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CDM Jessberger

A Division of Camp Dresser & McKee Inc.

geotechnical & environmental services

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November 13, 2000

Mr. Anthony Ordway
CertainTeed Roofing Products Group
6350 N.W. Front Avenue
Portland, Oregon 97210

242314

Subject: Geotechnical Investigation
Plant Building Improvement
6350 N.W. Front Avenue
Portland, Oregon

01-13589-00

Dear Mr. Ordway:

Camp Dresser and McKee Inc. (CDM) is pleased to present the results of our geotechnical investigation for the plant building improvements at the CertainTeed Roofing Products plant in Portland, Oregon. Our scope of work for this project was outlined in our proposal dated October 19, 2000. Authorization to proceed with our investigation was received on October 25, 2000.

Background Information

As shown on the Vicinity Map, Figure 1, the project site is located in the northwest industrial area on the west bank of the Willamette River.

The plant improvements will include the installation of a hopper near the beginning of the roofing production line. The hopper will have loads of approximately 170 kips on each of four columns. The roofing production line is located in an older, wood-framed metal building. We understand that the building was constructed in the 1930's and that the existing production line and hoppers are located on shallow concrete footings. The building appears to have functioned well, without noticeable settlement.

Field Investigation

One boring was drilled to a depth of 71.5 feet at the roofing products plant on October 29, 2000. Our original scope of work was to drill the boring using mud-rotary drilling methods, but because of access constraints, the boring necessarily was drilled using hollow-stem augers. The boring was drilled under subcontract to Geotech Explorations of Tualatin, Oregon with a truck-mounted Ingersoll Rand A-2000 drill rig. The approximate boring location is shown on the Site Exploration Plan, Figure 2.

The upper 10 feet of the boring was drilled without sampling because of the concern for buried structures. Below 10 feet, soil samples were taken at approximately 5-foot intervals using the Standard Penetration Test (SPT) method or thin-walled tube samples. The SPT method (ASTM D 1586) drives a 2-inch O.D. split barrel sampling tube into the soil using a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler each of three 6-inch increments is recorded and the sum of the blows for the final 12-inches of penetration is regarded as the N-value or SPT resistance. The N-value is a measure of the relative density of sands or the strength/consistency of clayey cohesive soils and is recorded as blows per foot (bpf). Following sampler removal from the boring, the sample was field classified and a representative soil sample was saved in airtight jars. The jars were labeled with the sample identification and returned to our laboratory.

A thin-walled "Shelby" tube sample was substituted in the sampling interval to obtain relatively undisturbed samples for in-situ unit weights. ASTM D 1587 methods were used. This procedure consists of hydraulically pushing a 3-inch diameter, thin-walled tubes below the base of the boring, and carefully removing the tube from hole. The ends of the tube were sealed to prevent moisture loss. The tube was then identified and returned to our laboratory for extrusion, classification, and testing, as required.

The boring was logged by an experienced CDM engineer who recorded sample depth and type, identified all samples, field-classified the soils, and developed preliminary field logs of the soil units encountered. The final boring log is presented on Figure 3.

Laboratory Testing

All jar samples were visually classified to refine, when necessary, the field soil classification. In additions, natural moisture contents were taken on all samples in accordance with ASTM D 2216. The moisture contents are expressed as a percentage of free water lost by evaporation compared to the dry weight of the soil. These results are presented graphically on the boring log.

The thin-walled, steel tube sample was extruded, classified, tested for relative strength with a Torvane and/or Pocket Penetrometer device, and tested for natural moisture content and unit weight. The unit weight determinations are tabulated on the log.

Subsurface Conditions

Soil Conditions

Based on the exploration, the subsurface profile can be described by six units. From the ground surface downward, these units are: 1) Sand FILL, 2) SAND, 3)Interbedded SILT and SAND, 4) Medium-stiff SILT, 5) SILT and SAND, and 6) Silty SAND to sandy SILT. A summary of the typical units is presented below.

Sand FILL: The site is mantled with approximately 14 feet of sand FILL with trace to some silt. Because of a concern about buried structures, the boring was augered to a depth of 10 feet without Standard Penetration Test (SPT) sampling. A sample of the auger cuttings was obtained within 10 feet of the floor slab. SPT blow counts below 10 feet were 9 and 17 blows per foot (bpf). Moisture contents above the groundwater level (12 feet) were 8 and 11 percent. Below the groundwater level, the moisture content was 25 percent. Upon removal of the auger at completion of the boring, the drilled hole collapsed below the groundwater level.

SAND: Very dense SAND that grades medium-dense with depth underlies the sand FILL. This unit extends from approximately 14 to 31 feet. SPT blow counts at 15 and 20 feet were 59 bpf and 75 blow per 11 inches of penetration. SPT blow counts at 25 and 30 feet were 22 and 16 bpf respectively. Moisture contents ranged from 25 to 33 percent and averaged 29 percent.

Interbedded SAND/SILT: From a depth of approximately 31 to 36 feet, interbedded SAND and SILT was encountered. A SILT layer from about 31 to 33 feet contained organics. One SPT in a sand layer at 35 feet was 12 bpf. Moisture contents ranged from 74 percent in the SILT with organics to 28 percent in the SAND.

Medium-stiff SILT: Medium-stiff SILT with trace clay was encountered at depths of approximately 36 to 45 feet. The SPT blow count at 40 feet was 4 bpf. Moisture contents were 32 percent.

SILT and SAND: Very stiff SILT to medium-dense silty SAND and sandy SILT was encountered at depths of 45 to 60 feet. SPT blow counts were 27 and 22 bpf. Moisture contents ranged from 28 to 40 percent and averaged 32 percent.

Silty SAND to sandy SILT: Below a depth of 60 feet to the maximum explored depth of 71.5 feet, dense to very dense silty SAND to sandy SILT was encountered. SPT blow counts ranged from 45 to 59 bpf and average 52 bpf. Moisture contents ranged from 35 to 39 percent and averaged 36 percent.

Groundwater

Groundwater was observed at a depth of 12 feet during drilling. We anticipate that the groundwater level will correspond closely with the Willamette River level and was near the seasonal low during drilling.

Comments and Recommendations

General

The site is mantled with approximately 14 feet of loose to medium-dense sand FILL overlying dense SAND and medium-stiff SILT. Very dense silty SAND to sandy SILT is located below a depth of 60 feet. The design groundwater level is assumed at 12 feet. We anticipate that three loose to medium-dense sand layers are susceptible to liquefaction.

There are several potential foundation alternatives to support the equipment including, micropiles, driven pipe piles, and drilled shafts. Micropiles are designed and built by specialty contractors. These foundation alternatives and other issues are discussed in the following paragraphs.

Micropiles

Micropiles are small diameter (typically less than 12 inches) drilled and pressure-grouted piles. They can be installed in limited access areas and produce minimal vibrations and noise during installation. The special drilling and grouting methods used in micropile installation enable high grout to ground bond, which allows for higher pile capacities than similar sized conventional drilled shafts. Micropiles are designed and installed by specialty contractors, many with low-clearance equipment.

Driven Piles

CDM developed a soil model for subsurface conditions based on the exploratory boring. We estimated pile capacities versus depth under static loading conditions using the Nordlund Method for cohesionless soils. In our opinion, this is the best available method for estimating static pile capacities. The designer should use the results carefully, realizing that the capacities are estimates and that subsurface conditions can vary from the conditions encountered at our boring.

The ultimate pile load will equal the sum of the live load and dead load multiplied by 3.0 (safety factor). We have assumed that two piles with 85 kip design capacities will be installed at each column. For design dead plus live loads of 85 kips, the required ultimate capacity is 255 kips. Estimated pile capacities for 12.75, 16, and 18-inch diameter, closed-end pipe pile versus depth are presented graphically on Figure 4.

