PACRIM GEOTECHNICAL INC.

FINAL GEOTECHNICAL REPORT
HOUGHTON, RENTON & ALGONA TRANSFER STATIONS ROOF REPLACEMENT PROJECT
KING COUNTY, WASHINGTON

Project No. 092-004
March 29, 2002

geotechnical
engineering
and
applied
earth
sciences

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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March 29, 2002
Project No. 092-004

ABKJ, Inc.
800 Fifth Avenue Suite 3800
Seattle, Washington 98104-3103

Attention: Mr. Tom Pittsford

Subject: **FINAL GEOTECHNICAL REPORT**
HOUGHTON, RENTON & ALGONA
TRANSFER STATIONS
ROOF REPLACEMENT PROJECT
KING COUNTY, WASHINGTON

Dear Tom:

We submit herewith three copies of our report titled *Final Geotechnical Report, Houghton, Renton & Algona Transfer Stations, Roof Replacement Project, King County, Washington*. Our work was authorized by a subconsultant agreement with ABKJ, Inc., dated February 2, 2000. We submitted a report titled *Draft Geotechnical Report, Houghton, Renton & Algona Transfer Stations, Roof Replacement Project, King County, Washington* on April 5, 2000. We have revised it to include issues discussed in our memorandum dated April 19, 2000.

We appreciate the opportunity to be of service to you on this interesting project. We look forward to continued involvement as the project proceeds through construction. Please call with any questions.

Sincerely,

PACRIM GEOTECHNICAL INC.

Harbans L. Chabra, P.E.
Principal

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FINAL GEOTECHNICAL REPORT
HOUGHTON, RENTON & ALGONA
TRANSFER STATIONS
ROOF REPLACEMENT PROJECT
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical engineering study by PacRim Geotechnical Inc. (PacRim) for design and construction of the Houghton, Renton and Algona Transfer Stations Roof Replacement Project in King County, Washington. The project locations are illustrated on Figures 1a, 1b, and 1c. The project owner is King County Solid Waste (Owner). The lead designer for the project is ABKJ, Inc, and PacRim is the geotechnical engineer-of-record.

The purpose of this study was to gather and review available existing subsurface information; conduct field explorations and laboratory testing to evaluate subsurface conditions at the site; and develop geotechnical conclusions and engineering recommendations for design and considerations for construction of the project.

1.2 REPORT ORGANIZATION

This report is organized in six sections. Section 1 provides a general description of the project, discusses previous geotechnical studies that have been completed at the project sites, and presents the limitations of this report. Section 2 describes our field and laboratory investigations. Section 3 describes site conditions at the project sites. Section 4 presents our conclusions and engineering recommendations. Section 5 presents our recommendations for document review and construction support. References are listed in Section 6. Summary tables are presented within the main body of text in this report. Figures are presented following the main body of text. Appendices are found at the end of this report. Appendix A includes descriptions and results of our field exploration, including summary exploration logs. Appendix B presents descriptions and results of our laboratory testing program. Summary logs and laboratory test results completed as part of earlier geotechnical studies for the sites are included in Appendix C. Shear, moment and deflection diagrams for pile types evaluated in this study are presented in Appendix D.

1.3 PROJECT UNDERSTANDING

The proposed project involves the replacement of canopy roof systems at Houghton, Renton and Algona Transfer Stations in King County, Washington. The existing canopies at these locations, which are of similar design and construction, will be removed

and replaced as part of this project. The new canopies will be designed to resist static and seismic loading. We understand column loads on the order of about 70 tons are anticipated at each column location. As part of the project, the foundation systems at all three sites will be either replaced or augmented. All shallow (spread footing) foundations will be replaced. Deep (pile) foundations will be installed at selected column locations at the Houghton and Algona facilities. In addition, we anticipate the project will involve replacing portions of the concrete pavement area beneath the existing canopies. The focus of this study is on the foundations and pavement subgrades. Based on our understanding of the project, no other development of the site is planned.

Limited information is available concerning the exiting foundation systems at the transfer station facilities. Existing foundation plans for Algona Transfer Station were available for review, and indicate the facility is constructed partially on spread footings and partially on timber piles. The length of the piles is unknown. Existing plans for the canopies at Houghton and Renton Transfer Stations were not available at the time of our study. The canopy at Houghton Transfer Station is thought to be constructed on shallow (spread) foundations (Hong West & Associates, Inc. [HWA], 1997). The results of previous studies at the Renton Transfer Station (HWA, 1997) suggest the canopy at that location is also supported on spread footings.

1.4 PREVIOUS GEOTECHNICAL STUDIES

Where appropriate, information from previous geotechnical studies completed at the transfer station sites was incorporated into our evaluations and analyses. The primary source of previous information was a seismic evaluation study completed for several King County transfer station facilities, including the facilities evaluated in this study (HWA, 1997). The scope of that study included the review of existing subsurface data, completion of limited field and laboratory investigations, and an evaluation of seismic hazards, including liquefaction, seismic settlement, ground fault, and seismically induced landslides. Recommendations were provided for seismic design of the facilities. Summaries of earlier studies completed at the transfer station sites were included in the HWA report, and were reviewed for this study. Relevant summary boring logs completed as part of earlier studies at the Houghton and Algona sites are included in Appendix C.

Golder Associates, Inc. (Golder) and Shannon & Wilson, Inc. (S&W) completed earlier studies at the Algona facility, which were mainly related to previous slope stabilization measures for the slope immediately west of the facility. The results of these earlier studies are summarized in the 1997 HWA report. To the best of our knowledge, no subsurface investigations were completed as part of the Golder and S&W studies.

1.5 AUTHORIZATION AND SCOPE OF WORK

PacRim's services were authorized by a subconsultant agreement with ABKJ, Inc., dated February 2, 2000. Our scope of work included gathering and reviewing existing subsurface information in the project vicinity; drilling and sampling exploratory borings; excavating backhoe test pits; performing laboratory testing; and completing engineering analyses to develop the geotechnical conclusions and recommendations presented in this report.

PacRim's scope of work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

1.6 LIMITATIONS AND USE OF THIS REPORT

We have prepared this report for use by ABKJ, Inc. and the Owner for this project. Experience has shown that subsurface soil and groundwater conditions can vary significantly over small distances. While the actual conditions encountered in the field are expected to be within the ranges discussed herein, the distribution of geologic conditions encountered will likely vary from those presented in this report. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, PacRim should be notified for review of the recommendations of this report, and revision of such if necessary. If there is a substantial lapse of time between the submission of this report and the start of construction, or if conditions have changed due to construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report is issued with the understanding that the information and recommendations contained herein will be brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and the necessary steps will be taken to verify that the contractor and subcontractors carry out such recommendations in the field.

PacRim does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the Contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein unsafe.

2.0 FIELD AND LABORATORY INVESTIGATIONS

2.1 FIELD INVESTIGATIONS

Our field explorations were completed between February 22 and 24, 2000, and included drilling two exploratory borings at the Houghton facility, two exploratory borings at the Algona facility, and excavating three backhoe test pits at the Renton facility. The exploratory borings were drilled to depths ranging from 29 to 39 feet, and the test pits were excavated to depths ranging from 5 to 6.5 feet. The approximate locations of the explorations are illustrated on Figures 2a, 2b and 2c. The exploration program was developed at the start of our study during site visits to the three transfer stations with King County and ABKJ personnel.

Specific details of the exploratory drilling, test pits and sampling methodologies are presented in Appendix A. Summary logs of the explorations are presented on Figures A-2 through A-8. Figure A-1 provides a key to symbols and terms used on the summary logs.

2.2 LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to assist in the characterization of certain engineering (physical) properties of the on-site soils. Laboratory tests completed at our in-house laboratory included determination of natural moisture content, fines content, and grain-size distribution testing. Laboratory tests were conducted in general accordance with appropriate American Society for Testing and Materials (ASTM) standards (ASTM, 1998). A discussion of laboratory test methodology and test results are presented in Appendix B. Test results are also displayed where appropriate on the exploration logs in Appendix A.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The transfer station facilities are of similar design and construction, and consist of an open canopy roof, a concrete paved tipping floor and a central tunnel beneath the tipping floor. The Houghton facility is constructed on the margin of a closed landfill. The closed landfill is located north and east of the facility, and surface elevations generally trend downward from the landfill to the transfer station site. The canopy area is elevated above the prevailing grades of the transfer station site to allow room for the central tunnel beneath the tipping floor.

The Renton facility is constructed on the site of a former gravel pit, portions of which were once used as a landfill. The central tunnel at the Renton site appears to have been

constructed by excavating below the prevailing transfer station grades. The western portion of the Renton facility is situated at the crest of an approximately 40-foot high slope, which ranges in inclination between about 1¼ to 1 (horizontal to vertical) to 1½ to 1. The column supporting the northwest corner of the canopy is situated within about 15 feet of the crest of the slope. At the time of our site visit, localized areas of erosion and rutting were observed where surface water is discharged onto the slope face, and/or where vegetation was absent.

The Algona facility is situated on the western margin of the Green River valley. The varied topography within a small area, coupled with the subsurface conditions, strongly influenced the foundation system used at the facility. Part of the facility is supported on shallow footings bearing directly on the lower portion of the steep slope, which rises about 300 feet to the west of the facility to an upland plateau. The remaining portion of the facility is founded on timber piles, which extend about 15 feet above grade to the tipping floor level. The site has experienced a history of shallow surface landslides. Measures have been undertaken to address instability in close proximity to the facility (HWA, 1997).

3.2 SUBSURFACE CONDITIONS

General geologic conditions at each of the transfer station facilities were presented in a report by others (HWA, 1997). The following report sections discuss site-specific subsurface conditions at each of the transfer station facilities, based on our review of existing information and the results of our field exploration. Summary logs of the explorations by PacRim and relevant summary logs by others, are presented in the appendices. The information from the explorations completed at the Houghton and Algona sites was interpolated to construct the generalized subsurface sections shown on Figures 3 and 4. The actual stratigraphic contacts are the result of complex geologic processes and/or construction activities, so they may be gradational or erratic in nature than as shown in the figures.

3.2.1 Houghton Transfer Station

PacRim explored the Houghton facility with 2 exploratory borings. The subsurface conditions observed in the explorations at the Houghton facility are described below, in order from youngest to oldest.

- **Fill:** Fill was encountered from the ground surface to depths of 5 and 21.5 feet in borings B-1 and B-2, respectively. The composition of the fill encountered our borings (in the canopy area) appears to be relatively uniform, and consists of moist, sand with varying amounts of silt and gravel. The fill encountered in boring B-1 also had scattered pockets of organics and wood fragments. The fill encountered in our borings appears to be consistent in appearance with natural

advance outwash soils found onsite, as described below. The unified soil classification system (USCS) designations for the fill include of SP and SP-SM. Standard Penetration Test (SPT) N-values* for the fill in these borings ranged from 12 to 35 with an average value of 18, suggesting the fill is predominantly medium dense.

No construction records documenting the placement of the fill in the area of the canopy structure are available at this time. Assuming the existing structure is founded on shallow footings, as suggested by others (HWA, 1997), it is likely that the fill was compacted to a relatively uniform density, considering the structure has performed adequately in the past. However, refuse fill has been documented in areas north, south and west of the structure, and is possible that it is present beneath the structure, as well.

- **Landfill Refuse:** Landfill refuse was encountered in several borings completed at the site by others, and typically consists of municipal solid waste of varying composition, mixed with soil fill. Landfill refuse was generally encountered beneath a 4 to 6-foot thick surficial layer of fill. Where encountered, landfill refuse was in direct contact with the underlying native soils as described in the following paragraph.

- **Advance Outwash:** Glacially overridden, advance outwash deposits were encountered in borings B-1 and B-2 below the fill, and extended to the maximum depth explored. Soil classified as advance outwash was also encountered in the borings by others below fill and landfill refuse. The outwash encountered in the PacRim borings is moist sand with varying amounts of silt and gravel, and USCS designations of SP and SP-SM. SPT N-values for the advance outwash in these borings ranged from 64 to in excess of 100, with an average value of about 93, suggesting that it is generally very dense.

  Advance outwash encountered in borings by others (HWA, 1992) appears to be of a similar composition to the material encountered in our borings. However, SPT N-values for the advance outwash soils appear to be slightly lower than those recorded during our investigation.

- **Groundwater:** Groundwater was not encountered in explorations completed in the immediate vicinity of the canopy structure. One boring completed by Hart Crowser, located about 250 feet northwest of the northwest corner of the canopy, encountered groundwater at the time of drilling at approximately Elevation 420

* N-values are defined as the number of blows required to drive a 2.0-inch outside diameter sampler one foot, using a 140-pound hammer falling a distance of 30 inches. Refer to ASTM D-1586 (ASTM, 1998)

(HWA, 1997), or about 25 to 30 feet below the surface elevation adjacent to the existing canopy structure.

3.2.2 Renton Transfer Station

PacRim explored the Renton facility with 3 exploratory test pits. An exploratory boring was completed by others (HWA, 1997) north of the northwest corner of the canopy structure. The subsurface conditions observed in the explorations at the Renton facility are described below, in order from youngest to oldest.

- **Fill:** Fill was encountered from the ground surface to depths ranging from 2 to 3.5 feet in test pits TP-1 TP-2 and TP-3, and was absent in boring BH-1 completed by others. However, the first sample collected from BH-1 was at a depth of 5 feet, and it is possible that a thin surficial layer of fill also exists at that location. The composition of the fill encountered in the canopy area is relatively uniform, and consists of medium dense, moist, sand and gravel with trace amounts of silt and varying amounts of cobbles. Scattered organics and construction rubble were also observed in the fill. The fill encountered in our test pits appears to be consistent in appearance with natural recessional outwash soils found onsite, as described below. The USCS designations for the fill include of SP, SM, GP and GW.

- **Recessional Outwash:** Recessional outwash deposits were encountered in the test pits TP-1 TP-2 and TP-3 below the fill, and extended to the maximum depth explored. The recessional outwash is medium dense, moist, gravel with sand, trace amounts of silt, varying amounts of cobbles, and a USCS designation of GP. Scattered, discontinuous layers of medium dense, moist, fine to medium sand, with trace amounts of silt and gravel (SP) were observed within the unit.

