GEODESIGN, INC.

GEOTECHNICAL, ENVIRONMENTAL, AND GEOLOGICAL CONSULTANTS

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REPORT OF
GEOTECHNICAL ENGINEERING SERVICES
Trumbull Asphalt Plant Expansion
Linnton, Oregon
GDI Project: OwensCorning-1

For
Owens Corning

17400 SW Upper Boones Ferry Rd., Suite 230 • Portland, Oregon 97224 • Phone (503) 968-8787 • Fax (503) 968-3068

GEODESIGN, INC.
GEOTECHNICAL, ENVIRONMENTAL, AND GEOLOGICAL CONSULTANTS

October 16, 1998

Owens Corning
c/o Norwest Engineering
4110 Northeast 122nd Avenue
Suite 207
Portland, Oregon 97230

Report of Geotechnical Engineering Services
Asphalt Plant Expansion
Linnton, Oregon
GDI Project: OwensCorning-1

Attention: Mr. John Deppa

GeoDesign, Inc. is pleased to submit three copies of our "Report of Geotechnical Engineering Services" for the proposed Trumbull Asphalt Plant expansion in Linnton, Oregon. Our services for this project were conducted in accordance with our proposal dated September 1, 1998.

We appreciate the opportunity to be of service to Norwest Engineering and Owens Corning. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Ryan White

Ryan White, E.I.T.
Geotechnical Staff

Scott V. Mills, P.E.
Senior Principal

RKW:DLR:SVM
Document ID: OwensCorning-1-geor

Attachments
Three copies submitted

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CONTENTS

REPORT OF GEOTECHNICAL ENGINEERING SERVICES PROPOSED TRUMBULL ASPHALT PLANT EXPANSION Linnton, Oregon

INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation of the site of the proposed Trumbull Asphalt Plant expansion in Linnton, Oregon. The site is located east of Highway 30 and west of the Willamette River, approximately a ¼ mile north of Kingsley Park. The general location of the site relative to surrounding physical features is shown in Figure 1.

Expansion of the plant will consist of the construction of four 12-foot diameter, 32-foot tall steel storage tanks, three 13-foot diameter, 36-foot tall steel storage tanks, a 22,500 square foot pre-engineered metal building, and a 100-foot tall stack. Based on conversations with John Deppa of Norwest Engineering, we understand the slab loads for the metal building will be approximately 500 pounds per square foot (psf) with column loads of approximately 40 kips. It is also our understanding that the proposed stack and asphalt storage tanks will be supported on pile foundations.

PURPOSE AND SCOPE

The purpose of our geotechnical engineering evaluation was to explore the subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction. The specific scope of our services for the geotechnical evaluation is summarized below.

- Coordinate and manage the field investigation, including utility locates, site access authorizations, scheduling of contractors and of GeoDesign's staff.
- Explore subsurface conditions by drilling up to three borings using mud rotary techniques and advancing one cone penetrometer probe to the Troutdale Formation with shear wave velocity measurements every meter. One boring and one cone penetrometer probe will be completed in the area of the proposed building, one boring in the area of the tanks, and one boring will be completed in the area of the proposed smoke stack.
- Obtain soil samples at approximately 2.5- to 5.0-foot intervals in the borings.
- Complete the following laboratory analyses on soil samples obtained from the explorations: a maximum of 30 moisture contents, one density, one consolidation test, and one #200 wash.
- Review information from previous geological and geotechnical studies conducted on and near the site.

- Provide geotechnical engineering recommendations for pile foundation design.
- Provide a discussion of seismic activity near the site, liquefaction potential and associated qualitative deformation estimates, and recommendations for the 1997 seismic coefficients and soil profile type.
- Provide a written report summarizing the results of our geotechnical evaluation.

SITE CONDITIONS

SURFACE CONDITIONS

The Trumbull Asphalt Plant is located east of Highway 30, north of Kingsley Park and bordered on the west by the Willamette River in Linnton, Oregon. The location of the proposed storage tanks (just north of the southern access road and south of tank T-3) is covered in sparse grass, sand, and gravel. Similar conditions exist at the site of the proposed stack on the southwestern edge of tank T-2. The pre-engineered metal building will be located south of the southern access road and east of the oil/water separator. This area is currently covered with grass and is mantled by up to 5 feet of fill.

SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling three soil borings (B-1 through B-3) and one cone penetrometer (P-1) at the approximate locations shown in Figure 2. The borings were drilled to approximate depths of between 30 and 92 feet below the ground surface. A description of our subsurface exploration program and the exploration logs are included in the appendix of this report.

In general, our explorations encountered between 5 and 20 feet of loose sand and soft to medium stiff silt fill containing small amounts of wood chips and organics. Underlying the fill the explorations encountered soft to medium stiff native silt containing varying amounts of fine sand and occasional layers and lenses of sand and gravel. Beneath the native silt, borings B-1 and B-2 encountered dense to very dense gravel (Troutdale Formations) at depths of 56 and 88 feet below the ground surface, respectively.

We were unable monitor depths to groundwater during drilling due to the presence of drilling mud in the holes. We assumed groundwater elevations were consistent with the level of the Willamette River nearby or approximately 5 feet below the existing ground surface.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on the results of our geotechnical investigation, we conclude that the stack, tanks, and pre-engineered metal building can be constructed as proposed. Piles should derive their capacity by end bearing in the very dense Troutdale Formation gravels that occurs at approximately 88 feet in the area of the stack and approximately 56 feet in the area of the tanks. It is our opinion that the pre-engineered metal building can be supported on shallow spread footings.

The following paragraphs present specific geotechnical recommendations for design and construction of the proposed development.

PILE FOUNDATIONS

We recommend that deep foundations derive their capacity from end bearing in the Troutdale Formation gravels that underlie the site. Based on the results of our site exploration, we anticipate that suitable embedment will be reached at less than 10 feet of penetration into the Troutdale Formation.

Pile Capacities

We have analyzed three pile sizes, 10.75, 12.75, and 14.0-inch diameter steel piles. Allowable capacities for downward axial loads are controlled by the structural capacity of the pile, assuming a rate of corrosion of 1 mil per year and a 50-year design life.

The following table presents our recommendations for pile design.

Pile Size, inches	Wall Thickness, inches	Downward Capacity, kips		Uplift Capacity, kips		Lateral Capacity, kips (1/2-inch deflection)
		(Empty)	(Concrete)	(stack)	(tanks)	
10.75	0.375	120	200	55	32	8.0
12.75	0.375	145	255	65	39	10.5
14.0	0.5	220	350	72	43	14.0

Pile spacing in the direction of lateral loading should be a minimum of three pile diameters, center-to-center. No reduction factor is necessary for pile groups of three or fewer because the site sands will be densified during pile driving. Our analysis assumed a free-head condition for the piles. Our analysis also indicates that for ½ inch deflections, the deflection profile is near zero at depths of more than 8 feet, and that moments are nominal below a depth of about 16 feet for an empty pile.

Resistance to lateral loads also can be developed by passive pressure on the face of pile caps, grade beams, tie beams and other buried foundation elements. Sliding friction on the base of pile-supported foundation elements should be limited to a coefficient of friction of 0.32 applied to a normal force equal to the fluid weight of the concrete. Assuming a maximum translation of ½-inch, the allowable passive resistance on the face of buried foundation elements may be computed using an equivalent fluid density of 280 pcf (triangular distribution) for foundation elements cast neat against the existing soil or backfilled with structural fill. At ¾ inches of translation this value may be increased to 400 pcf. The equivalent fluid density value may be increased to 350 pcf for ½-inch of translation or 500 pcf for ¾-inch if a zone of structural fill having a width at least equal to twice its thickness is placed against the foundation element. This zone of structural fill should be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D 1557. The allowable passive resistance should not be increased when considering earthquake or wind loads. Adjacent floor slabs, pavements or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

Piles should be driven to refusal in the underlying Troutdale Formation gravels using a hammer having adequate energy to penetrate through the overlying soils and properly found the piles. Our preliminary analysis indicates that a diesel or air-hammer developing approximately 30,000 ft-lb of driving energy should be adequate to develop the capacity of the pile. The piles should have sufficient structural capacity to withstand the stresses induced by pile driving; however we should be consulted to evaluate pile driving stresses and to develop terminal driving criteria once the hammer has been selected for the project.

