GEOTECHNICAL ENGINEERING REPORT

Milwaukie Riverfront Park - Phase I
Milwaukie, Oregon
GDI Project: Milwaukie-1

For
Atlas Landscape Architecture

July 24, 2000

Atlas Landscape Architecture
320 SW Sixth Avenue
Portland, OR 97204

Attention: Mr. Gill Williams

Geotechnical Engineering Report
Milwaukie Riverfront Park - Phase I
Milwaukie, Oregon
GDI Project: Milwaukie-1

GeoDesign, Inc. is pleased to submit our report for the Milwaukie Riverfront Park– Phase I in Milwaukie, Oregon. Our services for this project were conducted in accordance with our scope of services dated March 16, 2000, and our subsequent agreement dated June 22, 2000.

We appreciate the opportunity to be of service. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Don Rondema, PE
Principal

DLR:kt
Attachments
Three copies submitted
Document ID: Milwaukie-1-geor.doc

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INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation of the site of the proposed improvements for Milwaukie Riverfront Park - Phase I in Milwaukie, Oregon. The general location of the site relative to surrounding physical features is shown in Figure 1.

We understand that the improvements for Phase I will include the construction of a path with associated earthwork, landscaping, and other surface features. Subsequent phases will require further cutting and filling, with cuts up to 15.0 feet deep, as well as structures consisting of overlooks, associated paths and stairways, light poles, a pedestrian bridge over Kellogg Creek, and other features.

PURPOSE AND SCOPE

The purpose of our services is to evaluate near surface soil conditions and provide geotechnical recommendations for earthwork, drainage, pavements, and light foundation support, specifically for Phase I of this project. To make the best use of mobilization of drilling equipment, borings were also completed in future phases of the project.

The specific scope of our services includes the following:

START-UP PHASE

- Complete a site reconnaissance to plan explorations.
- Complete four drilled borings to depths of up to 21.5 feet and obtain soil samples at 2.5- to 5.0-foot intervals.
- Obtain representative soil samples from the explorations.
- Complete classification and moisture content testing of obtained samples.

DESIGN DEVELOPMENT

- Provide recommendations for site preparation, grading and drainage, stripping depths, fill type for imported materials, compaction criteria, trench excavation and backfill, use of on-site soils, and dry and wet weather earthwork.
- Recommend pavement thicknesses based on observed soil conditions and stated traffic loads.
- Provide recommendations for soil drainage subdrains including subdrain configuration and material types.
- Provide recommendations for foundation support of light poles and other light structures, including allowable bearing pressures for footings and piers, passive and sliding resistance to lateral loads, and estimated settlements/deflections.
- Provide a written report summarizing the results of our geotechnical evaluation.
- Review project specifications directly related to the preceding scope.

SITE CONDITIONS

SURFACE CONDITIONS

Johnson Creek forms the northern boundary of the proposed Phase I improvement area, and Kellogg Creek lies to the south. The central portion of the site is bisected north-south by the Jefferson Street Boat ramp, which is paved with asphalt concrete along with the associated parking areas to the south. North of Jefferson Street and adjacent to McLoughlin Boulevard lie a number of one- and two-story masonry and wood frame structures, some of which are to be demolished. Areas not covered with asphalt concrete or structures above elevation 10 are generally covered with grasses and landscape materials or separate areas of cottonwood trees. Areas below elevation 10 are generally exposed riverbank soil and rock, with scattered large boulders. Exposed riverbank soil along the Willamette River included silt with some fine sand. Weathered basalt bedrock was observed along the northern bank of Kellogg Creek near the fish ladder. Hard basalt was observed below the streambank vegetation on either side of Johnson Creek.

The site surface generally slopes down from McLoughlin Boulevard to the Willamette River, with an elevation change of about 40 feet on an overall slope averaging 3H:1V (horizontal to vertical). Slope inclinations are greater than 3/4H:1V along most of the south bank of Johnson Creek, and short steep slopes (likely corresponding to high water) are present below about elevation 20 north of Kellogg Creek. Slope cuts, benches for parking, and evidence of filling is present in the paved areas of the boat ramp parking, as well as along the old railroad alignment that is currently a paved bike path paralleling McLoughlin Boulevard.

