GEOTECHNICAL INVESTIGATION

METROPOLITAN EXPOSITION CENTER EXPANSION
PORTLAND, OREGON

Submitted To:

Metropolitan Exposition - Recreation Commission
777 N. E. Martin Luther King Jr. Boulevard
Portland, Oregon 97232

Submitted By:

AGRA Earth & Environmental, Inc.
7477 S. W. Tech Center Drive
Portland, Oregon 97223-8025

21-08598-00

March 1996
With April 18, 1996, letter containing clarifications
and additional recommendations

AGRA
Earth & Environmental

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APR 22 1996
Ans'd............

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April 18, 1996
21-08598-0

Mr. Mark Hunter
Construction/Capital Projects Manager
777 N.E. Martin Luther King Jr. Boulevard
Portland, Oregon 97232

Dear Mr. Hunter:

RE: CONSULTATION
METROPOLITAN EXPOSITION CENTER EXPANSION
PORTLAND, OREGON

This letter has been prepared to present additional recommendations for the subject project and to clarify previous recommendations for overexcavation of existing fill soils. AGRA Earth & Environmental, Inc. (AEE)'s geotechnical report of March 15, 1996 recommended that three feet of existing fill soil be excavated and recompacted prior to site development. This recommendation was based on the assumption that the new site elevations would not be substantially different from existing elevations. A review of preliminary grading plans for the project indicates that fills on the order of 2.5 to 3.5 feet will be placed over portions of the site. The three-foot-thick section of compacted soil is intended to provide firm soil support for the exposition center concrete slab. It is our opinion that the three-foot-thick section of compacted soil may come from imported fill soil, recompacted existing fill soil, or combinations of the two that result in a compacted fill section having a thickness of three feet. Areas of the exposition center building that will be increased in elevation three or more feet above existing elevations should have the grass stripped off, the exposed ground surface scarified and compacted, and import fill soil placed and compacted to finish grade elevations.

Within the pavement areas it is recommended that the upper one foot of subgrade soil be compacted per the recommendations of our report. Therefore, pavement areas which will receive one or more feet of import fill soil should have the grass stripped off and soil surface compacted as discussed above. As presented in our report, automobile parking pavement sections should consist of 2 inches of asphalt concrete underlain by 7 inches of base while driveway areas should consist of 3 inches of asphalt concrete underlain by 9 inches of base.

As discussed within our report of March 15, 1996 it is recommended that the exposition center building be supported on pile foundations. Discussions with representatives of KPFF Consulting Engineers indicate that additional columns will be located outside of the main building footprint and will support the entrance foyer area and meeting rooms east of the exposition center. Two foundation alternatives are available for these footings. The first alternative would consist of placing these foundations on piles per the recommendations of the previous report. The second alternative would consist of completely removing the existing fill soils within the area of the proposed footings and recompacting the soil as structural fill. This would require overexcavation of approximately 8 feet below the existing ground surface. Placement and compaction of fill soils should be in accordance with the recommendations contained in our March 15, 1996 report. Spread footings could then be used to support these columns. Spread footings should have a minimum width of 18 inches and be founded at least 18 inches below lowest adjacent soil subgrade. Footings with the above minimum dimensions may be designed for an allowable soil bearing pressure of 2,000 psf. A one-third increase may be assumed for transient wind or seismic loads. Lateral pressures may be resisted by a passive earth pressure equivalent fluid weight of 300 pcf and a friction factor of 0.3.

Continuous footings may be used to support the loading dock walls at the west end of the exposition center. Continuous footings should have a minimum width of 18 inches and be founded at least 18 inches below lowest adjacent subgrade. An allowable bearing pressure of 1500 psf is recommended for the continuous footings.

Pile foundation recommendations were presented in our report of March 15, 1996. At that time maximum pile lengths were estimated to be on the order of 100 feet and allowable pile loads were developed for these pile lengths. Discussions with the project contractor and structural engineer indicate that it will be more economical to utilize longer piles with higher pile capacities. Dense gravels were encountered during our geotechnical investigation at a depth of approximately 115 feet below the existing ground surface. Pipe piles having a minimum diameter of 12 inches may be designed for an allowable axial load of 120 tons if driven into the dense gravels at a depth of approximately 115 feet. The actual depth will depend on pile driving criteria developed from the pile contractors equipment.

It is recommended that the travel pathway that will be used by the crane to set the trusses for the exposition building receive at least 12 inches of pit run quarry rock (6- to 8-inch maximum size with no more than 5% by weight passing a No. 200 sieve). After placement, the rock should be compacted by a minimum of four complete passes with a moderately heavy steel drum roller.

If you have any questions regarding this letter or desire further information, please contact the undersigned at your convenience.

Sincerely,

AGRA Earth & Environmental, Inc.

A. Wesley Spang

A. Wesley Spang, Ph.D., P.E.
Associate

AWS/klp

c Mr. Nathan Charlton, KPFF Consulting Engineers
Mr. John Blumethal, Yost Grube Hall Architects
Mr. Dave Garske, Hoffman Construction

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March 15, 1996
21-08598-00

Mr. Mark Hunter
Construction/Capital Projects Manager
Metropolitan Exposition-Recreation Commission
777 N. E. Martin Luther King Jr. Boulevard
Portland, Oregon 97232

Dear Mr. Hunter:

RE: GEOTECHNICAL INVESTIGATION
METROPOLITAN EXPOSITION CENTER EXPANSION
PORTLAND, OREGON

In accordance with your authorization and our proposal dated February 9, 1996, AGRA Earth & Environmental, Inc. (AEE), is pleased to present this geotechnical investigation for the proposed Metropolitan Exposition Center Expansion in Portland, Oregon. As discussed within this report, it is recommended that the building columns be supported on deep foundations. We appreciate the opportunity to assist you and look forward to continued involvement on this and other projects.

If you have any questions regarding this report or desire further information, please contact the undersigned at (503) 639-3400 at your convenience.

Sincerely,

AGRA Earth & Environmental, Inc.

A. Wesley Spang

A. Wesley Spang, Ph.D., P.E.
Associate

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c Mr. Art Johnson, KPFF Consulting Engineers

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SUMMARY

It is the opinion of AGRA Earth & Environmental, Inc. (AEE), based on the results of our investigation, that the site is geotechnically suitable for the proposed construction. Key geotechnical design items are summarized below, and are discussed in greater detail in the following sections of this report.

SOILS: The project site is covered by a fill of silty gravel, building rubble, concrete etc. to a depth of approximately 7.5 to 8 feet. These materials are underlain by dredge fill materials composed of silty sand to sandy silts. Soft to medium stiff sandy silts, silty sands, and silty clays underlie the dredge soils to a depth of approximately 113 to 116 feet where gravels were encountered. Groundwater was interpreted at a depth of 15 to 17 feet based on the CPT piezocone soundings data.

Stripping Depths: The majority of the project site is covered with previously placed fill soils to a depth of 7.5 to 8 feet. It is recommended that the top 3 feet of existing fill materials be overexcavated and recompacted as structural fill to provide uniform soil support for the concrete floor slab.

Fills: Existing soils can be used for fill material provided they are free of organics, debris, or other deleterious material and can be adequately dried and compacted. Significant fills are not anticipated due to the relatively flat topography of the site.

Foundations: Building column loads will need to be supported on deep foundations to provide satisfactory foundation support.

Seismic Design: A site-specific seismic study was performed for the project site. It is recommended that the seismic design of the building incorporate a site soil coefficient of S3.

The preceding summary is intended for introductory and reference use only. This report should be read in its entirety in order to understand the details and basis of the recommendations.

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation prepared by AGRA Earth & Environmental, Inc. (AEE), for the proposed expansion of the Metropolitan Exposition Center in Portland, Oregon. The project location is shown on the Site Location Map, Figure 1. The purpose of our study was to characterize subsurface soil conditions within the proposed building location and provide geotechnical engineering design criteria for the project. The scope of our field work consisted of advancing two exploratory borings and four cone penetration soundings (CPT) at the approximate locations shown in the Site And Exploration Plan, Figure 2. Additionally, six pre-bored pressuremeter tests (PMT) were performed in one of the borings to assist in foundation design. Logs of the exploratory borings, CPT Soundings and PMT test results are presented in Appendix A. Laboratory test results of selected soil samples are presented in Appendix B.

This study has been accomplished in accordance with generally accepted geotechnical engineering practices for the exclusive use of the Metropolitan Exposition-Recreation Commission and their agents for specific application to the above described project.

2.0 SITE AND PROJECT DESCRIPTION

The site is located within the Multnomah County Fairgrounds and Exposition Center in Portland, Oregon. The Exposition Center property is fronted by North Marine Drive and North Broadcare Street to the north and south, respectively, and by North Force Avenue and I-5 to the west and east, respectively. It is understood that the proposed Exposition Center expansion will consist of constructing a steel-framed, clear span Exhibition Building approximately 340 feet by 360 feet in dimension located adjacent to and south of the existing South Hall. Discussions with the structural engineer (KPFF Consulting Engineers) indicate that the maximum column loads are on the order of 600 kips. The building will be a "Special Occupancy Structure" per ORS 455.447. Minimal fills are anticipated to raise the site to that of the existing South Hall floor elevation. Construction of parking areas, utilities, landscaping areas etc. are also anticipated for project development.

3.0 GEOLOGIC SETTING

The Multnomah County Exposition Center is located in North Portland between the North Portland Harbor Oregon Slough to the north and the Columbia Slough to the south. The subsurface materials within the general area consist of fine- to coarse-grained alluvium deposited by the Columbia River. The upper 10 to 20 feet of material is composed of dredged soils. Fill soils associated with construction activity are locally present.

4.0 SUBSURFACE CONDITIONS

Subsurface explorations were performed under the observation of a representative from AEE. The following descriptions of the site soils are based on the results of the borings, CPT soundings and PMT test results, and a review of available geotechnical/geologic literature.

Subsurface conditions and water levels at other locations may differ from the conditions at the locations where testing was conducted. The passage of time may also result in changes in the conditions interpreted to exist at the locations where testing was conducted. Based on the results of the subsurface exploration the following soil conditions are interpreted within the area of the proposed exhibition hall:

FILL: The majority of the project site is covered with previously placed fill soil consisting of silty gravel, building rubble, concrete etc. to a depth of approximately 7.5 to 8.0 feet. The central portion of the site is covered with saturated soil to a depth of 6 inches to 1 foot. This area is also underlain by the same fill described above. Concrete rubble was encountered at the base of the fill. The CPT soundings had to be predrilled through these fill materials. The presence of concrete, debris, and gravel in the fill may result in difficult pile installation operations near the ground surface.

DREDGE FILL: Dredge fill soils were encountered within the majority of the site below the fill layer. These soils consist primarily of silty sands to sandy silts and extend to depths of approximately 10 to 15 feet. Cone tip resistances and SPT blowcounts indicate the dredge fill soils are moderately dense.

SANDY SILTS AND SILTY SANDS: The dredge fill soils are underlain by soft to medium stiff sandy silts and silty sands. Based on interpretation of the CPT soundings and borings logs, these materials extend to depths of approximately 55 to 60 feet and also from approximately 80 to 114 feet. Analysis indicates that these soils are slightly preconsolidated (overconsolidation ratio between 1 to 2).

CLAYEY SILT TO SILTY CLAY: Medium stiff to stiff, saturated, blue to gray clayey silts to silty clay is present at a depth of approximately 60 to 80 feet below the ground surface.

DENSE GRAVEL: Dense to very dense, poorly-graded gravel was encountered at a depth of approximately 114 feet. A review of the "Geologic Map of the Portland Quadrangle" prepared by the Oregon Department of Geology and Mineral Industries (DOGAMI) indicates that the area is underlain at depth by Columbia River basalt.

5.0 GROUNDWATER

Interpretation of CPT piezocone soundings indicates a groundwater depth of 15 to 17 feet below the ground surface. However, it should be anticipated that perched groundwater may occur during wet periods in variable and unpredictable locations. Typical locations would be within interbedded silt layers, near landscaping areas, within existing utility trenches and in other similar locations where there are variations in soil permeability.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 SITE PREPARATION

Prior to beginning construction, all areas of the site that will receive buildings, pavement or fill should be overexcavated to a minimum depth of 3 feet and recompacted as structural fill. Soft soils encountered in the central portion of the site should be excavated and removed from the site. Isolated areas requiring deeper overexcavation and recompaction may be encountered. One such area was encountered near the location of CPT P-1 during our exploration program. The overexcavated material may be reused for structural fill if it meets the requirements of Section 6.2.

We have provided recommendations for both wet weather and dry weather construction. If possible, it is recommended that the site be prepared during relatively dry weather.

6.1.1 Dry Weather Construction

It is recommended that all building, pavement and fill areas be overexcavated to a depth of at least 3 feet and recompacted as structural fill. The structural fill should meet the specifications provided in Section 6.2. Excavated material that consists of concrete, debris, and other deleterious material should be exported off site. Import fill soils should consist of granular fill as described in Section 6.2. Even during dry weather it is possible that some areas of the exposed subgrade will become soft or will pump or weave. Any areas that cannot be effectively dried and/or compacted should be prepared in accordance with Section 6.1.2, Wet Weather Construction.

6.1.2 Wet Weather Construction

During wet weather, or when adequate moisture control and/or proper compaction cannot be obtained, it may be necessary to install a granular working blanket to support construction equipment and provide a firm base on which to place subsequent fills, floor slabs and pavements. Commonly the working blanket consists of pit run quarry rock (six inch to eight inch maximum size with no more than 5% by weight passing a No. 200 sieve). We recommend that we be consulted to approve the material before installation.

The working blanket should be installed on the excavated subgrade in a single lift with trucks end-dumping off an advancing pad of granular fill. After installation, the working blanket should be compacted by a minimum of four complete passes with a moderately heavy steel drum or grid roller.

The working blanket must provide a firm base for subsequent fill installation and compaction. It has been our experience that 12 to 18 inches of working pad is normally required, depending on the gradation and angularity of the working pad material. This assumes that the material is placed on a relatively undisturbed subgrade in accordance with the preceding recommendations, and that it is not subjected to frequent heavy construction traffic. A very conscientious contractor may be able to prepare the site without resulting in major subgrade disturbance. We recommend a minimum 12-inch-thick granular working pad be installed in all areas where scarification and compaction of the subgrade is not able to be accomplished.

Areas used as haul routes for heavy construction equipment may require a working pad thickness of two feet or more during extremely wet weather. If particularly soft areas are encountered, approved non-woven fabric installed on the subgrade is recommended before the placement of rock. If desired, we can provide you with sample specifications for the non-woven fabric. It should be anticipated that excavations may encounter locally perched groundwater during the winter months. This water may occur at variable and unpredictable locations.

6.1.3 Proof-rolling

Regardless of when the subgrade is prepared (i.e. wet weather or dry weather), we recommend that prior to fill placement or base course installation, the subgrade or granular working blanket be proof-rolled with a fully-loaded 10 to 12 yard dump truck. This pertains to all pavement and floor slab areas. Any areas that pump, weave or appear soft and muddy should be overexcavated and backfilled with compacted granular fill. If a significant length of time passes between completion of fill placement and commencement of construction operations, or if significant traffic has been routed across the site, we recommend that the site be similarly proof-rolled again before final placement of asphalt or concrete.