It is unlikely that vibrations induced during pile driving will damage nearby structures. However, if the nearby structures are particularly sensitive to vibration, it may be prudent to provide instrumentation during driving to monitor peak particle velocities. We should be consulted to evaluate proposed pile driving systems if vibrations are considered to be a critical concern.

It is important that each pile penetrate through the overlying soils and be properly founded in the Troutdale Formation. Therefore, we recommend that a member of our staff monitor pile installation to observe installation procedures, record pertinent data, and evaluate the adequacy of individual pile penetrations.

SHALLOW FOUNDATIONS

In our opinion, the pre-engineered, metal building can be supported on spread footings that bear on the medium dense sand or gravel, have a minimum width of 24 inches and founded at least 18 inches below the lowest adjacent grade. Continuous wall footings should have a minimum width of 18 inches.

Bearing Pressure and Settlement

Footings should be proportioned for a maximum allowable soil bearing pressure of 2,500 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. The weight of the footing and overlying backfill can be ignored in calculating footing loads.

For a 2,500 psf design bearing pressure, total settlement of footings 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 pcf for design purposes for footings confined by native silts and sands or structural fill. In order to develop this capacity, concrete must be poured neat in excavations or the adjacent confining structural fill must consist of granular soils compacted to 95% relative to ASTM D1557. 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 0.32 may be used when calculating resistance to sliding.

SEISMIC DESIGN

The site is currently rated as Seismic Zone 3. Site conditions correspond to a soil profile type S_D and a UBC 1997 seismic coefficients, $C_a = 0.36$ and $C_v = 0.54$.

LIQUEFACTION

Liquefaction is defined as the sudden loss of shear strength in a soil due to excessive buildup of pore water pressure during a seismic event. Liquefied layers densify as excess pore pressures dissipate, which can result in surface settlement, sand boils or ejections, and/or lateral spreading.

Liquefaction analysis of the site was conducted based on the information obtained from the CPT probes and borings. The results of each probe were analyzed to establish soil stresses resisting liquefaction using methods developed by Seed and De Alba Stark and Olsen. Stresses causing liquefaction were analyzed using methods developed by Seed. The ratio of stresses resisting and causing liquefaction was then used to determine a factor of safety for liquefaction. Where liquefaction was predicted to occur, methods outlined by Seed and Tokimatsu were used to estimate liquefaction-induced settlement.

Using these analytical procedures, less than 2 inches of liquefaction-induced is expected during a seismic event. Differential settlement associated with liquefaction is expected to be less than 1 inch.

Lateral spreading induced by the liquefaction of underlying sand layers in the explored areas could be as much as 14 inches. Differential lateral spreading from near river and far edges of the metal building is not expected to exceed a few inches.

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 Owens Corning and its consultants for the planned Trumbull Asphalt Plant expansion in Linnton, 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.

Drilled borings 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, 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.

◆ ◆ ◆

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.

