, 1998

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For
Classical Chinese Garden Trust
c/o City of Portland
Bureau of General Services

239 NW Everett St
1N1E34CA 3600

GeoEngineers

Consulting Engineers
and Geoscientists
Offices in Washington,
Oregon, and Alaska

November 25, 1998

Classical Chinese Garden Trust
c/o City of Portland
Bureau of General Services
1120 Southwest 5th Avenue, #1204
Portland, Oregon 97204

Attention: Mr. Dick Ragland

GeoEngineers is pleased to submit five copies of our Report of Geotechnical Engineering Services for the proposed Classical Chinese Garden located at the intersection of Northwest Everett Street and Northwest Second Avenue in Portland, Oregon. Our services were authorized by Mr. Gregg Kantor of the Chinese Classical Garden Trust and have been provided in accordance with the terms of our proposal dated October 26, 1998.

We appreciate the opportunity to provide these services. Please call if you have questions regarding this report or if we can be of further assistance.

Yours very truly,

GeoEngineers, Inc.

Douglas R. Schwarm, P.E.
Senior Engineer

DRS:mln
Document ID: 179201202R.doc

File No. 1792-012-02

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PROPOSED CLASSICAL CHINESE GARDEN
PORTLAND, OREGON
FOR
CLASSICAL CHINESE GARDEN TRUST
C/O CITY OF PORTLAND
BUREAU OF GENERAL SERVICES

INTRODUCTION

This report presents the results of GeoEngineers' geotechnical engineering evaluation of the proposed Classical Chinese Garden site located northwest of the intersection of Northwest Everett Street and Northwest Second Avenue in Portland, Oregon. Figure 1 shows the site location relative to surrounding topographic features.

GeoEngineers performed a geotechnical engineering evaluation for the previous proposed site, located one block north of the current proposed site. That evaluation is described in our November 20, 1997 geotechnical report.

Ms. Tamarra Walsh of KPFF Consulting Engineers provided us with a preliminary site development plan and other development information. It is our understanding that the project will consist of 16 small structures, a perimeter wall, rockeries, paths, pavers, and a lined pond. Site grading information was not provided; however, we anticipate that moderate cuts will be necessary for the pond, and minor fills will be necessary to achieve planned topographic contours. Figure 2 shows the proposed building, path, and pond locations.

PURPOSE AND SCOPE

The purpose of our services is to explore subsurface conditions at the site to provide geotechnical engineering recommendations for site development and foundation design. Our scope of work includes the following:

1. Explore subsurface conditions by drilling eight borings to a depth of 11.5 feet and one boring to a depth of 21.5 feet using hollow-stem auger equipment.
2. Obtain soil samples at selected depths from the borings.
3. Classify the materials encountered in the borings in general accordance with American Society for Testing and Materials (ASTM) Standard Practice D 2488. Maintain a detailed log of each exploration and observe ground water conditions.
4. Perform moisture content and dry density determinations, gradation tests, and one consolidation test on select soil samples.
5. Provide recommendations for demolition, site preparation, grading and drainage, stripping depths, fill characteristics for imported materials, compaction criteria, procedures for use of on-site soils, and wet and dry weather earthwork guidelines.
6. Provide recommendations for design and construction of shallow spread foundations, including allowable design bearing pressure and minimum footing depth and width.

7. Provide recommendations for design and construction of floor slabs, including an anticipated subgrade modulus.
8. Estimate settlement of the footings and floor slabs for the design loadings.
9. Provide recommendations for drainage of slabs and footings.
10. Provide lateral earth pressure recommendations for retaining wall and rockery design.
11. Provide geotechnical recommendations for design and construction of the pond liner.
12. Provide recommendations for the Uniform Building Code (UBC) site coefficients and seismic zone.
13. Provide a written report summarizing the results of our geotechnical evaluation.

SITE DESCRIPTION

SURFACE CONDITIONS

The site is comprised of the city block bounded by Northwest Second Avenue, Northwest Flanders Street, Northwest Third Avenue, and Northwest Everett Street. The site is currently occupied by a paved parking lot surrounded by sidewalk and landscape strips. The parking lot is graded to drain toward several catch basins, suggesting the presence of underground piping. Overhead lighting indicates the presence of buried electrical conduits.

Historically, we understand that the site has been developed several times since the late 1880s. At least two significant structures, a hotel and a parking garage, have occupied the site. Northwest Natural employees familiar with the parking lot construction indicated that the building foundations were not removed during demolition. We did not observe surface evidence of buried foundations or debris.

SUBSURFACE CONDITIONS

Subsurface conditions at the site were explored by drilling 9 borings to depths between 11.5 and 21.5 feet at the approximate locations shown on Figure 2. Select soil samples from the borings were tested to determine the moisture content, unit weight, gradation, and compressibility characteristics. Appendix A includes descriptions of the subsurface exploration and laboratory testing programs. The boring logs and the laboratory test results are included in Figures A-3 through A-11.

The borings encountered pavement and base course, a gravel fill subbase, highly variable random fill deposits, and native silt soils. Ground water was not encountered in the borings. The following sections provide additional detail regarding the subsurface conditions at the site.

Asphalt and Base Course

The site is covered with approximately 2 1/2 inches of asphaltic concrete. The asphalt overlies approximately 10 to 14 inches of base course consisting of subrounded to subangular crushed rock and sand.

Gravel Fill

Under the pavement and base course, borings B-4 through B-9 encountered a layer of compacted gravel or crushed rock approximately 1 1/2 feet thick. The gravel material is generally subrounded to subangular, moderately well-graded gravel with sand and silt, and has a maximum particle size of approximately 2 inches.

Existing Fill Materials

The gravel layer is underlain by variable fill materials to a depth of approximately 5.0 to more than 11.5 feet. The fill materials include brick rubble, wood, glass fragments, construction debris, silt, sand, and organic soils with no apparent layering or consistency. This material is most likely from demolition of previously existing buildings. After demolition, the basements of the buildings were probably backfilled with a mixture of the debris and on-site soils. Considering the loose and variable nature of the on-site fill, compaction efforts were minimal.

The consistency of the fill material ranges from medium stiff to stiff and moisture contents are between 11 and 36 percent by weight. The dry density of the fill varies from 78 to 115 pounds per cubic foot (pcf) as measured in three samples. These soils are compressible and weak, and will require some removal and replacement to provide adequate foundation support. Because of the variable nature of the fill soils, we expect that special subgrade preparation methods will be necessary for construction of the pond liner.

Native Silt and Silty Sand

The fill is underlain by medium stiff silt and loose silty sand to the maximum depth explored. The native soils have moderate compressibility characteristics and low to moderate strength. Laboratory tests indicate moisture contents between 29 and 37 percent and dry densities between 84 and 90 pcf.

GROUND WATER

A regional ground water table was not encountered in the borings, and ground water is not expected to affect the planned construction. Wet soil conditions were encountered at a depth of 21 feet in boring B-5.

Based on our experience in downtown Portland, near-surface water can flow preferentially in abandoned utility lines and pipe bedding, affecting localized portions of sites.

CONCLUSIONS

It is our opinion that the site is suitable for support of the planned garden structures and features provided the recommendations in this report are incorporated during design and construction. The following conditions are expected to affect the cost and feasibility of the planned development:

1. The site is underlain by 3 to more than 11.5 feet of variable fill soils, construction debris, ash, and organics. The random fill soils have widely varying strength and compressibility characteristics. Shallow foundations supporting buildings, retaining walls, and perimeter

walls will require overexcavation and replacement with compacted granular fill in order to reduce the potential for differential settlement.

2. Soft, unsuitable materials will provide poor support for construction equipment, particularly during periods of wet weather. Selective or widespread removals will likely be required, depending on the traffic management plan used during earthwork. Thick crushed rock haul roads will be required to support heavy construction equipment.
3. Although not encountered in the borings, foundations from previous structures are reportedly buried beneath the current asphalt pavement. Demolition of the foundations could be time-consuming and costly.
4. Rubble and debris may be present beneath the planned pond liner. Special subgrade preparation should be expected to provide uniform support for the planned flexible membrane under the pond.
5. Ground water was not encountered in the borings, and is not expected to affect the planned construction. Localized zones of saturated soils are known to occur in the general site area, typically caused by abandoned sewer lines conveying stormwater. Localized saturated zones should be expected.

RECOMMENDATIONS

GENERAL

We anticipate that the planned garden structures and boundary wall can be supported on shallow mat, pad, and pole foundations with provisions for removing unsuitable existing fill soils and replacing them with compacted granular fill. We expect that support for the pond lining will be affected by the character of the near-surface fill soils, and we suggest that flexible linings be considered. Demolition of buried foundations may be required. The following paragraphs present specific geotechnical recommendations for design and construction of the proposed development.

SITE PREPARATION

Demolition and Reuse of Existing Materials

Demolition, as used in this report, means complete removal of all existing improvements including buried piping, conduit, building foundations, pole foundations, abandoned foundations, and basement walls. The existing asphaltic concrete should be demolished and exported off site. It may be possible to reuse the asphalt as fill if it is crushed and can be placed in appropriate areas. We can provide further recommendations for reusing the asphalt, if requested. Existing utilities, such as storm sewer lines, should be located and removed, or protected if necessary.

Based on our experience in the downtown Portland area, near-surface water can be transmitted through abandoned pipes and through the bedding and backfill of active and inactive utilities. Any buried utilities that are on the site should be removed or grouted in place, and abandoned laterals should be grouted at the site perimeter.

We recommend that buried foundations and basement walls be exposed and demolished at least 3 feet below the depth of planned structures or features. The excavation that remains after demolition should be backfilled with compacted granular fill or landscaping soil, depending on the future use of the area.

The crushed rock base course and gravel fill that extends up to 2.5 feet below present grades is suitable for reuse as structural fill. The gravel fill should not be mixed with the underlying fill containing construction debris. When excavated, the underlying silty fill is not suitable for reuse as structural backfill due to the high moisture content. This material can be used as fill for landscaping areas; otherwise, it should be hauled off of the site.

Subgrade Evaluation

The existing fill soils will likely have soft, poorly-compacted pockets that were not specifically identified in the exploratory borings. After asphalt removal, demolition, and rough grading have been completed, the building and walkway subgrades should be evaluated by a GeoEngineers representative to identify soft or unsuitable subgrade areas. Typically, soft or unsuitable areas are overexcavated and backfilled with granular structural fill.

WET WEATHER CONSTRUCTION

Trafficability of the silty fill material will be difficult or impossible during periods of rainfall or when the moisture content is more than a few percentage points above optimum. When wet, this material is susceptible to disturbance and will provide inadequate support for construction equipment. If construction occurs during the wet season, excavation may need to be accomplished using track-mounted equipment, loading removed material into trucks traveling on crushed rock haul roads. Wet soil that has been disturbed during site preparation activities, or soft or unsuitable zones identified by a member of our geotechnical staff should be removed and replaced with suitable structural fill.

The use of granular haul roads and/or staging areas will be necessary for support of construction traffic during the rainy season. A 12-inch thickness of imported granular material generally should be sufficient for light staging areas and building pads but is generally not expected to adequately support heavy equipment or truck traffic. The existing 1.5-foot-thick gravel layer under the pavement section will likely provide adequate subgrade support for moderate truck traffic in areas that are not excavated. Haul roads and areas with repeated heavy construction traffic should be constructed with a minimum thickness of 18 inches of granular material. Ash, wood debris, or very soft soils should be removed from below haul road areas and replaced with compacted granular fill before building the haul road.

We recommend that a geotextile fabric be placed as a barrier between the subgrade and imported fill in areas of repeated construction traffic. The geotextile should have a minimum Mullen burst strength of 500 pounds per square inch (psi) and an apparent opening size (AOS) between a U.S. Standard No. 70 and No. 100 Sieve to reduce migration of fines into the rock. Amoco 2000 and Mirafi 500X are two fabrics that meet these specifications.

Imported granular material for haul roads should consist of crushed rock that is well-graded between coarse and fine sizes, contains no organic matter or unsuitable materials, no particles larger than 3 inches, and has less than 5 percent by weight passing the U.S. No. 200 Sieve. Imported granular material used for haul roads should be placed in one lift over the prepared subgrade and compacted using a smooth-drum roller without use of a drum vibrator. Vibratory compaction can be used after static rolling has densified the rock to at least 85 percent of the laboratory maximum density determined by ASTM Test Method D 1557.