- **Advance Outwash:** Natural soil identified by others (HWA, 1997) as advance outwash deposits were encountered in boring BH-1, and extended to the maximum depth explored. Based on the results of our recent investigations, we interpret the contact between recessional and advance outwash to be on the order of 5 to 10 feet below ground surface. The advance outwash was reported to consist of dense to very dense, moist, silty sand with gravel (SM) to a depth of about 19 feet, and very dense, moist, gravel with sand (GP) below 19 feet. SPT N-values recorded in the advance outwash ranged from 33 to in excess of 100, with an average value of about 76, suggesting that it is generally very dense.

- **Groundwater:** Groundwater was not encountered in explorations completed in the immediate vicinity of the canopy structure.

3.2.3 Algona Transfer Station

PacRim explored the Algona facility with 2 exploratory borings. Five exploratory borings were completed by others (HWA, 1997) south and east of the existing canopy structure. The subsurface conditions observed in the explorations at the Algona facility are described below, in order from youngest to oldest.

- **Valley Fill:** A surficial layer of valley fill was encountered east of the existing canopy structure from the ground surface to a depth of 7 feet in PacRim boring B-1. Fill of similar composition was also encountered in all 5 borings completed by others. The valley fill consists of medium dense to very dense, moist, sand and gravel with varying amounts of silt. Scattered organics were also observed in the valley fill. The USCS designations for the fill include of ML, SW, SM, GM and GW.

- **Fill/Colluvium:** Fill/Colluvium was encountered from the ground surface to a depth of 7 feet in boring B-2, located near the southwest corner of the existing canopy. It is uncertain whether this material is colluvium (soil deposited by gravity near the toe of slopes) or fill associated with prior grading activities reported to have occurred in this area of the site (HWA, 1997). The composition and engineering characteristics of fill or colluvium would be similar if native soils were used as fill. The composition of the fill encountered in boring B-2 consists of medium dense, moist, silty sand with gravel (SM).

- **Alluvium:** Alluvium was encountered below surficial valley fill in boring B-1 and to the total depths explored for all 5 borings completed by others. In boring B-1, alluvium was encountered to a depth of about 27 feet, and is underlain by glacial till (described below). The alluvium appears to vary in composition, and in boring B-1, it consists primarily of soft, wet, peat with scattered wood fragments and pockets of clay and silt. The peat in boring B-1 extends to a depth of about 23 feet, where a 4-foot thick layer of loose to medium dense gravel with sand is present. Alluvium encountered in borings by others consisted of soft peat, clay and silt, and loose to medium dense sand and silty sand, with varying amounts of gravel.

- **Glacial Till:** Glacially consolidated, lodgement till was encountered below the alluvium in boring B-1, and to the total depth explored. It was also encountered below surficial fill/colluvium in boring B-2 to a depth of 21 feet. The till consists of very dense, moist to wet, silty sand (SM), and sand with silt (SP-SM), with varying amounts of gravel. This material is characterized by high shear strength and low permeability.

- **Advance Outwash:** Advance outwash deposits were encountered below glacial till in boring B-2, and extended to the maximum depth explored. The advance outwash consists of very dense, wet, sand with silt and varying amounts of gravel (SW-SM). SPT N-values recorded in the advance outwash were in excess of 100.

- **Groundwater:** Groundwater was encountered within the alluvium deposits in the borings completed in the lowland area east of the existing canopy structure. In boring B-1, groundwater was encountered at a depth of about 7 feet at the time of drilling. The borings completed in the lowland area by others (HWA, 1997) encountered groundwater at depths ranging from 7 to 17 feet. Boring B-2, located near the southwest corner of the canopy structure, encountered groundwater within the advance outwash deposits at a depth of about 21 feet. Fluctuations in groundwater levels within the project area will likely occur due to seasonal rainfall, infiltration, and percolation of surface water. Groundwater levels are anticipated to be highest during the winter and spring months.

4.0 CONCLUSIONS AND ENGINEERING RECOMMENDATIONS

4.1 GENERAL

Based on the current subsurface investigation and laboratory testing, results of previous studies, and the analyses performed, it is our opinion that the proposed improvements are feasible from a geotechnical perspective, provided the recommendations of this report are incorporated in design and construction. The following report sections present recommendations for seismic considerations, site preparation and earthwork, and foundations.

Houghton Transfer Station: There is uncertainty regarding the fill composition and the existing foundation type beneath the southern portion of the Houghton facility, where undocumented fill was encountered. Landfill refuse was encountered in borings immediately south and west of the facility. To address these subsurface conditions, we provide recommendations for augercast piles for the southern portion of the Houghton facility that penetrate through the expected depth of fill, and into dense native soils. Recommendations for shallow foundations are provided for the northern portion of the facility where dense native soils are expected at foundation level.

Renton Transfer Station: Shallow foundation recommendations for the Renton facility are presented, since relatively dense native soils are expected beneath the entire facility. Analyses were completed to evaluate the static and seismic factors of safety of the slope immediately north of the Renton facility, and are discussed below.

Algona Transfer Station: Shallow foundations are considered appropriate for improvements planned for the northwestern corner of the facility since dense, glacially consolidated soils are present at foundation level. New piling will be installed to support roof columns on the remaining portions of the facility. Our recommendations reflect two different construction approaches. In the first scenario, drilled micropiles will be installed in the area beneath the tipping floor, which has limited overhead clearance. In the second scenario, driven steel pipe piles will be installed from above the tipping floor, with the piles penetrating the tipping floor as they are driven.

4.2 SEISMIC CONSIDERATIONS

The previously completed seismic evaluation (HWA, 1997) provides details on the regional seismicity and seismic hazards for the three transfer station sites. Based on our review of the HWA report, and our additional investigations and analyses, we concur with the seismic design parameters and hazard evaluations provided in that report. Seismic considerations that, in our opinion, warrant additional commentary include the "moderate" rating assigned to the Renton facility for seismic landslide hazard, and the "moderate" rating assigned to Algona facility for liquefaction, seismic settlement, and seismic landslide hazard. Table 1 presents our interpretation of the seismic risk at the transfer stations. Additional details are provided in the following sections for slope stability at the Renton facility and liquefaction, seismic settlement, and seismic landslide hazards at the Algona facility.

Table 1 – Summary of Relative Seismic Hazard (1)

Site	(2)Design Acceleration	(3)UBC Soil Type	Liquefaction Hazard	Seismic Settlement	Ground Fault	Land Slide
Houghton	0.30	S_c	low	low	low	low
Renton	0.30	S_c	low	low	low	moderate
Algona	0.30	S_c	low	low	low	moderate

Notes:

(1) Estimated relative seismic hazards rated as low/moderate/high

(2) Design accelerations taken from *USGS National Seismic Hazard Mapping Project*, 1996, representing bedrock accelerations and a 10 percent probability of exceedance in a 50-year period.

(3) Based on 1997 International Conference of Building Officials (ICBO) *Uniform Building Code (UBC)*.

4.2.1 Renton Transfer Station

A slope stability analysis was completed for the slope immediately northwest of the Renton facility using the UTEXAS3 computer program (Wright, 1990), and assuming

our interpretation of soil conditions, based on the results of investigations completed at the facility. Topographic conditions for the slope immediately adjacent to the northwest corner of the canopy were assumed, based on an undated site topographic map provided to us by ABKJ. The assumed engineering properties for soils that comprise the slope at the Renton facility are summarized in Table 2. Groundwater was assumed to be below the bottom elevation of the slope. Foundation loads of 3,000 pounds per square foot (psf) over a 7-foot square footing were included in the analyses for the column on the northwest corner of the canopy. Analyses were completed for both static and dynamic (seismic) cases. For the seismic case, a pseudostatic analysis was performed, using a pseudostatic coefficient, k_h, of 0.15, which coincides with a peak horizontal bedrock acceleration of above 0.30. The peak horizontal bedrock acceleration of 0.30 is representative of a 10 percent probability of exceedance in a 50-year period (HWA, 1997).

Table 2 - Assumed Soil Properties for Renton Transfer Station Stability Analyses

Property	Value
Angle of Internal Friction, ϕ	36 degrees
Cohesion	0 psf
Unit Weight	129 pcf

Note: psf = pounds per square foot; pcf = pounds per cubic foot

The UTEXAS3 program performs slope stability computations based on the modeled conditions, and calculates a factor of safety against slope failure, F, defined as:

$$F = s/\tau$$

where s is the available shear strength of the soil and τ is the shear stress required for "just-stable" equilibrium. A "just-stable" equilibrium condition would result in a factor of safety of one, while an unstable condition would result in a factor of safety less than one. UTEXAS3 program uses Spencer's procedure to determine the factor of safety. The results of our stability analyses are summarized in Table 3 below.

For the static case, the factor of safety against a surficial skin slide failure is marginally greater than 1. Slope instability related to this mode would manifest itself as minor surficial raveling and sloughing. The factor of safety against deeper slope failure that could influence improvements near the canopy (movement of a soil mass greater than about 2 feet in thickness) is greater than 1.5, which is generally considered acceptable for long-term stability. In our opinion, no further action is warranted to address deep-seated slope failure under static conditions. Erosion control measures should be implemented to address the possibility of shallow failure, as detailed in Section 4.4.

Table 3 - Summary of Slope Stability Analysis Results – Renton Transfer Station

Condition	Factor of Safety
Static case – surficial (less than about 1 foot) failure	1.02
Static case – deeper (greater than 2 feet) failure	>1.5
Seismic case – surficial (less than about 3-foot) failure	0.76 to 0.81
Seismic case – Moderate depth (between about 3 to 13-foot) failure	0.85 to 0.97
Seismic case – deeper (greater than 13 feet) failure	>1.3

The factor of safety against surficial and moderate depth failures under seismic conditions is less than one. This suggests that some earth movement can be expected on the face of the slope, and could possibly approach within a few feet of the foundation on the northwest corner of the canopy during the design magnitude earthquake. This movement could damage improvements located between the canopy footing and the slope. We do not anticipate that a potential slide would intercept the footing provided the edge of the footing is located at least 12 feet from the crest of the slope; however, lateral capacity of the footing could be compromised if soil next to the footing is lost to land sliding. The factor of safety against deeper-seated slope failure that could intercept the footing during a design level earthquake is greater than 1.3, which is generally considered acceptable.

4.2.2 Algona Transfer Station

The Algona facility was evaluated to have moderate hazard ratings for soil liquefaction, seismic settlement and seismic landslide hazard. The soil liquefaction concerns are related to the lowland area, beneath the pile supported portion of the facility. Layers of loose sand and silty sand were encountered in borings by others, in the vicinity of the site. Soils encountered beneath the facility in our investigations consist primarily of soft peat with varying clay and silt fractions. It is possible that localized, liquefiable lenses of

loose, saturated sand and silty sand may be present in the area. However, in general, the encountered soils are not considered susceptible to liquefaction, based on their relatively fine grain size distribution. The effects of possible localized liquefaction impacts will be appropriately mitigated by supporting the appropriate portions of the facility with a pile foundation as planned. Seismic settlement would impose downdrag forces on pile foundations if surrounding soils settle during a seismic event. Our pile foundation recommendations in Section 4.5 include consideration to downdrag forces.

The City of Algona and King County Sensitive Areas Ordinances identify the slope west of the Algona facility as an erosion and landslide hazard. Seismically induced landslide hazards were discussed in detail and assigned a moderate hazard in the 1997 HWA report. We concur with this evaluation, and with the statement that slope movement resulting from a seismic event would likely result in the accumulation of slide debris at the toe of the slope. We understand that measures have been taken to reduce the potential for future instability of the hillside (HWA, 1997).

4.3 SITE PREPARATION AND EARTHWORK

The proposed developments will require a certain amount of site preparation and earthwork activities related to construction of foundations and pavement areas.

4.3.1 Excavation and Temporary Shoring

Excavations will be required for the project to facilitate construction of foundation elements, possibly for underground utilities, and for other purposes. Based on the soil conditions observed in our explorations, we anticipate that the on-site soils can be excavated with conventional excavating equipment; however, care must be taken during construction to maintain stability of open excavations. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Any temporary cuts in excess of 4 feet in height should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155, or shored. The contractor is responsible for design of shoring. The existing fill materials generally classify as Type C Soil and, for planning purposes, temporary excavation side slopes may be assumed as steep as 1½ to1 (horizontal to vertical). This temporary slope inclination is applicable to excavations above the water table only. If groundwater is encountered during construction, dewatering may be required to lower the groundwater table below the base of the excavation.

Groundwater is not expected to be encountered at the Houghton or Renton sites. If excavations at the Algona site are performed deeper than about 5 feet, the contractor should anticipate encountering groundwater. If dewatering is necessary, a dewatering plan should be developed and implemented by the contractor, to enable completion of excavations in the dry. The type of dewatering system used will depend on the

contractor's methods, the depth of excavation below water and other factors. Regardless of the dewatering system used, it should be installed and operated such that natural soils are prevented from being removed along with the groundwater.

If groundwater is encountered within sand deposits during construction, reasonable care should be taken to prevent groundwater from flowing in from the bottom of the excavation, thereby creating a "quick" condition. Under quick conditions, the density of the natural soils will be reduced, resulting in increased settlement during and after construction. To reduce the risk of creating a quick condition, we recommend the groundwater level be kept at least 2 feet below the bottom of the excavation in areas where sand is encountered.

4.3.2 Subgrade Preparation

Subgrade preparation in areas supporting new structures and pavement should begin with the removal of all deleterious matter, asphalt, and concrete. The exposed subgrade soils should be evaluated by the geotechnical engineer. For large areas, this evaluation should be performed by proof-rolling the exposed subgrade with a fully loaded dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with imported structural fill, as described below. The depth of overexcavation, if required, should be evaluated by a qualified geotechnical engineer at the time of construction.

4.3.3 Structural Fill Materials and Compaction

Based on the results of our field exploration and laboratory testing program, it is anticipated that some on-site soils may be suitable for re-use as structural fill. The onsite soils at the Renton facility are relatively clean (low in fines content), and are considered a suitable all-weather fill. Onsite soils at the Houghton facility would be suitable for reuse as structural fill during dry periods, but the fines content (generally between 5 and 15 percent) will limit the usefulness of onsite soils during extended wet weather conditions. All near surface soils at the Algona facility are relatively high in fines content, or may contain deleterious materials, and are therefore not considered suitable for reuse. Any structural fill materials needed for the Houghton facility during wet weather construction and all structural fill materials at the Algona facility should be imported. Table 4 provides a summary of our conclusions and recommendations for reuse of onsite soils as structural fill.