Ryan White, E.I.T.
Geotechnical Staff

REGISTERED PROFESSIONAL ENGINEER
13701
OREGON
JULY 16, 1987
SCOTT V. MILLS

Scott V. Mills, P.E.
Senior Principal

USGS TOPOGRAPHIC MAP
LINNTON, OR QUAD

N

0 2000 4000 FT

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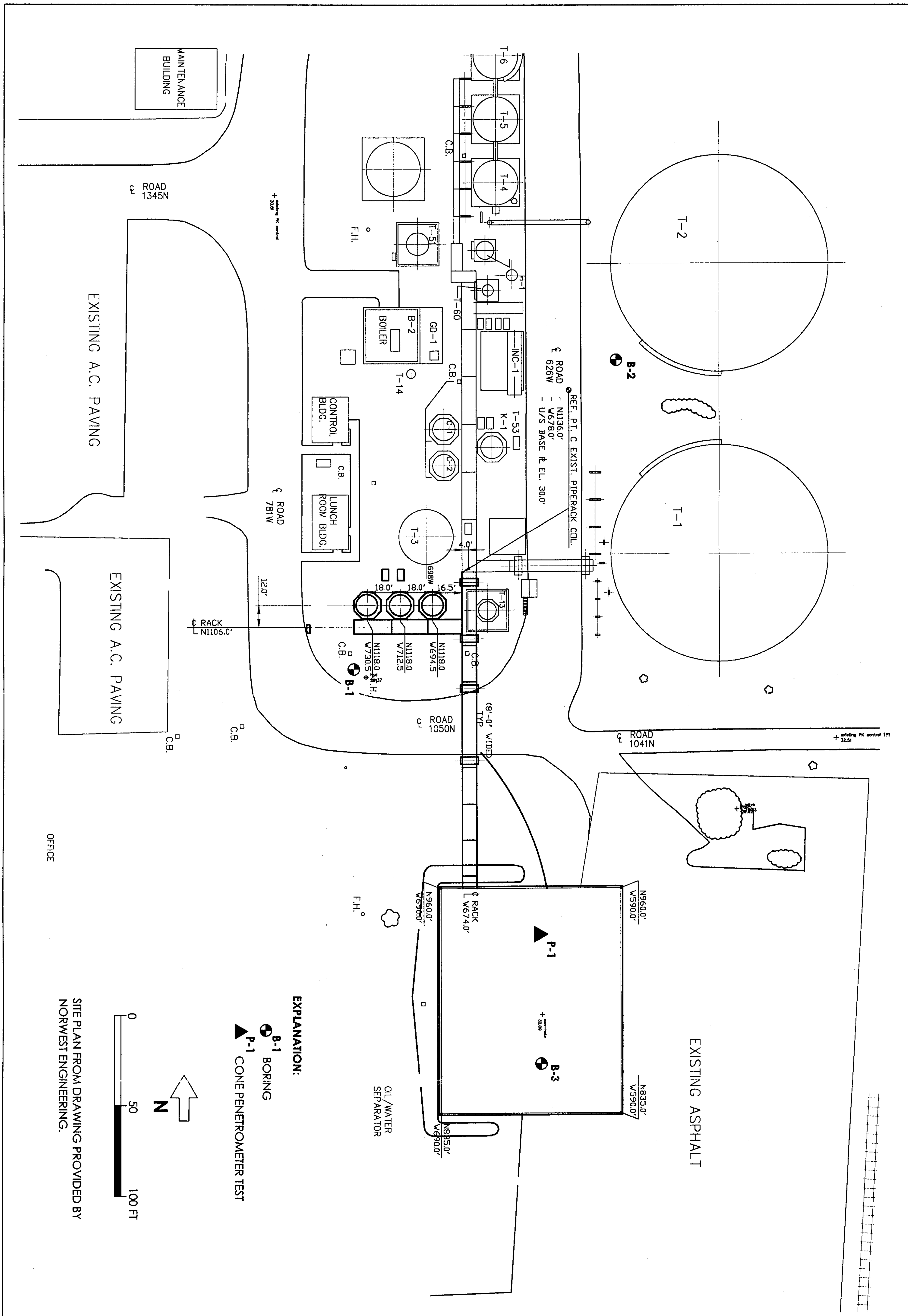
VICINITY MAP

OWENS CORNING-1	OCTOBER 1998	FIGURE 1



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SITE PLAN

OWENS CORNING-1	OCTOBER 1998	FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

We explored subsurface conditions at the site by drilling three geotechnical soil borings and advancing one cone penetrometer probe at the approximate locations shown in Figure 2. Van de Hey Soil Explorations advanced the cone penetrometers to a depth of 92 feet below the ground surface on September 15, 1998. GeoTech Explorations of Tualatin, Oregon, drilled the borings using mud-rotary drilling methods on September 14 and 15, 1998. The depths of the borings ranged between 30 and 92 feet below the ground surface.

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

We chose the boring locations based on a site plan provided to our office by Norwest Engineering. We determined the exploration locations in the field from existing site features. The locations shown on Figure 2 should be considered approximate. A qualified member of GeoDesign's staff observed and documented all field activities.

Classification, Moisture Content and Density

We classified the materials present in the samplers in the field in accordance with ASTM D 2488. The boring 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.

The soil samples were classified 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 natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The moisture contents are included in the logs presented in this appendix.

We tested selected soil samples to determine the in-situ dry density (dry unit weight). The tests were performed in general accordance with ASTM D 2937. The dry density is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry density typically is expressed in pounds per cubic foot. The dry densities are presented in the logs included in this appendix.

Consolidation Testing

A one-dimensional consolidation test was completed on one relatively undisturbed soil sample obtained from boring B-3. The test was conducted in general accordance with ASTM D 2435. The test measures the volume change (consolidation) of a soil sample under predetermined loads. The results of the consolidation testing are included Figures A-4 in this appendix.