As shown on the Figure 4, the larger diameter piles may achieve the ultimate capacity of 255 kips at depths of 15 to 20 feet. However, in an earthquake and liquefaction condition, the ultimate capacities are expected to drop and pile settlement may occur. For this reason, we recommend that piles be driven to the required ultimate capacities at depths greater than the anticipated liquefaction (40 feet). The estimated ultimate capacity of the piles considering only capacity developed below the potentially liquefiable layers is presented on Figure 5.

All piles should be driven in accordance with the ODOT Standard Specifications for Highway Construction, 1996, (ODOT-SSHC), Section 00520. Drive piles to the required minimum blow count using the required ultimate capacity and the Gates equation. Closed ends are recommended for all pipe piles.

We anticipate that the piles may not achieve capacity during initial driving. Easy driving occurs when large pore pressures build-up along the pile shaft causing the soils to liquefy and lose most of their strength. When the pore pressures dissipate due to drainage, the effective soil strength increases dramatically, increasing the side shear resistance of the pile shaft resulting in pile "freeze". Therefore, if the piles do not achieve capacity during the initial driving, we recommend that the "set period" criteria of ODOT 00520.42(d) be allowed.

Installation of driven piles produces noise and potentially damaging vibrations to surrounding equipment. In addition, because of the limited clearance special, continuous pile driving will not be possible. Splicing of short pile sections will be required.

The estimated uplift capacities for the pipe piles driven to an ultimate static compressive capacity of 255 kips are presented on the following table.

Pile Diameter (in)	Estimate Tip Depth (feet)	Static Ultimate Side Resistance (kips)	Liquefied Ultimate Side Resistance (kips)
12.75	50	110	55
16	44	135	55
18	44	162	85

We recommend that uplift to resist seismic forces be limited to the ultimate liquefaction side resistance presented in the above table. Greater uplift resistance can be achieved by driving the piles to higher capacities (deeper embedment).

Drilled Shafts
Using the soil profile model developed from the boring information, we estimated axial drilled shaft capacities for 16, 18, 20, and 24-inch diameter shafts using the procedures presented in the AASHTO Standard Specifications for Highway Bridges. A plot of the ultimate drilled shaft capacity versus embedment depth for the different diameter shafts is presented on Figure 6. The capacities assume that any casings are removed during construction and that a grout/concrete to soil bond achieved over the entire embedment length.

During an earthquake when some of the soil layers liquefy and loose strength, the drilled shaft capacity will decrease. For the liquefaction condition, we have assumed that the drilled shaft will develop its capacity only from the soil layers below the liquefiable zones (>36 feet). A plot showing the ultimate drilled shaft capacities for this liquefaction condition is shown on Figure 7.

AASHTO recommends that a 2.5 safety factor be applied to the ultimate axial compression loads to obtain the allowable design loads.

Uplift resistance can be achieved from a combination of the pier weight and the pier side resistance. Ultimate drilled shaft side resistance versus embedment depth for the different shaft diameters is presented on Figure 8. A plot of ultimate side resistance for the liquefaction condition is presented on Figure 9. AASHTO recommends that the ultimate uplift be reduced to 70 percent of the ultimate side resistance plus the shaft weight (Qult = 0.7Qs +W).

Drilled shafts can be constructed using several methods. For this site, we anticipate that augercast methods are the most practical method to install drilled shafts. Open-hole drilling techniques will not work in these soil/groundwater conditions.

Lateral Capacity

The lateral capacity of the driven pipe piles and drilled shafts were analyzed using a finite difference computer program modified from COM622. For our analyses, we assumed that the pile top was at the floor elevation and fixed against rotation. The results of our analyses are presented on Figure 109. The analysis indicates that the piles and drilled shafts develop their lateral capacity from the sand FILL soils. The lateral capacity of pile groups spaced less than eight diameters apart should be limited by the following reduction factors.

Pile Spacing D=Pile Diameter	Reduction Factor
8D	1.0
6D	0.70
4D	0.40
3D	0.25

Seismic Considerations

A seismic hazard evaluation of the site was not included in our scope or work. However, we reviewed the Relative Earthquake Hazard Map of the Portland Metro Region (DOGAMI IMS-1). This map identifies the relative earthquake hazards for slope stability, liquefaction, amplification, and overall hazard in the Portland area.

The liquefaction, amplification, and slope instability hazards are rated from 1 to 3 with 1 being the lowest hazard and 3 being the greatest. The site is mapped as a liquefaction hazard of 3, amplification hazard of 1, and no slope instability hazard. An amplification hazard of 1 indicates that bedrock accelerations will be amplified less than 1.25 times. The combined seismic hazards are mapped as Zone A through Zone D with Zone A having the greatest overall hazard. The site is mapped as Zone B.

The seismic hazards that would be anticipated from the soil conditions encountered in our boring are consistent with the relative seismic hazard map.

Liquefaction, or the loss of shear strength, can occur in loose saturated sands and some fine-grained soils during an earthquake. A formal liquefaction analysis was not included in our scope of work. Based on past experience, it is our opinion that three, medium-dense sand layers at approximate depths of 12 to 14 feet, 23 to 31 feet, and 33 to 36 feet are susceptible to liquefaction. Liquefaction of the sand would result in ground settlement on the order of 4 to 8 inches.

Most of western Oregon, including the site, is located in UBC Seismic Zone 3. Based on subsurface conditions encountered in our boring, it is our opinion that the subsurface profile is most similar to UBC Soil type S_D, Stiff Soil Profile. We recommend that soil type S_D be used.

UBC states that liquefiable soils require a type S_F classification and a site specific seismic evaluation. Deep foundations will be used for this project, mitigating the impact of estimated liquefaction. The selection of soil type S_D suggests a site amplification factor of 1.5 (Ca=0.24 for type S_A versus Ca=0.36 for type S_D), which conservatively exceeds the maximum amplification of 1.25 estimated by the ground response studies performed for the seismic hazard mapping. Therefore, the use of soil type S_D appears to be a conservative choice for static base shear calculations without a ground response analysis.

General Notes

This report was prepared solely for the Owner and Engineer for the design of the project. We encourage its review by bidders and/or the Contractor as it relates to factual data only (boring logand laboratory data). The opinions and recommendations contained within the report are not intended to be nor should they be construed to represent a warranty of subsurface conditions but are forwarded to assist in the planning and design process.

If, during construction, unexpected subsurface conditions are encountered, we should be notified at once so that we may review such conditions and revise our recommendations, if necessary. We request that we be retained to review the applicable portion of the plans and specifications for the project prior to bidding for conformance to our recommendations.

We would be pleased to provide additional input, as necessary, during the design process and to provide on-site observations during construction. Please feel free to contact us for this work as well as for any questions you might have regarding this report.

Very truly yours,

CAMP DRESSER & McKEE INC.