SUBSURFACE CONDITIONS

We explored subsurface conditions by drilling four borings to depths of 16.5 to 21.5 feet. Detailed boring logs are included in Appendix A. Each of the borings encountered fill, and in landscape and vegetated areas 4 to 6 inches of rooty topsoil. Boring B-1 to the north in Phase I encountered about 4.5 feet of stiff silt fill, while Borings B-2 and B-3 encountered generally medium stiff silt fill and loose sand fill to the 21.5-foot depths explored. In addition, Boring B-2 encountered gravel fill in the top 4 feet (this fill may represent old railroad bed/ballast materials) and a 1-inch thick layer of wood chips at a depth of 20.4 feet. Boring B-4 encountered sand and silt fill to depths of about 9 feet. The fill materials were generally inorganic, although at some depths included trace organics. Cobbles, boulders, and concrete fragments were also encountered in the borings at different depths within the fill.

Native silt and sand soils were encountered beneath the fill in Borings B-1 and B-4. Blowcounts in the native soils ranged from 3 to 13, and moisture contents were less than 20 percent in the cleaner sands and between 30 and 40 percent in the silts and silty sand.

CONCLUSIONS

Based on the results of our explorations, laboratory testing and analyses, it is our opinion that the proposed pathways and light pole foundations can be supported on native medium stiff or stiffer existing non-organic fill soils, or on structural fill that is properly installed during construction. Groundwater was encountered in B-3 at a depth of 20.0 feet. The following paragraphs present specific geotechnical recommendations for design and construction of the proposed facilities.

RECOMMENDATIONS

SITE PREPARATION

In all proposed areas of hard surfacing, foundations or slabs, and for a 2-foot margin around such areas, the existing root zone and organic landscape materials should be stripped and removed. Based on our explorations, the depth of stripping in such areas will be approximately 4 to 6 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

After stripping and required site cutting have been completed, we recommend that a member of our geotechnical staff probe the subgrade to identify soft or unsuitable areas.

CONSTRUCTION CONSIDERATIONS

Trafficability of the exposed subgrade may be difficult during or after extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum moisture content. When wet, the silty soils are easily disturbed and may provide inadequate support for construction equipment. Soils that have been disturbed during site preparation activities, or soft or loose zones identified during probing, should be removed and replaced with compacted structural fill.

STRUCTURAL FILL

On-site Materials

As the existing fill is in many layers, and the site has an extensive history of structures, cuts and fills, and varied uses, we expect that some of the fill materials encountered will contain appreciable organics and oversize materials that may be unsuitable for use in fills. A contingency should be built into the project budget and schedule to allow for such materials. The encountered native silts, sand with some silt, and fills with some or more silt are sensitive to small changes in moisture content and highly susceptible to disturbance when wet. We recommend completing construction in the dry season. If construction is planned for the wet season, careful consideration of the construction methods and schedule should be made to reduce overexcavation of disturbed site soils, and the project budget should reflect the recommendations for wet weather construction contained in this report.

Laboratory testing indicates that the moisture content of the on-site materials is generally greater than the anticipated optimum moisture content required for satisfactory compaction. Therefore, moisture conditioning will be required to achieve adequate compaction. We recommend using imported granular material for structural fill if the on-site material cannot be properly moisture-conditioned.

When used as structural fill, the on-site silts should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and be compacted to not less than 92 percent of the maximum dry density, as determined by American Society for Testing and Materials (ASTM) D 1557.

Imported Granular Material

If imported granular material is used as structural fill, this material should consist of pit or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine, contains no organic matter or other deleterious materials, has a maximum particle size of 3 inches, and has less than 5 percent passing the U.S. Standard No. 200 Sieve. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather. Imported granular material should be moisture conditioned to the approximate optimum moisture content for compaction, placed in 12-inch thick lifts, and compacted to not less than 95 percent of maximum dry density as determined by ASTM D 1557.

Trench Backfill

Trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material containing no organic or other deleterious material, having a maximum particle size of ¾-inch, and having less than 5 percent pass the U.S. Standard No. 200 Sieve.

Backfill for the pipe base and within the pipe zone should be placed in maximum 12-inch thick lifts and compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D 1557 or as recommended by the pipe manufacturer. Backfill above the pipe zone should be placed in maximum 12-inch-thick lifts and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D 1557. Trench backfill located within 2 feet of finish subgrade elevation should be placed in maximum 12-inch thick lifts and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

SHALLOW FOUNDATIONS

We recommend that spread footings bear on the medium stiff to very stiff silts, have a minimum width of 18 inches, and have the base of the footings founded at least 18 inches below the lowest adjacent grade. Continuous wall footings should have a minimum width of 12 inches, and be founded a minimum of 18 inches below the lowest adjacent grade. Drilled piers for light pole foundations should have a minimum diameter of 18 inches.