6.2 FILLS

Any fills on this project should be installed on a subgrade that has been prepared in accordance with the recommendations in Section 6.1 of this report. Fills should be installed in horizontal lifts not exceeding about eight inches in thickness, and should be compacted to at least 92% relative compaction for silty soils and 95% for sands and gravels per ASTM D-1557. This criteria may be reduced to 85% in non-structural landscaping or planter areas.

During dry weather, structural fills may consist of virtually any relatively well-graded soil that is free of debris, rubble, and organic matter and that can be compacted to the preceding specifications. It is anticipated that imported fill materials will be required due to the relatively level topography of the site. We should be contacted to approve any proposed fill materials before installation.

In order to achieve adequate compaction during wet weather, we recommend that fills consist of well-graded granular soils (sand or sand and gravel) that do not contain more than 5%

material by weight passing the No. 200 sieve. In addition, it is usually desirable to limit this material to a maximum six inches in diameter for future ease in the installation of utilities.

6.3 FLOOR SLABS

Subgrades for floor slabs on grade should be prepared in accordance with Section 6.1, Site Preparation. It is recommended that all floor slab areas be proof-rolled with a fully loaded dump truck. Any areas that pump, weave, or appear soft or muddy should be over-excavated and stabilized with compacted granular fill. Floor slab design thickness will be controlled by the subgrade soils in the building area. A layer of compacted crushed rock at least 6 inches thick should be installed over the prepared subgrade to provide a capillary barrier and to provide a strong base to minimize subgrade disturbance during construction. This crushed rock material should be well-graded, angular, and contain no more than 5% passing a No. 200 sieve.

It is anticipated that the majority of the floor slab will not receive moisture-sensitive floor coverings. However, it is recommended that an impermeable membrane (vapor barrier) be installed between the crushed rock and the floor slab in areas where floor moisture could be of concern. The purpose of the vapor barrier is to reduce the migration of water vapor through the floor slab. Many glued down floorings, carpets, hardwood, and finishes are damaged by floor moisture. Vapor barriers are commercially available from a number of suppliers. To maximize it's effectiveness, the membrane should be installed according to manufacture's recommendations. A thin layer of sand should be placed both below the barrier to protect it from punctures during construction, and above the barrier to reduce the potential for slab curling, cracking, and to improve curing.

6.4 RETAINING WALLS

At this time, it is our understanding that no retaining walls or basements are proposed. Should this change, we should be contacted for additional recommendations.

6.5 FOUNDATIONS

6.5.1 General

As previously discussed, the site is underlain by compressible soils. Site improvement techniques such as surcharging with fill soils, etc., are not anticipated to be feasible due to the costs and time required. Therefore, it is recommended that the exposition center expansion building be supported on a deep foundation system. It is anticipated that steel pipe piles and concrete driven grout piles will be the most economical foundation system for this project. AEE should be contacted if other pile types are to be considered.

6.5.2 Axial Pile Capacities

Allowable axial pile capacities as a function of depth of embedment below the ground surface are presented in Figure 3. Pile capacities have been calculated for 14-inch driven grout piles and 12-inch steel pipe piles. Pile capacities are based on analyses utilizing the CPT sounding data and PMT test results. Pile uplift capacity may be taken as 75 percent of the allowable downward axial load.

It is recommended that piles be designed with a center-to-center spacing of at least three pile diameters. Driven grout piles should not be installed for a minimum of 24 hours after the construction of an adjacent pile. Steel pipe piles should be driven closed-end to assist in densification of the soils during driving. Steel pipe piles should be filled with concrete after installation.

As previously discussed the upper 7 to 8 feet of material contains building debris, rubble, concrete, etc. Difficult pile installation operations should be anticipated within this material. Predrilling or other means of excavating the material at the pile locations may be needed to avoid damaging the piles during penetration of the upper fill materials.

6.5.3 Lateral Loads

Lateral loads on pile caps can be resisted by a combination of the lateral load capacity of the piles and passive earth pressures against pile caps. The lateral load resistance of the piles was determined based on the assumption that the piles will be structurally connected to the pile caps such that the pile head will be fixed against rotation. Additional assumptions include a minimum compressive strength of concrete of 4,000 psi and a steel pipe wall thickness of 3/8 inch. Lateral loads were calculated using the PMT results as suggested by Briaud (1989) and the computer program LPILE. Lateral load and moment diagrams are presented in Figures 4 through 6. In addition to the lateral load resistance of the piles, a passive earth pressure may be assumed to act against the pile caps and grade beams equivalent to a 250 pcf fluid pressure.

6.6 SEISMIC HAZARD STUDY

A site-specific seismic hazard study was performed for the property in accordance with Section 2905 of the Oregon Structural Specialty Code (OSSC). This amendment requires that sites which will contain certain structures (as defined by ORS 455.447) be investigated for susceptibility to seismic-induced geologic hazards. This section presents the results of our study and includes an evaluation of site ground shaking characteristics as well as other seismic-related hazards.

Seismicity and Earthquake Sources

The seismicity of the Portland Metropolitan area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The Cascadia Subduction Zone is located offshore and extends from Northern California to British Columbia. Within this zone the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers. The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon coast (Peterson et. al., 1993). Sequences of interlayered peats and sands have been interpreted to be the result of large subduction zone earthquakes occurring at intervals on the order of 300 to 500 years with the most recent event taking place approximately 300 years ago. A recent study by Geomatrix (1995) suggests that the maximum earthquake associated with the CSZ is moment magnitude[1] (M_W) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes which have occurred within subduction zones in other parts of the world. A moment magnitude 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones which have exhibited much higher levels of historical seismicity than the CSZ and is considered unlikely. For the purpose of this study an earthquake of moment magnitude 8.5 was assumed to occur within the Cascadia Subduction Zone.

The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below Western Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of subduction between Oregon and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone include the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

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[1] Moment magnitude is used by seismologists to measure larger earthquakes and is based on fault displacement and area of fault rupture. For smaller earthquakes the moment magnitude is approximately equal to the familiar Richter (or commonly called local) magnitude (M_L).

The third source of seismicity that can result in ground shaking within the greater Portland area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in western Oregon is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klammath Falls (magnitude 6.0) earthquakes were crustal earthquakes. Individual faults or fault zones which have been mapped by the Oregon Department of Geology and Mineral Industries (1991) and Geomatrix (1995) within the near-vicinity of the site include the following:

Fault System	Approximate Closest Distance To Site (miles)
Portland Hills Fault Zone	4
Bolton Fault	20
Grant Butte, Damascus-Tickle Creek Fault Zone	20
Helvetia Fault	11
Lacamas Creek Fault	12.5
Sandy River Fault	18
Mount Angel Fault	35
Newberg Fault	26
Gales Creek Fault	30

Seismic and geologic parameters such as slip rate, horizontal and vertical offset, rupture length, and geologic age have not been determined for the majority of the above faults. This is primarily due to the lack of surface expressions or exposures of faulting because of urban development and the presence of late Quaternary soil deposits which overlie the faults. The low level of historical seismicity (particularly for earthquakes greater than magnitude 5) and lack of paleo-seismic data results in large uncertainties when evaluating individual crustal fault maximum magnitude earthquakes and recurrence intervals. Thus it is considered prudent to also evaluate the potential for seismic shaking due to crustal earthquakes on a regional scale. Based on data presented by Geomatrix (1995) and DOGAMI (1991) the seismic exposure at the site from crustal zone sources is represented by an earthquake of magnitude 6.5.

Bedrock Acceleration

The acceleration within the underlying bedrock (at this site considered to be the Troutdale Formation) expected due to an earthquake occurring within the above-described seismic sources was calculated based on the maximum earthquake magnitude and empirical distance-attenuation relationships developed by Geomatrix (1995) and Joyner and Boore (1982). The following table presents the estimated peak bedrock accelerations at the site for the different earthquake sources:

Table I
Potential Earthquake Source Parameters

Earthquake Source	Maximum Moment Magnitude	Depth (km)	Epicentral Distance (km)	Peak Horizontal Acceleration (g)
Crustal Zone	6.5	10	10	0.26
Intraplate Zone	7.25	30	35	0.20
CSZ	8.5	15	120	0.10

The peak horizontal bedrock acceleration for the crustal zone earthquake (0.26g) was increased to 0.30g for conformance with the minimum level of bedrock acceleration specified per OSCC amended Section 2905.

Ground Acceleration

The potential ground accelerations at the site were evaluated by applying an acceleration time history record to the base of the site soil profile and determining the response at the ground surface. No recorded acceleration records are available from an earthquake within the Cascadia Subduction Zone. An acceleration record obtained from the 1985 Mexico City earthquake (a subduction zone event) was used to represent potential CSZ bedrock motions. Additionally, a synthetic time history record developed by Seed and Idriss (1969) to model a magnitude 8+ earthquake was used to assist in evaluating the response of the site to a CSZ magnitude 8.5 earthquake. Recorded time histories are available from the 1949 Olympia and 1965 Puget Sound earthquakes and were used to analyze the response of the site to an interplate zone earthquake. Ground shaking characteristics due to crustal zone sources were evaluated using the 1993 Scotts Mills acceleration time history as well as two near-surface California earthquake records. Parameters of the selected earthquake time histories are presented below:

Table II
Selected Earthquake Acceleration Time Histories

Earthquake Record	Magnitude	Epicenter Distance (km)	Peak Horizontal Acceleration (g)
Mexico City (1985)	8.1	131	0.17
Synthetic	8+	N.A.	0.32
Olympia (1949)	7.1	26	0.28
Puget Sound (1949)	6.5	62	0.20

Earthquake Record	Magnitude	Epicenter Distance (km)	Peak Horizontal Acceleration (g)
Scotts Mills (1993)	5.6	54	0.03
Castaic (1971)	6.5	30	0.32
Lake Hughes (1971)	6.5	32	0.15

The peak horizontal accelerations of the above time histories were scaled to match the peak horizontal accelerations shown in Table I.

Horizontal ground accelerations were calculated for the site by propagating the individual earthquake acceleration time history records up through the soil profile using the computer program SHAKE91 (Idriss, 1991). The following soil profile was developed from the exploratory borings, CPT shear wave velocity tests obtained from the CPT soundings, and geologic information presented by DOGAMI (1991) for the general vicinity of the site:

Depth (ft)	Geologic Unit	Shear Wave Velocity (ft/sec)
0 - 8	Fill	500
8 - 15	Dredged Sands & Silts	550
15 - 60	Silty Sand to sandy Silt	500
60 - 80	Clayey Silts to Silty Clays	500
80 - 114	Silty Sands to Sandy Silts	600
114-120	Gravel	800

Results of the analyses indicate that peak ground horizontal accelerations are approximately 0.23g, 0.17g, and 0.12g for the crustal zone, intraplate zone and CSZ earthquakes, respectively. These values of ground acceleration are in general agreement with a study by Wong and Madin (1993) for a site near the Portland Airport.

It is recommended that the seismic design of the building utilize the UBC Zone 3 response spectrum with a site soil coefficient (Table No. 23-J) of S_3.

6.7 FAULT DISPLACEMENT AND SUBSIDENCE

There are no mapped faults within the boundaries of the site or within adjacent properties. No evidence was encountered during the field investigation to suggest the presence of faults within the property. It is our opinion that the potential for fault displacement and associated ground rupture is extremely remote.

6.8 LANDSLIDES

There are no landslide or slope stability hazards at the site due to the relatively flat topography of the site and surrounding areas.

6.9 LIQUEFACTION

The potential for liquefaction within the site was evaluated using the SPT blowcounts from Boring B-1, CPT sounding data, and grain-size characteristics of the soils. Observed field performance of areas that have experienced earthquake shaking has resulted in empirical methods of analysis that provide a good assessment of a site's liquefaction potential. The analyses utilized herein are based on the work of Seed and DeAlba (1986) and Stark and Olsen (1995).

The seismic loading was assumed to consist of a Magnitude 6.5 earthquake with a peak horizontal ground acceleration of 0.25g. Liquefaction resistance was based on SPT blowcounts, CPT tip resistances, and percentage of soil passing the No. 200 sieve (% fines). The results of the analyses indicate that a portion of the underlying soils could experience liquefaction during a major seismic event. Manifestation of liquefaction can include a reduction in shear strength, settlement, and lateral spreading. Each of these consequences are discussed below.

Founding the building on driven piles will assist in mitigating the effects of liquefaction on the building. A reduction in soil shear strength due to liquefaction could result in settlement of the piles (approximately 1 to 2 inches) but would not compromise the structural integrity of the piles. Potential ground settlement within the site due to liquefaction was analyzed based on the method presented by Tokimatsu and Seed (1987). Analyses utilizing their methodology results in calculated settlements ranging from 4 to 6 inches. However, this procedure does not account for the beneficial effects of an overlying thickness of unliquefiable soil at the ground surface. Ishihara (1985) and Youd and Garris (1995) have shown that the presence of a 10- to 15-foot-thick layer of unliquefiable soil significantly reduces the potential for ground surface disturbance or settlement due to liquefaction of underlying layers. Thus it is our opinion that settlements at the ground surface would likely not exceed 1 to 2 inches. The potential for lateral spreading during an earthquake was evaluated based on the procedures presented by Bartlett and Youd (1995). This procedure incorporates earthquake and soil characteristics as well as site topography to estimate lateral movements due to earthquake shaking. The results of our analyses indicate that lateral spreading on the order of 1 to 3 inches could occur.

It is our opinion that the occurrence of liquefaction within the site would not compromise the life safety performance of the structure. The use of pile foundations and the presence of an overlying unliquefiable layer would mitigate the majority of the potential for significant building damage. Some building and/or floor slab remediation would likely be required after a major crustal earthquake.

It should be noted that the above analyses are deterministic based and do not include any information regarding the frequency or return interval of a seismic event which would cause liquefaction. A recent study by Geomatrix (1995) estimated return periods of 500, 1000, and 2500 years for peak bedrock accelerations in the Portland area of 0.19g, 0.27g, and 0.37g, respectively. The majority of these bedrock accelerations were attributed to crustal earthquake sources. A return period of 500 years roughly corresponds to the UBC return interval (10 percent probability of exceedence in 50 years). It can be seen that a bedrock acceleration of 0.30g (used in this report) has a return interval (per the Geomatrix study) of approximately 1000 years.

6.10 TSUNAMI AND SEICHE INUNDATION

There is no potential for tsunami- and seiche-related damage at the site due to the site's elevation and distance from coastal areas, waterways and lakes.

6.11 PAVEMENT DESIGN

Pavement designs for asphalt and portland cement concrete have been prepared in accordance with widely accepted AASHTO design methods. A range of pavement designs for various traffic conditions are presented in the table below, Pavement Designs. These designs assume that the subgrade will be prepared in accordance with Sections 5.1 and 5.2 except that the top eight inches should be compacted to 95% relative to ASTM D-1557. Specifications for pavements and base course should conform to current Oregon State Highway Department specifications, with the addition that the base rock should contain no more than 5% passing a No. 200 sieve, and that asphalt concrete be compacted to a minimum of 91% ASTM D-2041.