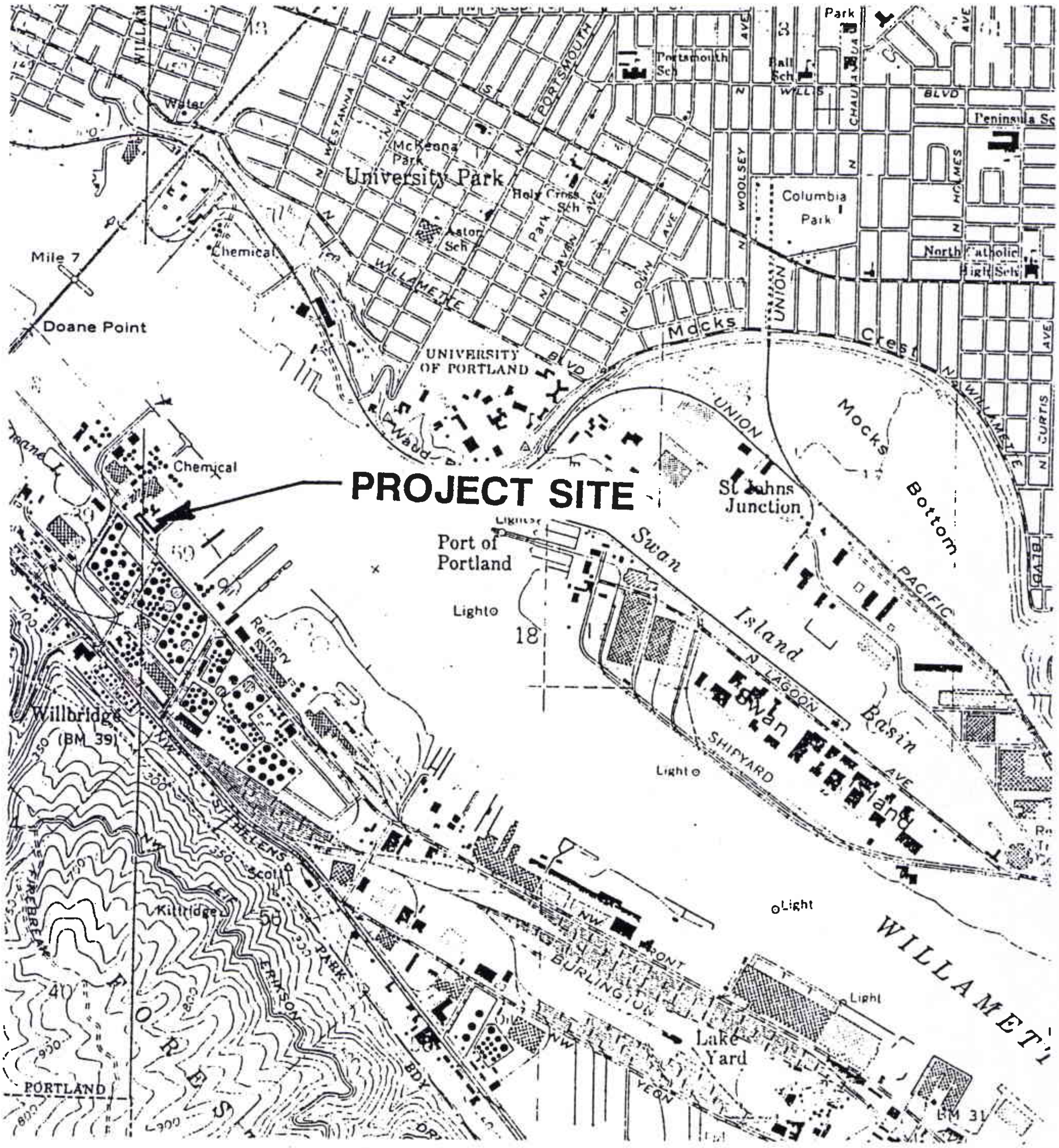
REGISTERED PROFESSIONAL ENGINEER
13146
OREGON
SEPT. 19, 1995
LLOYD J. REITZ
EXPIRES: 12-31-00
11-13-00

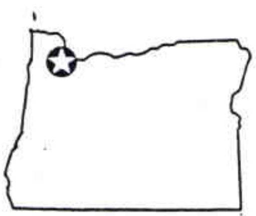
Lloyd J. Reitz, P.E.
Geotechnical Engineer

Robert J. Strazer, P.E.
Associate Geotechnical Engineer

LJR/RJS

Attachments: Figures 1 through 10





Index Map

Reference: USGS Portland Quadrangle 7.5-minute series, 1961, photorevised 1977

N

0 1000 2000

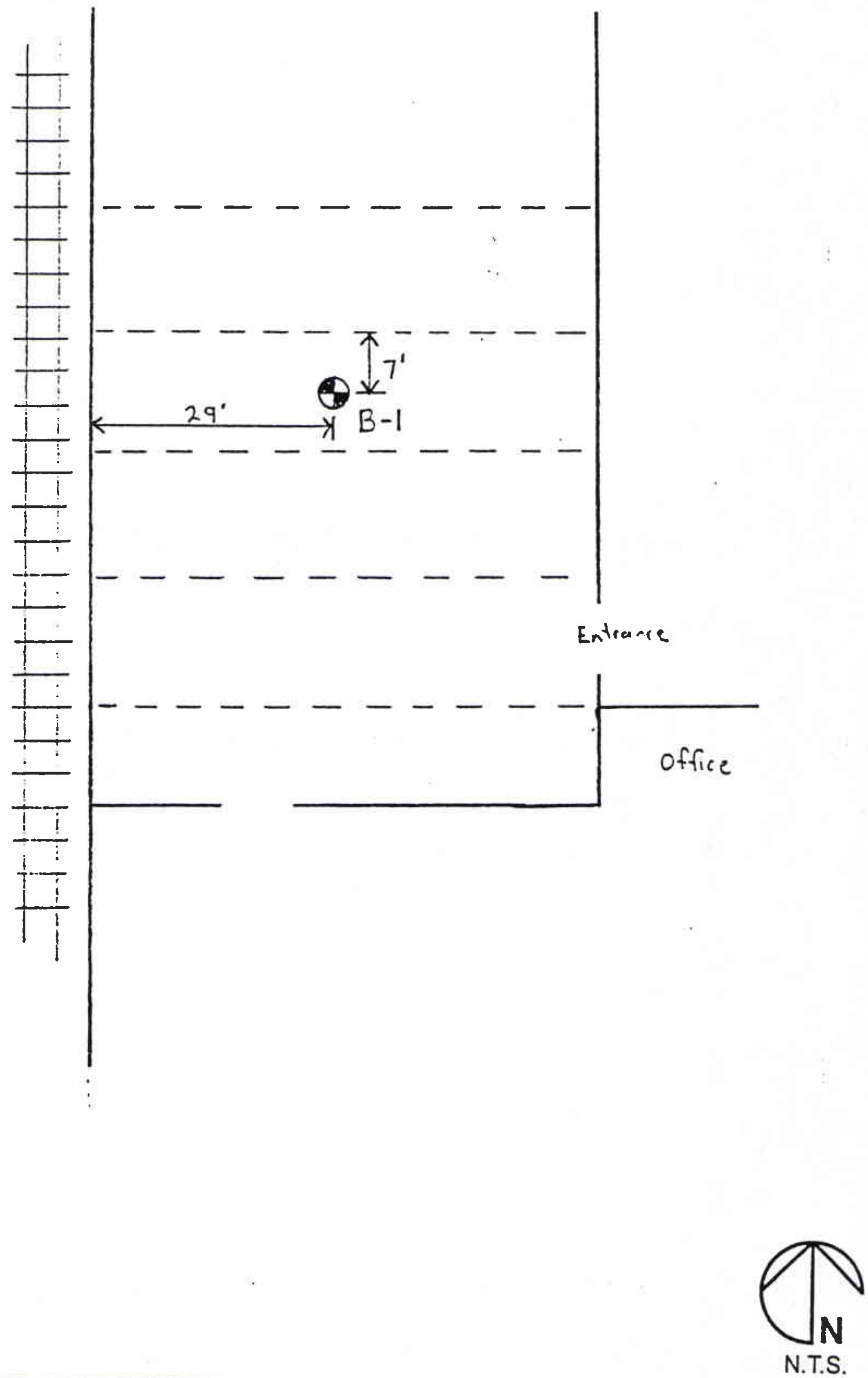
Scale in Feet

Vicinity Map

CertainTeed Roofing Products / Plant Building Improvements
Portland, Oregon

FIGURE 1

CDM Jessberger Inc.	DRAWING NAME	PROJECT NO	DRAWN	DATE	APPROVED
	Vicinity.cdr	20322-31233	LJR	09 Nov 00	