Footings founded as avove should be proportioned for a maximum allowable soil bearing pressure of 2,800 pounds per square foot (psf). Piers should be designed for at least 3 feet of embedment, and for shaft friction of 800 psf below the top 1 foot. These are net pressures and apply to the total of dead and long-term live loads and may be increased by 50 percent when considering earthquake or wind loads. The weight of the footings and piers, and overlying backfill, can be ignored in calculating footing loads.

For the preceding pressures with loads less than 50 kips for columns or piers and 3 kips per foot for walls, total settlement is anticipated to be less than about 1 inch. Differential settlements should not exceed ½ inch.

Lateral Capacity

We recommend using a passive pressure of 300 pounds per cubic foot for design purposes for footings and piers confined by native silt or structural fill. In order to develop this capacity, concrete must be poured neat in excavations or the adjacent confining material must consist of imported granular fill compacted to 92 percent relative to ASTM D 1557. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

A coefficient of friction equal to 0.35 may be used when calculating resistance to sliding at the base of footings.

DRAINAGE CONSIDERATIONS

Although groundwater was not encountered near proposed grades, the site soils consist primarily of silt, and sand with some silt, which generally provide poor drainage. Perched groundwater may develop in these soils during extended wet periods, and may result in cut slope seeps or ponded water in flat areas. Vegetated surfaces subject to pedestrian use should be sloped a minimum of 0.5 percent to allow for surface drainage/runoff. If seeps occur in cut slopes special drainage measures may be necessary to collect the water and prevent slope erosion. Such measures typically include installing a fabric-wrapped perforated pipe into the seeps, embedding it in clean gravel, and routing it to a suitable discharge. Composite drain materials, such as strip drains, can also be used for this purpose. Fabric should be non-woven and have an apparent opening size between a #70 and #100 sieve.

PAVEMENT

The pavement subgrade should be prepared in accordance with the previously described site preparation, construction considerations, and structural fill recommendations. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have assumed that traffic conditions will consist of fewer than 5 trucks and 200 cars per day.

A pavement section consisting of a thickness of at least 2.5 inches of asphalt concrete over at least 8 inches of crushed rock base course should be appropriate in areas where truck traffic is expected. If parking areas are limited to passenger automobiles only, the pavement section can be reduced to 2.5 inches of asphalt concrete over 6 inches of crushed rock. For portland cement concrete (pcc), we recommend a minimum 5.5 inches for plain-jointed pcc overlying 6 inches of crushed rock base.

Our pavement section recommendations are based on a California Bearing Ratio of 5 and the assumption that construction will be completed during a period of extended dry weather. An increased thickness of granular base course will be required if construction occurs during wet weather conditions.

OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. 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. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. 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.

We recommend that GeoDesign be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork and foundation construction are being met.

LIMITATIONS

We have prepared this report for use by Atlas Landscape Architecture and the City of Milwaukie, and their design teams for the proposed Milwaukie Riverfront Park - Phase I in Milwaukie, Oregon. The data and report can be used for bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Our explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, reevaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the building, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

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 this report was prepared. No warranty or other conditions, express or implied, should be understood.

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We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