Imported fill materials should meet the requirements for Gravel Borrow, as described in Section 9-03.14 of the 1998 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT *Standard Specifications,* 1998). Soils with fines contents higher than 7 percent may be acceptable if the earthwork is performed during

dry weather and the contractor's methods are conducive to proper compaction of the soil. The use of materials with fines contents in excess of 7 percent should be evaluated by the engineer on a case-by-case basis.

Table 4 – Reuse of Onsite Soils as Structural Fill

Facility	Anticipated Onsite Soil Classification*	Potential for Reuse as Structural Fill
Houghton	SP, SP-SM	Moderate potential for reuse. May present difficulties during extended wet weather events.
Renton	GP, SP	Very low fines content. Good all weather fill.
Algona (West Portion)	SM	Imported fill recommended; Low potential for reuse.
Algona (East Portion)	SM, GW	Imported fill recommended; Low potential for reuse.

* Anticipated unified soil classification system designation of near surface soils

Structural fill soils should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 95 percent of the MDD, as determined using test method ASTM D 1557 (Modified Proctor).

The procedure to achieve the specified minimum relative compaction depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. When size of the excavation restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough lifts to achieve the required compaction. A sufficient number of in-place density tests should be performed as the fill is placed to verify the required relative compaction is being achieved.

Generally, loosely compacted soils are a result of poor construction technique or improper moisture content. Soils with a high percentage of silt or clay are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty soils with moisture contents too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when control of soil moisture content is not possible, the following recommendations should apply:

- Earthwork should be accomplished in small sections to minimize exposure to wet weather. Excavations or the removal of unsuitable soil should be followed immediately by the placement and compaction of a suitable thickness of clean structural fill, as described below. The size of construction equipment used may have to be limited to prevent soil disturbance;

- Material used as trench backfill should consist of clean, granular soil, of which not more than 5 percent by dry weight passes the U.S. Standard No. 200 sieve, based on wet sieving the fraction passing the ¾ inch sieve. The fines should be non-plastic;

- The ground surface in the construction area should be sloped and sealed with a smooth drum roller to promote rapid runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent ponding of water;

- No soil should be left uncompacted so it can absorb water. Soils which become too wet for compaction should be removed and replaced with clean granular materials; and

- Excavation and placement of fill should be observed on a full time basis by a person experienced in wet weather earthwork to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved.

The above recommendations for wet weather earthwork should be incorporated into the contract specifications.

4.4 DRAINAGE AND EROSION CONSIDERATIONS

Surface runoff and erosion at the transfer station facilities can be controlled during construction by careful grading practices and observance of best management practices (BMPs). Such practices typically include the construction of shallow, upgrade perimeter ditches or low earthen berms, and the use of temporary sumps to collect runoff. Erosion at the sites during construction can be minimized by judicious use of straw bales and silt fences. If used, these erosion control devices should be in place and remain in place throughout construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly revegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets.

Permanent erosion control measures should be implemented at the Renton facility to reduce the potential for future erosion events. During our reconnaissance, two stormwater drain pipes were observed discharging onto the slope face. Those pipe outlets should be extended (tight lined) to a discharge point beyond the toe of the slope. The outlets should be protected with a suitable thickness of hand placed rip rap, splash mat or other appropriate means. Denuded areas should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

4.5 FOUNDATIONS

The facility improvements will include the construction of both shallow and deep foundations. Shallow foundations will be utilized at the north end of the facility at Houghton, the entire facility at Renton, and the northwestern corner of Algona. Pile foundations will be utilized in the remaining areas. Design footing types for specific locations can be evaluated once the location of the footing is known. We anticipate the use of augercast pile foundations at the southern portion of the Houghton facility, and either drilled micropiles or driven steel pipe piles at the eastern portion of the Algona facility.

4.5.1 Shallow Foundations

Spread footing foundations may be used to support the new structural elements at the aforementioned locations. Design parameters for shallow foundations at the three transfer station sites are summarized below on Table 5. The new footings at the sites should be designed using the allowable bearing pressures listed in Table 5 to limit differential settlement. Subgrades for new footings should be prepared as recommended in Section 4.3.2; structural fill should be placed as recommended in Section 4.3.3.

The spread footing along the western edge of the Renton facility should be situated so that the edge of the footing is at least 12 feet from the crest of the adjacent slope to reduce the potential of the footings being undermined as a result of slope instability during a seismic event.

The recommended maximum allowable bearing pressure assumes the undisturbed, native foundation bearing soils listed in Table 5. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. All exterior footings should be founded at least 18 inches below the

lowest adjacent finished grade; interior footings may be founded a minimum of 12 inches below top of slab. We recommend minimum footing widths of 18 and 24 inches for continuous strip and isolated column footings, respectively.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about ½ inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ¼ inch. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of values listed in Table 5 may be assumed along the interface between the base of the footing and subgrade soils. The recommended passive earth pressures listed in Table 5 may be assumed for soils adjacent to footings or other below-grade elements. The upper 1-foot of passive resistance should be neglected in design, unless the footing is protected by a floor slab or pavement. These values were calculated using a safety factor of approximately 1.5. Note that separate passive resistance values are provided for the Algona and Houghton facilities for lateral resistance of shallow foundations against undisturbed natural soil, and for appropriately designed grade beams or pile caps against structural fill, as discussed in Section 4.5.2.

Table 5 – Shallow Foundation Design Parameters

Facility	Foundation Bearing Soil(1)	Allowable Bearing Pressure(2) (psf)	Allowable Friction Factor(3)	Allowable Passive Resistance(4) (pcf)
Houghton	Advance Outwash	2,500	0.40	250
Houghton	Structural Fill(5)	Not Applicable	Not Applicable	200
Renton	Recessional Outwash	3,000	0.40	250
Algona	Glacial Till	3,000	0.33	250
Algona	Structural Fill(5)	Not Applicable	Not Applicable	200

Notes:

(1) Foundation bearing soils should be undisturbed.

(2) Allowable bearing pressure includes factor of safety of 3 to limit settlement.
(3) Allowable friction factor includes factor of safety of 1.5
(4) Allowable passive resistance includes factor of safety of 1.5 to limit deflection under lateral loading.
(5) Structural fill as defined in Section 4.3.3 of this report.

4.5.2 Deep Foundations

Deep foundations are anticipated at the south side of the Houghton facility and the east side of the Algona facility. Auger cast piles are recommended at selected locations for the Houghton facility, and either micropiles or driven steel pipe piles are recommended at selected locations for the Algona facility. The following sections present our recommendations for auger cast piles, drilled micropiles, and driven steel pipe piles.

Houghton Transfer Station

To avoid the risk of possible settlement of soil fill beneath the Houghton facility, we recommend the use of auger cast piles to provide adequate bearing support for the proposed canopy. Auger cast piles are not considered suitable for the Algona facility due to overhead clearance constraints, and the potential for construction difficulties associated with this pile type in peaty soils. The piles should be constructed to a minimum depth of 30 feet below existing grades to insure bearing in the dense advance outwash unit at depth. Assuming the auger cast piles are constructed to a minimum depth of 30 feet, allowable vertical capacity will be dependent on the as-constructed pile diameter. For a 12-inch diameter auger cast pile, an allowable vertical capacity of 40 tons in axial compression and 22 tons in axial tension may be assumed for each pile. For a 16-inch diameter pile, an allowable vertical capacity of 70 tons in axial compression and 28 tons in axial tension may be assumed. Piles should be constructed with a minimum center-to-center spacing of 3 pile diameters to maintain the full stated vertical capacity for each pile. The above allowable capacities include a factor of safety of 3, and are based on soil strength. Structural features of the piles could impose limitations on the available pile capacity; therefore, the structural engineer should verify the structural capacity of the piles.

The behavior of piles under lateral load was evaluated using the LPILE computer program (Reese et al, 1997), assuming both free and fixed head conditions, and vertical loads equal to the allowable vertical capacity for each pile. The results of the lateral capacity analyses are presented in Appendix D, and are presented as shear, moment, and deflection diagrams. The lateral pile response results in Appendix D are based on the soil parameters listed below in Table 6, and do not include a factor of safety.

Table 6 - Soil Parameters Used in LPILE Analyses – Houghton Transfer Station

Depth (BGS)		USCS Soil Type	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (psf)	Soil Strain Parameter, E_{50}	Soil Modulus Parameter, k (pci)
From	To						
0	19.5	SP, SP-SM	116	30	0	0	90(5)
19.5	30	SP-SM	132	40	0	0	225

Notes:
1) BGS = below existing ground surface
2) Maximum pile depth assumed as 30 BGS
3) Groundwater assumed below bottom of pile
4) pcf = pounds per cubic ft.; deg = degrees; psf = pounds per square ft.; pci = pounds per cubic in.
5) p-y criteria for sand (Reese et. al., 1974) applies to all soil layers

Auger cast piles are installed by drilling with a continuous flight hollow stem auger to the required depth, and pumping grout through the hollow stem as the auger is withdrawn. Once the auger is completely removed, steel reinforcement is placed in the grout-filled borehole. The rate at which the auger is withdrawn must be consistent with grout supply. If the auger is withdrawn too quickly, the pile will be under-grouted, resulting in "necking" of the pile, or contamination of grout materials with caving soil.

The quality of auger cast piles is highly dependent on the procedures and workmanship of the contractor who installs them; therefore, an experienced contractor is a necessity. Observation and monitoring of pile installations by an experienced geotechnical engineer is also recommended. A properly functioning pressure gage and pump stroke counter or flow meter should be provided on the grout pump to assist in monitoring auger cast pile installation. The pressure gage located at the pump is used to monitor the pressure of the grout to evaluate the rate at which the auger should be retracted, and if the auger or hoses

are plugged. The auger should be withdrawn with slow positive rotation at a slow steady pull and should not be pulled until the grout has been pumped several feet above the tip. The counter is used to determine the approximate volume of grout pumped by counting the number of strokes of a displacement-type pump. The pump should be calibrated prior to its use.

If the proposed structures are supported by a deep foundation system designed and constructed as recommended herein, total and differential settlements are anticipated to be within tolerable limits. Under the assumed loading conditions, we estimate less than about ½-inch total, and about ¼-inch differential settlement between adjacent foundation elements.

Algona Transfer Station

The existing timber pile foundation system on the eastern portion of the Algona facility will be augmented as part of the project. New piles will be installed to support the new canopy, which we understand will impose foundation loads on the order of 70 tons at each column location. Two approaches are possible for installing the additional piles. One approach is to install piles from beneath the existing tipping floor, which has an overhead clearance of about 15 feet. For this approach, we recommend micropiles. We envision 3 to 4 micropiles would be required at each column location. Another approach is to open holes through the tipping floor and install new driven piles from above. We envision 12-inch outside diameter steel pipe piles (2 at each column location) would be appropriate for this approach. The following paragraphs provide discussion and recommendations for piles using these two approaches.

Micropiles

Micropiles are small (4 to 12-inch) diameter, bored, grouted-in-place piles incorporating steel reinforcement. These piles are designed, installed and tested in a similar manner to tiebacks and soil nails. They are typically contractor-designed, based on a performance (capacity) specification. A non-production test pile is typically installed and destructively tested prior to production. Testing of a non-production pile allows the contractor to adjust the design if necessary. This initial pile is typically loaded to approximately two times the design load to confirm bearing capacity. Additionally, a specified percentage of production piles are tested in a non-destructive manner, similar to production testing of tiebacks. These piles are usually tested to lesser loads than the non-production test piles, typically 130 to 167% of the design load.

The installation methods are such that micropiles develop a high grout to ground bond along the periphery of the pile. To take advantage of this benefit, high capacity steel reinforcement elements are typically incorporated into the pile design, which serve as the principle load bearing element and sometimes can occupy up to 50 percent of the hole

volume. Micropiles are generally regarded as friction piles; end bearing is typically not relied upon.

The installation methods for micropiles allows their use in limited access and overhead situations. These piles can be installed with less than about 12 feet without special equipment adaptations, and they can be installed at any angle below horizontal (batter). Since they are drilled-in-place, they generate minimal vibration, noise and disturbance. The general construction sequence for micropiles as they apply to this project is as follows:

1) A boring (approximately 5.5-inch outside diameter) is advanced to the design depth either vertical, or at the specified inclination (in the case of battered piles). The design depth would be based on a minimum penetration into competent soils (also referred to as the bond zone). The boring is cased during drilling with flush-threaded, ½-inch wall thickness, 80 kips per square inch (ksi) steel casing in 6-foot lengths.

2) The specified reinforcement (either a solid steel bar or cage) is inserted into the drilled hole.

3) Neat cement grout (4,000 pounds per square inch [psi] or greater unconfined compressive strength at 28 days) is tremied into the boring as the casing is removed to expose the bond zone. The bond zone in this case would be the dense, glacially consolidated sediments underlying the soft alluvial (peat) sediments. The casing above the bond zone would be left in-place and become a permanent foundation element.

4) After the primary grout has cured for a sufficient time, high pressure secondary grout is then injected within the bond zone. This phase is optional, depending on the capacity requirements.

Discussion with local contractors who specialize in the installation of micropiles suggest micropiles for this project could be installed within about one to two weeks time (including mobilization and demobilization), and would cost on the order of $100 to $125 per lineal foot (including mobilization, time and materials).

We completed a preliminary capacity analysis for micropiles, based on methods suggested by Xanthakos et al (1994). We emphasize that these piles should be contractor designed, and their capacities verified in the field. For an allowable compressive (axial) capacity of 23.5 tons each, assuming a 3-pile array at each column location, we estimate the required vertical pile length to be on the order of 40 to 45 feet below ground surface, or 15 feet into the load bearing zone. This assumes a pile-soil adhesion value on the order of 2 ksf, and should be verified with a field testing program. For battered piles, this

length would require adjustment to account for the inclination. The uplift (tension) capacity would be equivalent to the compressive capacity. This estimated pile length is based on bond zone soil strength, and assumes a factor of safety of 3 for allowable axial loads. Structural features of the piles could impose limitations on the available pile capacity; therefore, the structural engineer should verify the structural capacity of the piles for both the working and testing loads.