STRUCTURAL FILL

Structural fill soils should be free of organic matter and other deleterious materials and particles larger than 3 inches in diameter. The existing pavement base and underlying gravel soils should be suitable for use as structural fill during most weather conditions. Additional imported material may be required during periods of prolonged wet weather. Cost and moisture conditions need to be taken into account when selecting the fill materials.

Imported Fill Characteristics

Granular material will be necessary for structural fill during wet weather or if the on-site materials cannot be properly moisture-conditioned. Granular material can be either the on-site crushed rock base, the existing gravel fill, or an imported granular fill. Imported fill should be pit or quarry run rock, crushed rock, or clean sand that has less than 5 percent passing the U.S. No. 200 Sieve. The fines content can be increased to 20 percent during extended dry weather and provided the fill material is within 3 to 4 percentage points of optimum moisture content.

Fill Placement and Compaction

Structural fill should be placed in uniform horizontal lifts and compacted to at least 95 percent of the maximum laboratory dry density as determined by ASTM Test Method D 1557. The optimum lift thickness will vary depending on the type of material and the compaction equipment being used, but should typically not exceed 8 inches in loose thickness. During the wet season or when wet subgrade conditions exist, the initial lift should have a maximum thickness of 15 inches and be compacted by rolling with a smooth-drum roller without use of a drum vibrator.

Fill Settlement

The existing fill soils that will underlie new structural fills are generally loose and soft, and the settlement characteristics are expected to vary widely. Embankment fills placed to raise site grades more than approximately 3 to 4 feet are expected to induce noticeable, nonuniform settlement. The amount of settlement will vary widely depending on the composition of the existing fill soils at each embankment location. We recommend that fills be placed as early as possible and that a settlement plate be established on each area that receives enough fill to raise site grades more than 4 feet above the existing grade. Figure 3 shows a typical settlement plate detail. Settlement plates should be surveyed on a weekly basis until three consecutive readings indicate that settlement is essentially complete.

UTILITY TRENCH EXCAVATION AND BACKFILL

Trench backfill in structural areas should consist of clean granular material with a maximum particle size of 3/4 inch and less than 5 percent passing the U.S. No. 200 Sieve. The material should be free of organic matter and other deleterious materials. Any of the on-site materials could be suitable for trench backfill in nonstructural areas, assuming the maximum particle size is less than 3/4 inch.

Backfill for pipe bedding and the pipe zone should be compacted to at least 90 percent of the maximum laboratory dry density determined by ASTM Test Method D 1557 or as recommended by the pipe manufacturer. Within building and walkway areas, trench backfill placed above the pipe zone should be compacted to at least 92 percent of the laboratory maximum at depths greater than 2 feet below the finished subgrade and as recommended for structural fill within 2 feet of finished subgrade. In nonstructural areas, trench backfill above the pipe zone should be compacted to at least 85 percent of the laboratory maximum dry density as determined by ASTM Test Method D 1557.

Trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor should be responsible for selecting trench excavation methods, monitoring the trench excavations for safety and providing shoring, as required, to protect personnel and adjacent improvements. Due to the loose nature of some of the granular fill materials, caving of trench excavations is expected. Proper shoring should be in place for all trench excavation work.

RETAINING WALLS AND ROCKERIES

We recommend that lateral earth pressures on rockeries and retaining walls be computed using an equivalent fluid unit weight of 35 pcf. This value assumes that the wall is free to rotate slightly at the top and that the wall backfill consists of free-draining crushed rock or sand. Retaining wall foundations should be designed and constructed using the recommendations provided in the "Foundation Support" section of this report. Walls should be provided with weepholes or foundation drains that convey collected water to a suitable outlet.

Rockeries higher than 3 feet should be designed in accordance with the Associated Rockery Contractors "Standard Rock Wall Construction Guidelines." A copy of the guidelines is included as Appendix B of this report. Rock walls will require a keyway and granular backfill to allow adequate drainage.

PERMANENT SLOPES

Permanent slopes should not exceed 3 horizontal to 1 vertical (3H:1V). Footings, buildings, and walkways should be located at least 5 feet horizontally from the face of slopes. The slopes should be planted with appropriate vegetation as soon as possible after grading to provide protection against erosion. Surface water should be collected and directed away from slopes to prevent water from running down the slope face.

PAVERS AND GRAVEL WALKWAYS

We recommend that precast concrete or stone pavers be set in sand underlain by at least 12 inches of compacted crushed rock or gravel. The existing pavement base and gravel fill materials should provide adequate subgrade support for pavers provided the materials are recompacted and proof-rolled immediately before placing the sand layer. A GeoEngineers representative should observe the proof-roll. The sand layer thickness should comply with the manufacturer's recommendations, but typically should be between 1 and 3 inches thick. We recommend that the paver subgrade preparation and paver placement be performed by an experienced specialist contractor familiar with the specific type of paver being used.

Drainage is particularly important in areas covered by pavers. We recommend that surface grades slope at least 2 percent to reduce the potential for localized depressions. Depressions can result from slight variations during installation, and can also result from long-term settlement of the random fills underlying the site. Area drains or perimeter drains should collect surface water and direct it to a suitable outlet.

We recommend that gravel walkways be at least 8 inches thick and that the gravel be separated from the underlying random fill soils by a geotextile filter fabric. The fabric is necessary to reduce long-term migration of fines up into the gravel section. Mirafi 140N and Amoco 4535 are two materials that should provide adequate separation.

FOUNDATION SUPPORT

The proposed buildings and boundary wall can be supported by continuous wall or isolated column footings founded on crushed rock fill placed over the existing random fill. Footings should be established at least 18 inches below the adjacent ground surface to avoid foundation movement due to frost. Design of shallow footings should consider both bearing capacity and settlement issues.

Foundation Preparation

The existing fill soils are highly variable and compressible, resulting in a high potential for differential settlement. We recommend that all footings that will impose a net, long-term foundation bearing pressure less than 500 pounds per square foot (psf) be underlain by at least 6 inches of compacted crushed rock. The net bearing pressure is the difference between the foundation pressure (including backfill above the footing) and the present vertical effective stress at the foundation base. The present effective stress is the product of the unit weight and the depth to the bearing grade. We recommend using unit weights of 135 pcf for footing backfill materials and 115 pcf for the existing fill soils.

Foundations that exceed the threshold pressure should be underlain by at least 2 feet of compacted crushed rock below the planned bearing elevation. The crushed rock pad should be at least 12 inches wider than the footing on each side. Consequently, all foundations are expected to require concrete forms. The crushed rock material should have a maximum particle size of 1 1/2 inches and less than 5 percent passing a U.S. No. 200 Sieve.

Foundation overexcavation should be monitored by a GeoEngineers representative qualified to evaluate the consistency of the subgrade soils. It is possible that very soft soils may be encountered during excavation, and that additional removals will be required.

Bearing Capacity

We recommend that foundations be proportioned using a maximum allowable bearing pressure of 1,500 psf. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. Isolated column and continuous wall footings should have minimum widths of 24 inches and 18 inches, respectively.

The net foundation bearing pressure is calculated by subtracting the weight of excavated soil from the foundation bearing pressure due to building loads and footing backfill. We recommend that a unit weight of 115 pcf be used to calculate the pressure reduction due to foundation excavation and a unit weight of 135 pcf for backfill on top of the footings.

All shallow foundations should be founded on a prepared surface consisting of 6 inches to 2 feet of compacted crushed rock as described in the previous section. Loose or disturbed materials should be removed before placing the crushed rock base.

We recommend that a GeoEngineers representative observe all foundation bearing surfaces before placing reinforcing steel. This should confirm that adequate bearing surfaces have been achieved and that soil conditions are as anticipated in the recommendations presented in this report. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, it should be removed and footing subgrade should be reevaluated be a member of our staff.

Pole Foundations

We recommend that pole foundations embedded in crushed rock fill be designed using an allowable lateral equivalent fluid unit weight of 300 pcf. Pole foundations that extend deeper than 4 feet below present grades should be designed using an equivalent fluid unit weight of 200 pcf due to the soft and variable nature of the lower fill soils. These allowable values include a safety factor of 2.0.

FOUNDATION SETTLEMENT

Shallow foundations designed in accordance with the recommendations presented in this report are expected to settle less than 1 inch. Differential settlements of up to 1/2 inch can be expected between adjacent footings with similar loads. We expect that approximately half of the settlement will occur during construction as loads are applied, and the remainder will be essentially complete within 3 months.

in addition to the settlement caused by the footing loads. We recommend that foundations placed on fill embankments be delayed until settlement monitoring, as described in the "Fill Settlement" section of this report, indicates the fill settlement is essentially complete.

LATERAL RESISTANCE

Lateral loads on footings can be resisted by passive earth pressure on the sides of footings and by friction on the base of the footings. We recommend a passive earth pressure of 275 psf per foot of depth for footings confined by structural fill or constructed in direct contact with the gravel site soils. This value is based on the assumptions that the adjacent confining structural fill or adjacent materials are level and that static ground water remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or is inside a building.

We recommend a friction coefficient of 0.40 for computing the friction capacity of building foundations founded on gravel. The passive and friction resistance may be combined provided that the passive component does not exceed two-thirds of the total.

These values do not include safety factors. We recommend a safety factor of 3 when designing for dead loads plus frequently applied live loads and a safety factor of 2 be applied when considering transitory loads such as wind and seismic forces.

FLOOR SLAB SUPPORT

Satisfactory subgrade support for building floor slabs supporting up to 150 psf areal loading can be obtained provided the building areas are prepared as described previously. A subgrade modulus value of 200 pounds per cubic inch (pci) may be used to design floor slabs.

A 6-inch-thick layer of compacted imported granular material should be placed over the prepared subgrade to provide uniform support and to assist as a capillary break. Imported granular material should be crushed rock or angular gravel and sand that is fairly well-graded between coarse and fine, contains no organic matter and other deleterious materials, has a maximum particle size of 1 1/2 inches, and less than 5 percent passing the U.S. No. 200 Sieve. The imported granular material should be placed in one lift and compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D 1557.

Vapor barriers are often required by flooring manufacturers to protect flooring and flooring adhesives. A typical vapor barrier consists of plastic sheeting covered with 2 inches of sand. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

DRAINAGE CONSIDERATIONS

Foundation drains are not required at this site provided the path and landscape areas surrounding the buildings are graded to drain away from the foundations. We recommend that all roof drains for larger buildings and subsurface drains outlet to a non-perforated pipe leading to the storm drain or stormwater disposal system. Pavement surfaces and open space areas should be sloped such that the surface water runoff is collected and routed to suitable discharge points.

POND AND LINER

Some of the random fill soils found on site are comprised of rubble and gravel that has a characteristically high coefficient of permeability. Consequently, the site is not conducive to the maintenance of a pond without an impermeable liner. We recommend that the pond liner be designed by a qualified specialist consultant. The following paragraphs summarize our geotechnical recommendations for design and construction of the pond.

The site soils are highly variable and subject to differential settlement. Although the pond will not impose additional loads on the existing soils, it is likely that future soil movements will occur that are not associated with consolidation settlement. We recommend that the pond liner consist of a flexible system that can tolerate movements of up to 3 inches over the design life. Thick polymer membranes and polymer/bentonite composites could provide adequate long-term performance. Membranes should be installed in accordance with the manufacturer's recommendations.

We recommend that the pond area be prepared by covering the excavated grade with at least 4 inches of clean sand that is moderately compacted to retain its shape during construction. It is possible that rough rubble surfaces could be exposed in the pond excavation. We recommend that all porous surfaces be covered with a geotextile filter fabric before placing the sand base. Mirafi 140N and Amoco 4551 are two materials that should provide adequate filtration. The Amoco fabrics lighter than 4551 are not recommended because of the low bust strength relative to the rough subgrade materials.