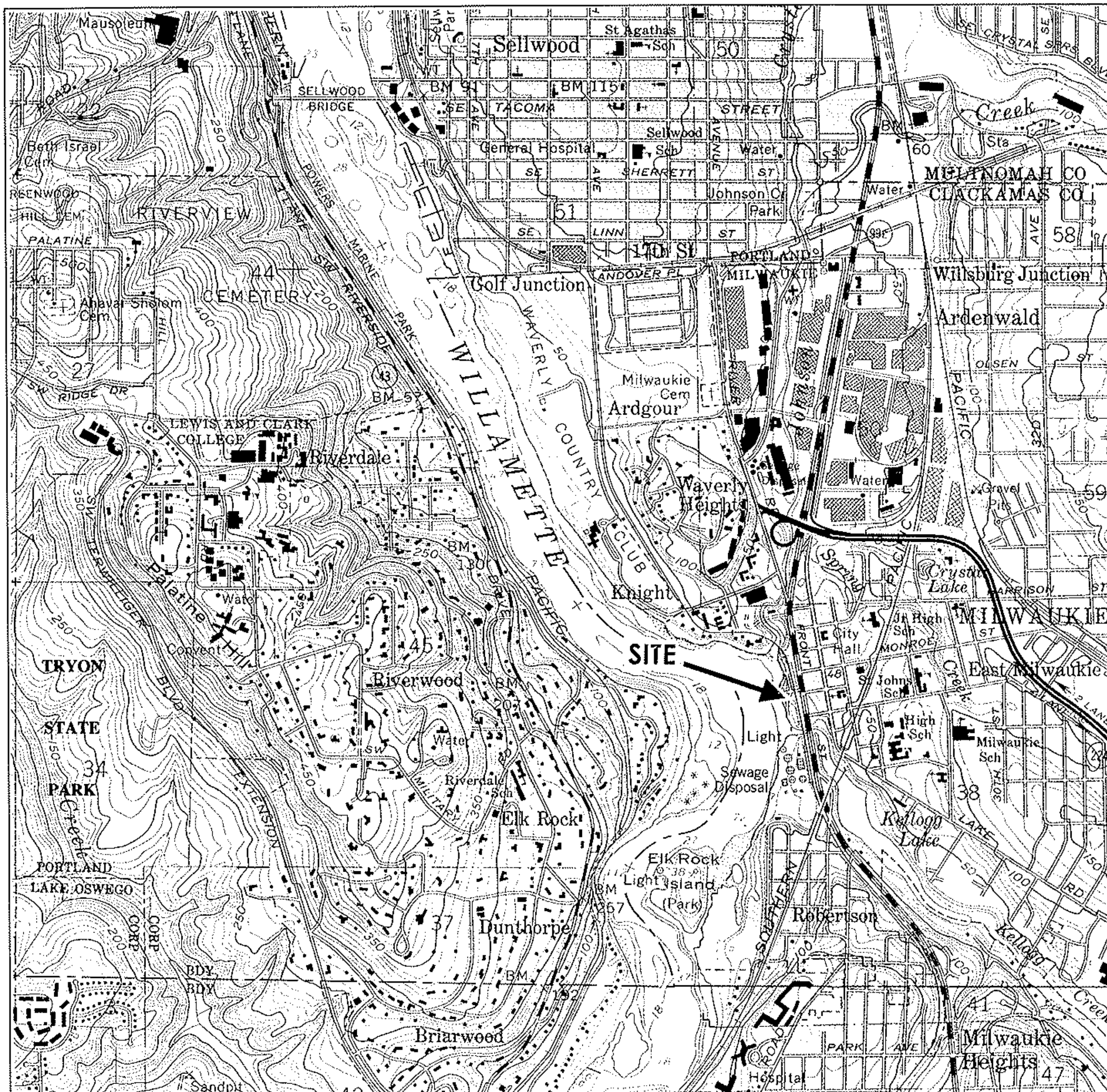
Sincerely,

GeoDesign, Inc.