ASPHALT CONCRETE PAVEMENT

Approx. No. Trucks/day (each way)	Approx. No. 18 k design axles (x1000)	Asphalt Concrete Thickness (in.)	Crushed Rock Base Thickness (in.)
Auto Parking	10	2.0	7
5	22	2.5	8
10	44	2.5	9
15	66	2.5	10
25	110	3.0	10
50	220	3.5	11
100	440	4.0	12

PORTLAND CEMENT CONCRETE PAVEMENT4,5

Approx. No. Trucks/day (each way)	Approx. No. 18 k design axles (x1000)	P.C.C. Thickness (in.)	Crushed Rock Base Thickness (in.)
5	22	5.0	6
10	44	5.5	6
15	66	6.0	6
25	110	6.5	6
50	220	6.0	6
100	440	7.0	6

Notes:

1) All pavement sections were designed using AASHTO design methods.

2) All pavement sections assume an AASHTO reliability level « of 90%, with a terminal serviceability of 2.0 for asphalt concrete, and 2.5 for portland cement concrete.

3) The 18 kip design axle loads are estimated from the number of trucks per day using State of Washington typical axle distributions for truck traffic and AASHTO load equivalency factors, and assuming a 20 year life.

4) Concrete design is based on a modulus of rupture equal to 550 psi, and a compressive strength of 4000 psi.

5) Concrete sections assume plain jointed or jointed reinforced sections with no load transfer devices at the shoulder.

If possible, construction traffic should be limited to unpaved and untreated roadways, or specially constructed haul roads. If this is not possible, the pavement design selected from the above table should include an allowance for construction traffic.

7.0 CONSTRUCTION OBSERVATIONS

The analysis, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and on the assumption that the results obtained from the subsurface exploration are representative of the subsurface conditions throughout the site. It is the nature of geotechnical work for soil conditions to vary from the conditions identified during the geotechnical investigation, even when a normally acceptable program of exploration has been implemented.

While some variations may appear slight, their impact on the performance of structures and other improvements can be significant. It is therefore recommended that AEE be retained to observe the portions of this project relating to geotechnical engineering, particularly the construction of the building foundations and soil compaction. This will allow AEE to correlate observations and findings with actual soil conditions encountered during construction and to evaluate construction conformance with respect to the recommendations in this report.

Unanticipated soil conditions frequently require additional expenditures to attain a properly constructed project. It is therefore prudent to allow for such unforeseen conditions in both the project schedule and construction budget.

8.0 LIMITATIONS

The recommendations in this report are based on information gathered in our office review and on site conditions observed at the time of the field exploration. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at, or adjacent to the site, it is recommended that AEE be requested to review this report to evaluate the conclusions and recommendations considering the lapse of time or changed conditions.

AEE requests that a copy of the plans and specifications be forwarded to AEE for review, so that AEE may evaluate any specific conceptual, architectural, or construction details which might affect the validity of AEE's recommendations, and ensure that AEE's recommendations have been appropriately interpreted.

AGRA Earth & Environmental, Inc.

Rajiv Ali
Geotechnical Engineering Staff

A. Wesley Spang

A. Wesley Spang, Ph.D., P.E.
Associate

AWS/skh

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Table B-2
Hydrometer Analysis

Boring Number	Depth (feet)	% Silt Size	% Clay Size
B-1	15 - 16.5'	89 %	11 %
B-1	35 - 36.5'	93 %	7 %
B-1	60 - 61.5'	87 %	13 %
B-1	90 - 91.5'	95 %	4 %

ATTERBERG LIMITS

Atterberg limits of selected soil samples were determined in accordance with ASTM D 4318. The Atterberg Limits of these soil samples are presented in the table below:

Table III
Atterberg Limits

Boring Number	Depth (feet)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B-1	15 - 16.5'	Non Plastic		
B-1	35 - 36.5'	Non Plastic		
B-1	50 - 51.5'	Non Plastic		
B-1	60 - 61.5'	37	29	8
B-1	90 - 91.5'	Non Plastic		

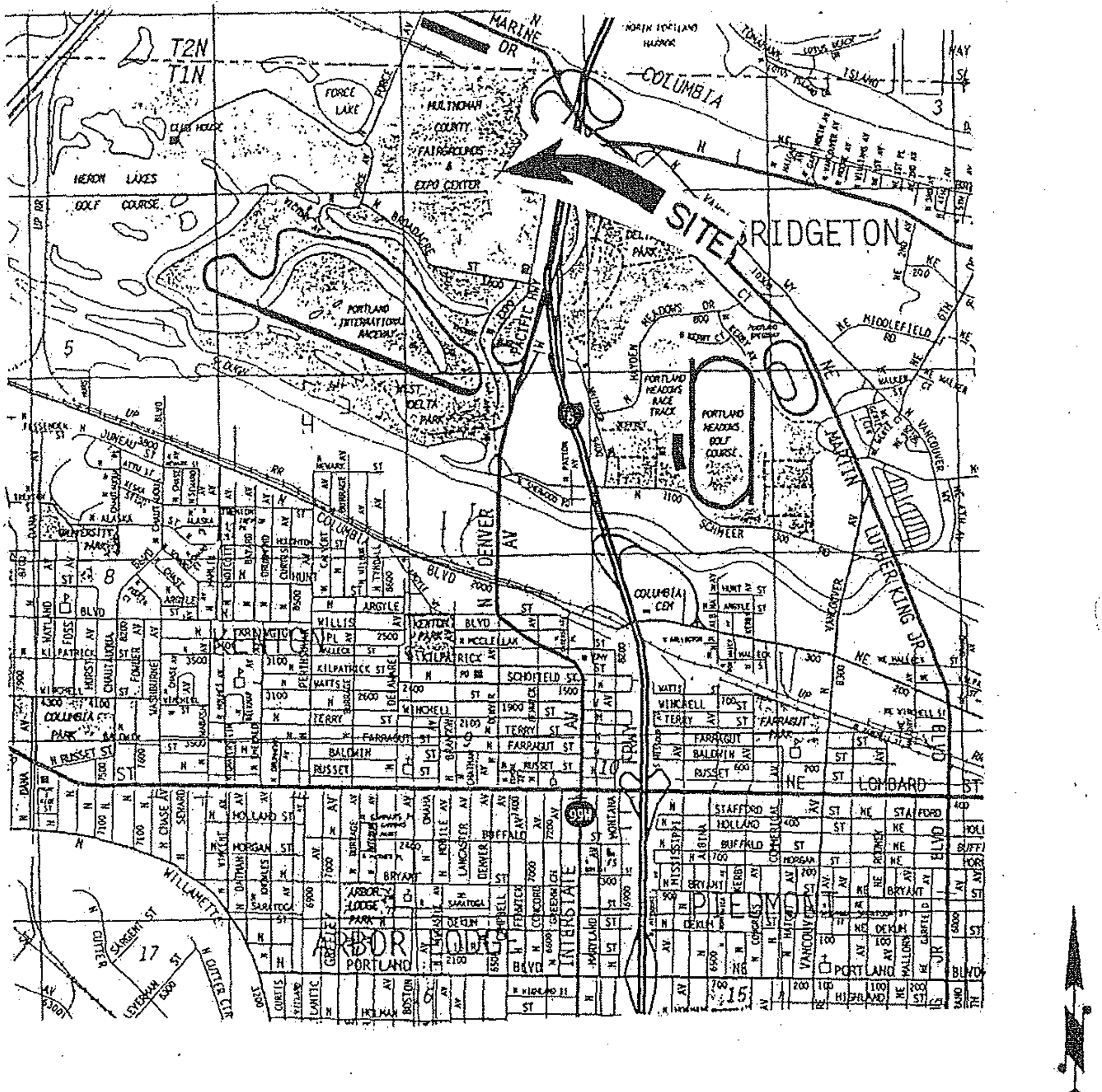


FIGURE 1

AGRA
Earth & Environmental
7477 S.W. Tech Center Drive
Portland, OR, U.S.A. 97223-8025

W.O. 6-617-08598-0
DESIGN RA
DRAWN GW
DATE FEB 1996
SCALE

METROPOLITAN EXPOSITION-RECREATION COMM.
2060 N. MARINE DR
PORTLAND, OREG

SITE LOCATION MAP

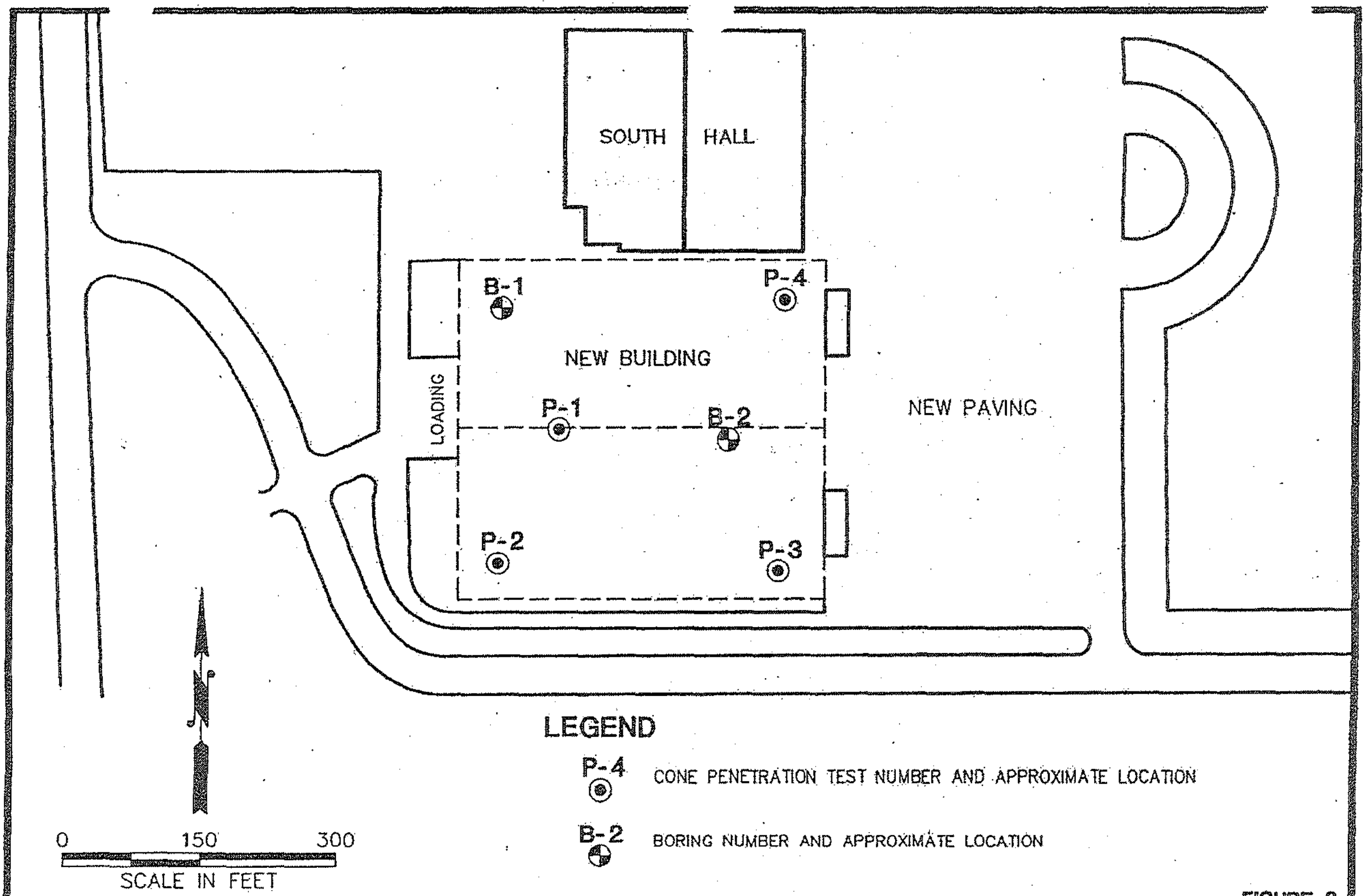


FIGURE 2

AGRA
Earth & Environmental
7477 S.W. Tech Center Drive
Portland, OR, U.S.A. 97223-8025

W.O. 6-617-08598-0
DESIGN RA
DRAWN GW
DATE FEB 1996
SCALE 1"=150'