Grain-Size Testing

Grain-size testing was completed on selected samples. Testing included percent fines determinations in general accordance with ASTM C 136 and ASTM D 1140 and hydrometer analysis in general accordance with ASTM C 136 and ASTM D 422. The results of the percent fines testing are included in the logs presented in this appendix.

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, elastic 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-1

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 OR SHELBY TUBE SAMPLER
	LOCATION OF THIN WALL OR SHELBY TUBE SAMPLING ATTEMPT WITH NO SAMPLE RECOVERY
	LOCATION OF SAMPLE OBTAINED USING DAMES AND MOORE SAMPLER AND 300 POUND HAMMER
	LOCATION OF DAMES AND MOORE SAMPLING ATTEMPT (300 POUND HAMMER) WITH NO SAMPLE RECOVERY
	LOCATION OF GRAB SAMPLE
	WATER LEVEL

GEOTECHNICAL TESTING EXPLANATIONS

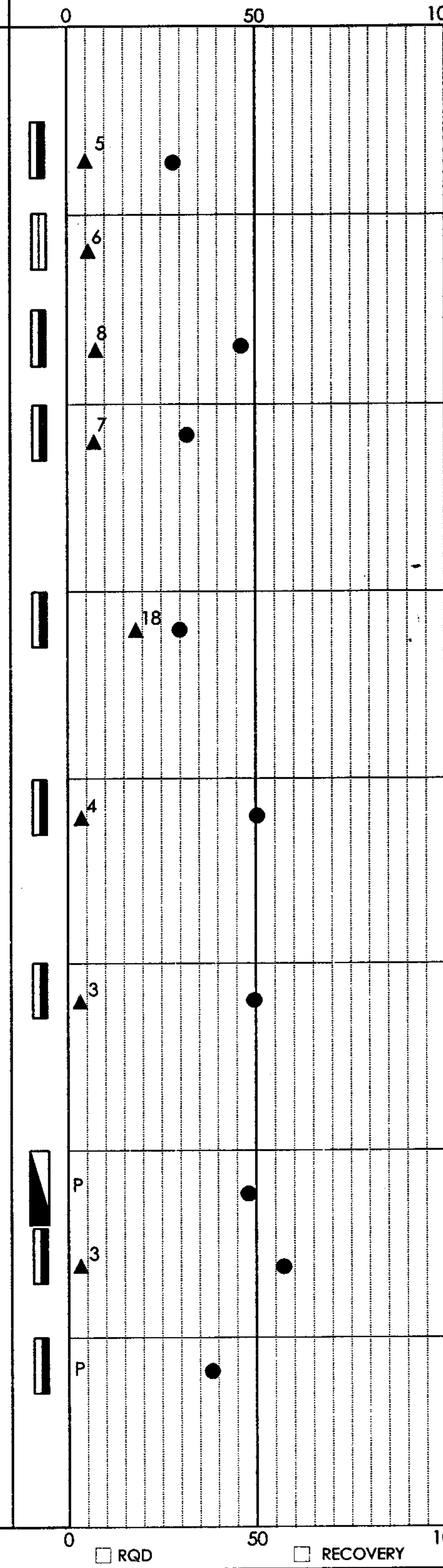
PP	POCKET PENETROMETER	LL	LIQUID LIMIT
TOR	TORVANE	PI	PLASTIC INDEX
CONSOL	CONSOLIDATION	PCF	POUNDS PER CUBIC FOOT
DS	DIRECT SHEAR	PSF	POUNDS PER SQUARE FOOT
P200	PERCENT PASSING U.S. NO. 200 SIEVE	TSF	TONS PER SQUARE FOOT
W	MOISTURE CONTENT		
DD	DRY DENSITY		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	CHEMICAL ANALYSIS
NA	NO VISIBLE SHEEN
SS	SLIGHT SHEEN
MS	MODERATE SHEEN
HS	HEAVY SHEEN
ND	NOT DETECTED