Don Rondema, P.E.
Principal

REGISTERED PROFESSIONAL
ENGINEER
16,073
OREGON
JULY 21, 1992
DON RONDEMA

Expires 12/31/00



USGS TOPOGRAPHIC MAP
LAKE OSWEGO, OR QUAD

N

0 2000 4000 FT

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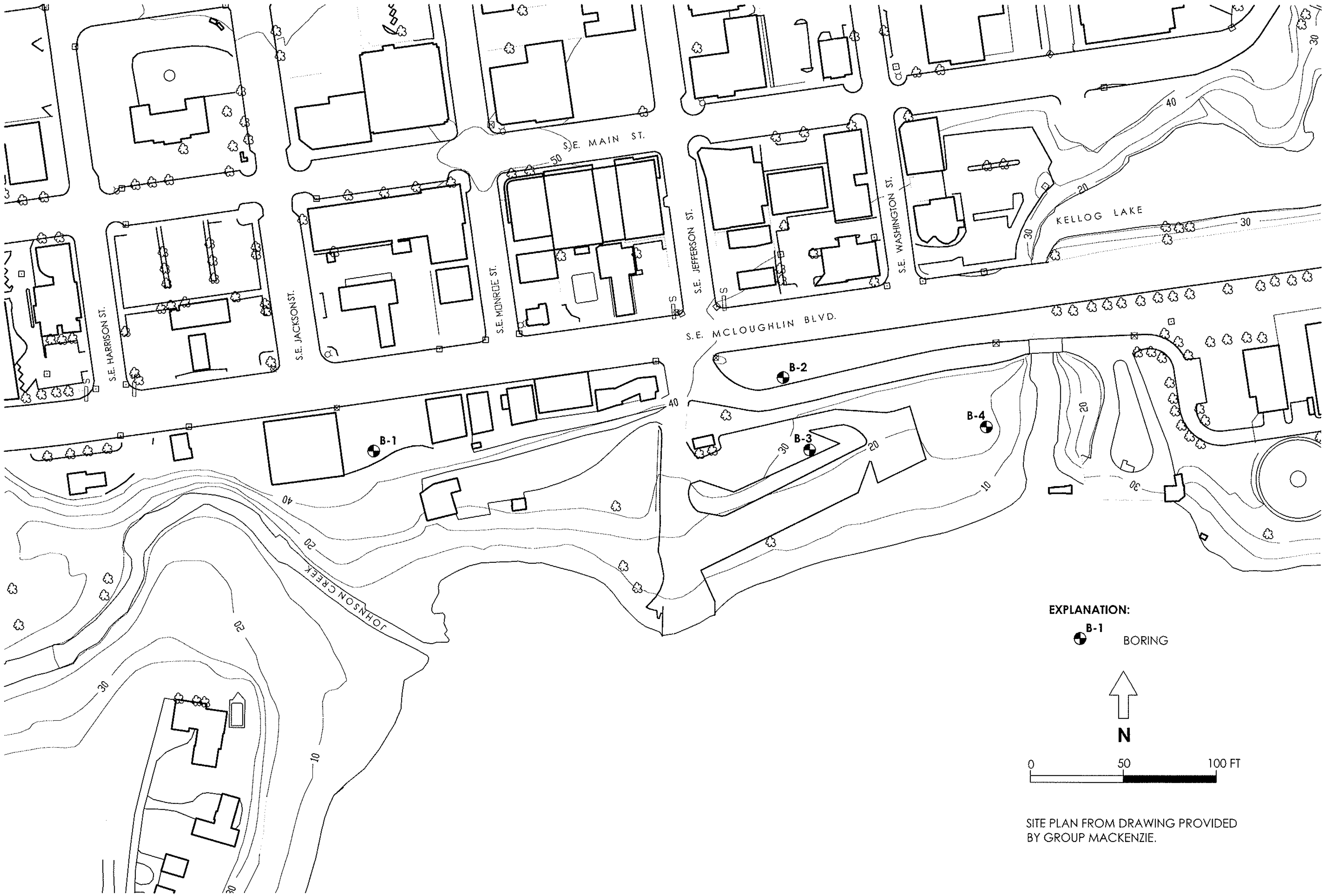
VICINITY MAP

MILWAUKIE-1	JULY 2000	FIGURE 1



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SITE PLAN

MILWAUKIE-1

JULY 2000

FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

We explored subsurface conditions at the site by drilling four borings (B-1 through B-4) at the approximate locations shown in Figure 2. Subsurface Technologies drilled the boring with hollow stem auger methods and SPT safety hammer sampling on July 6, 2000.

Boring locations were based on a site plan provided to our office by Gill Williams of Atlas Landscape Architecture. We determined the exploration locations in the field from existing site features. The locations shown on Figure 2 should be considered approximate.

We obtained representative samples of the various soils encountered for geotechnical laboratory testing. Classifications and sampling intervals are shown on the logs included in this appendix.

We classified the materials present in the samplers in the field in accordance with ASTM D 2488. The logs indicate the depths at which the soils or their characteristics change, although the change actually may be gradual. If the change occurred between sample locations, the depth was interpreted.

LABORATORY TESTING

We classified soil samples in the laboratory to confirm field classifications. The laboratory classifications are included in the boring logs if those classifications differed from the field classifications.

We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The moisture contents are included in the boring logs in this appendix.

KEY TO TEST PIT AND BORING LOG SYMBOLS

SYMBOL	SOIL DESCRIPTION
	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test
	Location of SPT sampling attempt with no sample recovery
	Location of sample obtained using thin wall, shelby tube, or Geoprobe® sampler in general accordance with ASTM D 1587
	Location of thin wall, shelby tube, or GeoProbe® sampling attempt with no sample recovery
	Location of sample obtained using Dames and Moore sampler and 300 pound hammer or pushed
	Location of Dames and Moore sampling attempt (300 pound hammer or pushed) with no sample recovery
	Location of grab sample
	Rock Coring Interval
	Water level

GEOTECHNICAL TESTING EXPLANATIONS

PP	Pocket Penetrometer	LL	Liquid Limit
TOR	Torvane	PI	Plasticity Index
CONSOL	Consolidation	PCF	Pounds Per Cubic Foot
DS	Direct Shear	PSF	Pounds Per Square Foot
P200	Percent Passing U.S. No. 200 Sieve	TSF	Tons Per Square Foot
W	Moisture Content	P	Pushed Sample
DD	Dry Density	OC	Organic Content