Although near-surface ground water does not affect the site on a persistent basis, we recommend that the pond liner be designed to resist uplift forces caused by ground water within 1 foot of the surrounding street grade. Uplift resistance can be provided by covering the liner with soil or rock, by providing pressure relief ports, or by installing a subsurface drainage system under the pond.

SEISMIC DESIGN PARAMETERS

We recommend that seismic design be performed using the static lateral force procedure outlined in the Uniform Building Code (UBC). The following parameters should be used in computing seismic base shear forces:

Parameter		Value
Seismic Zone Factor	Z	0.30
Soil Profile Type	S	S_D
Seismic Coefficient	C_a	0.36
Seismic Coefficient	C_v	0.54

The site is not affected by persistent high ground water, and does not have continuous layers of clean sandy soils within the upper 20 feet. The potential for liquefaction and ground failure at the site is considered to be negligible.

CONSTRUCTION MONITORING

Satisfactory foundation and earthwork performance depends to a large degree on the quality of construction. Soil conditions at this site are difficult because of the nature and extent of the random fills. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that a qualified geotechnical engineer be retained to observe excavation and general fill placement and to review laboratory compaction, consolidation and field moisture-density information.

Subsurface conditions observed during construction should be compared with those encountered in the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated and to provide recommendations for mitigation, if appropriate.

LIMITATIONS

We have prepared this report for use by the Classical Chinese Garden Trust, KPFF Consulting Engineers and other members of the design and construction team for the proposed Classical Chinese Garden located in Portland, Oregon. The conclusions and recommendations presented in this report are based on explorations that indicate the soil conditions only at those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thicknesses or water level variations that may exist between explorations. If subsurface conditions differing from those described are observed during the course of excavation and construction, reevaluation will be necessary.

We recommend that the final design and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If there are changes in the grades, location, configuration or type of construction for the buildings, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide necessary changes or additional recommendations.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

◆

We appreciate the opportunity to work with you on this project. Please call if you have questions concerning this report or if we can be of additional assistance.

Respectfully submitted,

GeoEngineers, Inc.

Douglas R. Schwarm, P.E.
Senior Engineer

David L. Thielen, P.E.
Associate

REGISTERED PROFESSIONAL ENGINEER
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OREGON
DEC. 1, 1992
DOUGLAS ROBERT SCHWARM
EXP 6.30.00

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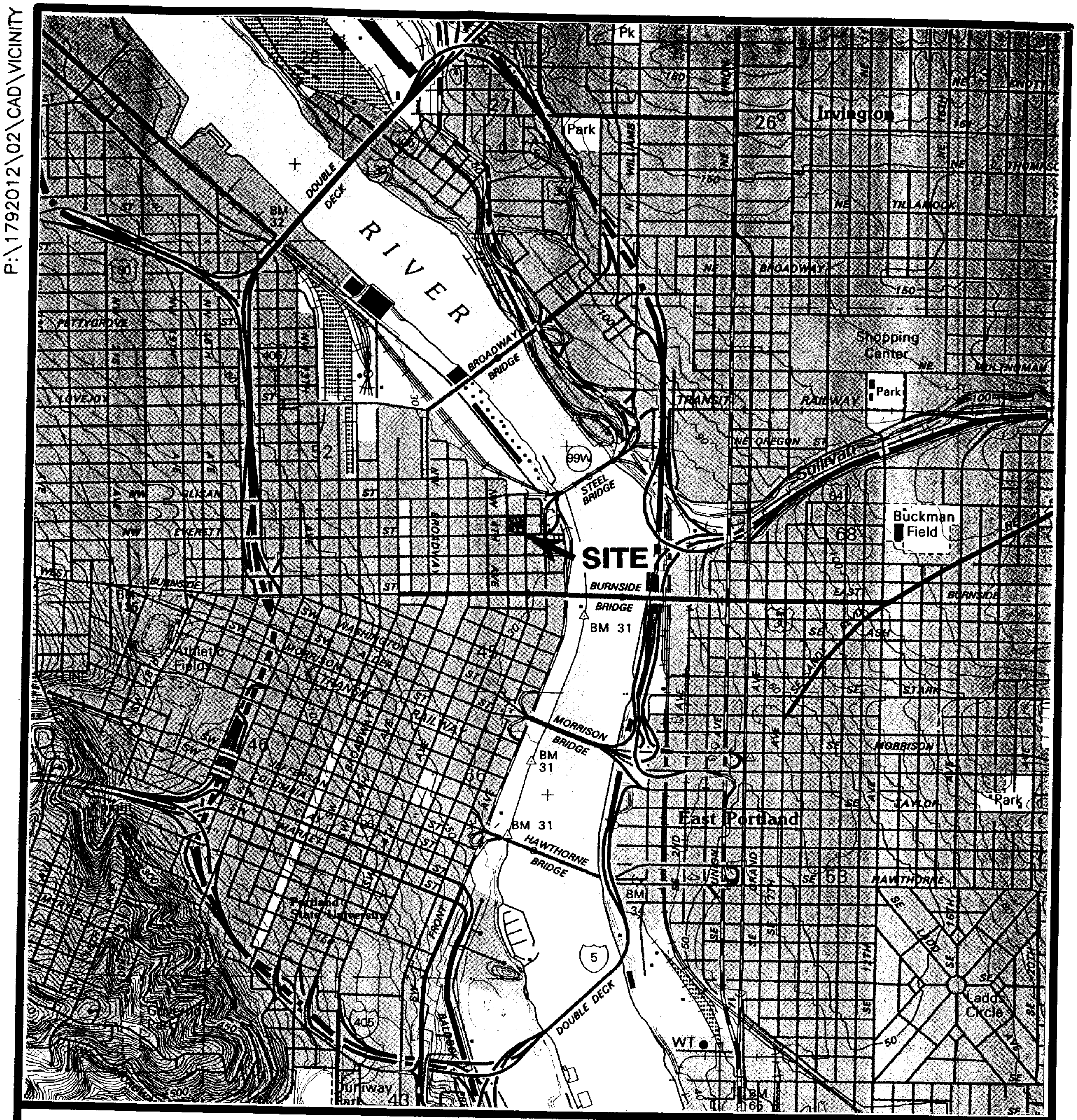

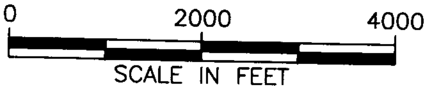

Reference: USGS 7.5' topographic quadrangle map, "Portland, OR–WA," dated 1990.

GeoEngineers

VICINITY MAP

FIGURE 1

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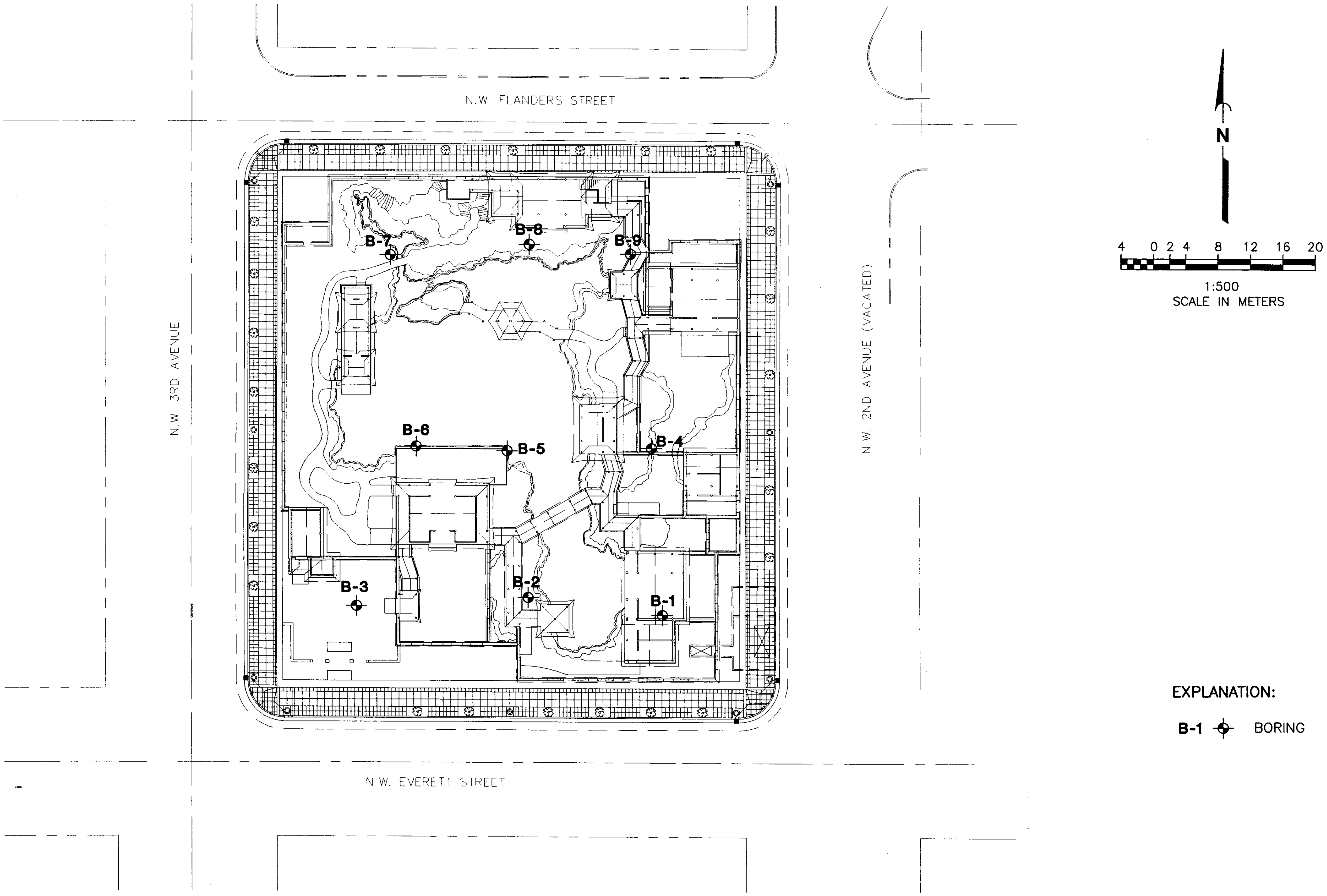

Reference: Base drawing produced from undated AutoCad file, "6237Base.DWG", provided by KPFF.

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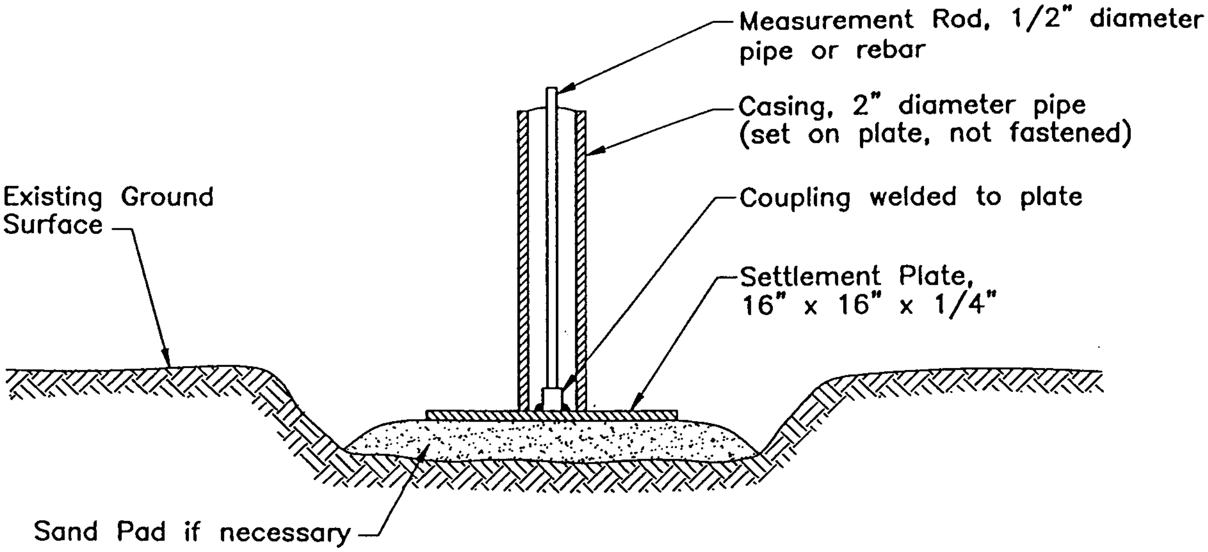
SITE PLAN

FIGURE 2

C:\DETAILS\SETPLATE



(Not to Scale)

NOTES:

1. Install markers on firm ground or on sand pads if needed for stability. Take initial reading on top of rod and at adjacent ground level prior to placement of any fill.
2. For ease in handling, rod and casing are usually installed in 5-foot sections. As fill progresses, couplings are used to install additional lengths. Continuity is maintained by reading the top of the measurement rod, then immediately adding the new section and reading the top of the added rod. Both readings are recorded.
3. Record the elevation of the top of the measurement rod in each marker at the recommended time intervals. Each time, note the elevation of the adjacent fill surface.
4. Read the marker to the nearest 0.01 foot, or 0.005 foot if possible. Note the fill elevation to the nearest 0.1 foot.
5. The elevations should be referenced to a temporary benchmark located on stable ground at least 100 feet from the embankment.