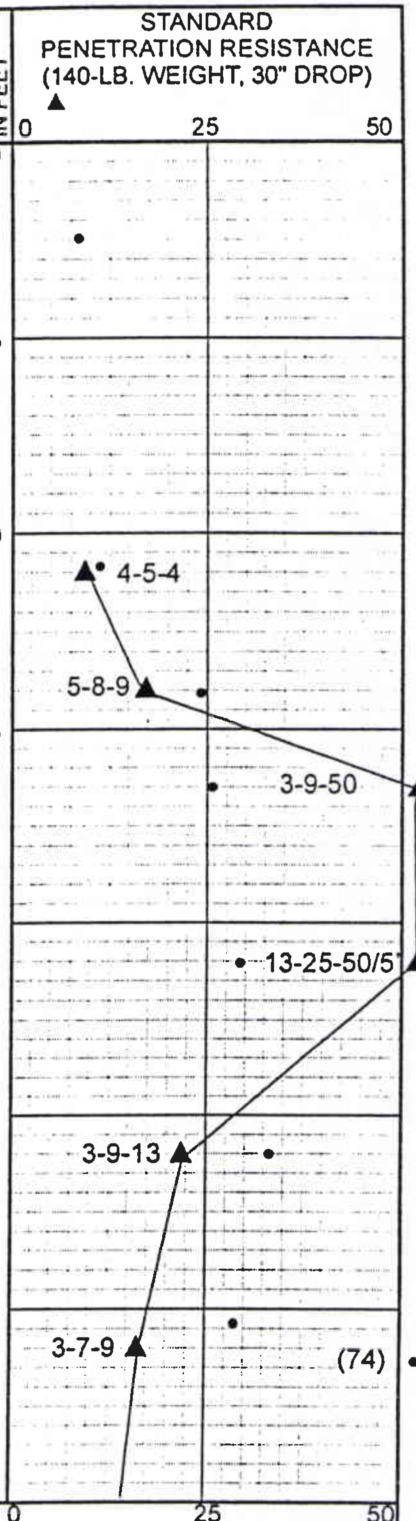
Site Exploration Plan

CertainTeed Roofing Products / Plant Building Improvements
Portland, Oregon

FIGURE
2

CDM Jessberger Inc.	DRAWING NAME	PROJECT NO	DRAWN	DATE	APPROVED
	Site.cdr	20322-31233	LJR	12 Nov 00	

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Top of slab		SAMPLE	GROUND WATER	DEPTH IN FEET
	7" thick concrete slab	0.58			0
	-grab sample. Loose to medium-dense, brown, fine SAND, SP, trace to some silt, dry to damp. (FILL)		S-1		5
			S-2	∇ 10-29-00	10
	-grades wet, leaf in sample.	14.0	S-3		
	Medium-dense to dense, brown to black, fine SAND, SP, wet. (NATIVE) (sampler may be pushing a rock)		S-4		15
	- very dense.		S-5		20
	- grades medium-dense		S-6		25
	- increasing organics and trace fine gravel.	31.0	S-7-		30
	Stiff, brown, SILT, ML, trace clay and organics, wet, low plasticity	33.0			
	Medium-dense, gray, fine to medium-grained SAND, SP, wet.				35



2.0" O.D. SPLIT SPOON SAMPLE
3.25" O.D. SPLIT SPOON SAMPLE
3.0" O.D. THIN WALL SAMPLE
CORE RUN SAMPLE

P SAMPLE PUSHED
* SAMPLE NOT RECOVERED

ATTERBERG LIMITS
- LIQUID LIMIT
- NATURAL WATER CONTENT
- PLASTIC LIMIT

IMPERVIOUS SEAL
∇ WATER LEVEL
OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE. ACTUAL CHANGES MAY BE GRADUAL.
2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

Log of Boring B-1

CertainTeed Roofing Products / Plant Building Improvements
Portland, Oregon

FIGURE **3a**

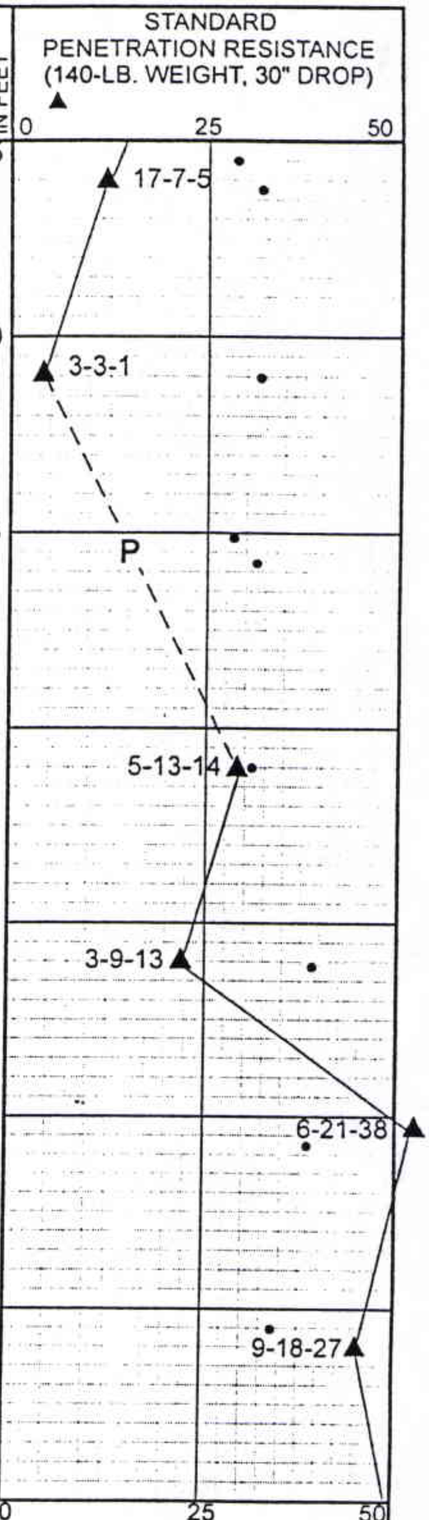
CDM Jessberger Inc.

DRAWING NAME	PROJECT NO	DRAWN	REVISED	DATE
Boring.cdr	20322-31233	LJR		09 Nov 00

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Top of slab		SAMPLE	GROUND WATER	DEPTH IN FEET
	Medium-dense, gray, fine to medium-grained SAND, SP, wet.	36..0	S-8		35
	Stiff, gray SILT with trace clay, ML, wet, low plasticity.				
	- grades soft to medium-stiff.		S-9		40
			S-10		45
	- grades very stiff.		S-11		50
		52.0			
	Medium-dense, gray, silty fine SAND to sandy SILT, SM/ML, wet.		S-12		55
	- grades very dense.		S-13		60
	- grades dense, 2" thick clay lens.		S-14		65
					70

LAB DATA

Sample Number	Depth (ft)	γw (pcf)	γd (pcf)	wc (%)
S-10A	45.0 - 45.6	129.34	99.8	`29.6



2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

CORE RUN SAMPLE

P SAMPLE PUSHED

* SAMPLE NOT RECOVERED

ATTERBERG LIMITS
- LIQUID LIMIT
- NATURAL WATER CONTENT
- PLASTIC LIMIT

IMPERVIOUS SEAL

WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE, ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

Log of Boring B-1

CertainTeed Roofing Products / Plant Building Improvements
Portland, Oregon

FIGURE **3b**

CDM Jessberger Inc.