GEODESIGN, INC.	KEY TO TEST PIT AND BORING LOG SYMBOLS
	TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING
0		**SP-FILL** Loose, poorly graded, brown medium SAND FILL; moist.			
5		**ML-FILL** Medium stiff to stiff, gray SILT FILL with some organics (wood chips): moist.			
10		**SP-FILL** Loose, poorly graded, coarse black SAND FILL with some organics (wood chips); moist.			
15					
20		**ML** Soft to medium stiff, gray SILT; moist.			
25					
30					DD = 67 PCF
		ML Soft, dark gray SILT with trace fine sand; moist (organic odor).			
35					
40					



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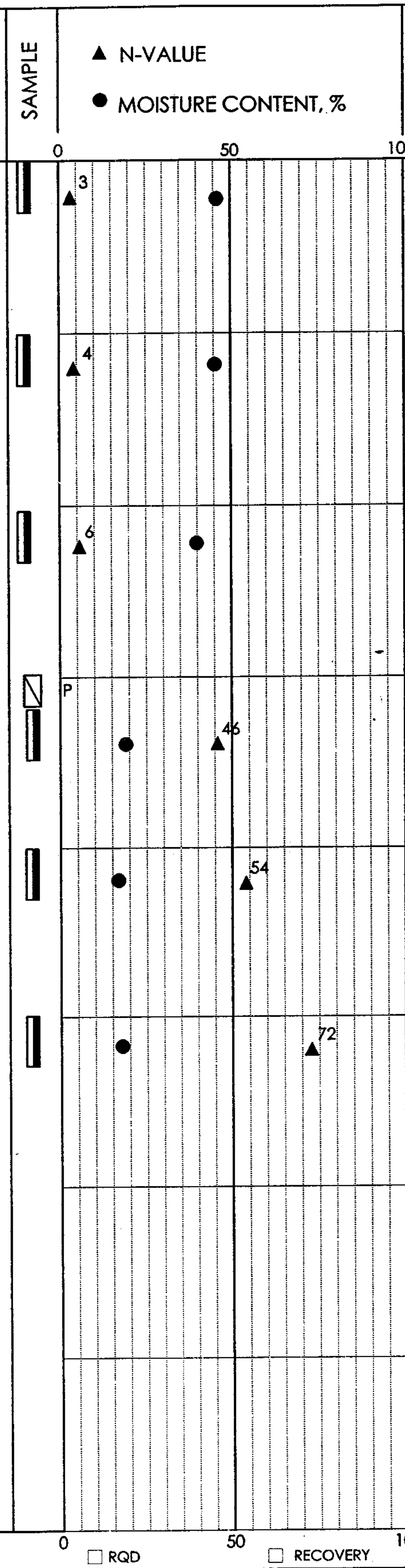
BORING B-1

OWENS CORNING-1	OCTOBER 1998	FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, %	ADDITIONAL TESTING
45		with trace organics at 45.0 feet			
50		grades to some sand at 50.0 feet			
56		**GW** Dense, sub rounded GRAVEL, with trace coarse, black silty sand; moist.			
66.5		Boring completed at 66.5 feet on September 14, 1998.			



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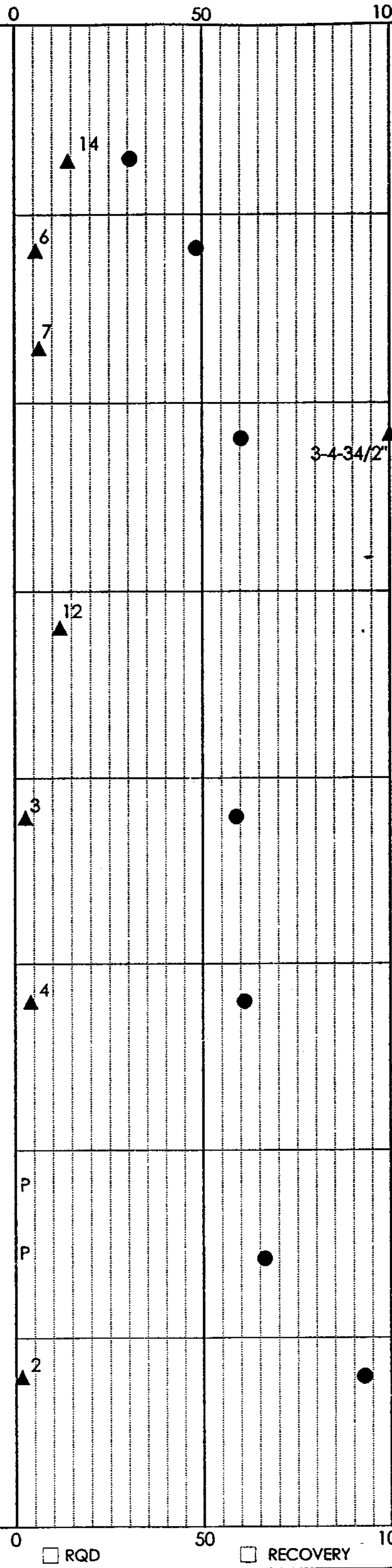
BORING B-1 (CONT.)