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SETTLEMENT PLATE DETAIL

FIGURE 3

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface conditions at the site were explored by drilling 9 soil borings at the approximate locations shown in Figure 2. Exploration locations were chosen based on a preliminary site plan provided to our office by KPFF Consulting Engineers. The boring locations were determined in the field by measuring from existing site features. Exploration locations should be considered accurate only to the degree implied by the method used.

The borings were completed using a hollow-stem auger drill rig owned and operated by Crisman Drilling of Tigard, Oregon. The borings were completed on November 11, 1998 to depths ranging between 11.5 and 21.5 feet. None of these borings met refusal conditions.

Representative soil samples were obtained at selected depths within each boring. Relatively undisturbed samples were taken using a Dames & Moore sampler or a split-spoon sampler. The materials observed were classified in the field in accordance with ASTM Test Method D 2488, which is described in Figure A-1. The boring logs indicate the depths at which the soils or their characteristics change, although the change actually may be gradual. Figure A-2 provides a description of the boring log forms. Soil classifications and sampling intervals are shown in the boring logs (Figures A-3 through A-11).

LABORATORY TESTING

Representative samples obtained from the borings were examined visually to confirm or modify field classifications. Selected soil samples were tested to determine the natural moisture content in general accordance with ASTM Test Method D 2216, in-situ dry density in general accordance with ASTM Test Method D 2937. The laboratory test results are summarized in the boring logs.

One consolidation test was performed in general conformance with ASTM Test Method D 2435 to evaluate the compressibility characteristics of the foundation soils. Figure A-12 presents the test results.

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More Than 50% Retained on No. 200 Sieve	GRAVEL More Than 50% of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More Than 50% of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More Than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less Than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or More	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is based on ASTM D2487-90.
3. Descriptions of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry -	Absence of moisture, dusty, dry to the touch
Moist -	Damp, but no visible water
Wet -	Visible free water or saturated, usually soil is obtained from below water table

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SOIL CLASSIFICATION SYSTEM

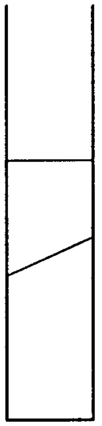
FIGURE A-1

G:\ACAD\DETAILS\GTKBR300

LABORATORY TESTS:

AL	Atterberg limits
CP	Compaction
CS	Consolidation
DS	Direct Sheer
GS	Grain – size
%F	Percent fines
HA	Hydrometer analysis
SK	Permeability
SM	Moisture content
MD	Moisture and density
SP	Swelling pressure
TX	Triaxial compression
UC	Unconfiend compression
CA	Chemical analysis
CTX	Cyclic triaxial testing

SOIL GRAPH:



SM Soil Group Symbol (See Note 2)

Distinct Contact Between Soil Strata

Gradual or Approximate Location of Change Between Soil Strata

∇ Water Level

Bottom of Boring

BLOW-COUNT/SAMPLE DATA:

Blows required to drive a 2.4–inch I.D. split–barrel sampler 12 inches or other indicated distances using a 300–pound hammer falling 30 inches.

22 ■ Location of relatively undisturbed sample

12 ☒ Location of disturbed sample

17 ☐ Location of sampling attempt with no recovery

Blows required to drive a 1.5–inch I.D. (SPT) split–barrel sampler 12 inches or other indicated distances using a 140–pound hammer falling 30 inches.

10 ◧ Location of sample obtained in general accordance with Standard Penetration Test (ASTM D 1586) procedures

26 ◫ Location of SPT sampling attempt with no recovery

"P" indicates sampler pushed with weight of hammer or against weight of drill rig.

P Ⅰ Location of relatively undisturbed sample obtained using a 3–inch–diameter thin–wall sample tube. Sample obtained in general accordance with ASTM D 1587.

▤ Location of grab sample

NOTES:

1. The reader must refer to the discussion in the report text, the Key to Boring Log Symbols and the exploration logs for a proper understanding of subsurface conditions.
2. Soil classification system is summarized in Figure A–1.

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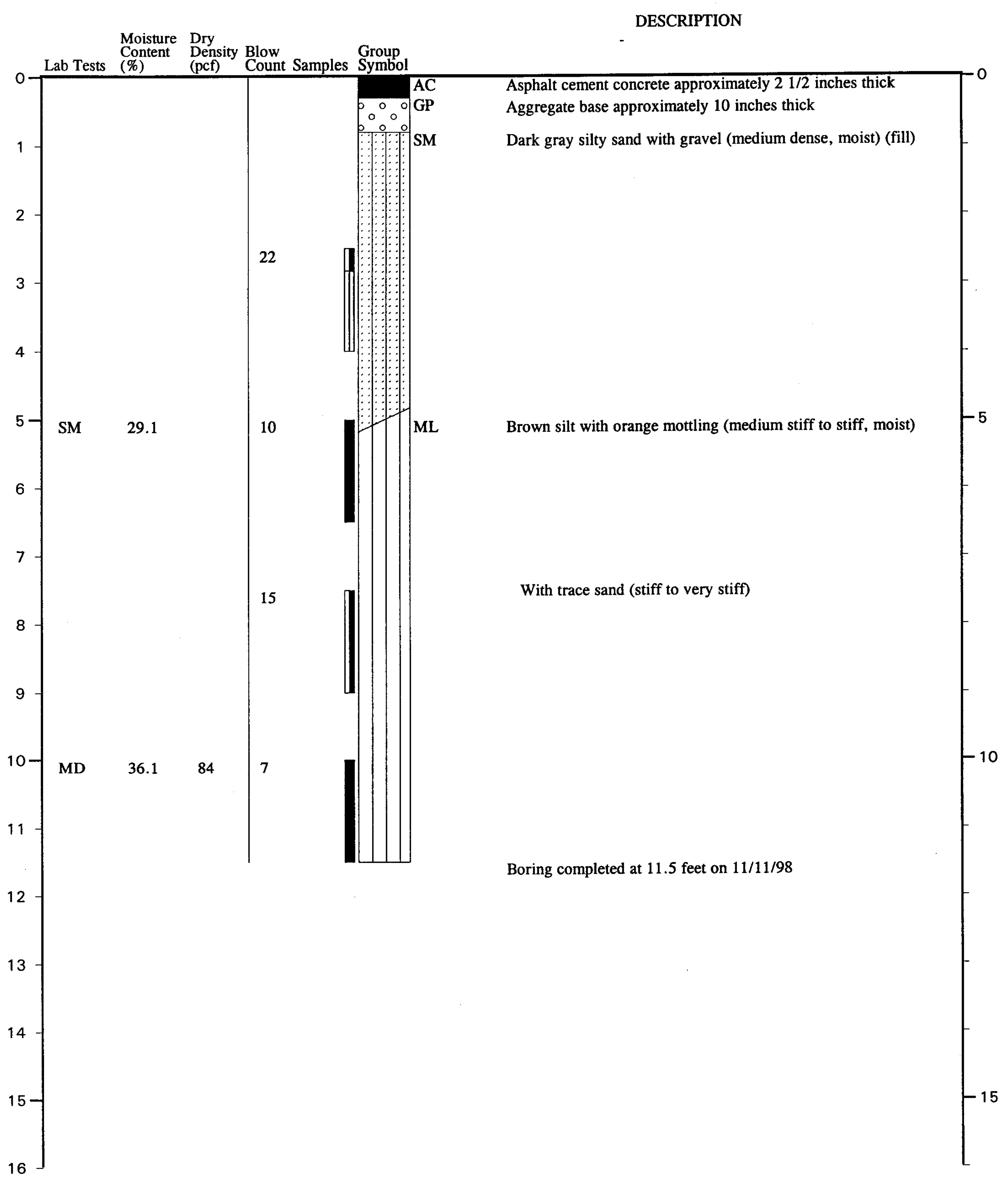
KEY TO EXPLORATION LOG SYMBOLS

FIGURE A-2

GEI 121–90

BORING B-1



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	Description
				AC	Asphalt cement concrete approximately 2 1/2 inches thick
				GP	Aggregate base approximately 10 inches thick
				SM	Dark gray silty sand with gravel (medium dense, moist) (fill)
			22		
SM	29.1		10	ML	Brown silt with orange mottling (medium stiff to stiff, moist)
			15		With trace sand (stiff to very stiff)
MD	36.1	84	7		
					Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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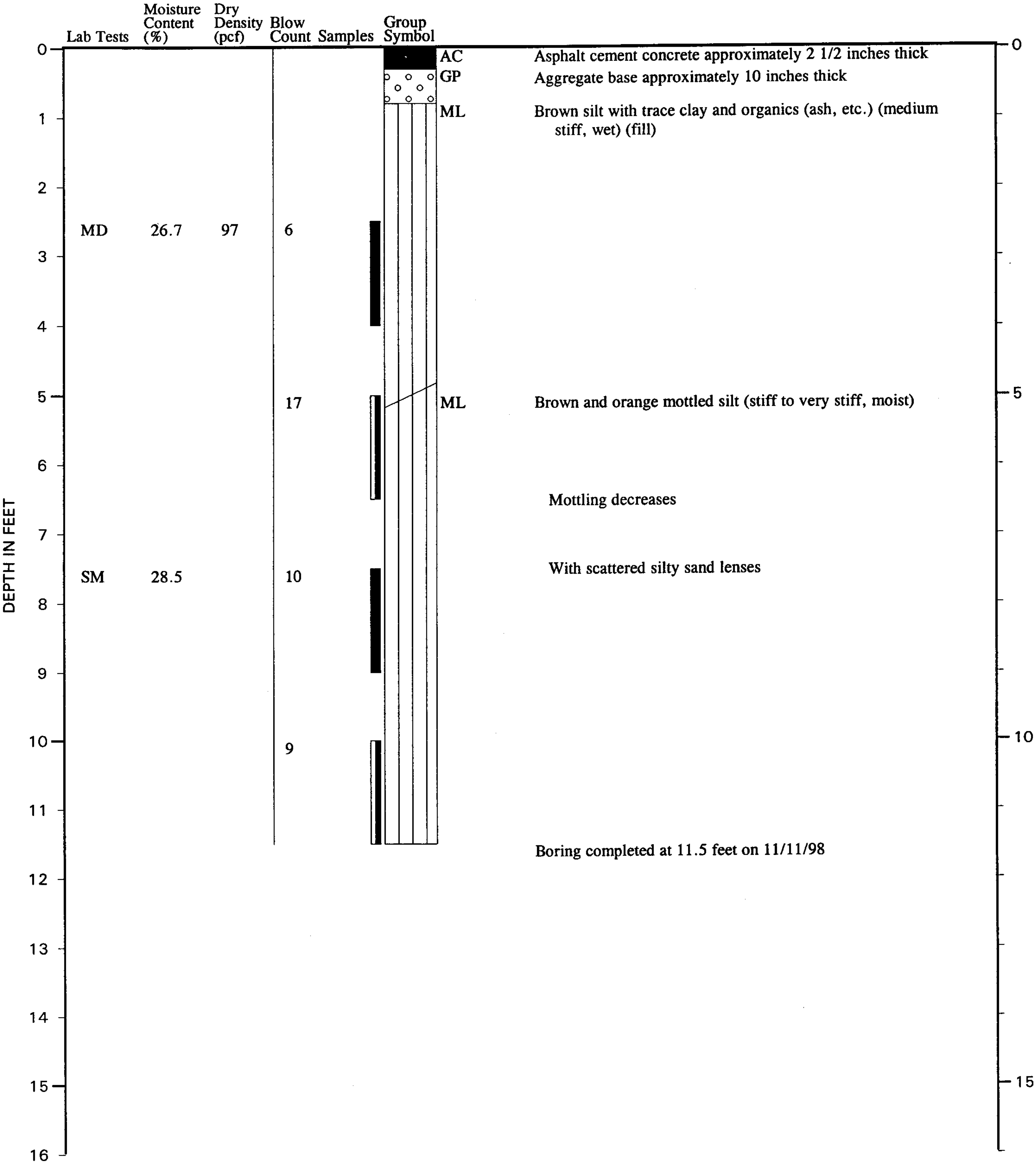
LOG OF BORING