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
PID	Photoionization Detector Headspace Analysis	NS	No Visible Sheen
		SS	Slight Sheen
PPM	Parts Per Million	MS	Moderate Sheen
MG/KG	Milligrams Per Kilogram	HS	Heavy Sheen
P	Pushed Sample		

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KEY TO TEST PIT AND BORING LOG SYMBOLS

TABLE A-1

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	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, 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, 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	Low plasticity silt
			CL	Low plasticity clay
		Organic	OL	Organic silt, organic clay
	Silt and Clay Liquid Limit greater than 50%	Inorganic	MH	High plasticity silt
			CH	High plasticity clay, fat clay
		Organic	OH	Organic clay, organic silt
Highly Organic Soils			PT	Peat

SOIL CLASSIFICATION GUIDELINES

GRANULAR SOILS		COHESIVE SOILS		
Relative Density	**Standard Penetration Resistance**	**Consistency**	**Standard Penetration Resistance**	**Unconfined Compressive Strength (tsf)**
Very Loose	0 – 4	Very Soft	Less than 2	Less than 0.25
Loose	4 – 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	10 – 30	Medium Stiff	4 - 8	0.50 - 1.0
Dense	30 – 50	Stiff	8 - 15	1.0 - 2.0
Very Dense	More than 50	Very Stiff	15 - 30	2.0 - 4.0
		Hard	More than 30	More than 4.0

GRAIN SIZE CLASSIFICATION

Boulders	12 - 36 inches
Cobbles	3 - 12 inches
Gravel	¾ - 3 inches (coarse) ¼ - ¾ inches (fine)
Sand	No. 10 - No. 4 Sieve (coarse) No. 10 - No. 40 Sieve (medium) No. 40 - No. 200 Sieve (fine)

Subclassifications

	Percentage of other material in sample
Clean	0 – 2
Trace	2 – 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

Dry = very low moisture, dry to the touch; Moist = damp, without visible moisture; Wet = saturated, with visible free water.

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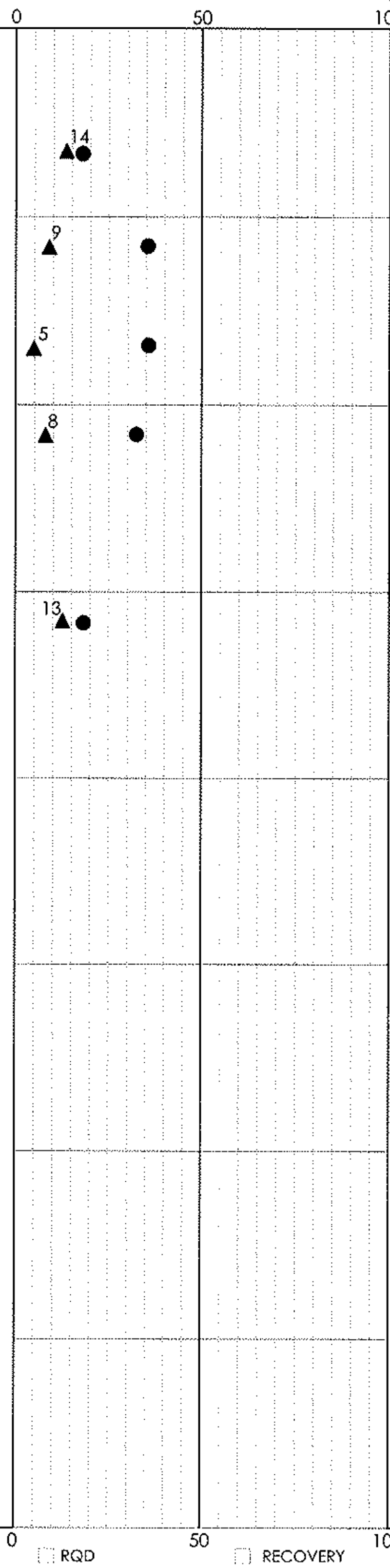
SOIL CLASSIFICATION SYSTEM AND GUIDELINES

TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING
0	ML-FILL	Stiff, brown SILT FILL with trace sand and gravel; moist.			
5	ML	Stiff, brown SILT; moist.			
		becomes medium stiff at 7.5 feet			
10	SP	Loose to medium dense, brown, medium SAND; moist.			
15		with some basalt gravel at 15.0 feet			
	ML	Stiff, brown, sandy SILT; moist.			
		Boring completed at 16.5 feet on July 6, 2000.			
20					
25					
30					
35					
40					