METROPOLITAN EXPOSITION-RECREATION COMM.
2060 N. MARINE DRIVE
PORTLAND, OREGON

SITE AND EXPLORATION PLAN

AGRA EARTH & ENVIRONMENTAL, INC. DRAWING NO. \PROJECTS\21\08598\

METRO Exposition Expansion
Axial Pile Capacity

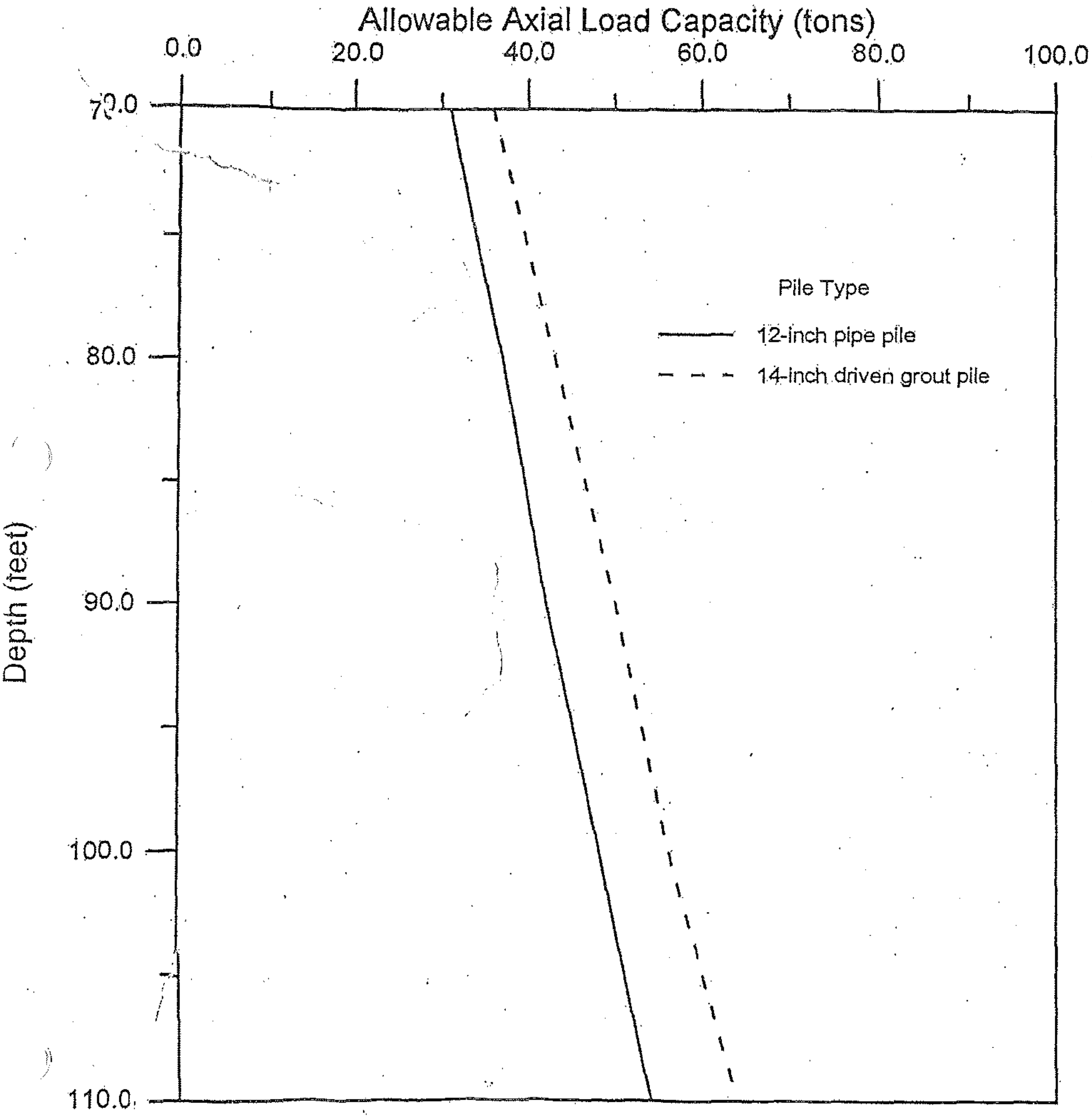


Figure 3

METRO Exposition Expansion
Lateral Load Capacity

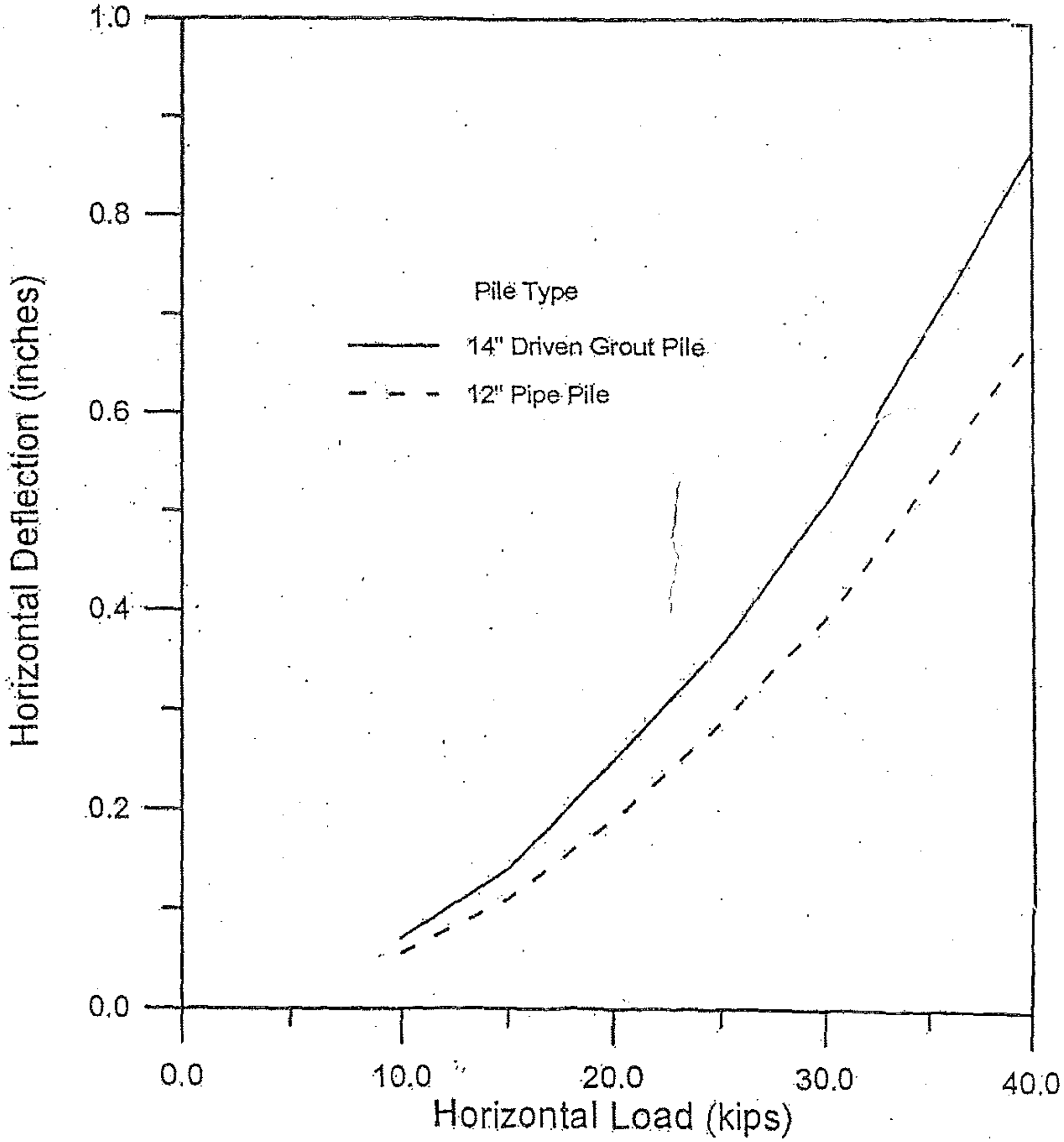


Figure 4

METRO Exposition Expansion
Pile Moment Diagram

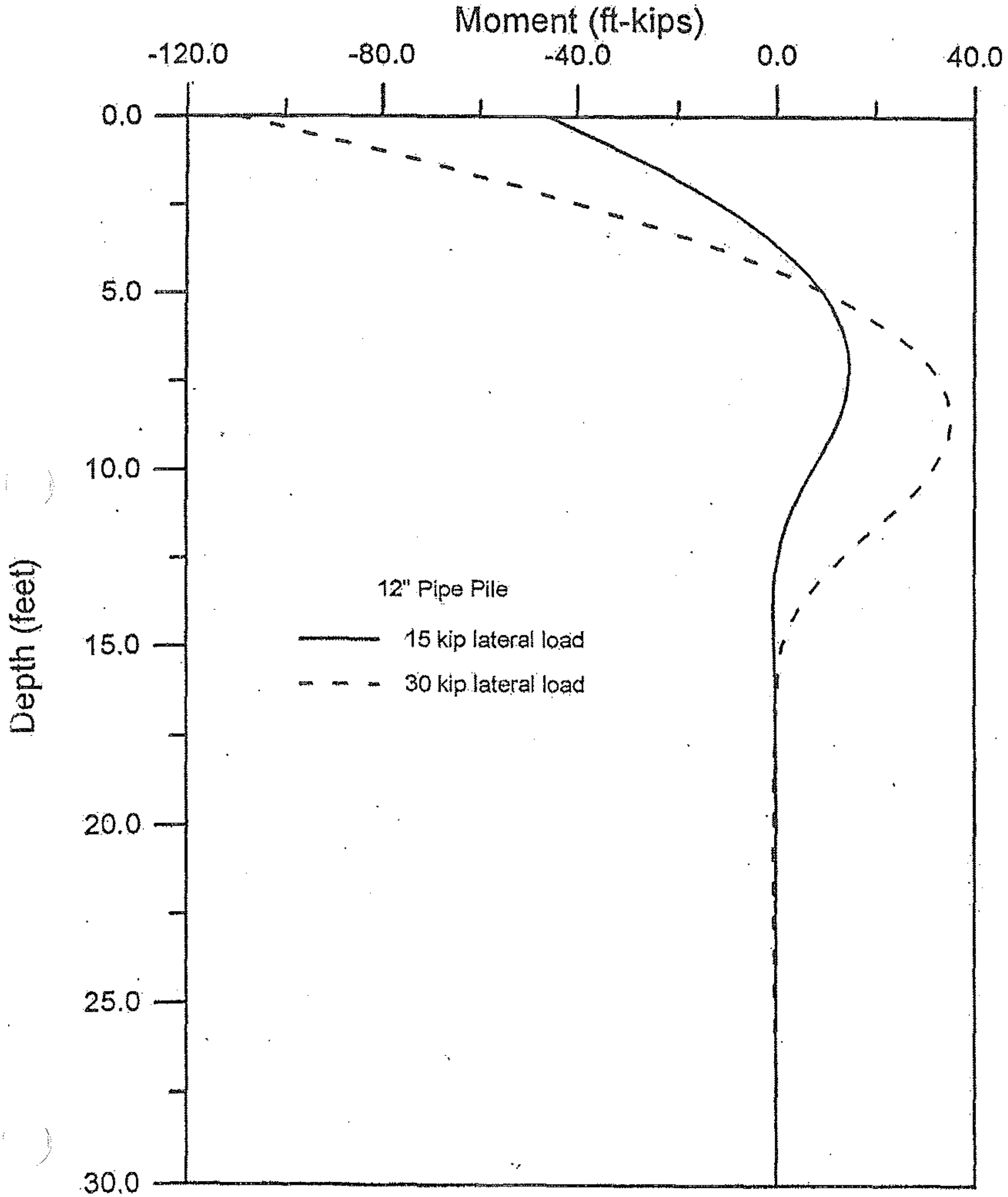


Figure 5

METRO Exposition Expansion
Pile Moment Diagram

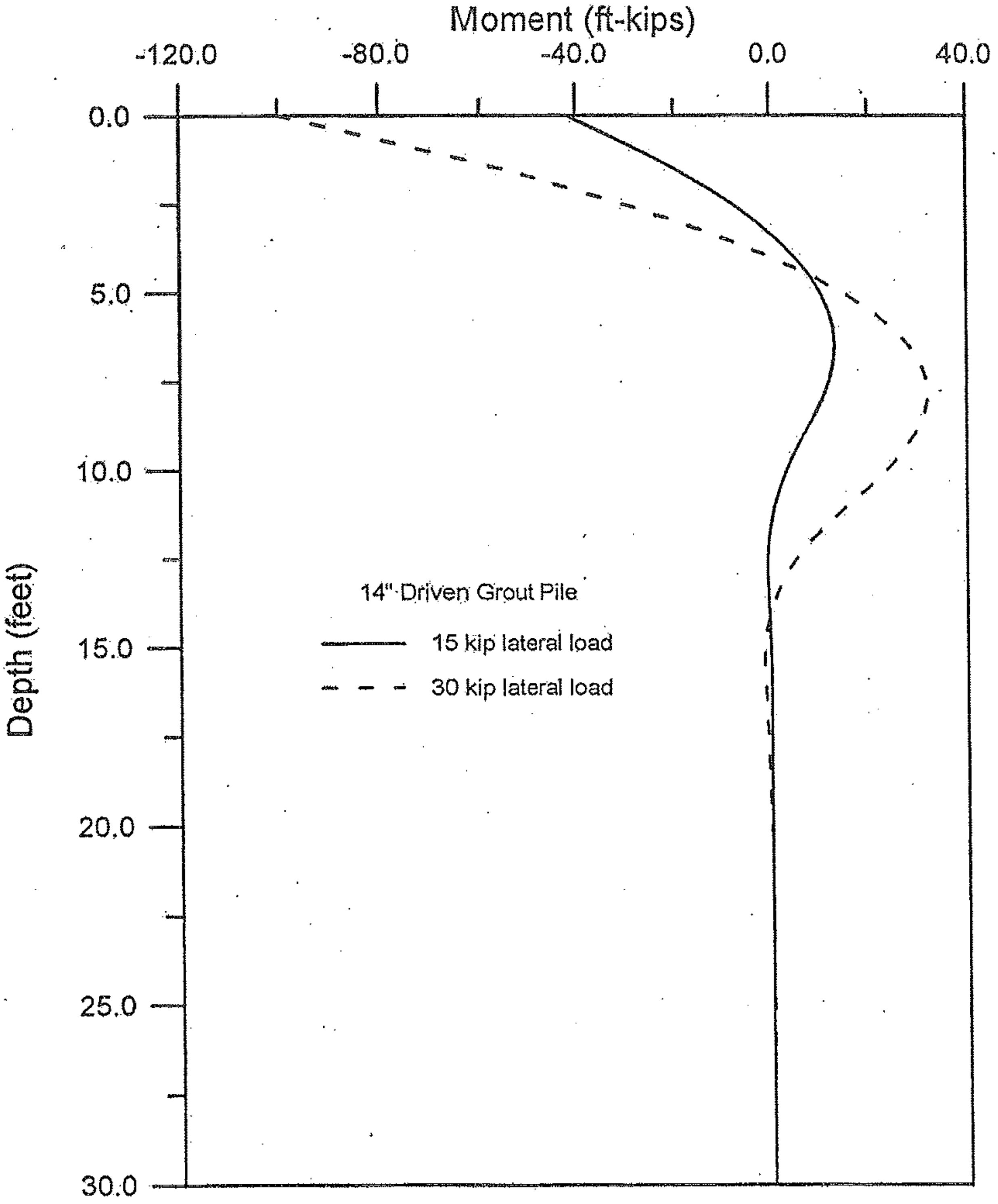


Figure 6

APPENDIX A

Field Investigation

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed from February 21 through 23, 1996 and consisted of the excavation of 2 exploratory borings and 4 CPT soundings at the approximate locations shown in Figure 2. The borings were advanced to depth of 116 feet with a CME 55 drill rig using mud-rotary drilling techniques. Soil samples were obtained at selected depths for classification and laboratory testing. Logs of the borings are presented on the following pages.

Six prebored pressuremeter tests were performed within Boring 2 by Insitu Tech (Dr. Trevor Smith at Portland state University). Pressuremeter testing consists of expanding a membrane laterally against the borehole sidewall. The resulting membrane expansion versus pressure data results in a determination of soil modulus and limit pressure. These parameters are then used for evaluating axial and lateral pile capacity and settlement. Graphs of the pressuremeter tests are presented herein.

Four cone penetration tests (CPT) were also advanced within the building footprint at the approximate locations shown in Figure 2. Resistances to cone penetration and sleeve friction were recorded at intervals of 0.3 feet. Cone tip resistances and sleeve friction recordings are presented herein. Shear wave velocities were determined at 10-foot-intervals within CPT-1 and are also presented herein.

PROJECT: Metropolitan Exposition Center **PROJECT NUMBER: 6617-08598-0** **PAGE 1**

Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET)

SOIL DESCRIPTION

0

FILL of gravel, building rubble, silts, etc. Concrete layer at bottom of this fill.

5

Medium dense, moist to wet, gray, poorly graded fine SAND.

10

Medium stiff to stiff, moist to wet, gray, SILT.

15

Medium dense, wet to saturated, gray, poorly graded, fine silty SAND.