The compressible soils above the bond zone will continue to settle under their own weight over the life of the facility, mobilizing a downward frictional load on the portions of the pile embedded in those layers. This downdrag force would be minimized by keeping the steel casing permanently in-place above the bond zone. The above stated estimation of pile length considers an approximately 1.5-ton downdrag force.

Being small in diameter, lateral capacity of micropiles is relatively small when compared to larger diameter piles. Lateral resistance can be gained from passive soil pressure acting against the pile caps, using the design parameters provided in Section 4.5.1 and Table 5. For added lateral capacity, micropiles are typically installed battered so a component of the axial capacity can be utilized to resist lateral loads.

The behavior of vertical micropiles under lateral load was evaluated using the LPILE computer program (Reese et al, 1997), assuming both free and fixed head conditions, and vertical loads equal to the allowable vertical capacity for each pile. The results of the lateral capacity analyses are presented in Appendix D, and are presented as shear, moment, and deflection diagrams. The lateral pile response results in Appendix D are based on the soil parameters listed below in Table 7, and do not include a factor of safety.

Table 7 - Soil Parameters Used in LPILE Analyses – Algona Transfer Station

Depth (BGS)		USCS Soil Type	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (psf)	Soil Strain Parameter, E_{50}	Soil Modulus Parameter, k (pci)
From	To						
0	7	GW, SM	120	34	0	0	175(5)
7	23	PT	0	0	250	0.030	15(6)
23	27	GW	62	32	0	0	60(5)
27	46	SM	78	38	0	0	125(5)

Notes:
1) BGS = below existing ground surface
2) Maximum pile depth assumed as 46 feet BGS

3) Groundwater level assumed at 7 feet BGS
4) pcf = pounds per cubic ft.; deg = degrees; psf = pounds per square ft.; pci = pounds per cubic in.
5) p-y criteria for sand (Reese et. al., 1974)
6) p-y criteria for soft clay

The piles will penetrate potentially corrosive soils (peat), and they should be protected to inhibit corrosion. Corrosion protection using a protective coating would not be practical for this application since the piles are drilled-in-place with the drill casing acting as a permanent element of the pile. Therefore, we recommend corrosion protection be considered in design either by providing cathodic protection or by including sacrificial steel and/or corrosion resistant concrete.

The piles will be drilled through compressive soils above the bond zone. We believe that the drilling equipment is stiff enough, and the peat is firm enough to obtain a reasonably constant batter for the anticipated pile depths.

If micropiles are selected as the preferred alternative, a field verification testing program should be developed and specified in the contract documents. One non-production test pile should be installed for performance testing. We recommend conducting tension tests, since compression testing requires the installation of two or more anchor piles and is more expensive and time consuming than tension testing. Tension testing requires no additional piles and is believed to be appropriate since micropiles are considered friction piles with the tensile capacity equaling the compressive capacity. Tension testing is typically completed in a similar manner to tieback testing, with a performance test being conducted on the dedicated test pile, and production testing being completed on one pile at each column location. Testing would be completed in incremental fashion, in general accordance with the recommendations presented by the Post-Tensioning Institute (PTI, 1996). The performance test pile should be installed using the same equipment that will complete the production piles. The actual lengths of the production piles should be based on the results of the performance testing program. One pile at each column location should be proof-tested to a load that slightly exceeds the design load. The results of the proof tests should be compared to the results of the performance tests. If any significant variation from the performance test results is observed, as determined by the Engineer, then the design capacity of this and subsequent piles should be re-evaluated. The elastic deformation and the permanent set for each load increment should be measured to aid in the interpretation of the behavior of the micropile.

Driven Steel Pipe Piles

In our opinion, driven steel pipe piles may be a suitable alternative for the new canopy foundation. These piles would have a higher capacity and greater stiffness than micropiles. Because of their higher capacity, we envision two piles installed at each column location. However, since they must be installed from above the tipping floor

with a crane and drive hammer, they are likely to interrupt other construction activities and operations more than micropiles. Their method of installation is also more likely to impact existing improvements at the site from vibration.

We evaluated the axial capacities (compression and uplift) of 12-inch diameter, 32-foot long driven steel pipe piles using the SPILE computer program (Urzua, 1993). The piles should be constructed to a minimum depth of 32 feet below existing grades and a minimum penetration of 5 feet into dense glacially overridden soils to insure adequate bearing capacity, and to develop a suitable for lateral load resistance. To achieve this penetration, we recommend steel pipe piles be driven open ended. The depth to glacially overridden soils is expected to change across the site, but it should be relatively close to the depth depicted in our boring. Assuming the steel pipe piles are constructed to a minimum depth of 32 feet, we expect the allowable vertical capacity of 49 tons (compression) and 5 tons (tension) may be assumed for each pile. These values consider a downdrag load of about 3.5 tons. Piles should be constructed with a minimum center-to-center spacing of 3 pile diameters to maintain the full stated vertical capacity for each pile. The above allowable capacities include a factor of safety of 3, and are based on soil strength. Structural features of the piles could impose limitations on the available pile capacity; therefore, the structural engineer should verify the structural capacity of the piles. As with micropiles, we recommend driven steel pipe piles be protected with corrosion protection measures.

The behavior of plain (no concrete in-filling), open-end steel pipe piles under lateral load was evaluated using the LPILE computer program (Reese et al, 1997), assuming both free and fixed head conditions, and vertical loads equal to the allowable vertical capacity for each pile. The soil parameters listed above in Table 7, which do not include a factor of safety, were used in this analysis. Our analysis assumed 12-inch outside diameter, ½-inch wall thickness, grade A36 steel piles. The results of the lateral capacity analyses are presented in Appendix D, and are presented as shear, moment, and deflection diagrams. The lateral pile response results in Appendix D.

If driven steel pipe piles are selected for support of the column loads, we recommend a wave equation analysis of pile driving (WEAP) be performed prior to construction to determine refusal criteria for pile driving. This analysis is performed once the pile section and pile driving equipment is known, to make sure that the pile will not be overstressed during driving. We can supply suitable forms for the contractor to indicate in advance what equipment they will use for pile driving. This information, which would be used in the WEAP analysis, should be supplied at least one week in advance of construction.

5.0 DOCUMENT REVIEW AND CONSTRUCTION SUPPORT

Our subconsultant agreement includes time for PacRim to review the final Contract Documents to verify that the recommendations presented herein have been interpreted and implemented as intended. If micropiles are selected as the preferred alternative for the Algona facility, we should be consulted during the development of the specifications pertaining to that portion of the work. In addition, PacRim should be retained during construction to review the geotechnical aspects of Contractor submittals for pile driving, dewatering and shoring, as necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided by PacRim during construction to confirm that the conditions encountered are consistent with those indicated by explorations, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated. Construction consultations would include addressing geotechnical issues during construction when requested by ABKJ or the Owner

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03/29/02

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6.0 REFERENCES

American Society for Testing and Materials, 1998, Annual Book of ASTM Standards, Vol. 4.08, Soil and Rock (1): D420-D4914, Philadelphia: ASTM.

Hong West and Associates, Inc., 1992, *Geotechnical Investigation, Houghton Transfer Station Queuing Improvements, Kirkland, Washington*, Consultant report prepared for R.W. Beck & Associates.

Hong West and Associates, Inc., 1997, *Geotechnical Report, King County Transfer Stations, Seismic Evaluation, First Ave. NE, Houghton, Factoria, and Renton Facilities, King County, Washington*, Consultant report prepared for ABKJ, Inc.

International Conference of Building Officials, 1997, *Uniform Building Code, Structural Engineering Design Provisions,* Vol. 2.

Post-Tensioning Institute, 1996. Recommendations for Prestressed Rock and Soil Anchors, Post-Tensioning Institute, U.S.A.

Reese, L.C., Wang, S. T, Arrellaga, J. A., Hendrix, J., 1997, Documentation for Computer Program LPILE Plus, Version 3.0 for Windows, Ensoft, Inc., Austin, Texas.

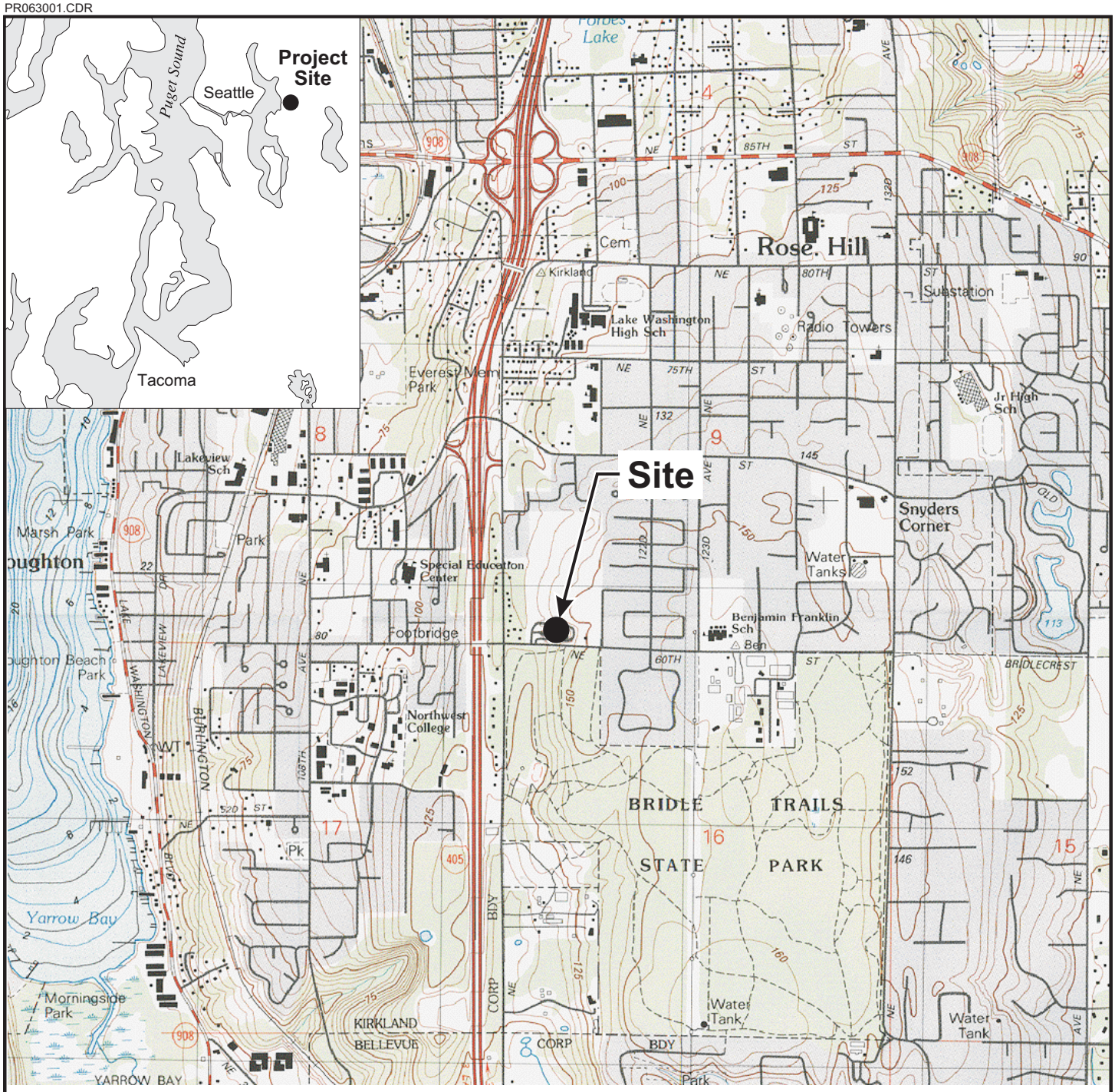
Urzua, A., 1993, SPILE, *A Microcomputer Program for Determining Ultimate Vertical Static Pile Capacity*, Publication No. FHWA-SA-92-044, U.S. Department of Transportation, Federal Highway Administration, Washington D.C.

Washington State Department of Transportation, 1998, *Standard Specifications for Road, Bridges, and Municipal Construction*.

Wright, S. G., 1990, Documentation for Computer Program UTEXAS3, A Computer Program for Slope Stability Calculations, Shinoak Software, Austin, Texas.

Xanthakos, P. P., Abramson, L. W., and Bruce, D. A., 1994, *Ground Control and Improvement*, John Wiley & Sons, Inc., New York, 910 pages.

PR063001.CDR



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SCALE IN FEET

0 2,000 4,000

King County Solid Waste Transfer Stations
King County, Washington

ABKJ, Inc.