DRAWING NAME	PROJECT NO	DRAWN	REVISED	DATE
Boring.cdr	20322-31233	LJR		09 Nov 00

GRAPHIC LOG	SOIL DESCRIPTION SURFACE ELEVATION = Top of slab		SAMPLE	GROUND WATER	DEPTH IN FEET	STANDARD PENETRATION RESISTANCE (140-LB. WEIGHT, 30" DROP) ▲ 0 25 50
	Very dense, gray, silty fine SAND to sandy SILT, SM/ML, wet.	71.5	S-15		70	8-18-34
	Bottom of boring at 71.5 ft, completed 10/29/00 Groundwater encountered at approximately 12 ft				75	
					80	
					85	
					90	
					95	
					100	
					105	0 25 50

2.0" O.D. SPLIT SPOON SAMPLE

3.25" O.D. SPLIT SPOON SAMPLE

3.0" O.D. THIN WALL SAMPLE

CORE RUN SAMPLE

P SAMPLE PUSHED

* SAMPLE NOT RECOVERED

ATTERBERG LIMITS

LIQUID LIMIT

NATURAL WATER CONTENT

PLASTIC LIMIT

IMPERVIOUS SEAL

WATER LEVEL

OBSERVATION WELL TIP

• WATER CONTENT IN PERCENT

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE. ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

Log of Boring B-1

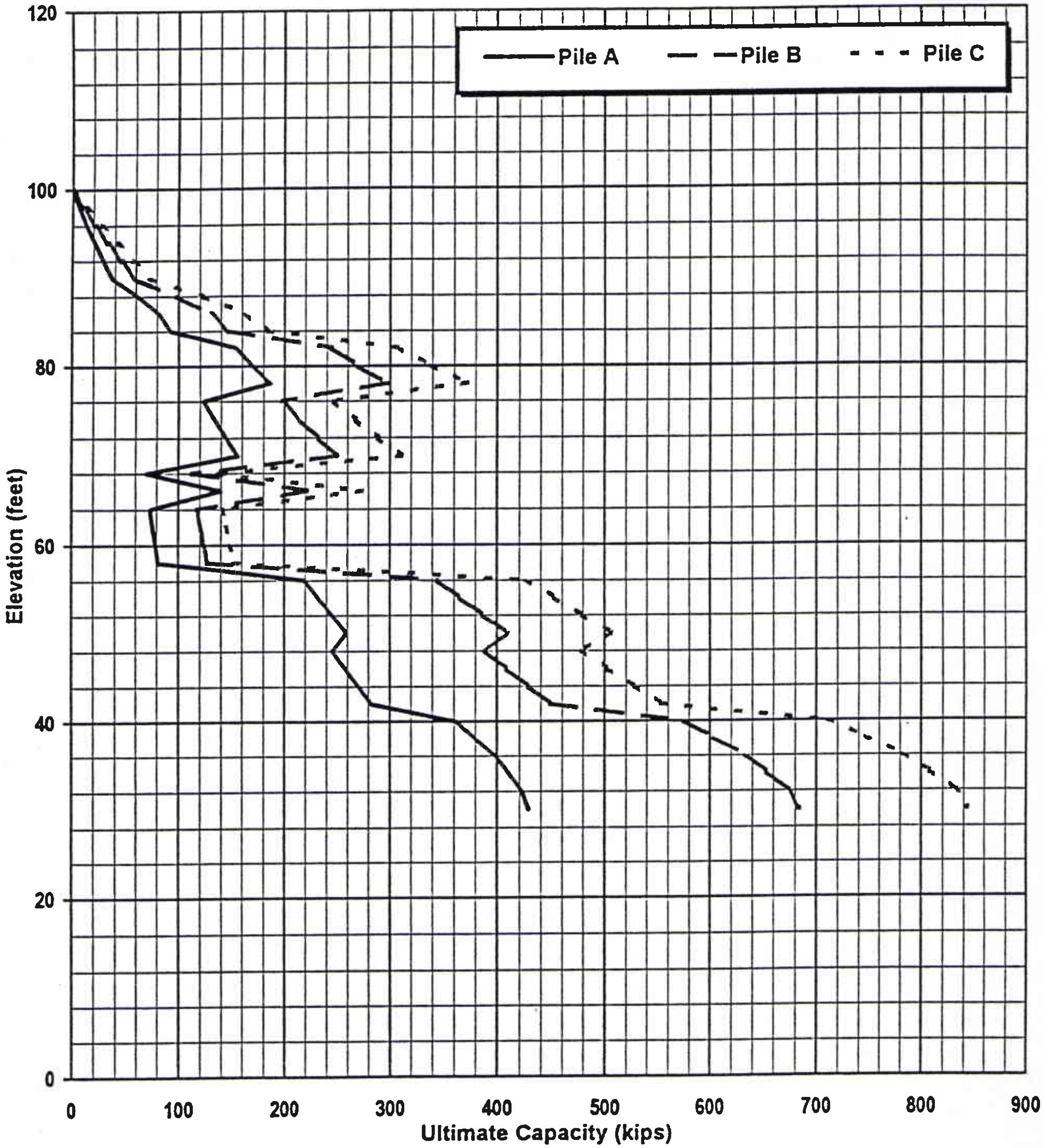
CertainTeed Roofing Products / Plant Building Improvements

Portland, Oregon

FIGURE **3c**

CDM Jessberger Inc.

DRAWING NAME	PROJECT NO	DRAWN	REVISED	DATE
Boring.cdr	20322-31233	LJR		09 Nov 00



Pile A - 12.75-inch pipe
Pile B - 16-inch pipe
Pile C - 18-inch pipe

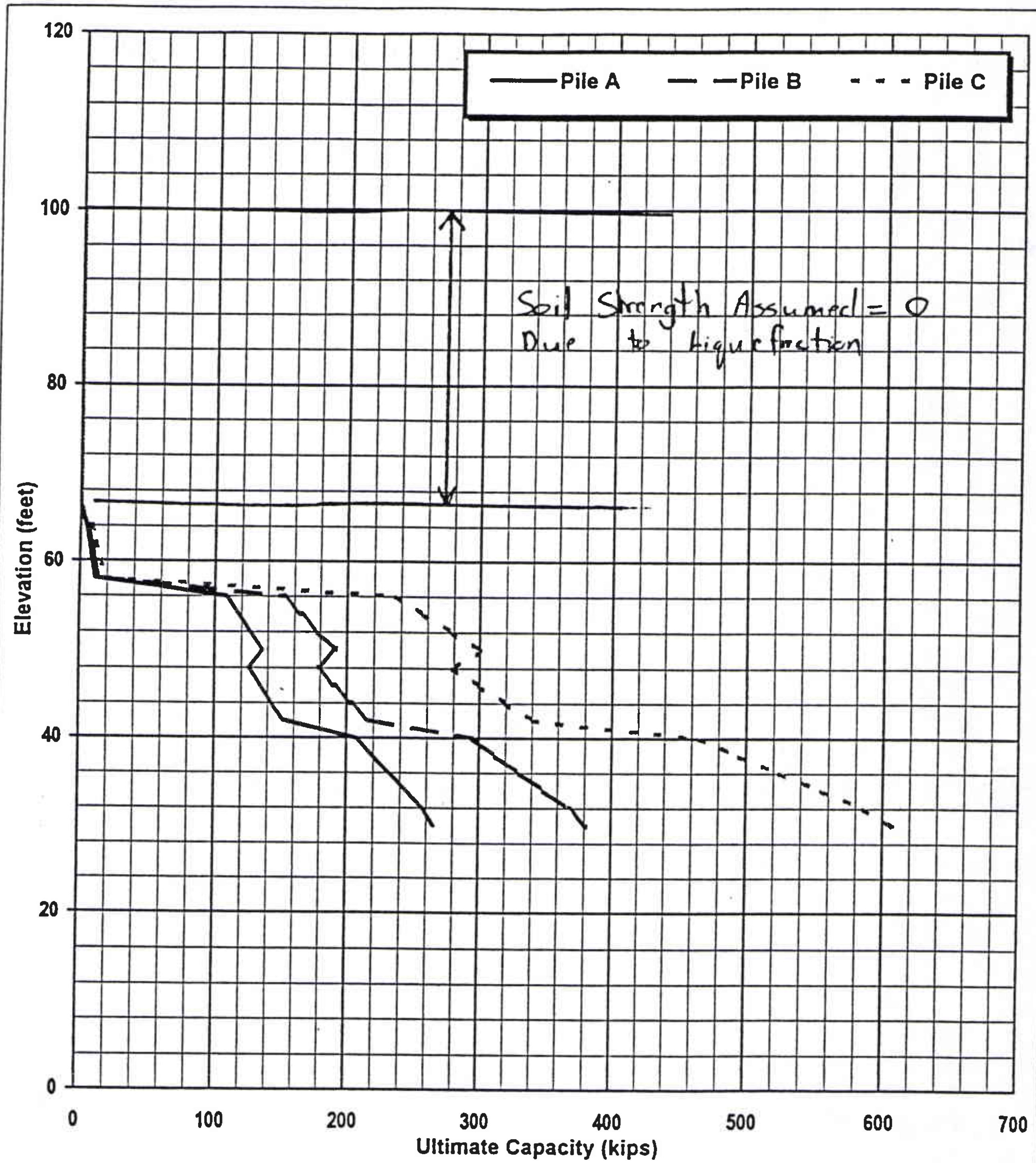
Note: Floor slab elevation assumed +100 feet.