20

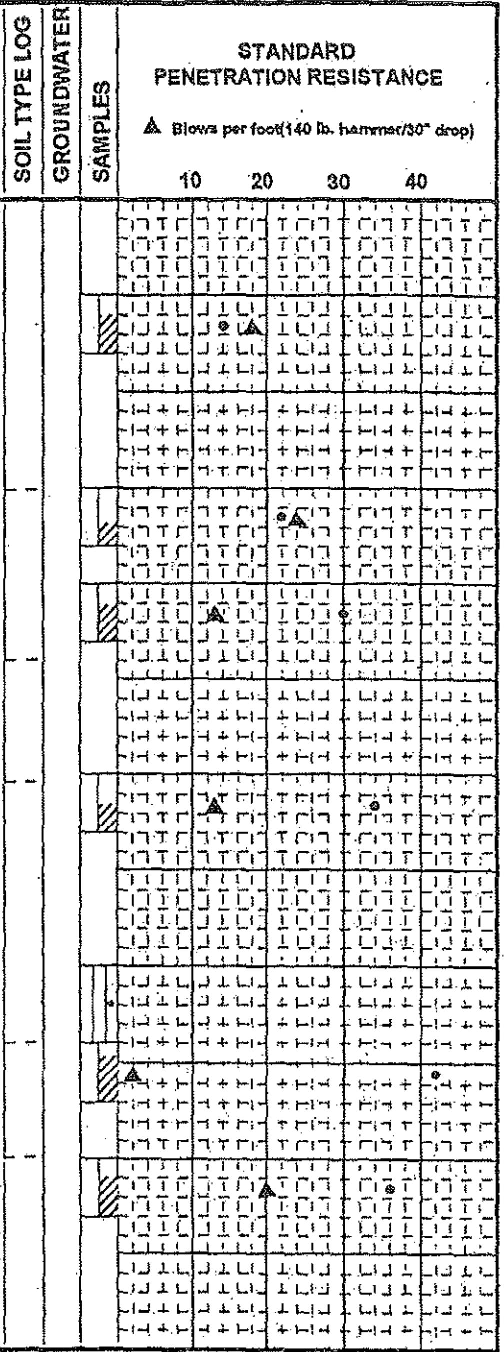
No sample recovered in Shelby Tube.

Very soft, saturated, gray to black, SILT with some sand.

25

Medium dense/stiff, saturated, gray to brown, silty SAND to sandy SILT.

30



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- ▽ Water level fluctuation
- ▽ Static water level
- ▼ WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

Metropolitan Exposition Center
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Portland, Oregon

AGRA EARTH AND ENVIRONMENTAL INCORPORATED

7477 S.W. Tech Center Drive
Portland, Oregon 97223
Phone: (503) 639-3400 Fax: (503) 620-7892

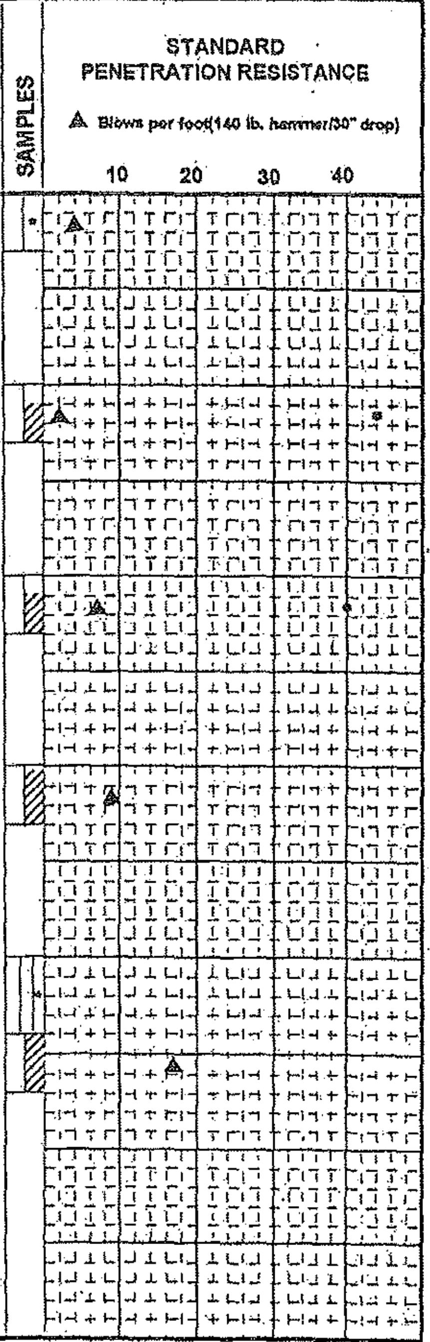
Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\8598B1.DRW

Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET)	SOIL DESCRIPTION	SOIL TYPE LOG	GROUNDWATER	SAMPLES
30	Loose/soft, saturated, gray, silty SAND to sandy SILT. No sample recovered from 30 to 31.5 feet.			
35				
40				
45	Medium stiff/loose, saturated, gray, sandy SILT to silty SAND with conspicuous organics.			
50	No sample recovered in Shelby Tube.			
55	Medium dense/stiff, saturated, gray, silty SAND to sandy SILT with conspicuous organics.			
60				



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- ▽ Water level fluctuation
- ▽ Static water level
- ▼ WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

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Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\8598B1P2.DRW

PROJECT: Metropolitan Exposition Center PROJECT NUMBER: 6617-08598-0 PAGE 3

Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET)

SOIL DESCRIPTION

60

65

Medium stiff to stiff, saturated, blue to gray, clayey SILT to silty CLAY.

70

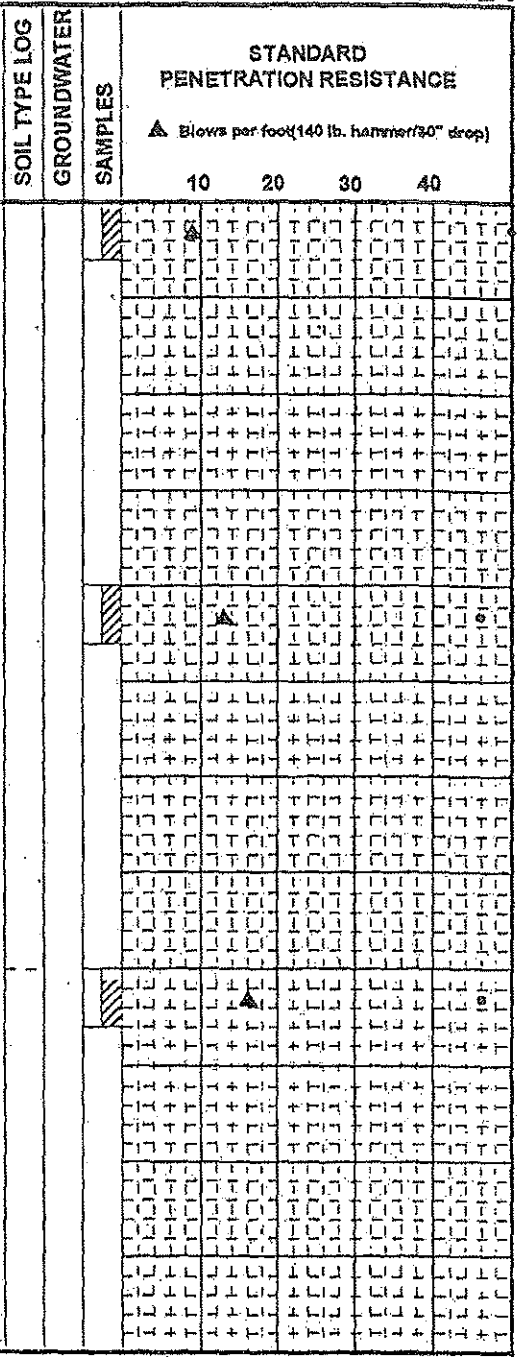
75

80

Stiff, saturated, gray, clayey SILT with some sand.

85

90



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- Water level fluctuation
- ▽ Static water level
- WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

Metropolitan Exposition Center
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Phone: (503) 639-3400 Fax: (503) 620-7892

Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\859881P3.DRW

PROJECT: Metropolitan Exposition Center PROJECT NUMBER: 6617-08598-0 PAGE 4

Boring Number: B-1

Boring Method: Mud Rotary

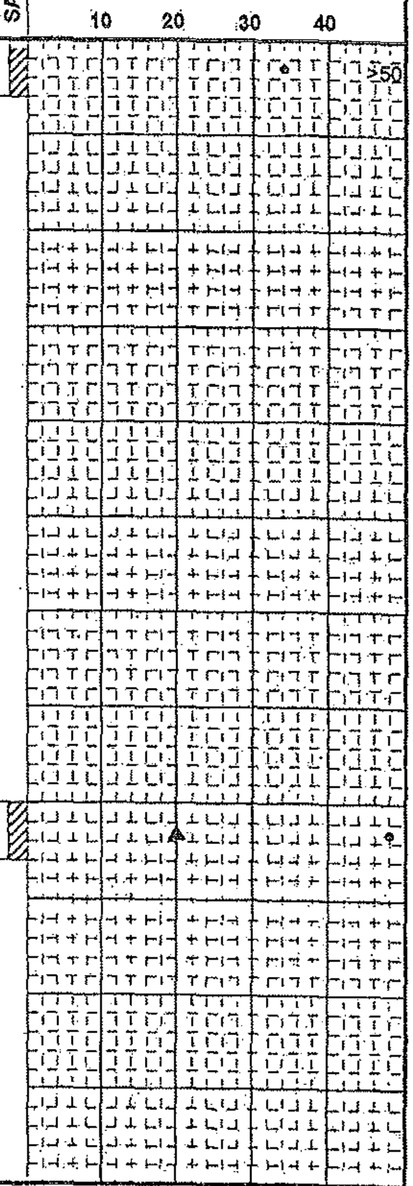
Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET)	SOIL DESCRIPTION
90	
95	Dense to very dense, saturated, gray, silty SAND to sandy SILT.
100	
105	
110	Medium dense, saturated, gray, silty SAND.
115	Very dense, saturated, GRAVEL and cobbles. (By drill cuttings and action of drill rig.)
	Boring terminated at 116.0 feet.
120	

SOIL TYPE LOG | GROUNDWATER | SAMPLES

STANDARD PENETRATION RESISTANCE

▲ Blows per foot(140 lb. hammer/30" drop)



LEGEND

2.0" O.D. split spoon sampler with percent recovered

3.0" O.D. undisturbed sampler with percent recovered

☒ 3.0" I.D. Universal sampler

☐ 3.0" I.D. Ring sampler

G Grab sample interval

L(C) Laboratory/chemical analysis

Piezometer tip

P Sampler pushed

● % moisture content

* Sample not recovered

Water level fluctuation

▽ Static water level

▼ WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

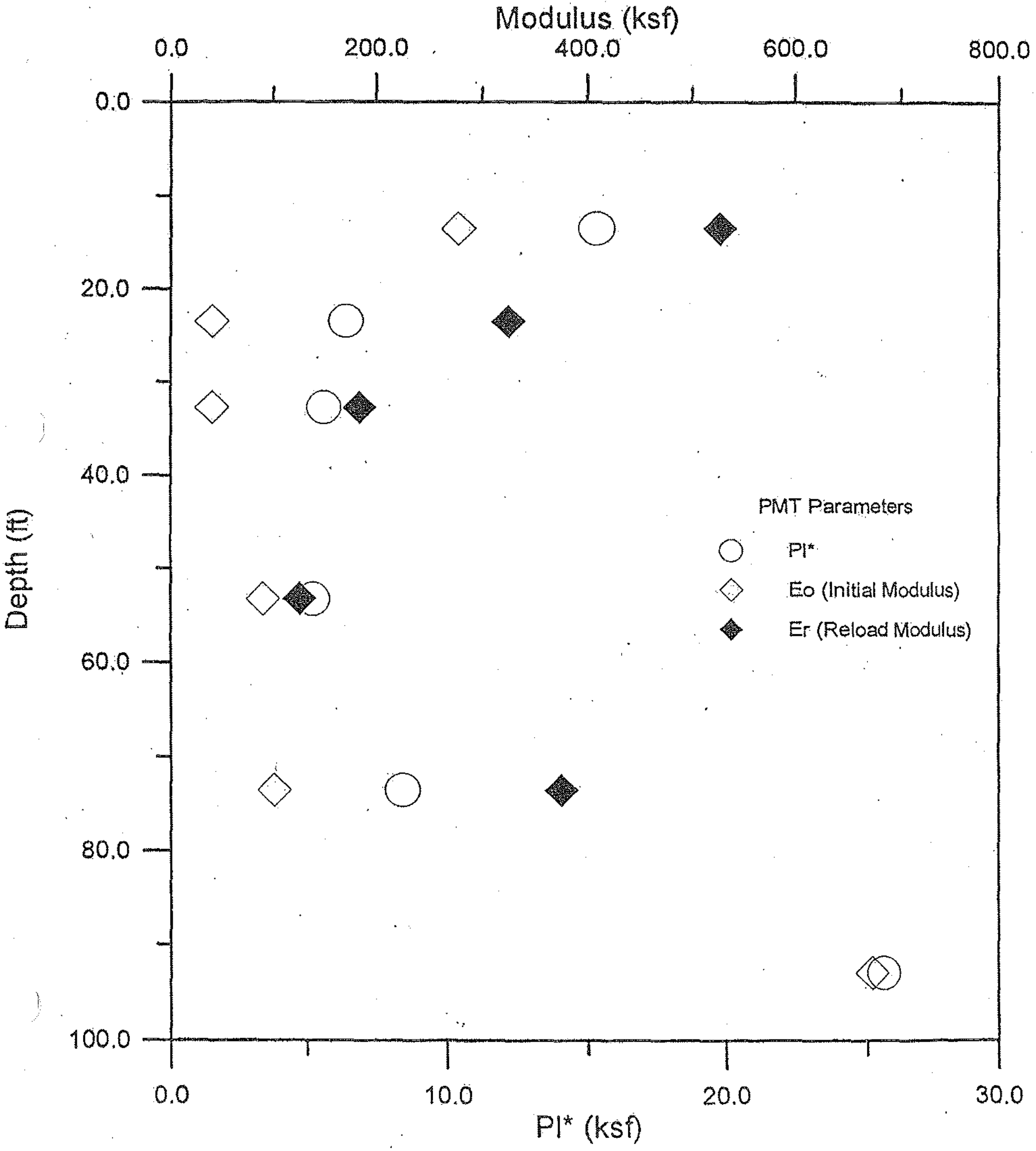
Metropolitan Exposition Center
2060 N. Marine Drive
Portland, Oregon