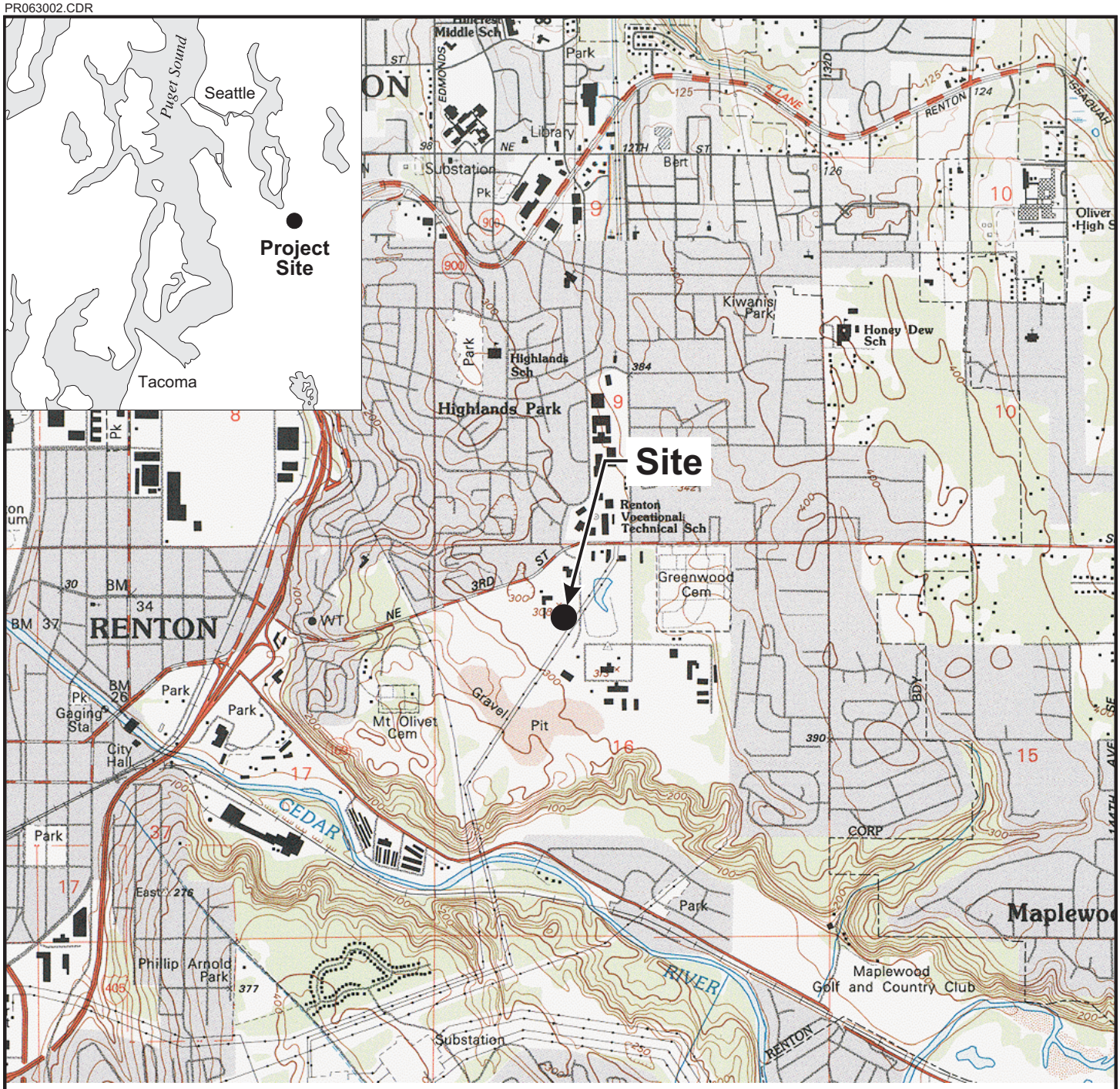
PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

VICINITY MAP
HOUGHTON TRANSFER STATION

Project No.: 092-004

FIGURE 1a

PR063002.CDR



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SCALE IN FEET

0 2,000 4,000

King County Solid Waste Transfer Stations
King County, Washington

ABKJ, Inc.

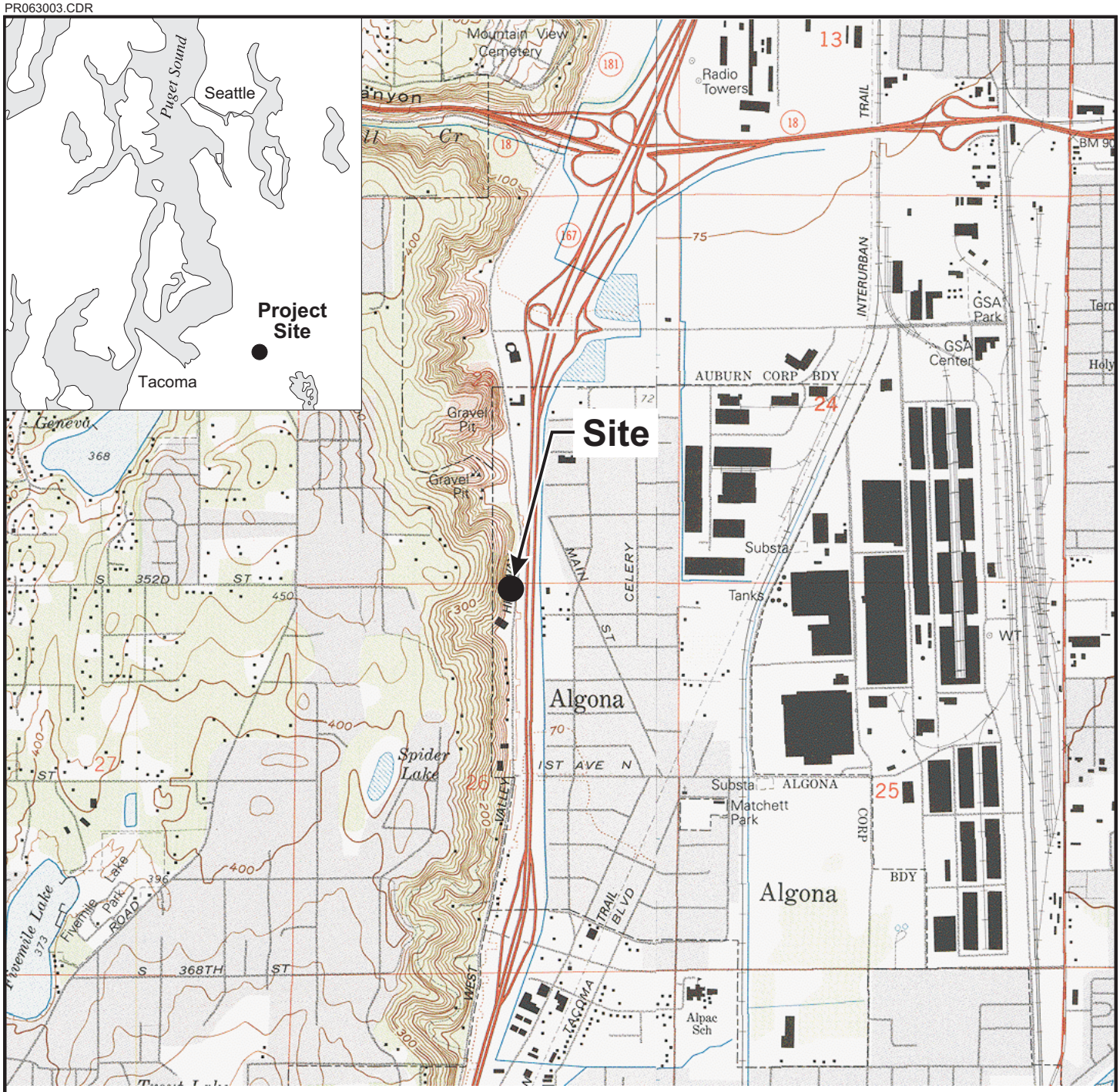
PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

VICINITY MAP
RENTON TRANSFER STATION

Project No.: 092-004

FIGURE 1b

PR063003.CDR



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SCALE IN FEET

0 2,000 4,000

King County Solid Waste Transfer Stations
King County, Washington

ABKJ, Inc.

PACRIM GEOTECHNICAL INC.

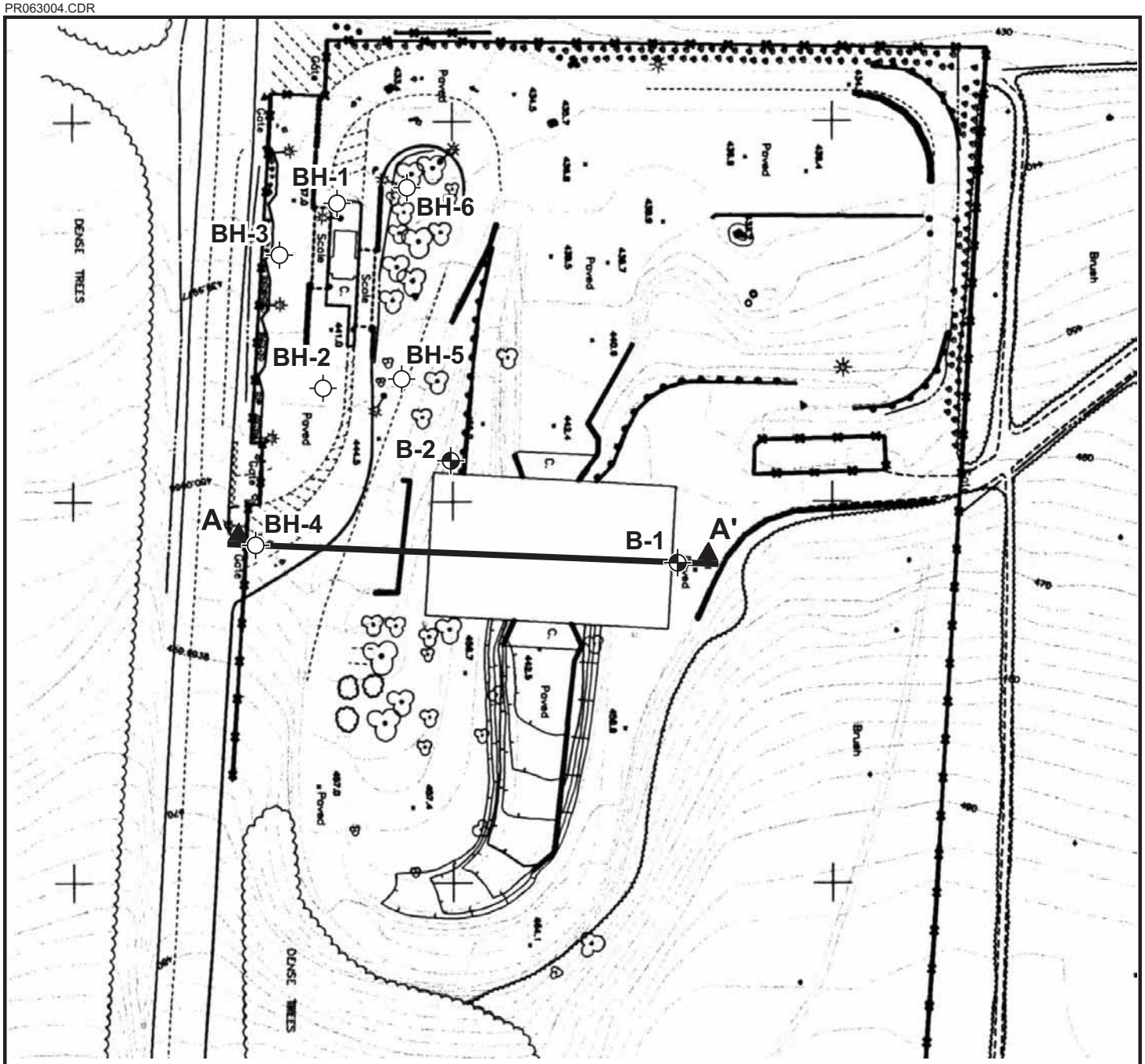
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

VICINITY MAP
ALGONA TRANSFER STATION

Project No.: 092-004

FIGURE 1c

PR063004.CDR



LEGEND

Approximate location and designation of borings completed for this study

Approximate location and designation of borings completed by others (HWA, 1992)

A A' Location of subsurface cross section (see Figure 3)

SOURCE: Degross Aerial Mapping for King County Solid Waste

SCALE IN FEET

0 100 200

King County Solid Waste Transfer Stations
King County, Washington

ABKJ, Inc.

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

SITE AND EXPLORATION PLAN FOR HOUGHTON TRANSFER STATION

Project No.: 092-004

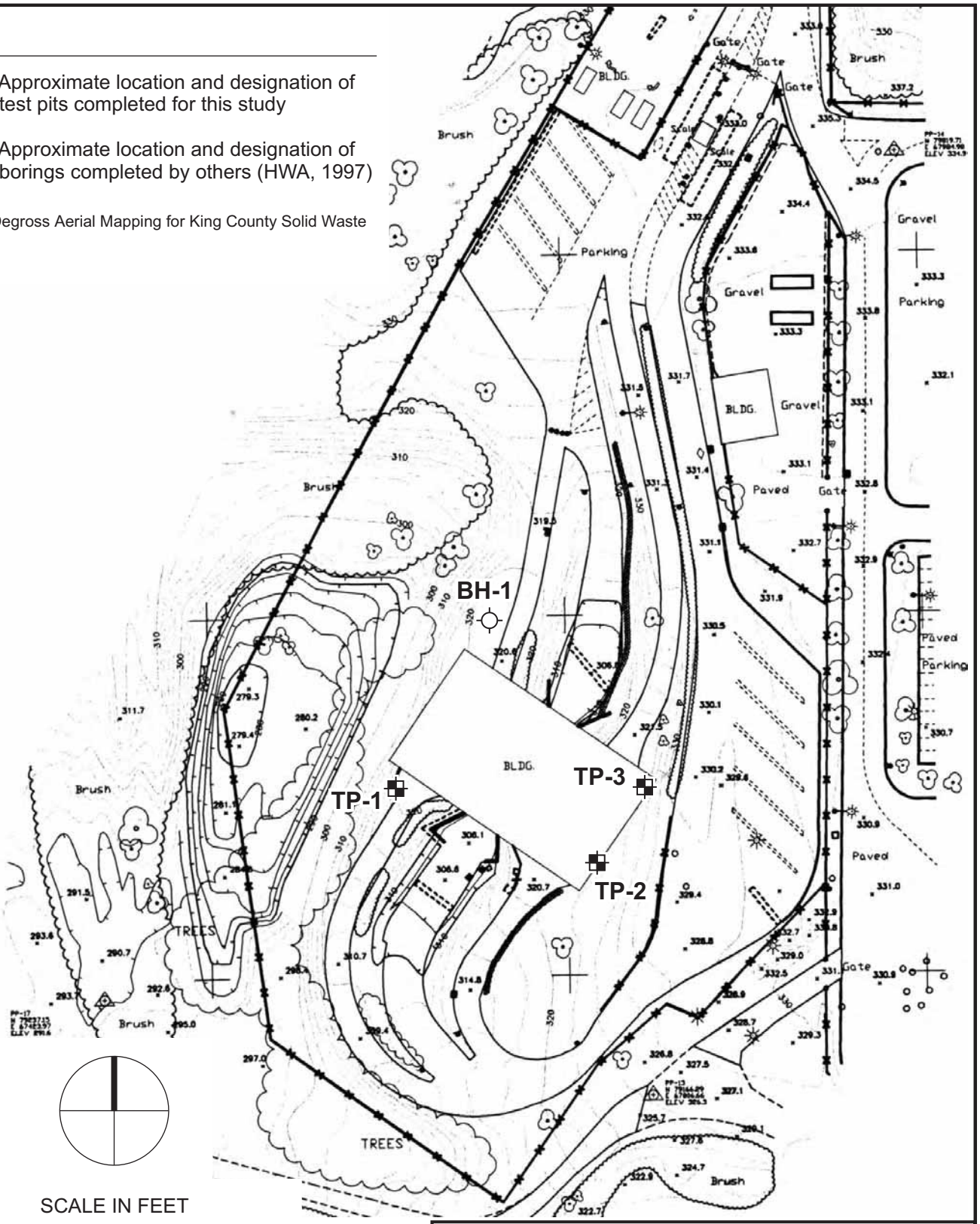
FIGURE 2a

PR063005.CDR

LEGEND

- Approximate location and designation of test pits completed for this study
- Approximate location and designation of borings completed by others (HWA, 1997)

SOURCE: Degross Aerial Mapping for King County Solid Waste



SCALE IN FEET

0 100 200

King County Solid Waste Transfer Stations
King County, Washington

ABKJ, Inc.