FIGURE A-3

TEST DATA

BORING B-2

DESCRIPTION



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	Description
				AC	Asphalt cement concrete approximately 2 1/2 inches thick
				GP	Aggregate base approximately 10 inches thick
				ML	Brown silt with trace clay and organics (ash, etc.) (medium stiff, wet) (fill)
MD	26.7	97	6		
			17	ML	Brown and orange mottled silt (stiff to very stiff, moist)
					Mottling decreases
SM	28.5		10		With scattered silty sand lenses
			9		
					Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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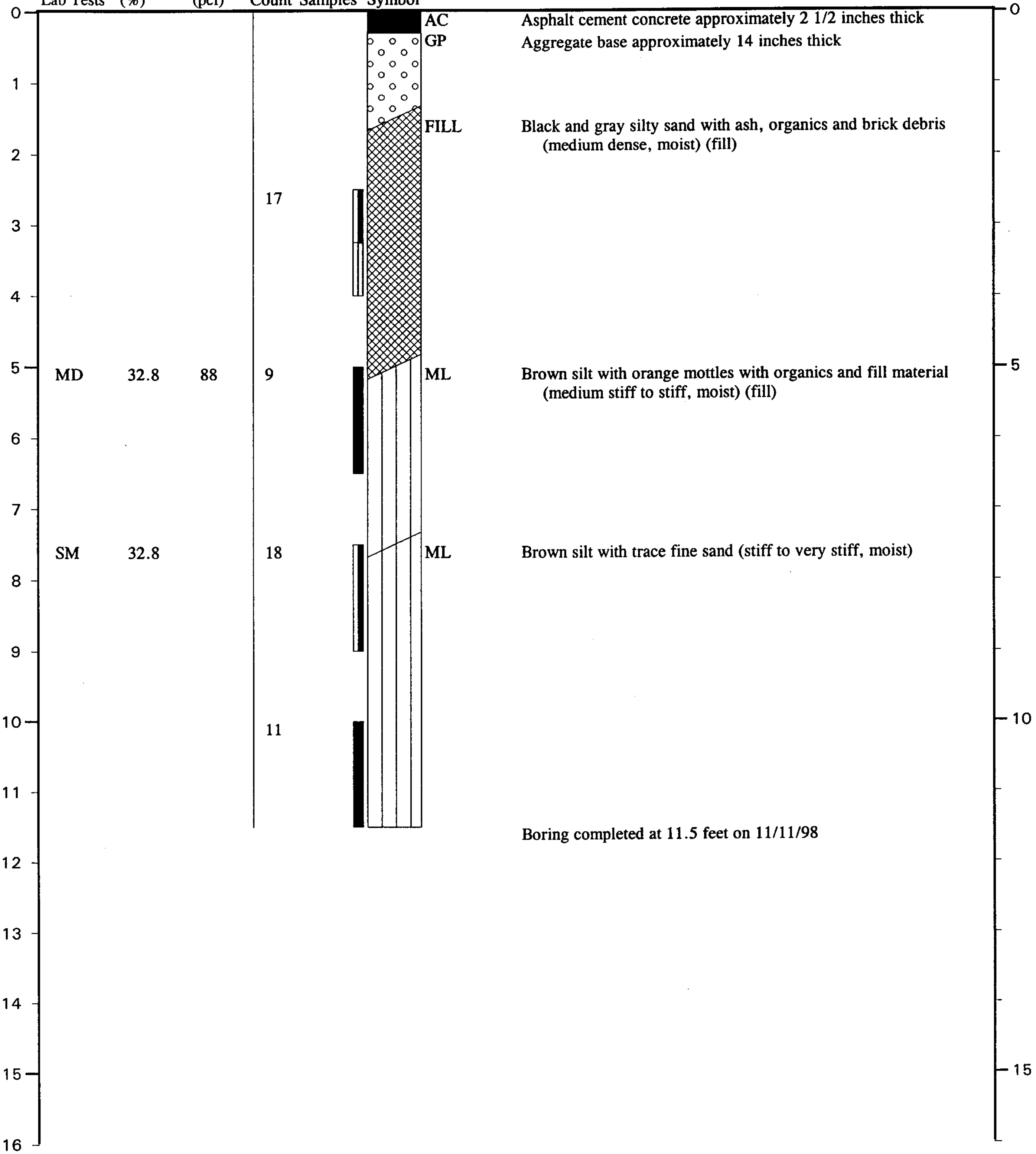
LOG OF BORING

FIGURE A-4

TEST DATA

BORING B-3



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	DESCRIPTION
				AC	Asphalt cement concrete approximately 2 1/2 inches thick
				GP	Aggregate base approximately 14 inches thick
				FILL	Black and gray silty sand with ash, organics and brick debris (medium dense, moist) (fill)
			17		
MD	32.8	88	9	ML	Brown silt with orange mottles with organics and fill material (medium stiff to stiff, moist) (fill)
SM	32.8		18	ML	Brown silt with trace fine sand (stiff to very stiff, moist)
			11		
					Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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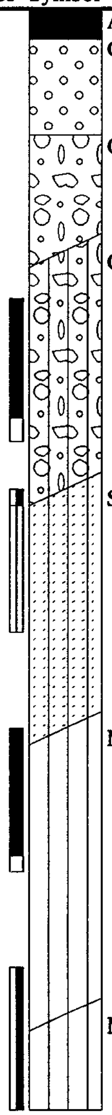
LOG OF BORING

FIGURE A-5

TEST DATA

BORING B-4



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	DESCRIPTION
				AC	Asphalt cement concrete approximately 2 1/2 inches thick
				GP	Aggregate base approximately 12 inches thick
				GW	Dark gray silty gravel (medium dense, moist) (fill)
				GM	Gray silty gravel (loose, moist) (fill)
MD	36.4	78	6		Grades to sand with ash (loose, moist)
			4	SM	Gray and brown silty coarse sand with organics and brick debris (very loose to loose, moist) (fill)
			4	ML	Brown silt with ash and organics (soft, moist) (fill)
			8		
				ML	Light brown silt with trace sand (medium stiff, wet)

Boring completed at 11.5 feet on 11/11/98

DEPTH IN FEET

Note: See Figure A-2 for explanation of symbols

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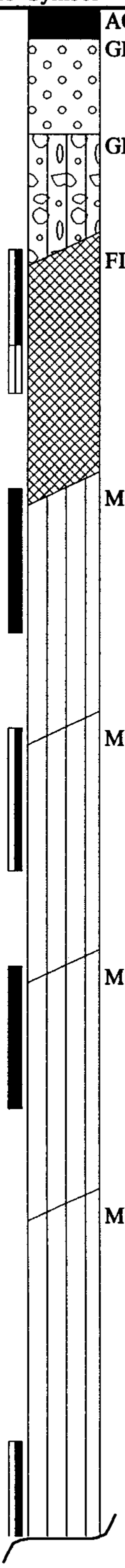
LOG OF BORING

FIGURE A-6

BORING B-5

TEST DATA

DESCRIPTION



DEPTH IN FEET	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
0						AC	Asphalt cement concrete approximately 2 1/2 inches thick
						GP	Aggregate base approximately 11 inches thick
						GM	Brown silty sandy gravel (medium dense, moist) (fill)
				51		FILL	Dark brown silty sand with ash, brick and other organics (loose, moist)
5	CS			11		ML	Brown silt with brick and other debris (stiff, moist) (fill)
	SM	29.9		15		ML	Brown silt (medium dense, moist)
10	SM	36.8		7		ML	Brown to gray silt with trace clay (medium stiff, moist)
						ML	Brown sandy silt, micaceous (medium stiff, wet)
15				12			

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

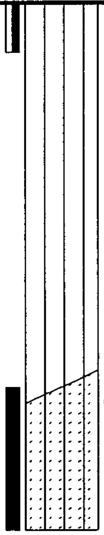
FIGURE A-7

TEST DATA

BORING B-5
(Continued)

DESCRIPTION

DEPTH IN FEET	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
20	SM	26.5		8		SM	Brown silty fine sand, slightly micaceous (loose, wet)
21.5							Boring completed at 21.5 feet on 11/11/98



Note: See Figure A-2 for explanation of symbols

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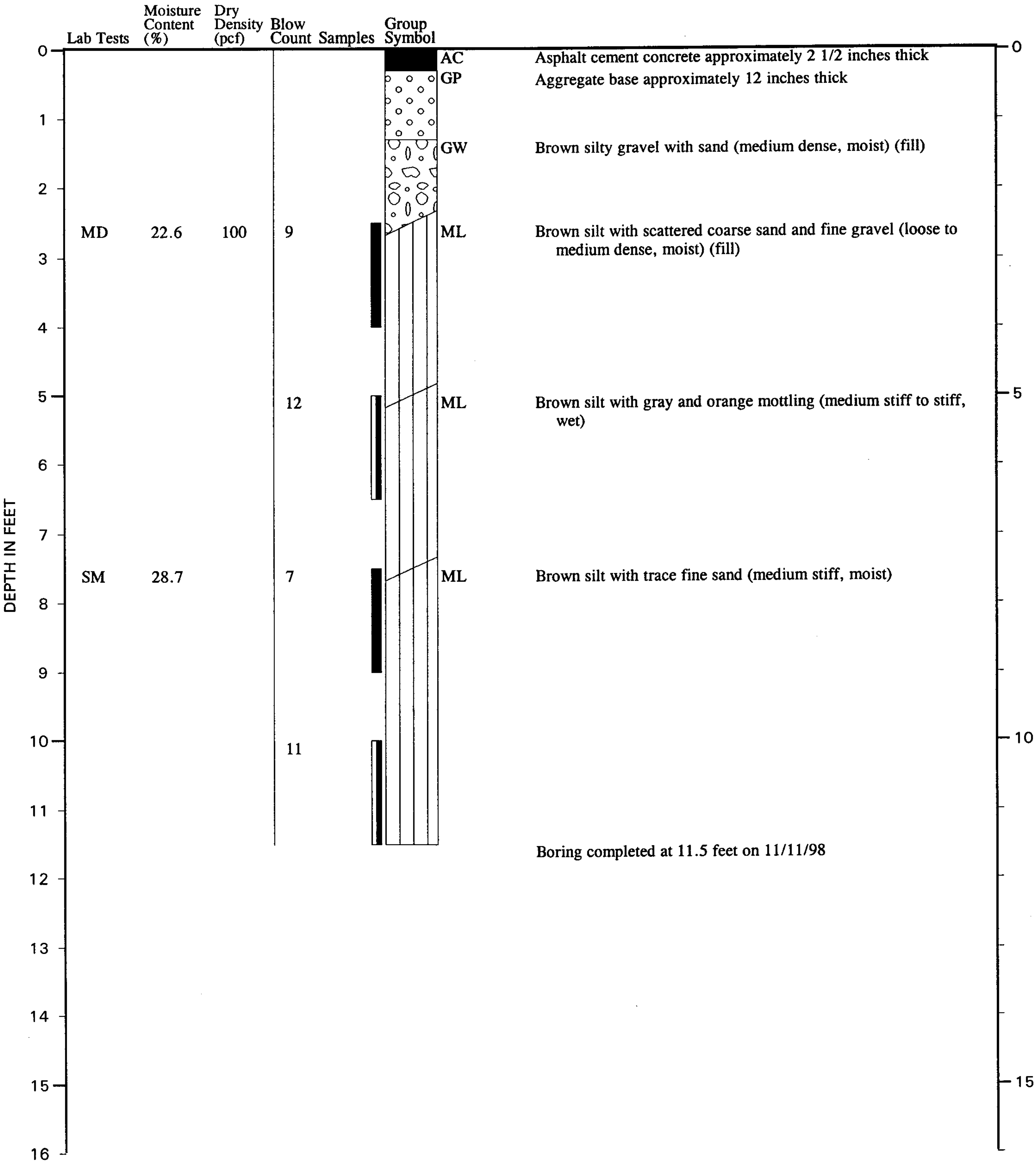
LOG OF BORING