OWENS CORNING-1	OCTOBER 1998	FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, % (0 – 50 – 100)	ADDITIONAL TESTING
0		**SP-FILL** Loose, poorly graded, brown SAND FILL; moist.			
5		**SM-FILL** Loose, black silty SAND FILL with trace organics; moist.			
		GM-FILL Very dense, black silty GRAVEL FILL; moist (loosing drilling mud).			
10					
		with some wood chips in cuttings at 13.0 feet			
15		decaying sawdust (12-inch thick) at 15.0 feet			
20		**ML** Soft, gray SILT with some organics (wood chips); moist.			
25		becomes soft to medium stiff at 25.0 feet			
30		becomes stiff at 31.0 feet	P		PP = 1.8 TSF
		becomes very soft at 33.0 feet	P		
35					
40					



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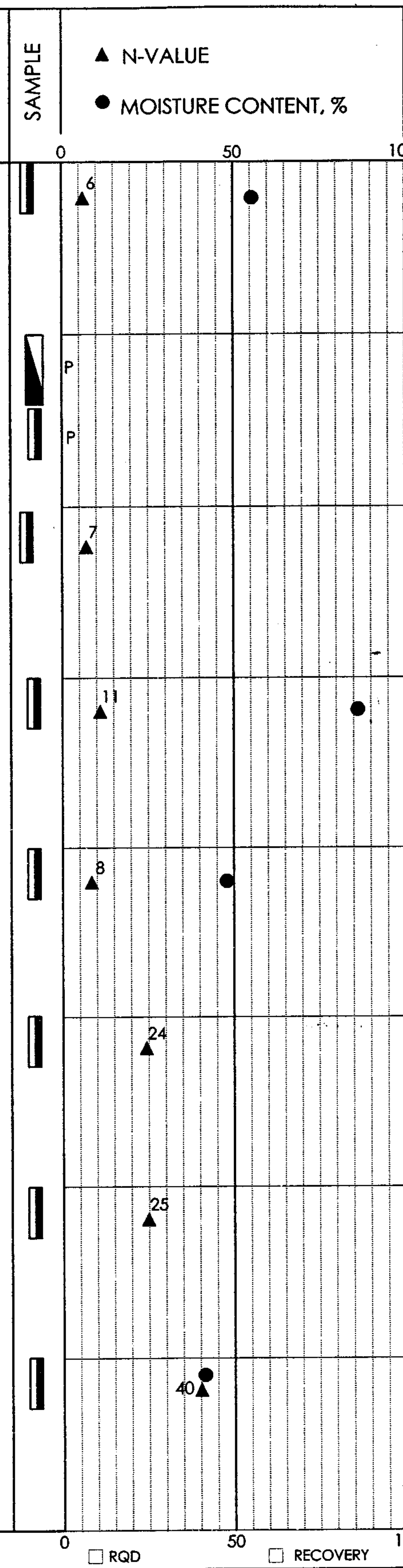
BORING B-2

OWENS CORNING-1	OCTOBER 1998	FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION
40		
45		
		becomes very soft at 48.0 feet
50		with trace organics at 50.0 feet
	ML	Stiff, blue-gray SILT; moist.
55		
	ML	Medium stiff to stiff, gray-brown SILT with trace fine sand; moist.
60		
65		becomes sandy at 65.0 feet
		gravel lens at 68.0 feet
70		
75		
	SM	Medium dense, dark gray silty SAND; moist.
80		



ADDITIONAL TESTING

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BORING B-2 (CONT.)