AGRA EARTH AND ENVIRONMENTAL INCORPORATED

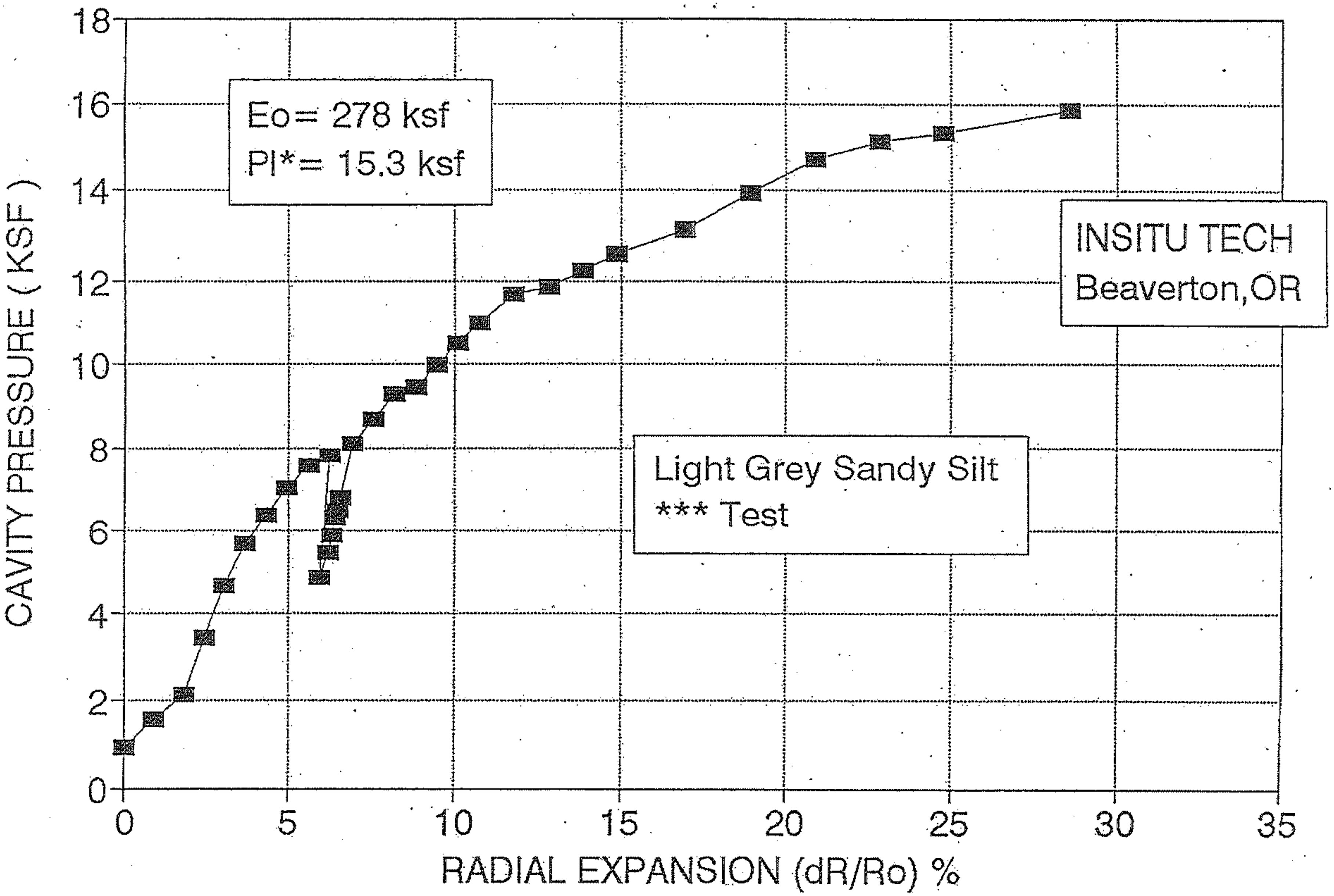
7477 S.W. Tech Center Drive
Portland, Oregon 97223
Phone: (503) 639-3400 Fax: (503) 620-7892

Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\859881P4.DRW

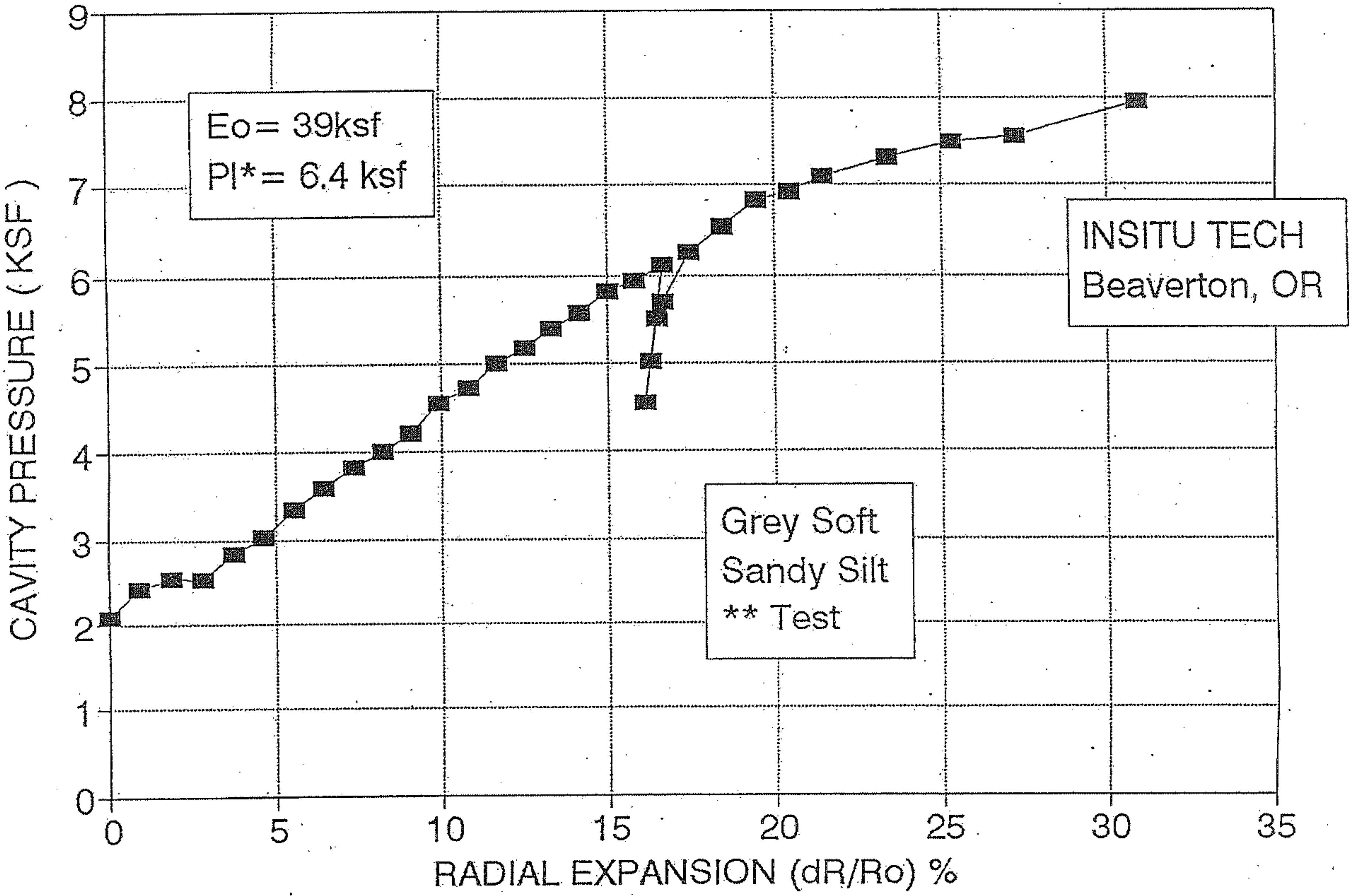
METRO Exposition Expansion
Pressuremeter Results