FIGURE A-7

BORING B-6

TEST DATA

DESCRIPTION



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
					AC	Asphalt cement concrete approximately 2 1/2 inches thick
					GP	Aggregate base approximately 12 inches thick
					GW	Brown silty gravel with sand (medium dense, moist) (fill)
MD	22.6	100	9	▮	ML	Brown silt with scattered coarse sand and fine gravel (loose to medium dense, moist) (fill)
			12	▯	ML	Brown silt with gray and orange mottling (medium stiff to stiff, wet)
SM	28.7		7	▮	ML	Brown silt with trace fine sand (medium stiff, moist)
			11	▯		

Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

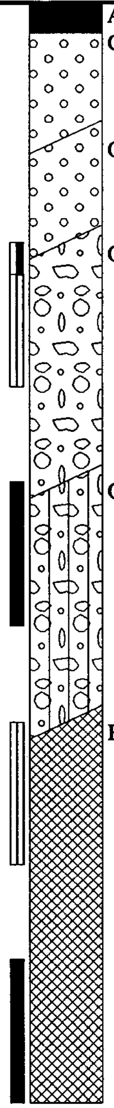
FIGURE A-8

DRS:JJB:mln 11/24/98

TEST DATA

BORING B-7

DESCRIPTION



Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
					AC	Asphalt cement concrete approximately 2 1/2 inches thick
					GP	Aggregate base approximately 13 inches thick
					GP	Dark gray sandy gravel with concrete and brick debris (medium dense, moist) (fill)
			15		GW	Gray silty gravel with sand and silt clods (loose to dense, moist) (fill)
MD	11.0	115	9		GM	Gray silt with gravel (loose to medium dense, moist) (fill)
			17		FILL	Red brick and organic fill (medium dense to dense, dry) (fill)
			31			

Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

FIGURE A-9

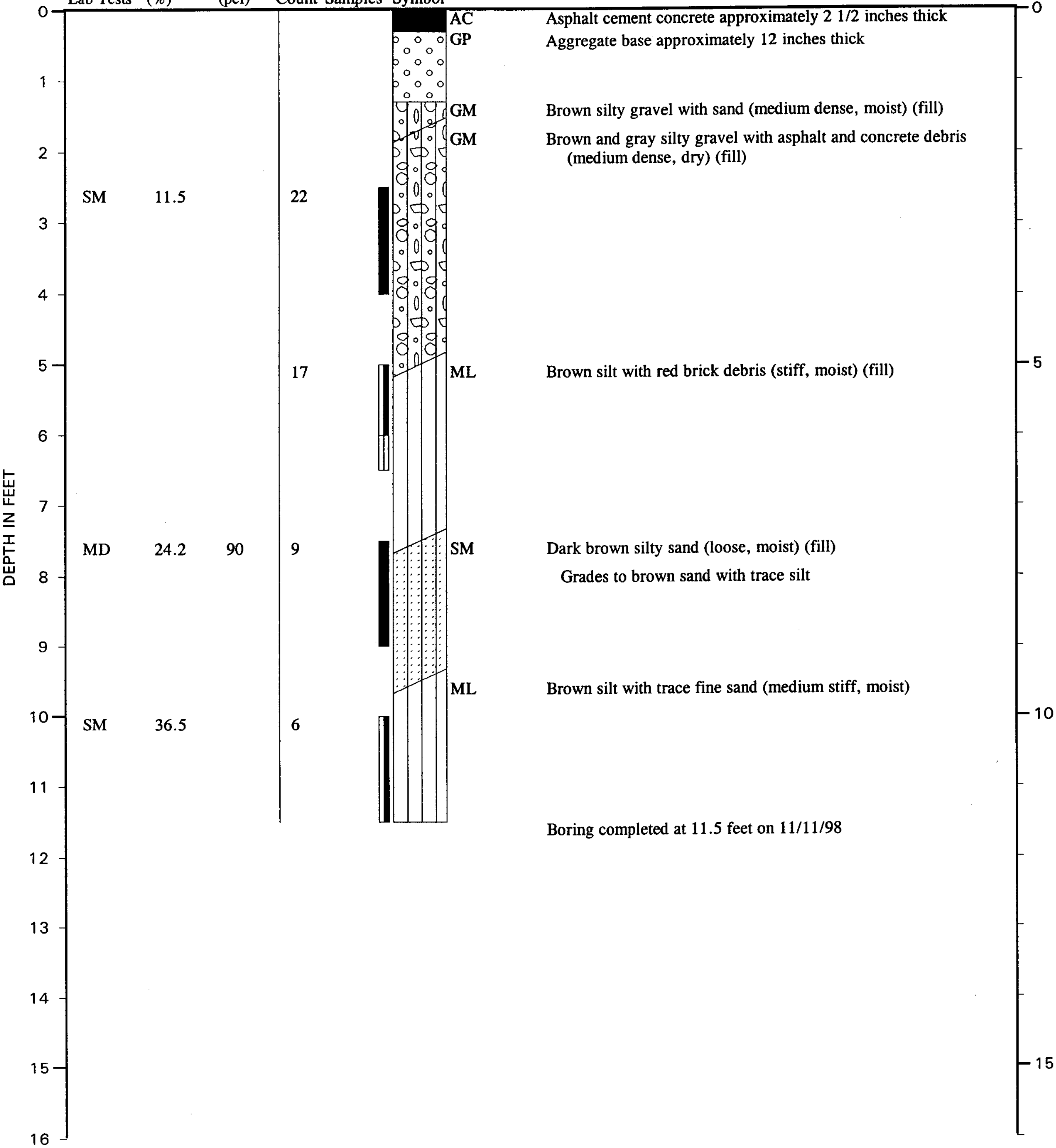
BORING B-8

TEST DATA

DESCRIPTION

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Group Symbol	Description
				AC	Asphalt cement concrete approximately 2 1/2 inches thick
				GP	Aggregate base approximately 12 inches thick
				GM	Brown silty gravel with sand (medium dense, moist) (fill)
				GM	Brown and gray silty gravel with asphalt and concrete debris (medium dense, dry) (fill)
SM	11.5		22		
			17	ML	Brown silt with red brick debris (stiff, moist) (fill)
MD	24.2	90	9	SM	Dark brown silty sand (loose, moist) (fill) Grades to brown sand with trace silt
				ML	Brown silt with trace fine sand (medium stiff, moist)
SM	36.5		6		

Boring completed at 11.5 feet on 11/11/98



Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

FIGURE A-10

TEST DATA

BORING B-9

DEPTH IN FEET	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
0						AC	Asphalt cement concrete approximately 2 1/2 inches thick
						GP	Aggregate base approximately 11 inches thick
						GM	Gray silty gravel (medium dense, moist) (fill)
						ML	Brown silt with gravel (soft, moist) (fill) 3/4-inch-thick organic silt layer (soft, wet) (fill)
	SM	29.6		4		FILL	Ash, wood debris, and charcoal with brick debris
						ML	Brown silt with trace clay (soft, wet)
5	SM	28.3		5			
				14			Brown clayey silt (medium stiff to medium dense, moist)
10	SM	33.7		12			With trace fine sand
							Boring completed at 11.5 feet on 11/11/98

Note: See Figure A-2 for explanation of symbols

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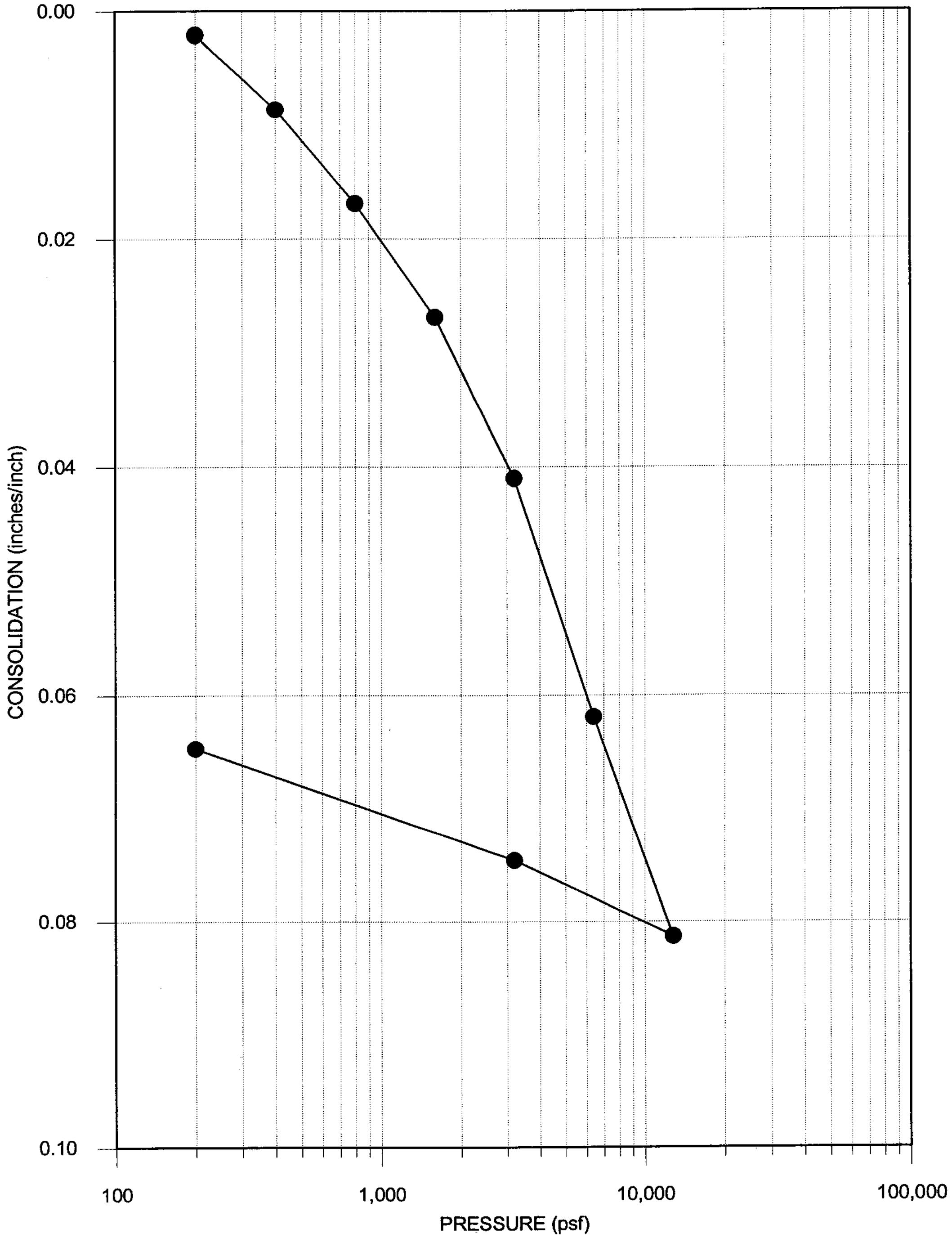
LOG OF BORING

FIGURE A-11



KEY	BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT^3)
●	B-5	5.0	Dark Brown silt, (ML)	33.4%	88.5

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CONSOLIDATION TEST RESULTS

FIGURE A-12

APPENDIX B

ARC

Associated Rockery Contractors

Standard
Rock Wall Construction
Guidelines

P.O. Box 1794 • Woodinville, Washington 98072

Association Representatives
(206) 481-3456 or (206) 481-7222

Contents

ARC STANDARD ROCKERY CONSTRUCTION GUIDELINES

1.01 Introduction:

1.01.1 Historical Background These standard rock wall construction guidelines have been developed in an effort to provide a more stringent degree of control on 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 Associated Rockery Contractors (ARC).