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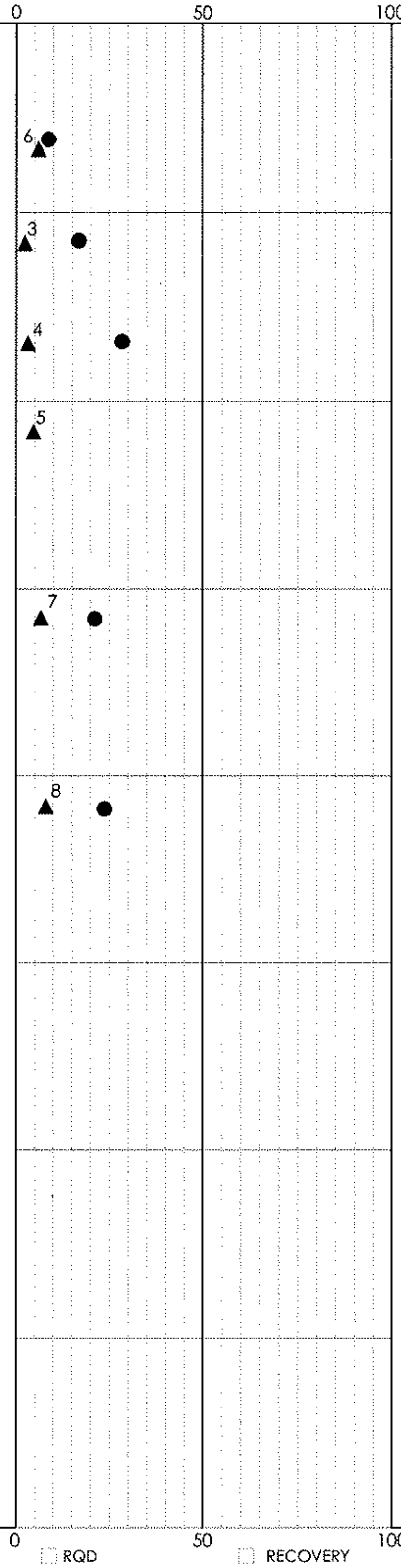
BORING B-1

MILWAUKIE-1	JULY 2000	FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING
0		**GW-FILL** Loose, brown GRAVEL with some sand and silt; moist.			
5		**SM-FILL** Very loose, brown, silty fine SAND FILL with occasional gravel; moist.			
		SP-FILL Loose, brown, fine SAND FILL with trace to some silt; moist.			
10		**ML-FILL** Medium stiff, brown SILT FILL; moist.			
15		becomes sandy with trace gravel and fine organics at 15.0 feet			
		SP-FILL Loose, brown SAND FILL with some silt; moist. 1-inch thick wood chip layer encountered at 20.4 feet becomes gray at 20.5 feet			
20					
		Boring completed at 21.5 feet on July 6, 2000.			
25					
30					
35					
40					



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BORING B-2

MILWAUKIE-1	JULY 2000	FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING

AC ASPHALT CONCRETE (4.5-inches thick).

GW GRAVEL baserock (7-inches thick).

ML-FILL Medium stiff, brown and gray SILT FILL with trace gravel; moist.