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

SITE AND EXPLORATION PLAN FOR RENTON TRANSFER STATION

Project No.: 092-004

FIGURE 2b

PR063006.CDR

LEGEND

Approximate location and designation of borings completed for this study

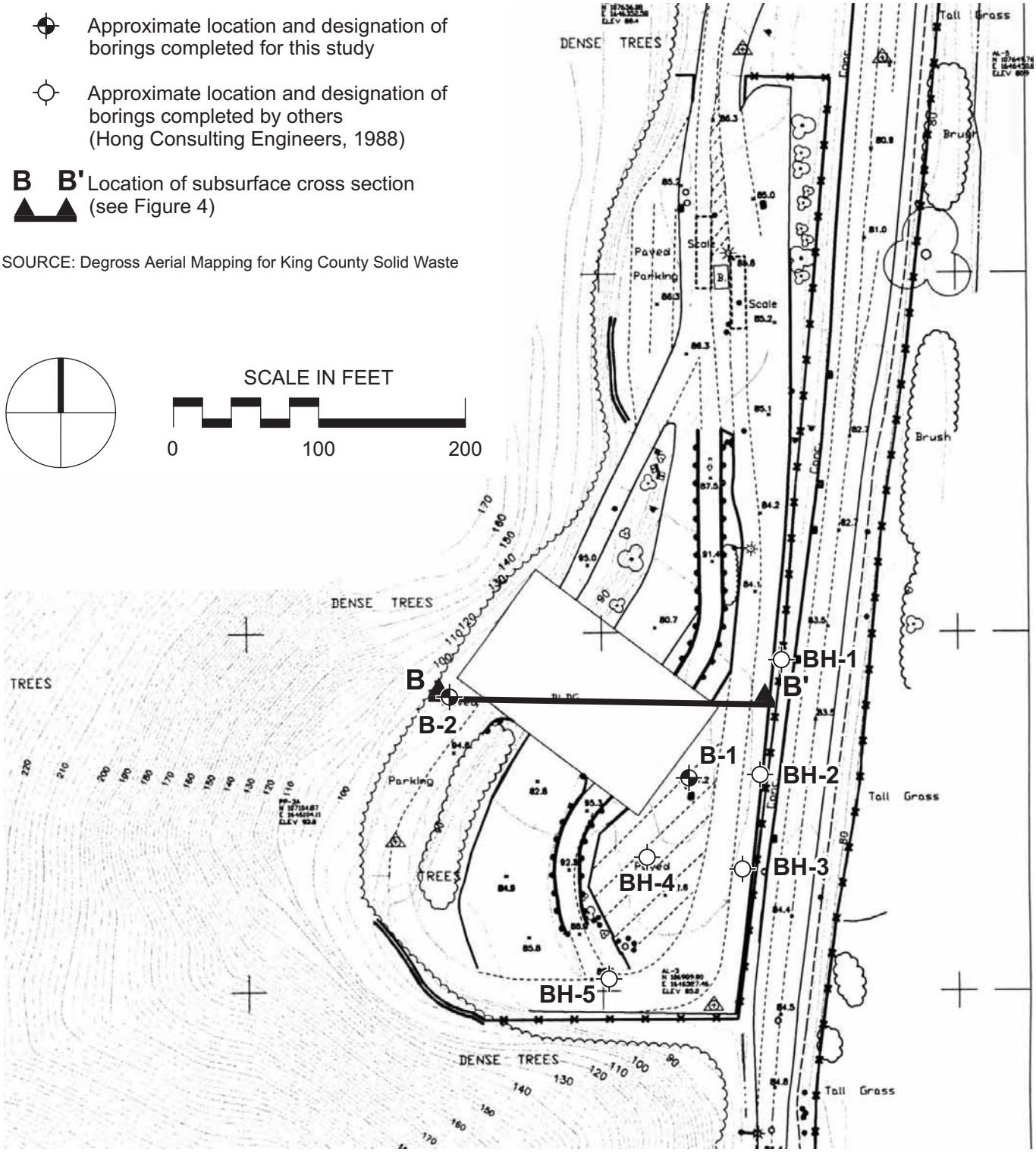
Approximate location and designation of borings completed by others (Hong Consulting Engineers, 1988)

B B' Location of subsurface cross section (see Figure 4)

SOURCE: Degross Aerial Mapping for King County Solid Waste

SCALE IN FEET

0 100 200



King County Solid Waste Transfer Stations
King County, Washington

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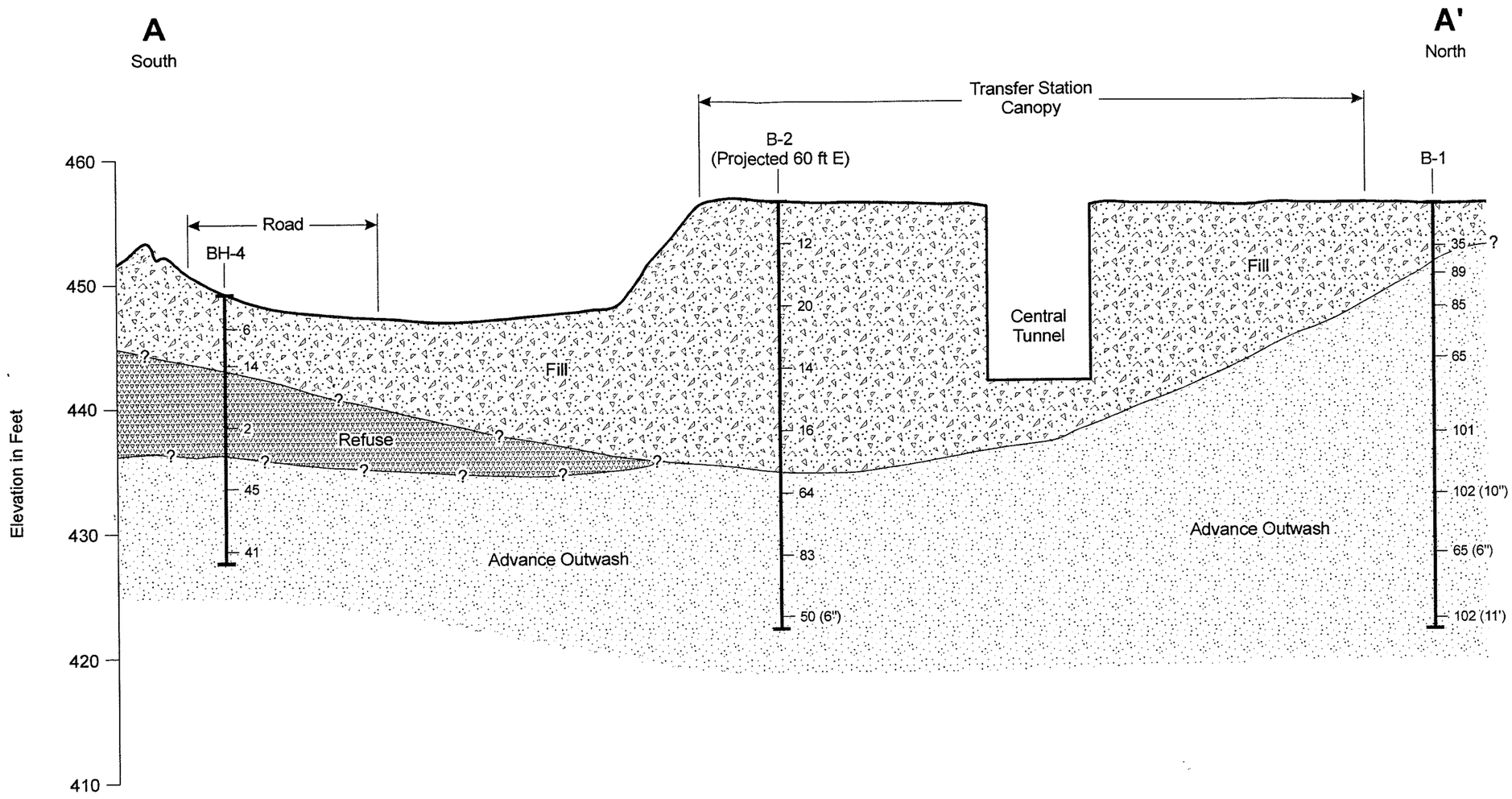
PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

SITE AND EXPLORATION PLAN FOR ALGONA TRANSFER STATION

Project No.: 092-004

FIGURE 2c

PR063007.CDR



LEGEND

B-1 Boring location

65 SPT N-Value

DEPOSITIONAL UNITS	PREDOMINANT CHARACTERISTICS (USCS SYMBOL)
Fill	Loose to medium dense sand, sand with silt, silty sand and clayey sand. (SP, SW, SP-SM, SM)
Refuse	Municipal solid waste of varying composition mixed with sandy soil.
Advance Outwash	Dense to very dense sand and sand with silt, with varying amounts of gravel. (SP, SW, SP-SM)

NOTES

1. Refer to Figure 2a for location of section.
2. Groundwater not encountered in borings.

HORIZONTAL SCALE IN FEET
VERTICAL EXAGGERATION = 3x

0 30 60

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

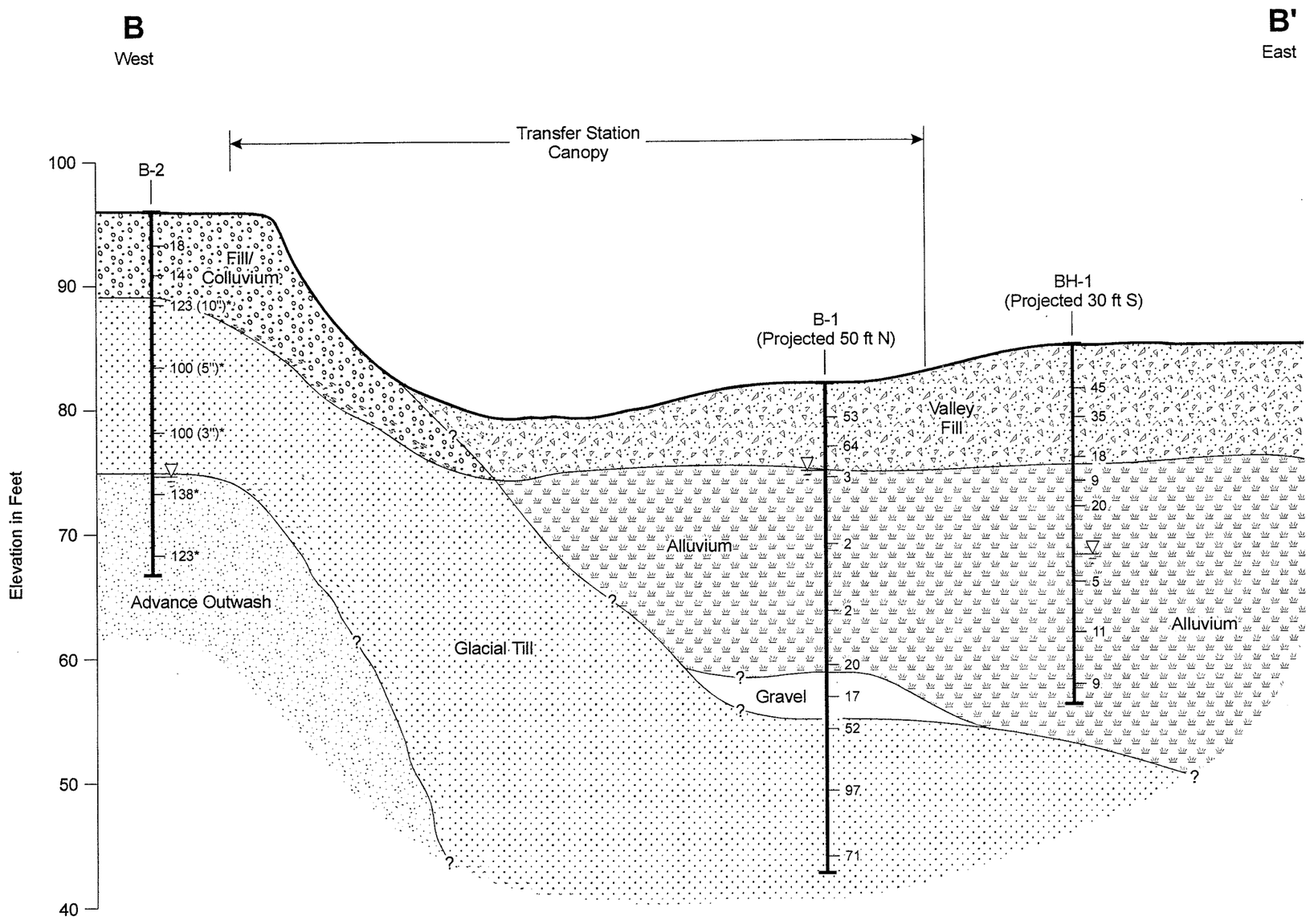
King County Solid Waste Transfer Stations
King County, Washington
ABKJ, Inc.

GENERALIZED SUBSURFACE SECTION
HOUGHTON TRANSFER STATION

Project No.: 092-004

FIGURE 3

PR063008.CDR



DEPOSITIONAL UNITS	PREDOMINANT CHARACTERISTICS (USCS SYMBOL)
Fill/Colluvium	Medium dense, silty sand with gravel. (SM)
Valley Fill	Medium dense to dense, sand with gravel, gravel with sand, varying amounts of silt. (SM, SW-SM, GW)
Alluvium	Soft peat, silty sand and clayey silt, varying amounts of gravel, scattered layers of sand and gravel. (PT)
Glacial Till	Very dense, silty sand with gravel, sand with silt. (SP-SM, SM)
Advance Outwash	Very dense sand with silt, with varying amounts of gravel. (SW-SM)

NOTES

1. Refer to Figure 2c for location of section.
2. SPT N-values with "*" notation represent blows per last foot (or fraction thereof) using a 140 lb. hammer falling 30 inches and a 3-inch OD/2.5-inch ID sampler.