1.01.2 Goal The primary goals of this document are to standardize the methods of construction for rock walls over four feet in height, and to provide a means of verifying the quality of 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, angular ledge rock that is resistant to weathering. 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 ASTM C127 AASHTO T-85	*Not more than 2.0% for igneous and metamorphic rock types and 3.0% for sedimentary rock types.*
b.	Accelerated Expansion *(15 days)* CRD-C-148 *1, *2	*Not more than 15% breakdown.*
c.	Soundness *(MsS04 at 5 cycles)* ASTM C88 CRD-C-137	*Not greater than 5% loss.*
d.	Unconfined Compressive Strength ASTM D 2938	*Intact strength of 6,000 psi, or greater.*
e.	Bulk Specific Gravity (155pcf) ASTM C127 AASHTO T-85	*Greater than 2.48*

*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.".

*2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock.

2.01.2 Frequency of Testing Quarry sources 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 (12,000) tons of wall rock supplied to establish that specific rock source. The tests shall then be performed once a year, every 40,000 tons, 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 equal to, or greater than, one hundred fifty-five (155) pcf. Typically, rocks used for rock wall construction shall be sized approximately as follows:

Rock Size	Rock Weight	Average Dimension
One man	50-200 pounds	12 to 18 inches
Two man	200-700 pounds	18 to 28 inches
Three man	700-2000 pounds	28 to 36 inches
Four man	2000-4000 pounds	36 to 48 inches
Five Man	4000-6000 pounds	48 to 54 inches
Six Man	6000-8000 pounds	54 to 60 inches

In rock walls eight feet and over in height, it should not be possible to move the large sized rocks (four to six-man size) with a pry bar. If these rocks can be moved, the rock wall 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 pry bar to achieve the "best fit".

2.01.4 Submittals The rock source shall present current geologic and test data for the minimum guidelines described in Section 2.01.1 on request by either the rock wall contractor, the owner, or the applicable agency.

3.01 Rock Wall Construction:

3.01.1 General Rock wall construction is a craft and depends largely on the skill and experience of the builder. A rock wall is a protective system which helps to retard the weathering and erosion process acting on an exposed cut or fill soil face. While by its nature (the mass, size and shape of the rocks) it will provide some undetermined 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 rock wall being constructed. The larger the rock, the more competent the rock wall. To accomplish an appropriate degree of competency, all rock walls in excess of four feet in height should be built on a "mass" basis, i.e. by the ton.

To provide a competent and adequate rock wall structure, all rock walls constructed in front of either cuts or fills eight feet and over in height should be bid and constructed in accordance with these standard guidelines and the geotechnical engineer's supplemental recommendations. Both the standard guidelines and the supplemental geotechnical recommendations should be provided to prospective bidders before bidding and the start of construction.

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

3.01.3 Responsibility The ultimate responsibility for standard rock wall construction should remain with the rock wall builder. However, rock walls 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 eight feet or more in height, all represent special design conditions and require consultation and/or advice from qualified experts.

3.01.4 Workmanship All workmanship is guaranteed by the rock wall contractor and all materials are guaranteed by the 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, temporary excavation of ditches or trenches for any utility within a distance of less than five feet from the back of the top of the rock wall; excavation made either within a distance equal to at least two thirds of the free-standing wall height in front of the toe of a rock wall, or that will penetrate an imaginary line extended at a 1H:1V (Horizontal: Vertical) slope from the front edge of the rock wall toe (see Figure A); removal of any material from the subgrade in front of the wall, excavation of material from any location behind the rock wall within a distance at least equal to the rock wall's height, the addition of any surcharge or other loads within a similar distance of the top of the rock wall, or surface or subsurface water forced, directed, or otherwise caused to flow behind the rock wall in any quantity.

3.01.6 Slopes Slopes above rock walls should be kept as flat as possible, but should not exceed 2H:1V unless the rock wall is designed specifically to provide some restraint to the load imposed by the slope. Any slope existing above a completed rock wall should be covered with vegetation 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, will typically 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 geotextile, to help maintain the seed in-place until the root mat has an opportunity to germinate and take hold.

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

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

3.01.8 Fill Compaction Where rock walls 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 seven percent fines (silt and clay sized particles) passing the number 200 mesh sieve.

All fills should be placed in thin lifts not exceeding ten (10) 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 that this degree of compaction is being achieved.

3.01.9 Fill Construction Reinforcement There are two methods of constructing a fill. The first, which typically applies to rock walls of less than eight feet in height, is to overbuild and then cut back the fill. The second, which applies to all rock walls eight feet and over in height, is to construct the fill using a geogrid or geotextile 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 rock wall.

For the higher rock walls the use of a geogrid or geotextile fabric to help reinforce the fill results in construction of a more stable fill face against which to construct the rock wall. This form of construction leads to a longer lasting and more stable rock wall 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, the amount of lapping and the construction sequence must be determined on a case by case basis.

3.01.10 Rock Wall Keyway

The first step in rock wall construction, after general excavation, is to construct a keyway in which to build the rock wall. The keyway shall comprise a shallow trench of at least twelve (12) inches in depth, extending for the full length of the rock wall. The keyway subgrade should be slightly inclined back towards the face being protected. It is typically dug as wide as the rock wall (including the width of the rock filter layer). If the condition of the cut 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 day's work.

The competency of the keyway subgrade to support the rock wall shall be verified by probing with a small diameter steel rod. The rod shall have a diameter of between three-eighths 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, with ease, shall indicate a "soft" subgrade and one that could require treatment. Shallow soft areas of the subgrade can be "firmed up" by tamping a layer of coarse quarry spalls into the subgrade.

3.01.11 Keyway Drainage

Upon 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 key way. A minimum four-inch diameter perforated or slotted rigid 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 rock wall rock. This drain pipe should be installed with sufficient gradient to initiate flow, and the outfall should be connected to a positive and permanent discharge.

Positive and permanent drainage should be considered to ~~mean~~ connect to 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 Rock Wall Thickness

The individual rock wall thickness should be equal to the thickness of the recommended size of rock plus the thickness of the drain rock layer. This thickness, which will be determined on a case by case basis, will be dependant on the specific rock sizes recommended for each individual rock wall. For example, if four-man rock is used the rock wall thickness will be approximately five feet.

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 rock wall 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 rectangular shape and selected in accordance with Section 2.01.3. Any rocks of basically rounded or tetrahedral form should be rejected or used for filling large void spaces.

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. The bottom of the first course of rock should be a minimum of twelve (12) inches below the lowest adjacent site grade.

As the rock wall is constructed, the rocks should be placed so that there are no continuous joint planes in either the vertical or lateral direction. Wherever possible, 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 (the top surface of rock), should slope back towards the cut face and away from the face of the rock wall.

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

3.01.15 Face Inclination The face of the rock wall should be inclined at a gradient of about 1H:6V back towards the face being protected. The inclination should not be constructed flatter than 1H:4V.

3.01.16 Voids Because of the nature of the product used to construct a rock wall, 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 rock wall construction. As the size of rock used to build a rock wall increases, i.e. to six-man size, the void spaces between individual rocks should be expected to be larger.

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

3.01.17 Drain Rock Layer In order to provide some degree of drainage control behind the rock wall, and as a means of helping to prevent loss of soil through the face of the rock wall, a rock drainage filter shall be installed between the rear face of the rock wall and the soil face being protected. This drain rock layer should be at least twelve (12) inches thick; and for rock walls eight feet in height or higher, it should be at least eighteen (18) inches thick. It should be composed of 4 to 2-inch sized crushed rock quarry spalls, crushed concrete, or other material approved by the geotechnical engineer. If a random wall rock extends back to the exposed soil face, it is not necessary that the filter rock layer extend between it and the soil face.

Depending on soil type and potential water seepage, a geotextile fabric may or may not be required. This can be determined on a case by case basis by the geotechnical engineer during design and prior to bidding.

3.01.18 Surface Drainage It is the owner's responsibility to intercept surface drainage from above the rock wall and direct it away from the rock wall to a positive and permanent discharge well below and beyond the toe of the rock 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.

GLEN MANN
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

Post Construction Guidline

Schematic Only - Not to Scale

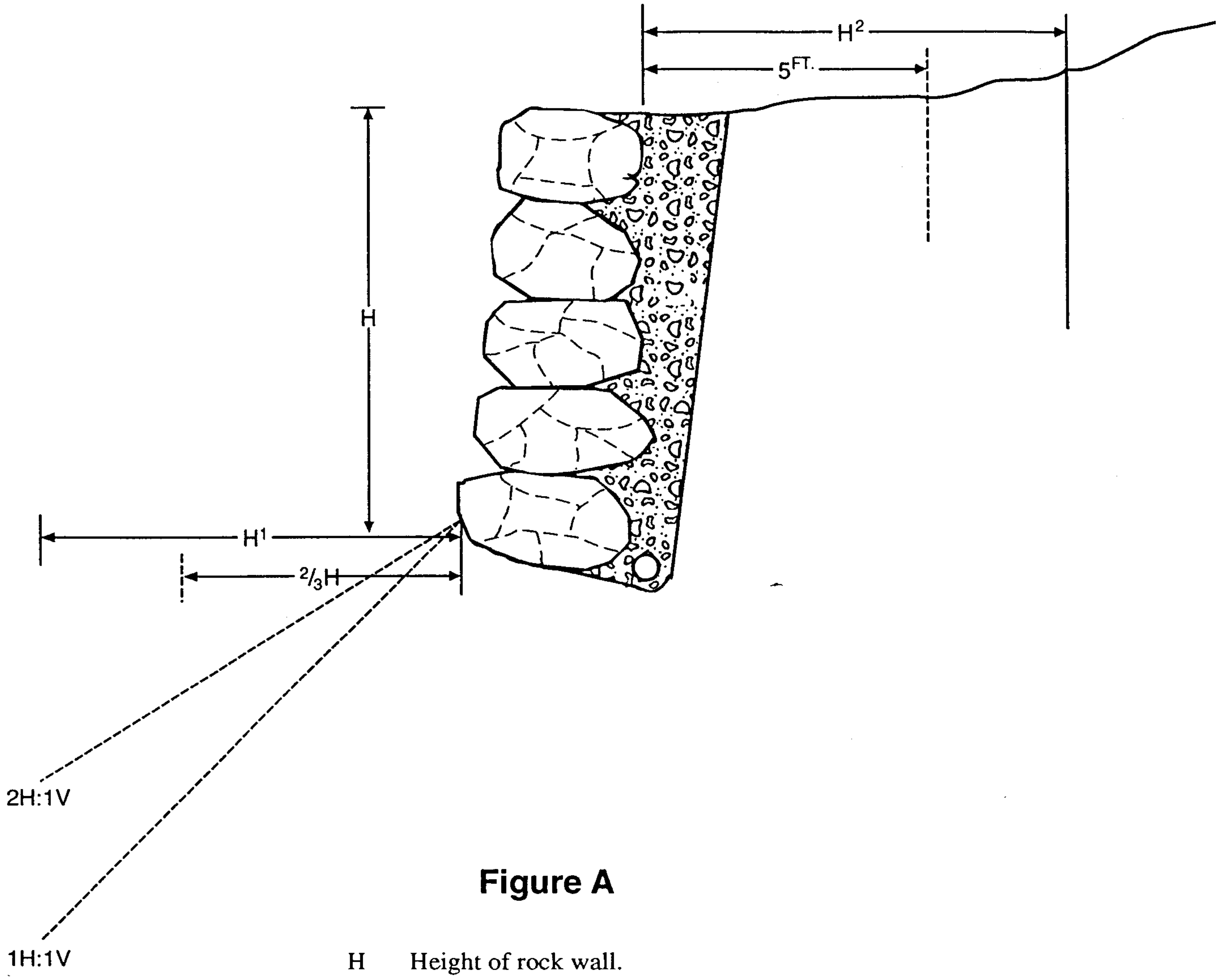
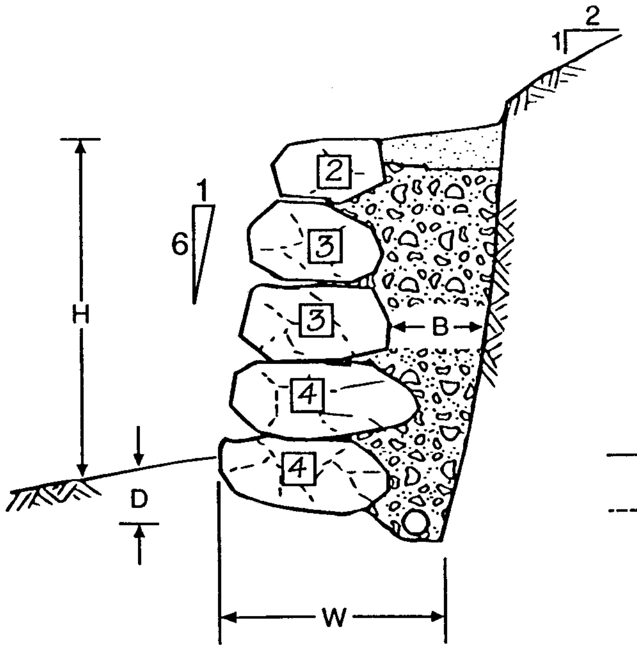


Figure A

H	Height of rock wall.
5^{FT}	No temporary excavation or trenching.
$^2/_3H$	No temporary excavation or trenching.
H^1	No permanent removal of subgrade allowed.
H^2	No excavation of material or any surcharge allowed.
1H:1V	No excavation below allowed.
2H:1V	Maximum finished grade or permanent excavation .