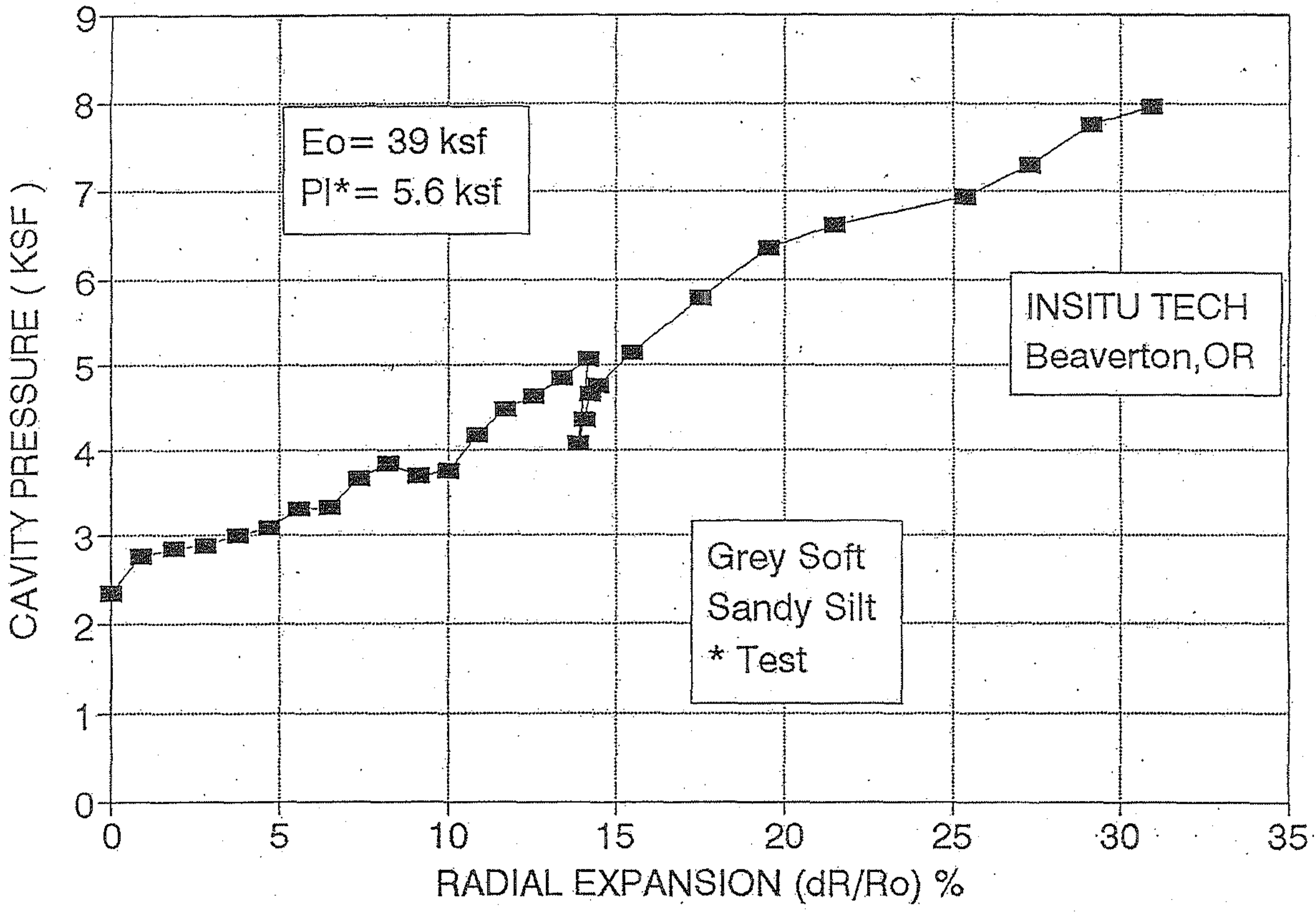
EXPO CENTER PMT TEST2 13.5 Ft



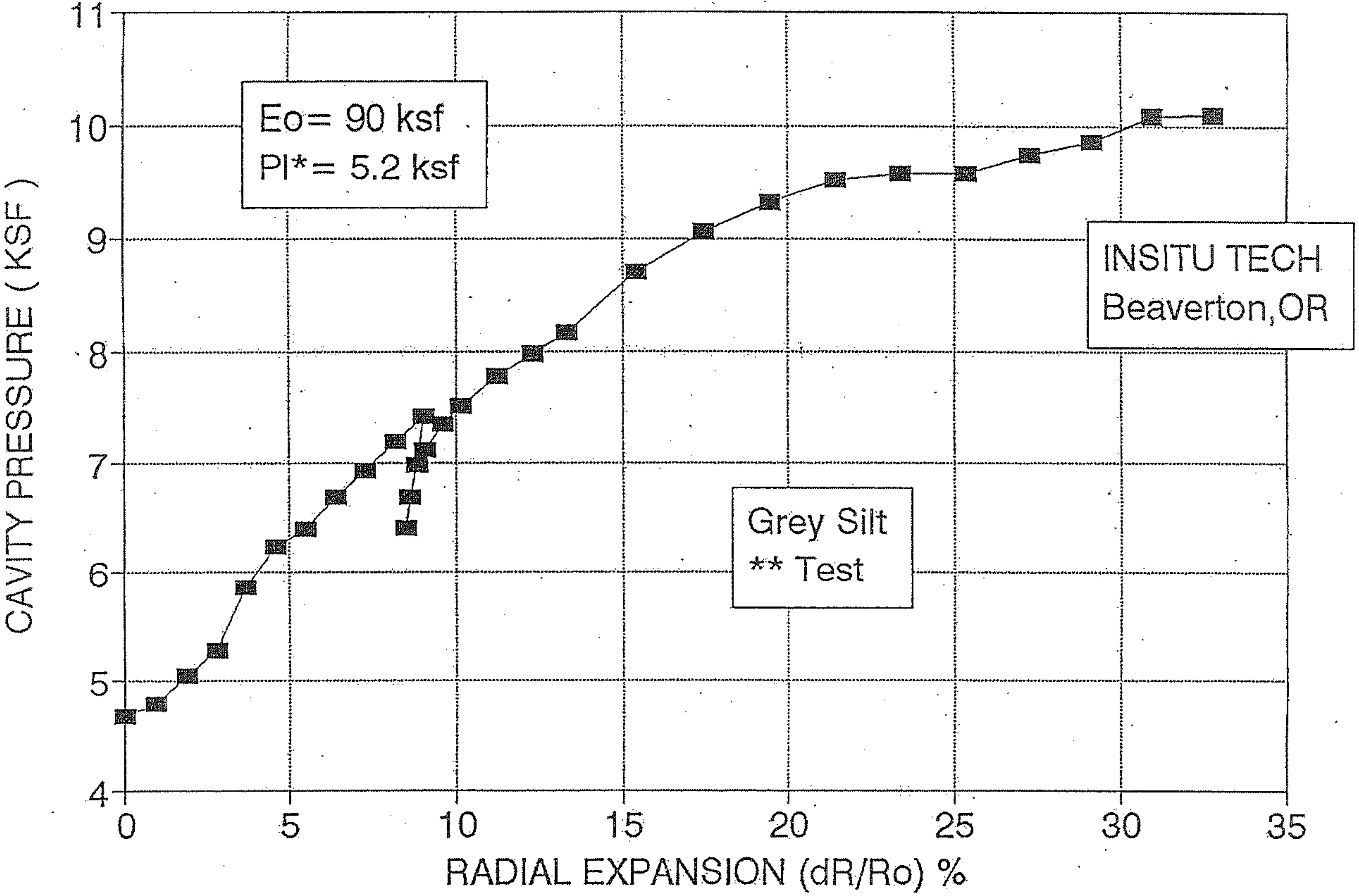
EXPO CENTER PMT TEST3 23.5 Ft



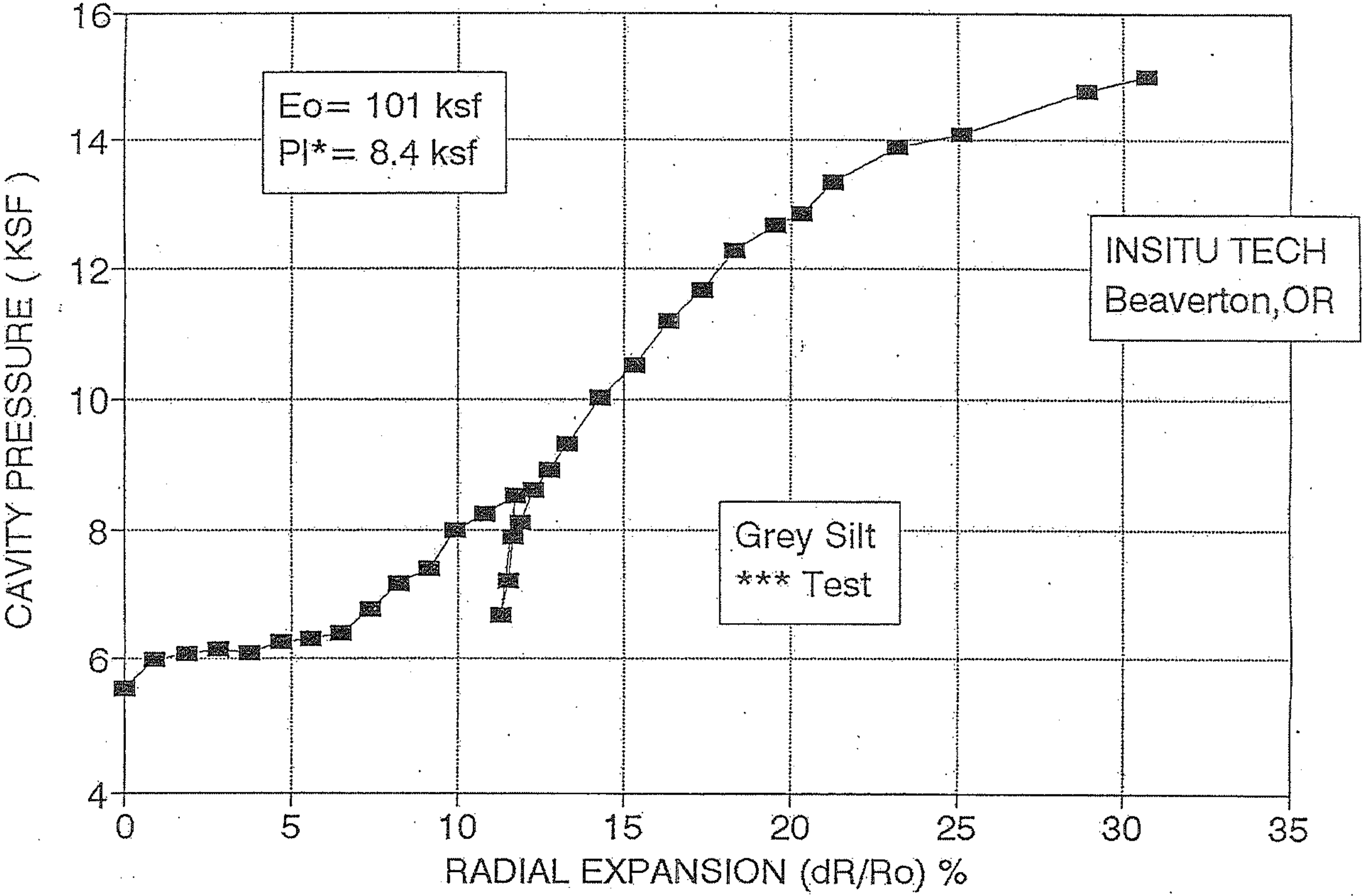
EXPO CENTER PMT TEST5 32.7 Ft



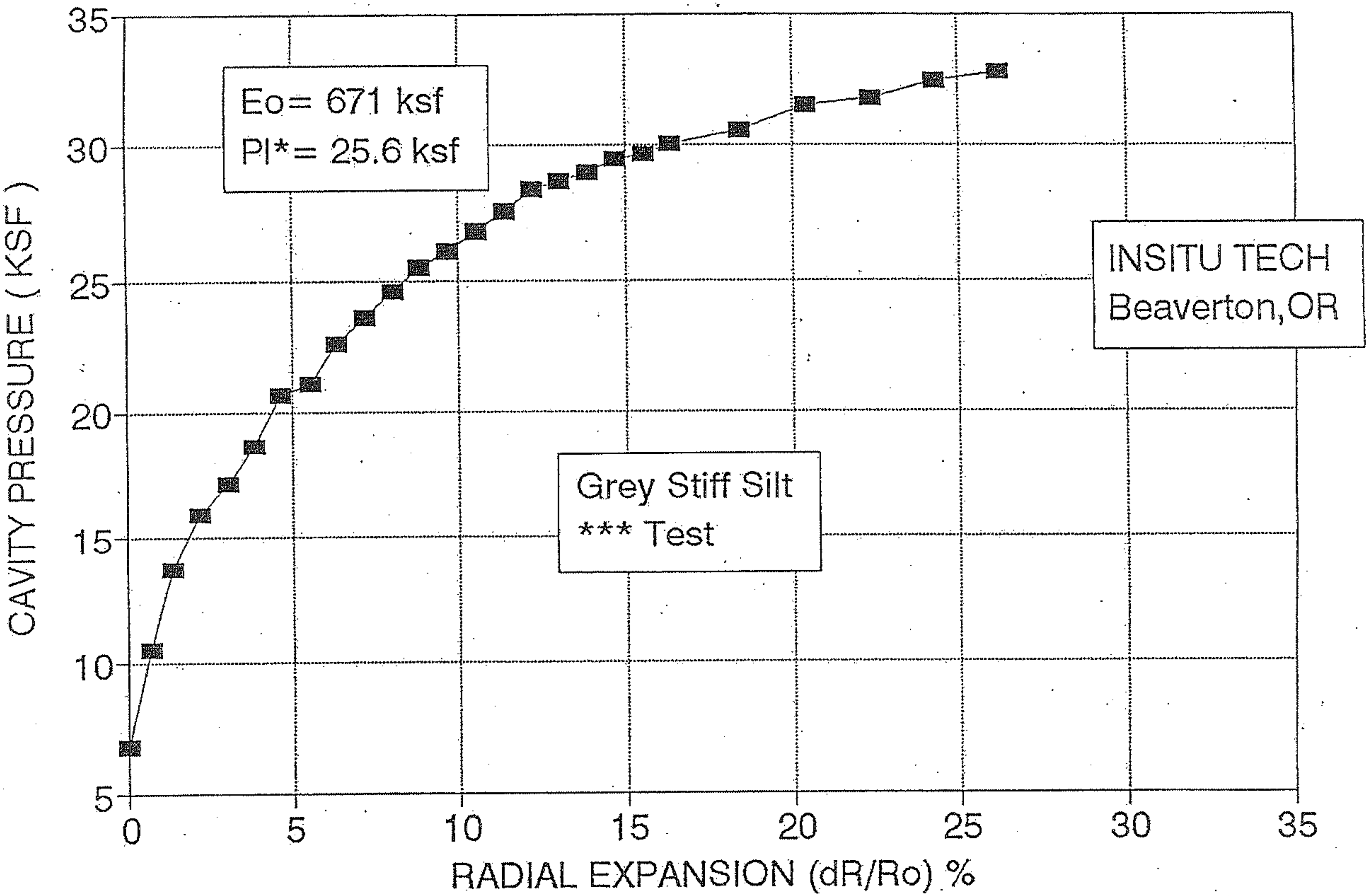
EXPO CENTER PMT TEST6 53.2 Ft



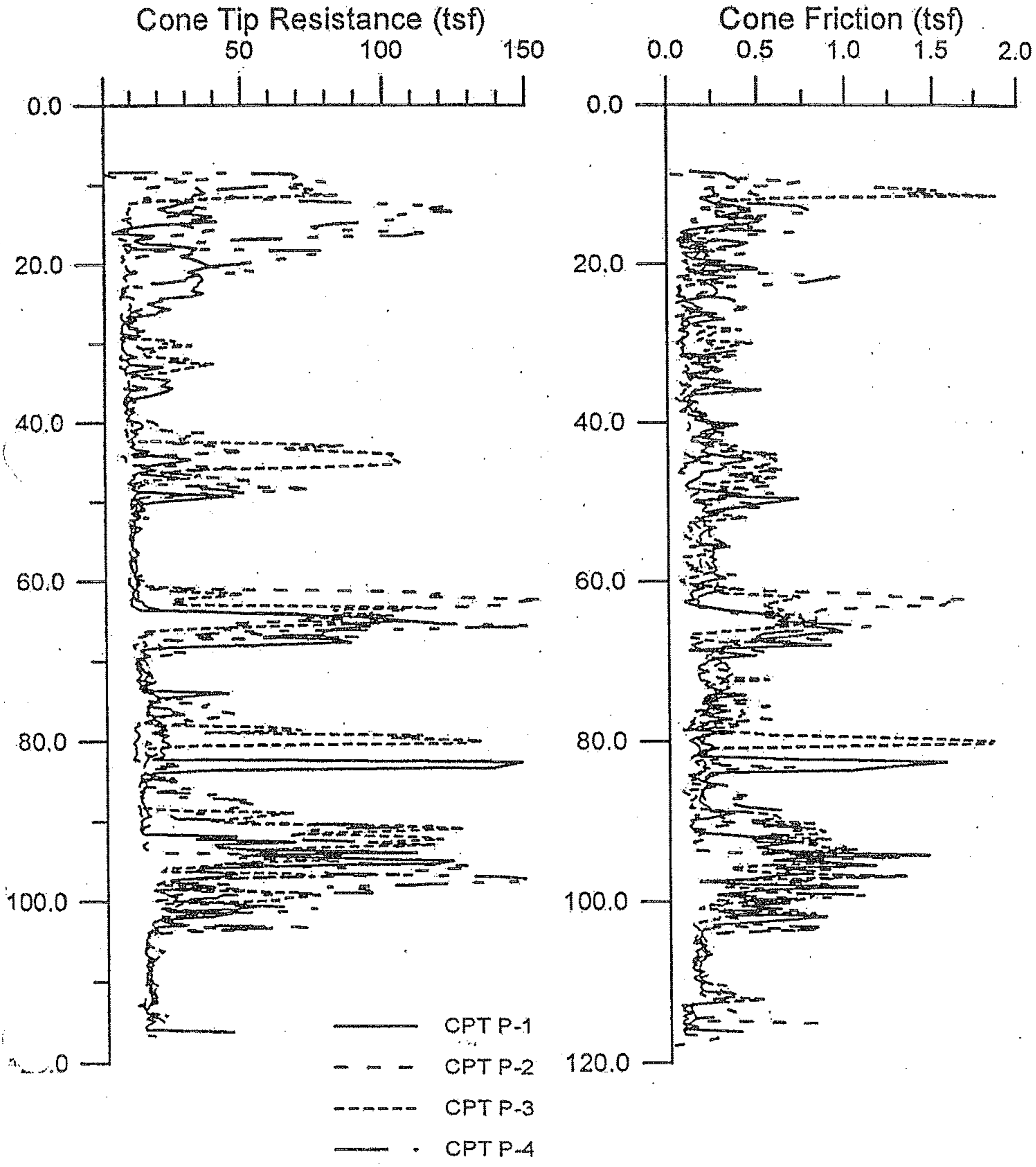
EXPO CENTER PMT TEST7 73.5 Ft



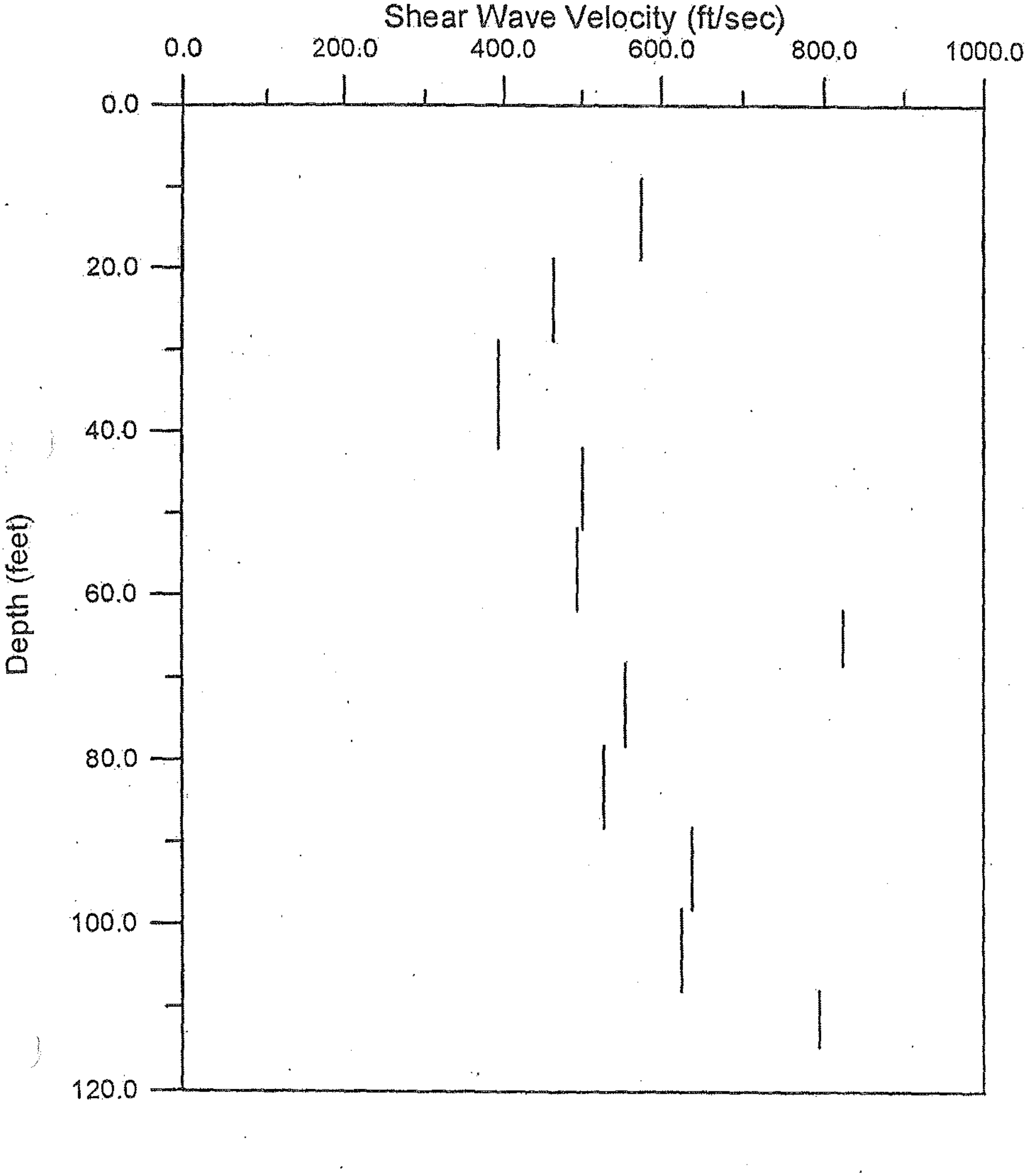
EXPO CENTER PMT TEST8 92.9 Ft



METRO Exposition Expansion
CPT Results



METRO Exposition Expansion
Shear Wave Profile



APPENDIX B

Laboratory Testing

Table B-2
Hydrometer Analysis

Boring Number	Depth (feet)	% Silt Size	% Clay Size
B-1	15 - 16.5'	89 %	11 %
B-1	35 - 36.5'	93 %	7 %
B-1	60 - 61.5'	87 %	13 %
B-1	90 - 91.5'	95 %	4 %