Ultimate Pile Capacity - Static Condition

CertainTeed Roofing
Portland, Oregon

FIGURE **4**

CDM Jessberger	DRAWN	DATE	PROJECT NO.	FILE NAME
	LJR	13-Nov-00	20322-31233	Certain Teed.xls



Pile A - **12.75-inch pipe**
Pile B - **16-inch pipe**
Pile C - **18-inch pipe**

Note: Floor slab elevation assumed +100 feet.

Ultimate Pile Capacity - Liquefaction Condition

CertainTeed Roofing
Portland, Oregon

FIGURE **5**

	DRAWN	DATE	PROJECT NO.	FILE NAME
CDM Jessberger	LJR	13-Nov-00	20322-31233	Certain Teed Liquefied.xls

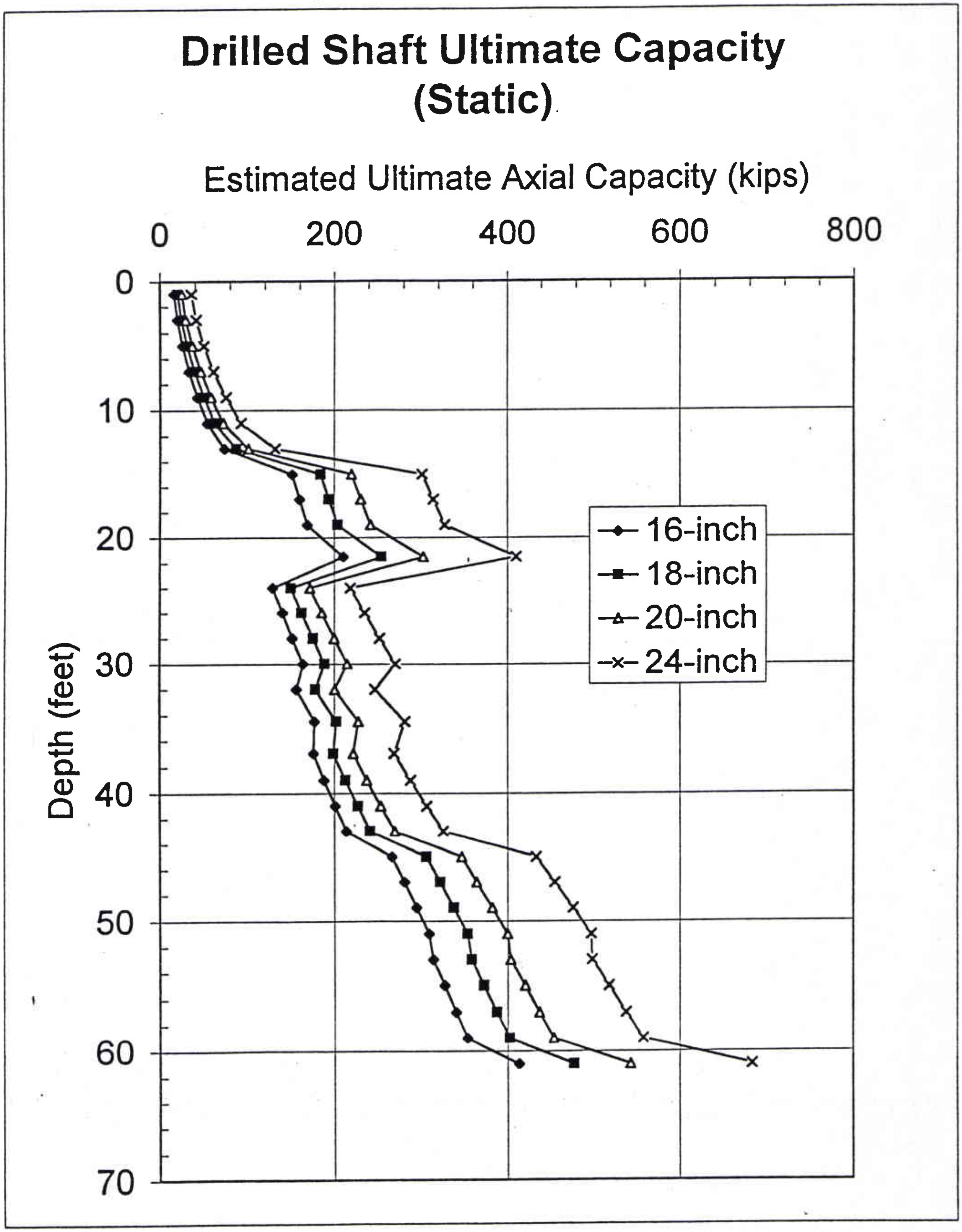
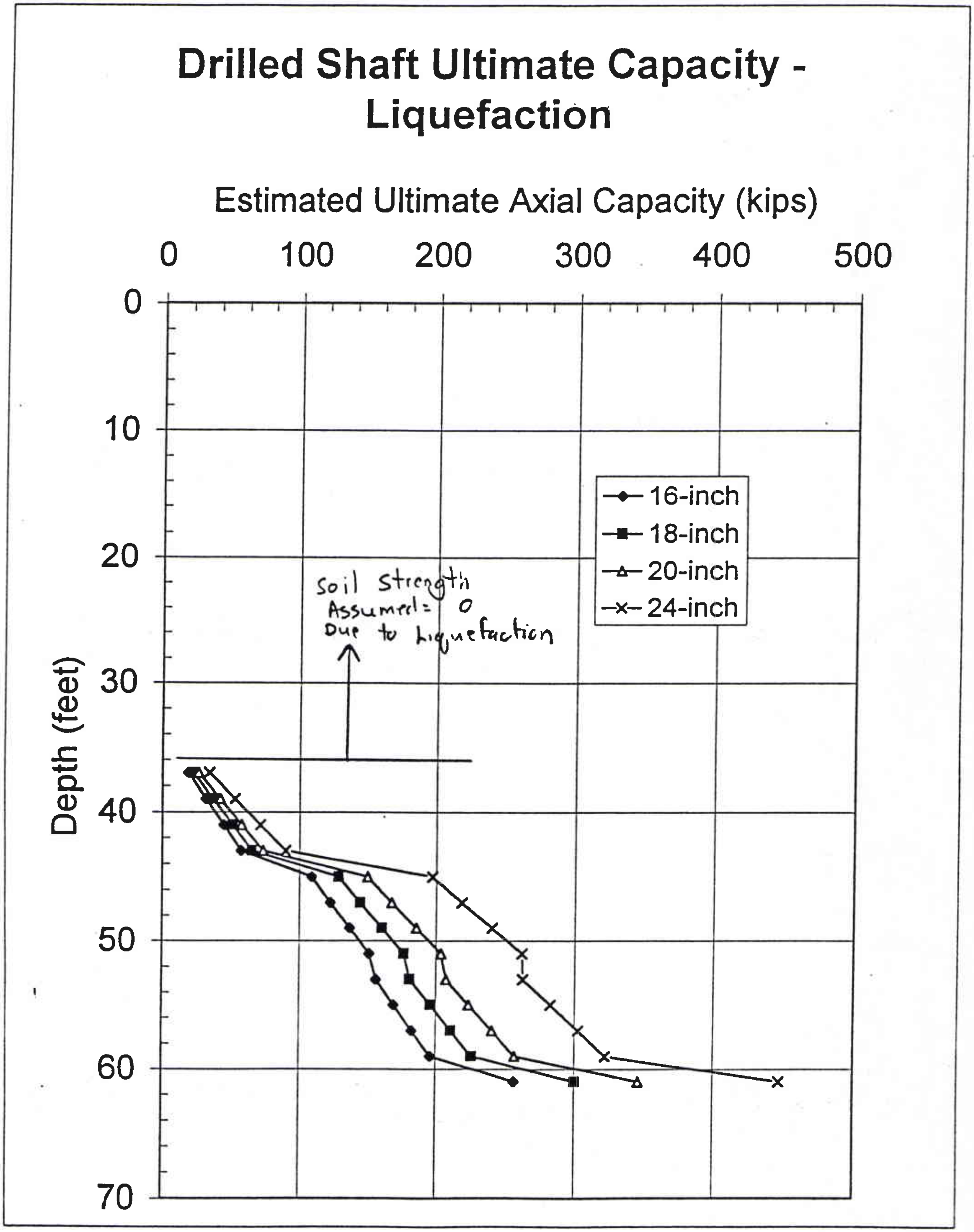