OWENS CORNING-1	OCTOBER 1998	FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	▲ N-VALUE ● MOISTURE CONTENT, % (0 – 50 – 100)	ADDITIONAL TESTING
80		**SP** Medium dense, dark gray to black, poorly graded SAND; moist.		10	
85				27	
		GW Very dense, black gravel with trace SILT; moist.			
90				50/4"	
		Boring completed at 92.0 feet on September 15, 1998.			
95					
100					
105					
110					
115					
120				0 – 50 – 100 ☐ RQD ☐ RECOVERY	

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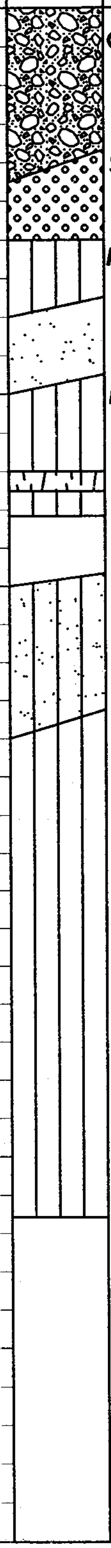
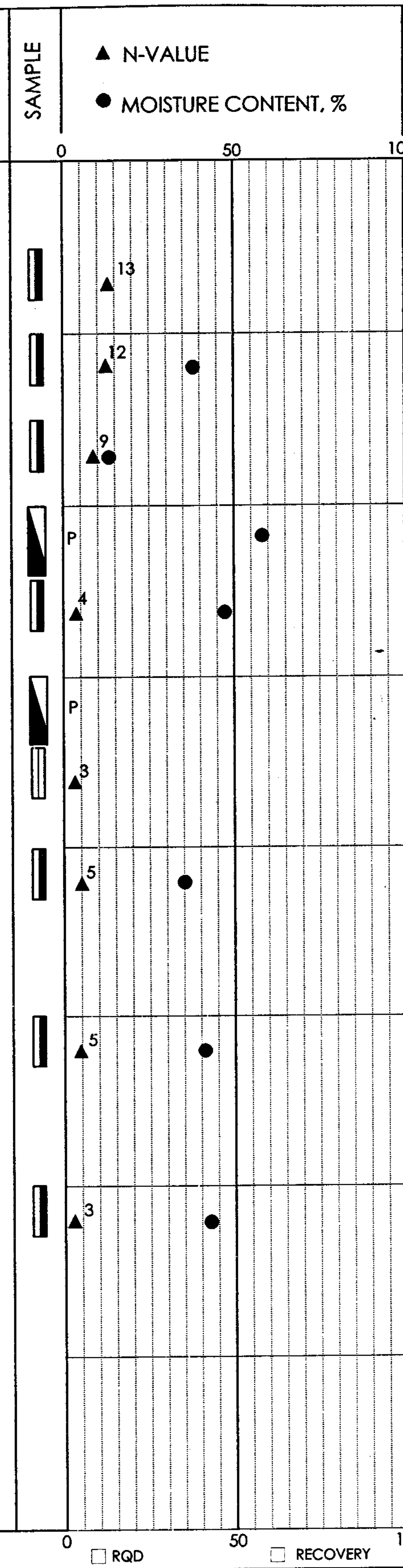
BORING B-2 (CONT.)

OWENS CORNING-1	OCTOBER 1998	FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION
0		**GW-FILL** Medium dense, angular GRAVEL FILL with some brown silt; moist.
5		**SW-FILL** Medium dense, gray-brown SAND FILL with trace brick fragments; moist.
		ML-FILL Medium stiff, gray SILT FILL; moist.
		SP-FILL Loose, dark gray, poorly graded SAND FILL with trace silt; moist.
10		**ML-FILL** Soft to medium stiff, gray SILT FILL; moist.
		decaying sawdust (4-inch thick)
		SP Medium dense, poorly graded fine SAND with trace silt; moist.
15		**SM** Dense, gray silty SAND; moist.
		ML Medium stiff, gray SILT; moist.
20		
		grades to brown with trace gray mottling at 23.0 feet
25		
		becomes soft and sandy at 28.0 feet
30		
		Boring completed at 31.5 feet on September 15, 1998.
35		
40		



ADDITIONAL TESTING

DD = 67 PCF

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BORING B-3

OWENS CORNING-1	OCTOBER 1998	FIGURE A-3

Owens Corning-1 B3@10.0'