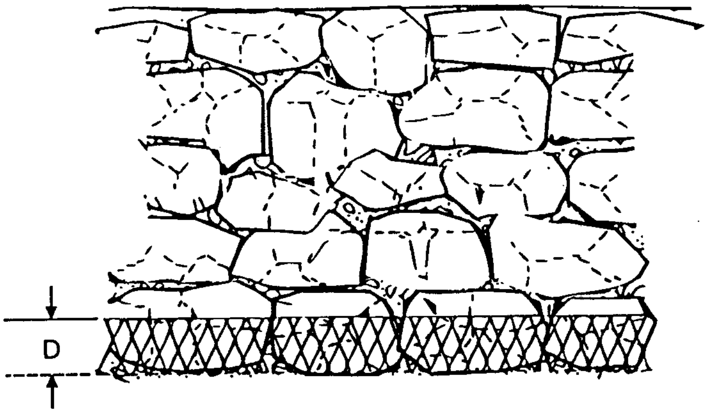
Schematic Only - Not to Scale



Rock Wall Section

NOTES:

- Rock wall construction is a craft and depends largely on the skill and experience of the builder.
- A rock wall is a protective system which helps retard the weathering and erosion process on an exposed soil face.
- While by its nature (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 dependent on the size of the rock used; that is, the mass or weight, and the height of the wall being constructed. The larger the rock, the more competent the rock wall should be.
- Rock walls should be considered maintenance items that will require periodic inspection and repair. They should be located so that they can be reached by a contractor if repairs become necessary.
- Maximum inclination of the slopes above and behind rock walls should be 2:1 (Horizontal:Vertical).
- Minimum thickness of rock filter layer B = 12 inches, Minimum embedment D = 12 inches undisturbed native soil or compacted fill placed in accordance with report recommendations.
- Maximum rock wall height H = _______ feet.
- Rock walls greater than 8 feet in height to be installed under periodic or full time observation of the geotechnical engineer.
- Rock should be placed to gradually decrease in size with increasing wall height in accordance with geotechnical engineers recommendations.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.



Rock Wall Elevation

- The long dimension of the rocks should extend back towards the cut or fill face to provide maximum stability. Rocks should not be stacked like shoe boxes. They should be placed to avoid continuous joint planes in vertical or lateral directions. Whenever possible each rock should bear on two or more rocks below it, with good flat-to-flat contact.
- All rock walls over 4 feet in height should be constructed on basis of wall mass, not square footage of face.

Size	Approximate Weight - lbs.	Approximate Diameter
1 Man	50 - 200	12 - 18"
2 Man	200 - 700	18 - 28"
3 Man	700 - 2000	28 - 36"
4 Man	2000 - 4000	36 - 48"
5 Man	4000 - 6000	48 - 54"
6 Man	6000 - 8000	54 - 60"

Reference: Local quarry weight study using average weights of no less than six rocks of each man size conducted in January 1, 1988.

LEGEND:

▨	Drainage materials to consist of clean angular 4 to 2 inch spalls, or other material , approved by the geotechnical engineer
░	Surface seal: may consist of impervious soil or a fine free draining granular material.
▤	Undisturbed firm Native soil
○	Drainpipe: 4-inch minimum diameter, perforated or slotted rigid plastic ADS pipe laid with a positive gradient to discharge under control well away from the wall.
$\boxed{4}$	Designates size of rock required, i.e. 4 man.

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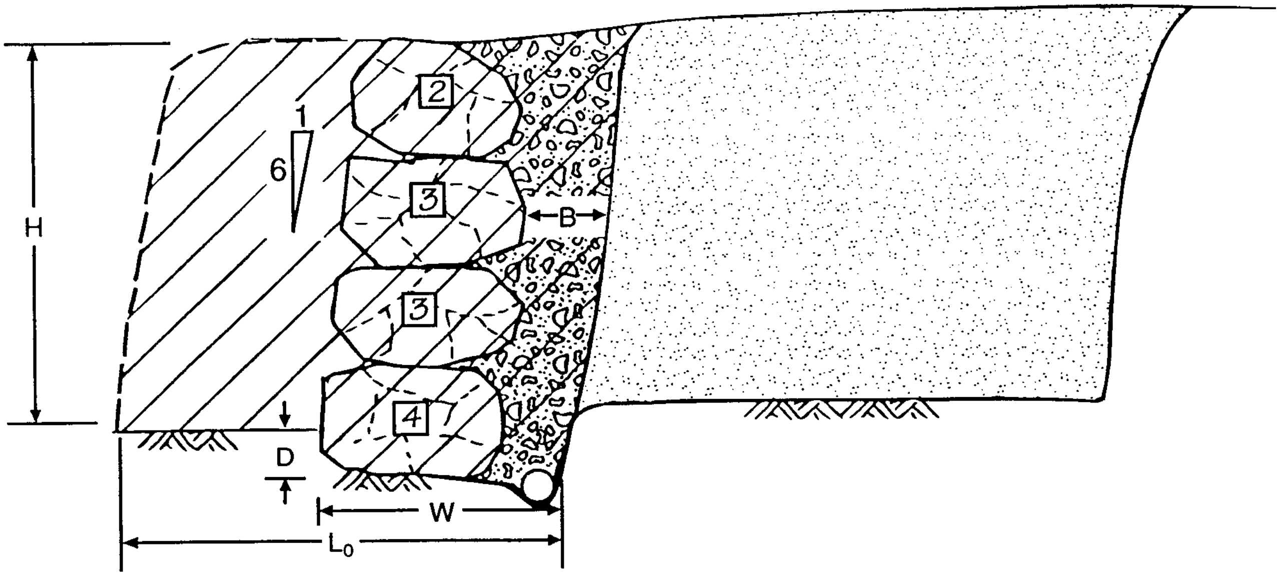
CEO
Geotechnical Engineers, Geologists & Environmental Scientists

TYPICAL DETAIL
NATIVE CUT, ANY HEIGHT OVER 4 FEET

Plate
A

Schematic Only - Not to Scale



LEGEND

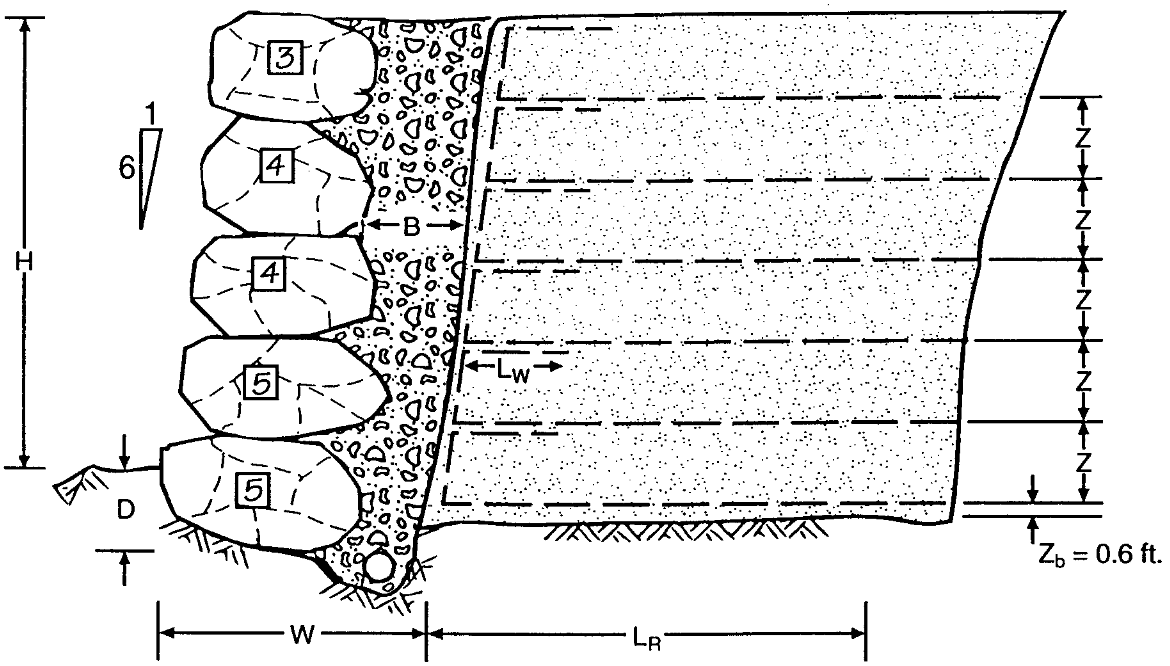
▒	Crushed rock or other apporved material ranging between 4 to 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sleve).
▨	Structural fill overbuild, compacted to at least 95% of maximum dry density as determined by ASTM Test Method D-1558-78 (Modified Proctor).
░	Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to at least 95 percent of ASTM D-1557-78 maximum dry density.
▭	Undisturbed firm Native soil
○	Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.
$\boxed{4}$	Designates size of rock required, i.e. 4 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Thickness of crushed filter rock layer, B, should be no less than 12 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches.
- Height of rock wall, H, should not exceed _______ feet.
- Lateral extent of fill overbuild, L_0 should be no less than H feet.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

Creative Engineering Options INC. CEO Geotechnical Engineers, Geologists & Environmental Scientists	**TYPICAL DETAIL OVERBUILD FILL CONSTRUCTION ROCK WALL LESS THAN 8 FEET IN HEIGHT**	Plate **B**

Schematic Only - Not to Scale



LEGEND

▨ Crushed rock or crushed concrete drain rock material ranging between 4 and 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sieve).

░ Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to a least 95 percent of ASTM D-1557-78 maximum dry density.

☐ Undisturbed firm Native soil

— — · Geogrid reinforcement approved by geotechnical engineer.

○ Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.

$\boxed{5}$ Designates size of rock required, i.e. 5 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Minimum length of geogrid wrap over top of fill, L_W, should be no less than 3 feet.
- Length of reinforcing geogrid, L_R shall be ______ feet.
- Geogrid reinforcement layer spacing Z shall be ______ feet as determined by the geotechnical engineer's design.
- Height of rock wall, H, should not exceed ______ feet.
- Thickness of crushed drain rock layer, B should be no less than 18 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

Creative Engineering Options INC.
Geotechnical Engineers, Geologists & Environmental Scientists

TYPICAL DETAIL
GEOGRID REINFORCED FILL CONSTRUCTION
ROCK WALL 8 FEET OR MORE IN HEIGHT

Plate
C