Drilled Shaft Ultimate Capacity
(Static)
Estimated Ultimate Axial Capacity (kips)
0
200
400
600
800
Depth (feet)
0
10
20
30
40
50
60
70
16-inch
18-inch
20-inch
24-inch

Figure 6



Drilled Shaft Ultimate Capacity - Liquefaction
Estimated Ultimate Axial Capacity (kips)
0
100
200
300
400
500
Depth (feet)
0
10
20
30
40
50
60
70
16-inch
18-inch
20-inch
24-inch
Soil Strength Assumed = 0 Due to Liquefaction

Figure 7

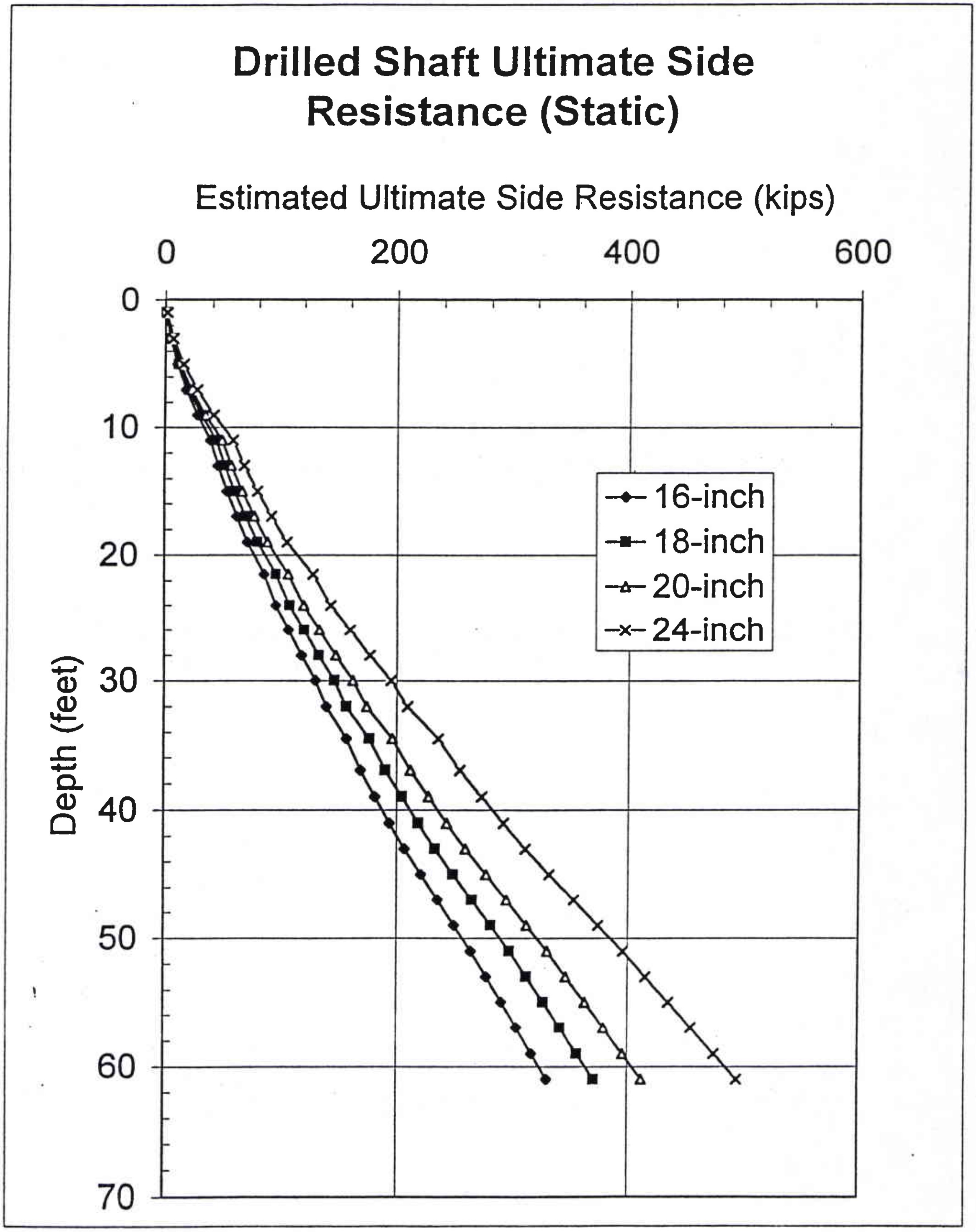

Drilled Shaft Ultimate Side Resistance (Static)
Estimated Ultimate Side Resistance (kips)
0
200
400
600
0
10
20
30
40
50
60
70
Depth (feet)
16-inch
18-inch
20-inch
24-inch