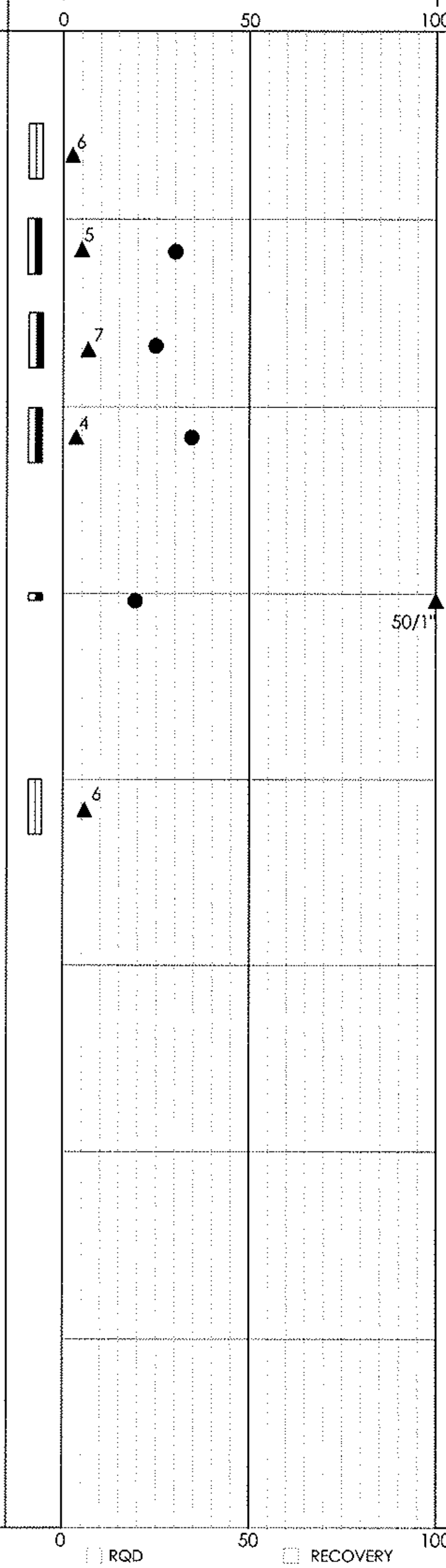
with some sand and trace fine organics at 7.5 feet

organics grade out at 10.0 feet

with some basalt gravel and possible cobbles and boulders at 15.0 feet

▽

Boring completed at 21.5 feet on July 6, 2000.



Depth (feet): 0, 5, 10, 15, 20, 25, 30, 35, 40

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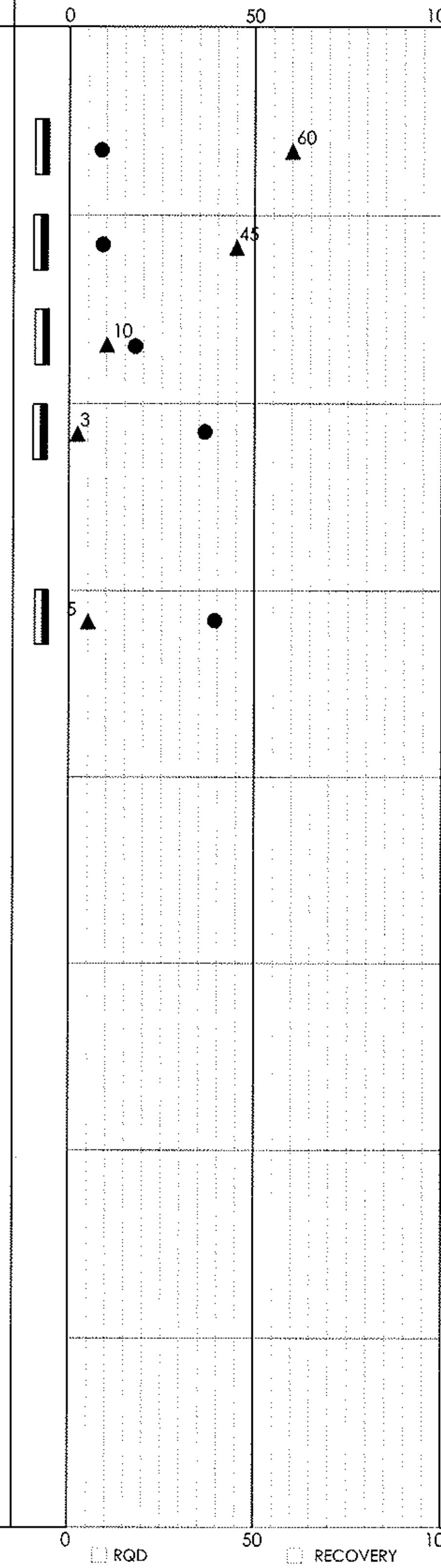
BORING B-3

MILWAUKIE-1	JULY 2000	FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION		SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING
0		**SP-FILL**	Very dense, brown gravelly SAND FILL with trace organics; dry.			
5		**SP-FILL**	Dense, brown, medium SAND FILL with trace to some basalt gravel, concrete and trace organics; moist.			
		ML-FILL	Stiff, brown mottled SILT FILL with trace to some gravel; moist.			
10		**SM**	Very loose, gray, silty fine SAND; moist.			
15		**SM-SP**	Loose, gray, silty SAND grades to medium SAND with trace silt and organics; moist.			
			Boring completed at 16.5 feet on July 6, 2000.			
20						
25						
30						
35						
40						



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BORING B-4

MILWAUKIE-1	JULY 2000	FIGURE A-4