ATTERBERG LIMITS

Atterberg limits of selected soil samples were determined in accordance with ASTM D 4318. The Atterberg Limits of these soil samples are presented in the table below:

Table III
Atterberg Limits

Boring Number	Depth (feet)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B-1	15 - 16.5'	Non Plastic		
B-1	35 - 36.5'	Non Plastic		
B-1	50 - 51.5'	Non Plastic		
B-1	60 - 61.5'	37	29	8
B-1	90 - 91.5'	Non Plastic		

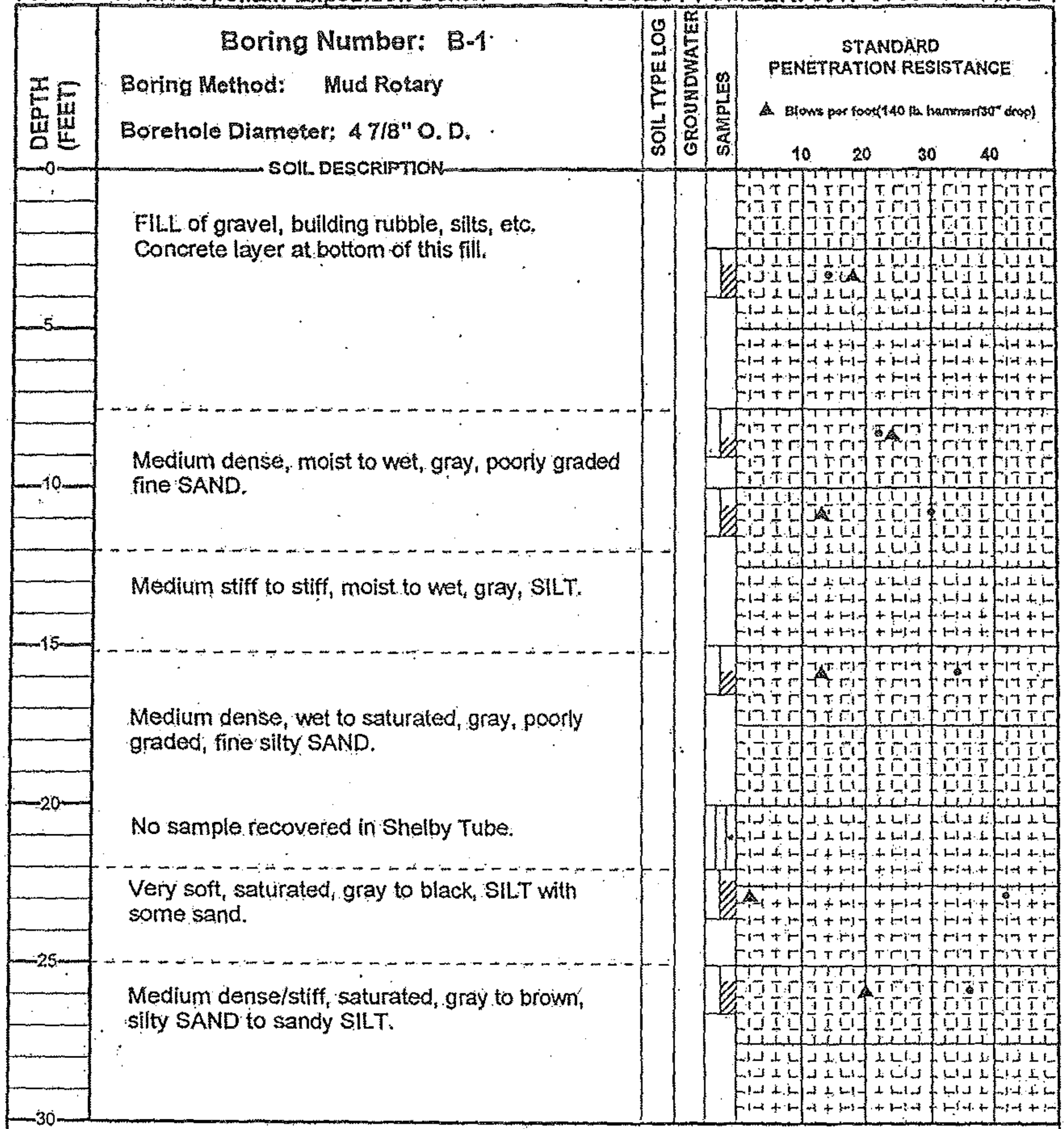
Logs of Previous Subsurface Explorations (AGRA, 1996)

PROJECT: Metropolitan Exposition Center PROJECT NUMBER: 6617-08598-0 PAGE 1

Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.



FILL of gravel, building rubble, silts, etc. Concrete layer at bottom of this fill.

Medium dense, moist to wet, gray, poorly graded fine SAND.

Medium stiff to stiff, moist to wet, gray, SILT.

Medium dense, wet to saturated, gray, poorly graded, fine silty SAND.

No sample recovered in Shelby Tube.

Very soft, saturated, gray to black, SILT with some sand.

Medium dense/stiff, saturated, gray to brown, silty SAND to sandy SILT.

LEGEND

▨	2.0" O.D. split spoon sampler with percent recovered	P	Sampler pushed
▯	3.0" O.D. undisturbed sampler with percent recovered	●	% moisture content
☒	3.0" I.D. Universal sampler	*	Sample not recovered
☐	3.0" I.D. Ring sampler	▽?	Water level fluctuation
G	Grab sample interval	▽	Static water level
L(C)	Laboratory/chemical analysis	▽ WD	Groundwater level at time of drilling
¦	Piezometer tip		

AEE Project Number: 6617-08598-0

Metropolitan Exposition Center
2060 N. Marine Drive
Portland, Oregon

AGRA EARTH AND ENVIRONMENTAL INCORPORATED

7477 S.W. Tech Center Drive
Portland, Oregon 97223
Phone: (503) 639-3400 Fax: (503) 620-7892

Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\859881.DRW

Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET)

SOIL DESCRIPTION

SOIL TYPE LOG

GROUNDWATER

30

Loose/soft, saturated, gray, silty SAND to sandy SILT. No sample recovered from 30 to 31.5 feet.

35

40

45

Medium stiff/loose, saturated, gray, sandy SILT to silty SAND with conspicuous organics.

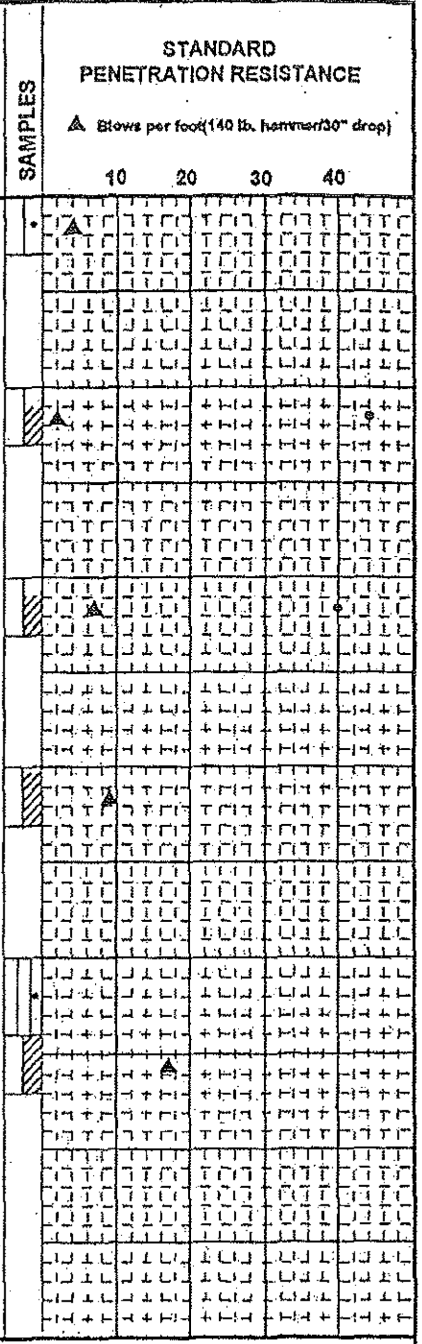
50

No sample recovered in Shelby Tube.

55

Medium dense/stiff, saturated, gray, silty SAND to sandy SILT with conspicuous organics.

60



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- Water level fluctuation
- Static water level
- WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

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Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\85988 1P2.DRW

Boring Number: B-1

Boring Method: Mud Rotary

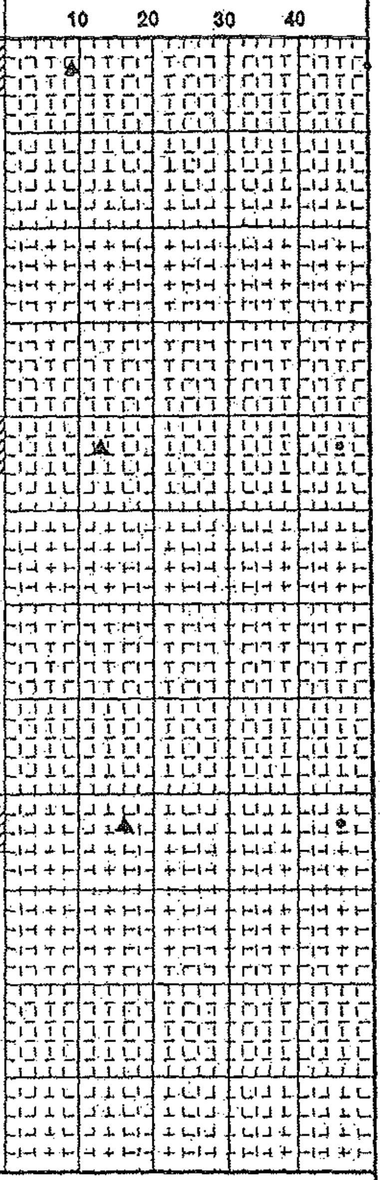
Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET) | SOIL DESCRIPTION | SOIL TYPE LOG | GROUNDWATER | SAMPLES

DEPTH (FEET)	SOIL DESCRIPTION
60	
65	Medium stiff to stiff, saturated, blue to gray, clayey SILT to silty CLAY.
70	
75	
80	Stiff, saturated, gray, clayey SILT with some sand.
85	
90	

STANDARD PENETRATION RESISTANCE

▲ Blows per foot(140 lb. hammer/30" drop)



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- i Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- Water level fluctuation
- Static water level
- WD Groundwater level at time of drilling

AEE Project Number: 6617-08698-0

Metropolitan Exposition Center
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Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA

b:\GEOTECH\MEC\85980B1P3.DRW

PROJECT: Metropolitan Exposition Center **PROJECT NUMBER:** 6617-08598-0 **PAGE 4**

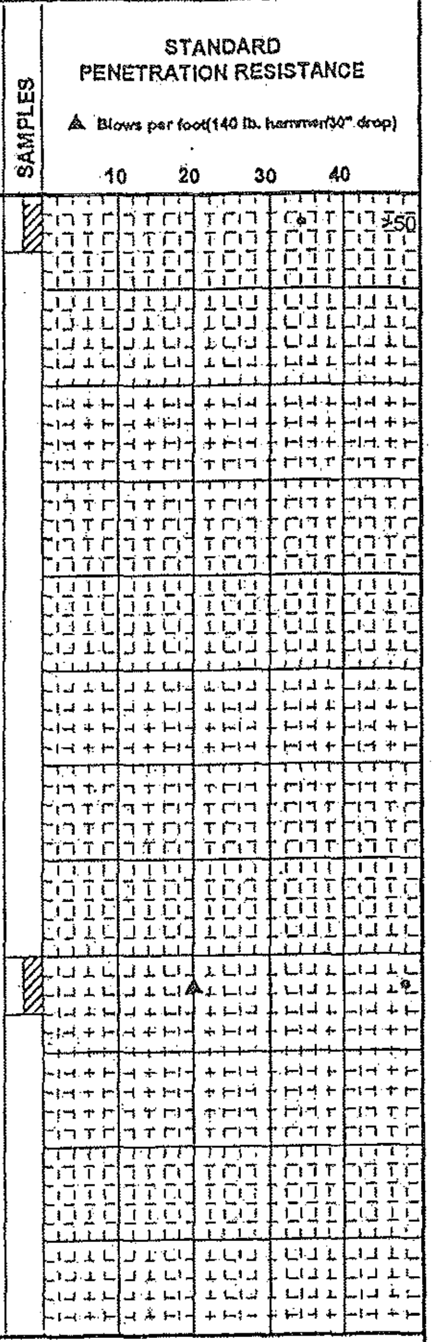
Boring Number: B-1

Boring Method: Mud Rotary

Borehole Diameter: 4 7/8" O. D.

DEPTH (FEET) | SOIL DESCRIPTION | SOIL TYPE LOG | GROUNDWATER | SAMPLES

DEPTH (FEET)	SOIL DESCRIPTION
90	
95	Dense to very dense, saturated, gray, silty SAND to sandy SILT.
100	
105	
110	Medium dense, saturated, gray, silty SAND.
115	Very dense, saturated, GRAVEL and cobbles. (By drill cuttings and action of drill rig.)
	Boring terminated at 116.0 feet.
120	



LEGEND

- 2.0" O.D. split spoon sampler with percent recovered
- 3.0" O.D. undisturbed sampler with percent recovered
- ☒ 3.0" I.D. Universal sampler
- ☐ 3.0" I.D. Ring sampler
- G Grab sample interval
- L(C) Laboratory/chemical analysis
- Piezometer tip
- P Sampler pushed
- ● % moisture content
- * Sample not recovered
- Water level fluctuation
- Static water level
- WD Groundwater level at time of drilling

AEE Project Number: 6617-08598-0

Metropolitan Exposition Center
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Portland, Oregon

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Drilling Started: 2/22/96 Drilling Completed: 2/23/96 Logged By: RA a:\GEOTECH\MEC\8598B1P4.DRW

METRO Exposition Expansion
CPT Results