Figure 8

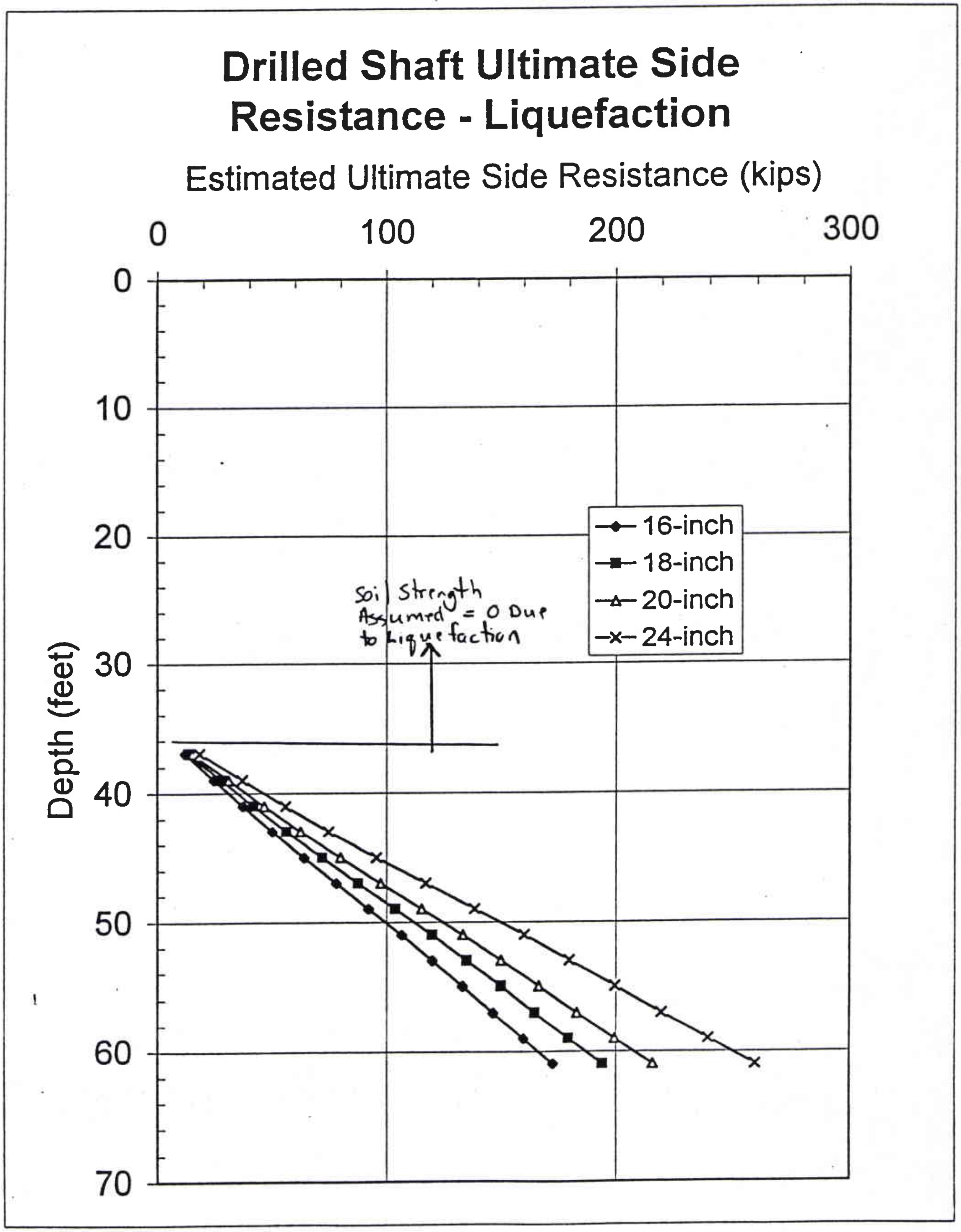

Drilled Shaft Ultimate Side Resistance - Liquefaction
Estimated Ultimate Side Resistance (kips)
0
100
200
300
Depth (feet)
0
10
20
30
40
50
60
70
16-inch
18-inch
20-inch
24-inch
Soil Strength Assumed = 0 Due to Liquefaction

Figure 9

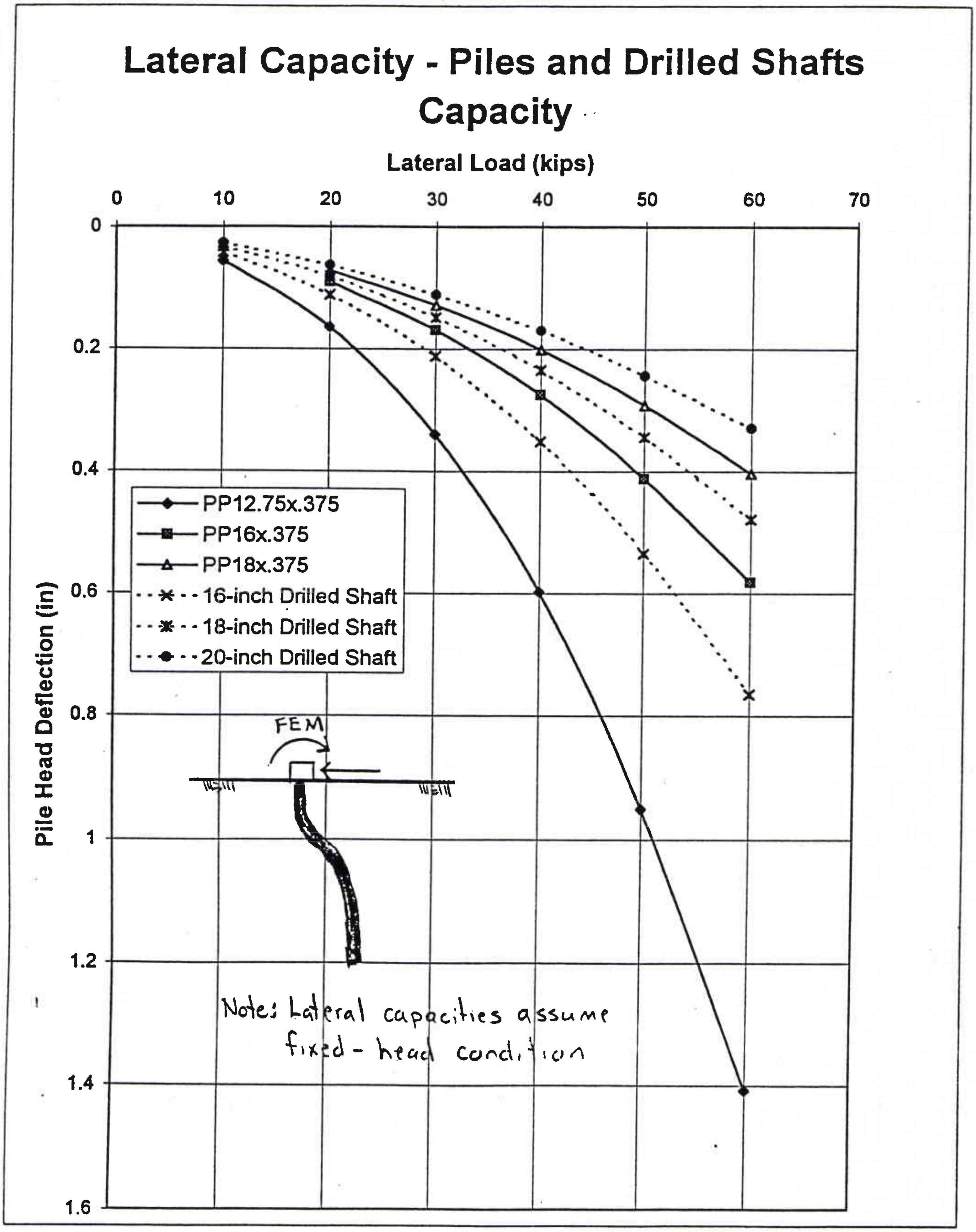

Lateral Capacity - Piles and Drilled Shafts
Capacity
Lateral Load (kips)
0
10
20
30
40
50
60
70
Pile Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
PP12.75x.375
PP16x.375
PP18x.375
16-inch Drilled Shaft
18-inch Drilled Shaft
20-inch Drilled Shaft
FEM
Note: Lateral capacities assume fixed-head condition

Figure 10