LEGEND

B-1 Boring location

65 SPT N-Value

Groundwater level at time of drilling

HORIZONTAL SCALE IN FEET
VERTICAL EXAGGERATION = 3x

0 30 60

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

King County Solid Waste Transfer Stations
King County, Washington
ABKJ, Inc.

GENERALIZED SUBSURFACE SECTION
ALGONA TRANSFER STATION

Project No.: 092-004

FIGURE 4

APPENDIX A

FIELD INVESTIGATIONS

APPENDIX A

FIELD INVESTIGATIONS

DRILLING AND TEST PITS

Subsurface conditions were explored using hollow stem auger and mud rotary drilling techniques, and by excavated test pits. Four borings and three test pits were completed at selected locations within the project areas. At the Renton Transfer Station, three test pits were excavated to depths ranging from 5 to 6.5 feet on February 22, 2000. Two boring were completed to depths of 34 feet each at the Houghton Transfer Station on February 23, 2000, and two borings were completed drilled to depths of 29 feet and 39 feet at the Algona Transfer Station on February 24, 2000. The approximate locations of the explorations are illustrated on Figure 2 in the main body of the text.

The borings were drilled by Geo-Tech Explorations of Tualatin, Oregon, and the test pits were completed by Custom Backhoe of Bellevue, Washington, under subcontract to PacRim. The equipment and exploration methods utilized for the borings and test pits are summarized on the individual summary boring and test pit logs, which are included in this appendix as Figures A-2 through A-8. A key to the symbols and terms used on the summary logs is presented as Figure A-1.

Soil samples were obtained from all borings at depth intervals ranging from 2.5 to 5 feet using a 2 inch OD Standard Penetration Test Sampler, and a 3-inch outside diameter (2½-inch inside diameter) split barrel sampler. Sampler types used for each boring are graphically indicated on the individual summary logs at the appropriate sample interval. Specific samplers used for each sample are indicated graphically on the summary logs. The sampler was driven into the soil a distance of 18 inches using a 140-pound hammer falling a distance of 30 inches. The hammer was operated using a rope-and-cathead system. Recorded blows for each 6 inches of sampler penetration (blow counts) are shown on the summary logs in this appendix. The blow counts provide a qualitative measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. For the purpose of analyses, field blow counts are corrected to Standard Penetration Test (SPT) blow counts, also referred to as N-values. Blow count corrections are based on the energy delivered to the sampler from the drive hammer, and the cross-sectional area of the sampler. SPT $N_{(60)}$-values represent SPT N-values normalized to an effective energy delivered to drill rods equal to 60 percent of the theoretical free-fall energy. In test pits TP-1, TP-2, and TP-3, disturbed soil samples were obtained at selected intervals. Representative portions of all recovered samples were placed in sealed containers and transported to our laboratory for further observation and testing.

A PacRim Geotechnical Inc. representative was present throughout the field exploration program to observe the explorations, assist in sampling, and to prepare descriptive logs of the explorations. Soils were classified in general accordance with ASTM D-2488 *Standard Practice for Description and identification of Soils (Visual-Manual Procedure)* (ASTM, 1998). The summary exploration logs represent our interpretation of the contents of the field logs and the results of laboratory testing. The stratigraphic contacts shown on the individual summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The

subsurface conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density (%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	**Gravel and Gravelly Soils** More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	**Silt and Clay**	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILTor CLAY
	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILTor CLAY
Highly Organic Soils			PT	PEAT

DESCRIPTORS FOR SOIL STRATA AND STRUCTURE

General Thickness or Spacing		Structure		General Attitude	
Parting:	less than 1/16 in.	Pocket:	Erratic, discontinuous deposit of limited extent	Near horizontal:	0 to 10 deg.
Seam:	1/16 to 1/2 in.	Lens:	Lenticular deposit	Low angle:	10 to 45 deg.
Layer:	1/2 to 12 in.	Varved:	Alternating seams of silt and clay	High angle:	45 to 80 deg.
Stratum:	greater than 12 in.	Laminated:	Alternating seams	Near vertical:	80 to 90 deg.
Scattered:	less than 1 per ft.	Interbedded:	Alternating layers		
Numerous:	more than 1 per ft.				

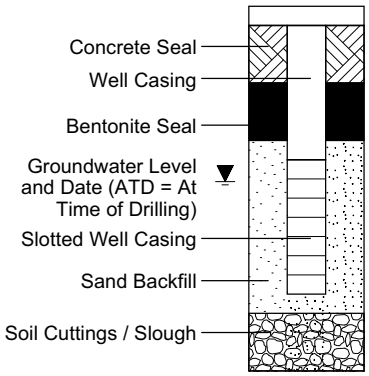
LABORATORY TEST SYMBOLS

AL	Atterberg Limits
FC	Fines Content
GSD	Grain Size Distribution
MC	Moisture Content
MD	Moisture Content/Dry Density
Comp	Compaction Test (Proctor)
SG	Specific Gravity
CBR	California Bearing Ratio
RM	Resilient Modulus
Perm	Permeability
TXP	Triaxial Permeability
Cons	Consolidation
VS	Vane Shear
DS	Direct Shear
UC	Unconfined Compression
TXS	Triaxial Compression
HYD	Hydrometer
UU	Unconsolidated, Undrained
CU	Consolidated, Undrained
CD	Consolidated, Drained

SAMPLE TYPE SYMBOLS

- Std. Penetration Test (2.0" OD)
- Ring Sampler (3.25" OD)
- California Sampler (3.0" OD)
- Undisturbed Tube Sample
- Grab Sample
- Core Run
- Non-standard Penetration Test (with split spoon sampler)

GROUNDWATER WELL COMPLETIONS



Notes:

1. Sample descriptions in this report are based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide. Where laboratory data are available, soil classifications are in general accordance with ASTM D 2487.

2. Solid lines between soil unit descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the unit.

PACRIM EXPLORATION LEGEND 092-004.GPJ PACRIM.GDT 4/1/02

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

King County Transfer Stations
King County Washington

ABKJ, Inc.

KEY TO EXPLORATION LOGS

Project No. 092-004

FIGURE A-1

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Asphalt - 4 inches thick.			
									Dense brown, fine to medium SAND, few gravel, trace silt, scattered pockets of black organics and wood fragments; moist.	455		
	1		14	8 20 15			SP		**(FILL)**			
						5			Very dense, brown SAND with silt, few fine gravel; moist; gravel is subrounded.	450		
	2		7	10 37 52					**(ADVANCE OUTWASH)**			
GSD	3		7	24 35 50					Becomes gray-brown.			
						10				445		
	4		7	16 26 39					Becomes gray; fine to medium, scattered pockets of sandy silt.			
						15				440		
GSD	5		10	23 51 50								
						20	SP-SM			435		
	6		17	36 52 50/4"					Becomes blue-gray, fine.			
						25				430		
	7		7	37 65/6"					Becomes gray-brown.			
						30				425		
	8		7	25 52 50/5"					Grades fine to medium.			
						35			Bottom of boring at 34 feet. Boring backfilled with cuttings. Top 8 inches sealed with concrete. No groundwater encountered.	420		

Surface Elevation: 456 feet
Date Completed: 2-23-00
Logged By: CJN
Equipment: Mobile Drill B-59
Drilling Method: 4-inch ID Hollow Stem Auger
Hammer System: Rope and Cathead, 140 lb. Hammer

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Houghton Transfer Station
Kirkland, Washington

ABKJ, Inc.

LOG OF BORING B-1

SHEET 1 OF 1

Project No. 092-004-001

FIGURE A-2

BORING WELL HOUGHTON.GPJ PACRIM.GDT 4/1/02

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Grass and topsoil 4 inches thick.			
									Medium dense, brown, SAND with silt, trace coarse sand, few to little gravel; moist.	455		
GSD	1		7	6 6 6					**(FILL)**			
						5				450		
	2		10	9 9 11								
						10	SP-SM			445		
	3		11	24 7 7								
						15				440		
	4		10	8 9 7								
						20				435		
									Very dense, blue-gray, fine to medium SAND with silt; moist.			
GSD	5		18	17 28 36					**(ADVANCE OUTWASH)**			
						25				430		
							SP-SM					
FC	6		21	25 41 42								
						30				425		
									Grades cleaner.			
	7		10	32 50/6"			SP					
									Bottom of boring at 34 feet. Boring backfilled using cuttings. No groundwater encountered.			
						35				420		

Surface Elevation: 456 feet
Date Completed: 2-23-00
Logged By: CJN
Equipment: Mobile Drill B-59
Drilling Method: 4-inch ID Hollow Stem Auger
Hammer System: Rope and Cathead, 140 lb. Hammer

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Houghton Transfer Station
Kirkland, Washington

ABKJ, Inc.

LOG OF BORING B-2

SHEET 1 OF 1

Project No. 092-004-001

FIGURE A-3

BORING WELL HOUGHTON.GPJ PACRIM.GDT 4/1/02

EXCAVATION COMPANY: Custom Backhoe
EQUIPMENT: Case 580
SURFACE ELEVATION: 321 feet

DATE COMPLETED: 2-22-00
LOGGED BY: CJN

Depth (feet)	Symbol	USCS	DESCRIPTION	Sample	Sample No.	Moisture Content (%)	Other Tests	Groundwater
0			Asphalt 4 inches thick.					
		GW	5/8-inch minus crushed rock base.					
		GP, SP	Medium dense, brown, GRAVEL with sand, trace silt, trace cobbles, scattered organics; moist.	G	S3			
		GP	**(FILL)**					
2			Medium dense, gray, fine to medium SAND, trace silt, trace fine gravel; moist.	G	S1			
		SP, GP	**(RECESSIONAL OUTWASH)**					
			Medium dense, brown, GRAVEL with sand, trace silt, trace cobbles; moist.					
4			Medium dense, gray, fine to medium SAND, trace silt, trace fine gravel; moist.					
			Medium dense, brown, GRAVEL with sand, trace silt, trace cobbles; moist.	G	S2	7		
6								
8								
10								

Sample S3 composited from excavation spoils removed from upper 3 ft. Sieve preformed on S3.

Test pit backfilled using native excavation spoils, compacted using bucket of backhoe and hoepack. Crushed rock base course material placed over top 6 inches, compacted using hoepack.

No groundwater encountered.

S ETC O SOUT SIDE O PIT

ORI ONT DIST NCE (feet)

0 2 4 6 8 10

Depth (feet)

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Renton Transfer Station
Renton, Washington

ABKJ, Inc.

LOG OF TEST PIT

TP-1

Project No. 092-004-002

FIGURE A-4

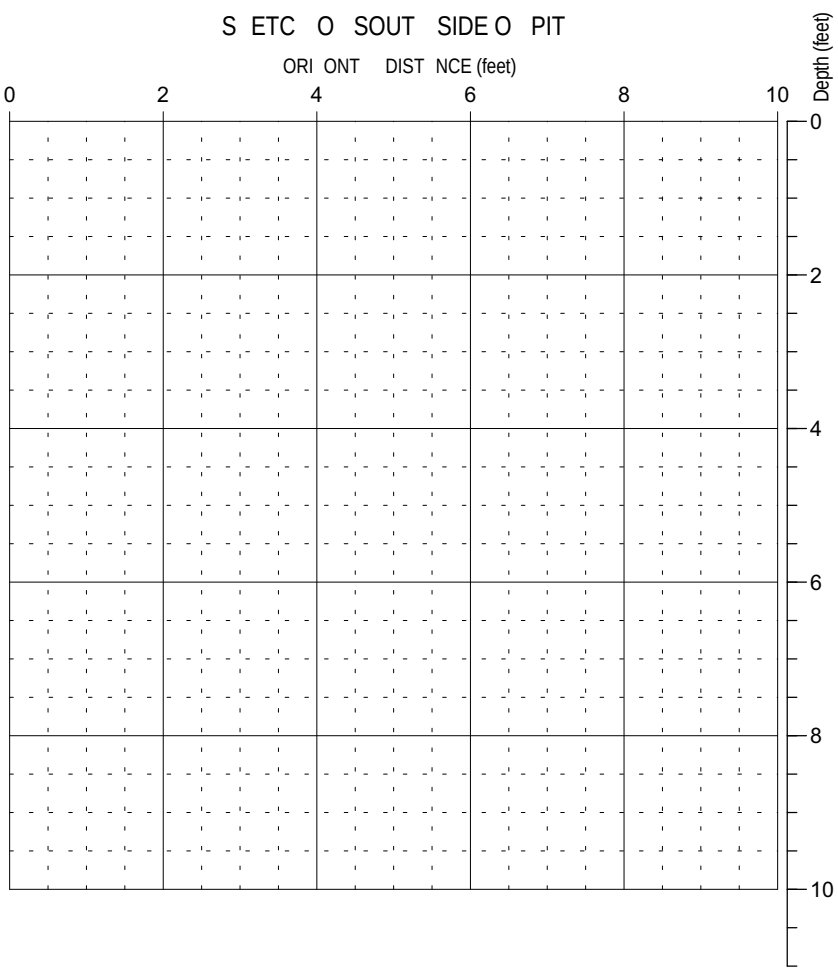
EXCAVATION COMPANY: Custom Backhoe
EQUIPMENT: Case 580
SURFACE ELEVATION: 322 feet

DATE COMPLETED: 2-22-00
LOGGED BY: CJN

Depth (feet)	Symbol	USCS	DESCRIPTION	Sample	Sample No.	Moisture Content (%)	Other Tests	Groundwater
0			Grass and topsoil 4 inches thick.					
		GP	Medium dense, brown, GRAVEL with sand, trace silt, trace cobbles, scattered organics; moist. **(FILL)**					
2		SP	Medium dense, gray, fine to medium SAND, trace silt, trace fine gravel; moist.	G	S1			
		GP	**(RECESSIONAL OUTWASH)** Medium dense, brown, GRAVEL with sand, trace silt, trace cobbles; moist.					
4				G	S2	7		
6				G	S3	3		
8								
10								

Testpit backfilled using native excavation spoils, compacted using the bucket of backhoe.

No groundwater encountered.



PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Renton Transfer Station
Renton, Washington

ABKJ, Inc.

LOG OF TEST PIT

TP-2

Project No. 092-004-002

FIGURE A-5

EXCAVATION COMPANY: Custom Backhoe
EQUIPMENT: Case 580
SURFACE ELEVATION: 324 feet

DATE COMPLETED: 2-22-00
LOGGED BY: CJN

Depth (feet)	Symbol	USCS	DESCRIPTION	Sample	Sample No.	Moisture Content (%)	Other Tests	Groundwater
0		SM	Medium dense, brown, silty fine SAND, scattered organics, moist.					
		SP	**(FILL)** Medium dense, gray, fine to medium SAND with gravel, trace silt, trace cobbles, scattered organics; moist.					
2				G	S1			
		GP	Medium dense, brown, GRAVEL with sand, trace silt; moist.					
4			**(RECESSIONAL OUTWASH)**					
		SP	Medium dense, gray, fine to medium SAND, trace silt, trace fine gravel; moist.					
6				G	S2	3		
8								
10								

Testpit backfilled using native excavation spoils, compacted using the bucket of the backhoe.

No groundwater encountered.

S ETC O SOUT SIDE O PIT

ORI ONT DIST NCE (feet)

0 2 4 6 8 10

Depth (feet)

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Renton Transfer Station
Renton, Washington

ABKJ, Inc.

LOG OF TEST PIT

TP-3

Project No. 092-004-002

FIGURE A-6

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Asphalt - 4 inches thick.			
									Dense, brown, GRAVEL with sand, trace silt; moist. **(VALLEY FILL)**	80		
	1			10 19 34			GW		(Blow count not representative due to gravel)			
						5			Very dense, gray, silty SAND with gravel, trace organics and roots; moist; organic odor.			
	2		10	20 43 21			SM			75	ATD	
	3			4 1 2					Soft, brown, PEAT, trace fine gravel, trace silt and clay; moist to wet; organic odor, fibrous. **(ALLUVIUM)**			
						10				70		
	4			0 1 1					Becomes mottled brown and black; scattered pockets of gray clay.			
						15	PT			65		
	5			1 1 1					No recovery at 17.5 feet.			
						20			Scattered wood fragments noted at 21 feet based on drill action.	60		
	6			8 11 9					No recovery at 22.5 feet. Medium dense, gray, GRAVEL with sand; wet.			
	7			2 9 8		25	GW			55		
FC	8		12	4 26 26					Very dense, gray, silty SAND with gravel; wet; gravel is subrounded. **(GLACIAL TILL)**			
						30				50		
	9		9	50 46 51			SM		Becomes brown, moist.			
						35				45		
	10			36 29 42								
						40			Bottom of boring at 39 feet. Boring backfilled with bentonite chips. Top 6 inches sealed with concrete.	40		

Surface Elevation: 81 feet
Date Completed: 2-24-00
Logged By: CJN
Equipment: Mobile Drill B-59
Drilling Method: 4-inch ID Hollow Stem Auger
Hammer System: Rope and Cathead, 140 lb. Hammer

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Algona Transfer Station
Algona, Washington

ABKJ, Inc.

LOG OF BORING B-1

SHEET 1 OF 1

Project No. 092-004-003

FIGURE A-7

BORING WELL ALGONA.GPJ PACRIM.GDT 4/1/02

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Asphalt - 3 inches thick.			
									Medium dense, mottled brown and gray, silty SAND with gravel; moist.	95		
									(FILL/COLLUVIUM)			
	1			15 10 8			SM					
						5			No recovery at 5 feet.			
	2			7 7 7						90		
									Very dense, mottled gray and brown, SAND with silt and fine gravel; moist; gravel is subrounded.			
GSD	3		12	33 73 50/4"					**(GLACIAL TILL)**			
						10				85		
	4			100/5"			SP-SM					
						15				80		
	5			100/3"					No recovery at 17.5 feet.			
						20					ATD	
									Very dense, brown, fine to medium SAND with silt, trace coarse sand and fine gravel; wet.	75		
GSD	6		22	47 59 79					**(ADVANCE OUTWASH)**			
						25	SW-SM					
										70		
	7		21	38 54 69								
									Bottom of boring at 29 feet. Boring backfilled with bentonite chips. Top 8 inches sealed with concrete.			
						30				65		
						35				60		

Surface Elevation: 96 feet
Date Completed: 2-24-00
Logged By: CJN
Equipment: Mobile Drill B-59
Drilling Method: 4-inch ID Hollow Stem Auger
Hammer System: Rope and Cathead, 140 lb. Hammer

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Algona Transfer Station
Algona, Washington

ABKJ, Inc.

LOG OF BORING B-2

SHEET 1 OF 1

Project No. 092-004-003

FIGURE A-8

BORING WELL ALGONA.GPJ PACRIM.GDT 3/29/02

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY INVESTIGATIONS

GENERAL

PacRim Geotechnical Inc. personnel performed laboratory tests on soil samples in our in-house laboratory. Laboratory index tests included moisture content, fines content, and grain size distribution. All laboratory index tests were conducted in general accordance with appropriate ASTM test methods (ASTM, 1998). The test procedures and test results are discussed below.

Moisture Content Determinations

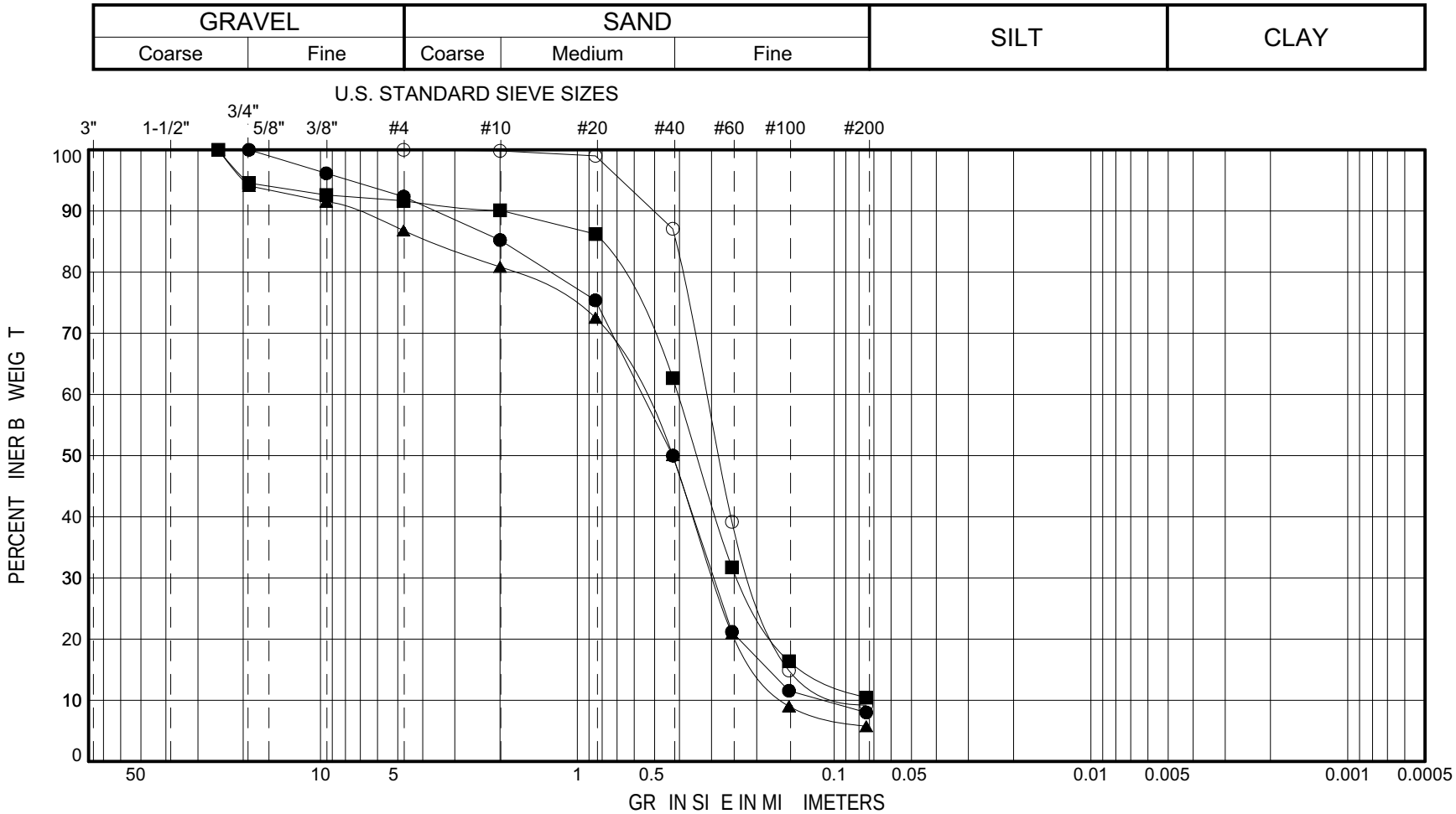
Moisture content determinations were conducted on selected soil samples in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate summary logs in Appendix A.

Fines Content Determinations

Selected soil samples were washed through the U.S. Standard No. 200 sieve in general accordance with ASTM D 1140 to determine the percentage of fines. The test results are plotted on Figures B-2 and B-4. The soil samples analyzed for fines content are indicated on the summary logs in Appendix A in the column labeled "Other tests".

Grain Size Analyses

Grain size analyses of selected soil samples were evaluated in general accordance with the procedures outlined in ASTM D-422. The test results are plotted on Figures B-1 through B-4. The soil samples analyzed for grain size distribution are indicated on the summary logs in Appendix A in the column labeled "Other tests".



S MBO	S MP E		DEPT (ft)	C SSI IC TION	% MC		P	PI	% Gravel	% Sand	% ines
●	B-1	3	7.5 - 9.0	Brown, SAND with silt, few fine gravel (SP-SM)	7				8	84	8
■	B-1	5	17.5 - 19.0	Brown, SAND with silt, few gravel (SP-SM)	10				8	81	10
▲	B-2	1	2.5 - 4.0	Brown, SAND with silt, few to little gravel (SP-SM)	7				13	81	6
○	B-2	5	22.5 - 24.0	Gray, fine to medium SAND with silt (SP-SM)	18				0	91	9

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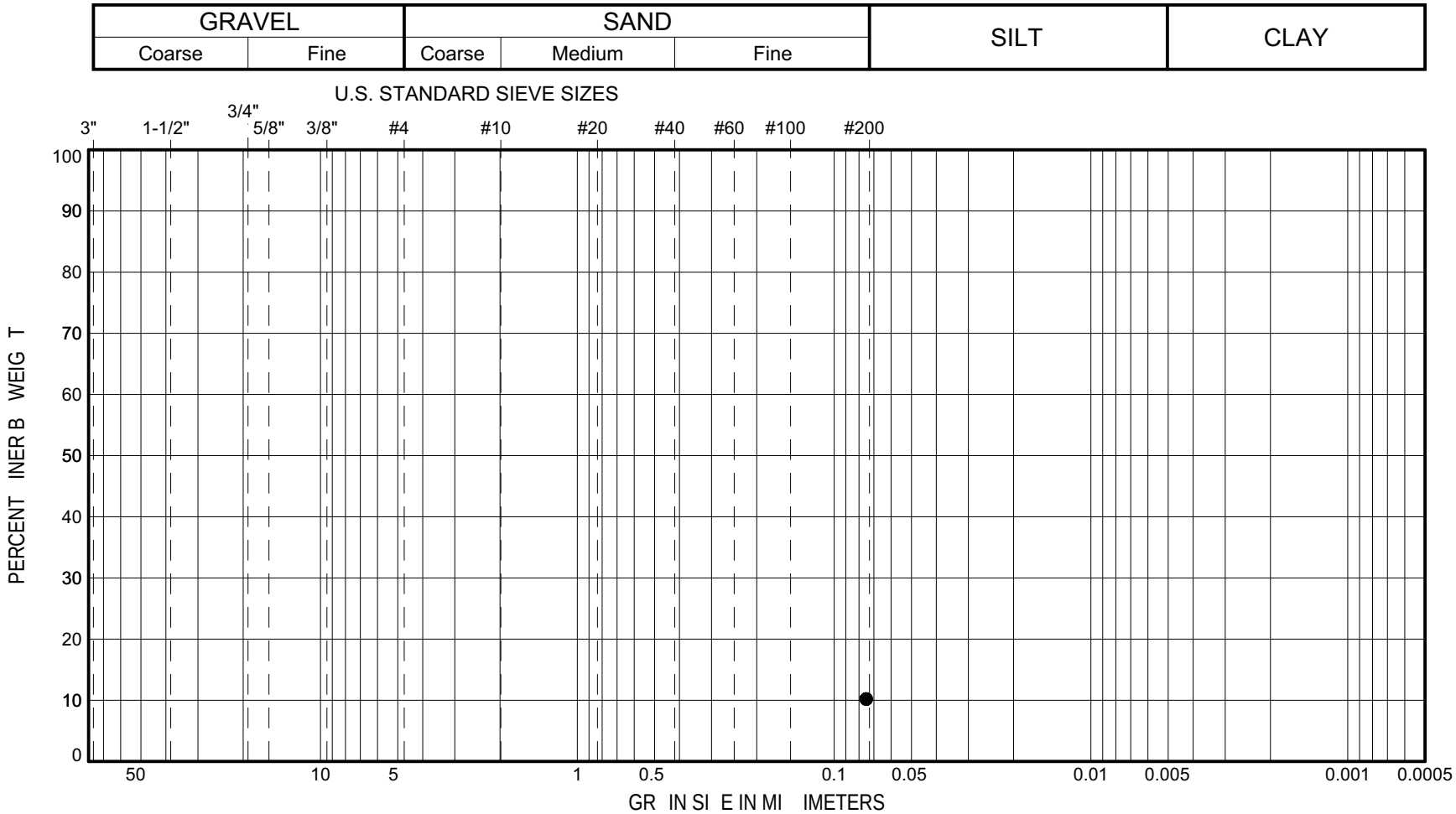
Project No. 092-004-001

FIGURE B-1



S MBO	S MP E		DEPT (ft)	C SSI IC TION	% MC		P	PI	% Gravel	% Sand	% ines
●	B-2	6	27.5 - 29.0	Gray, fine to medium SAND with silt (SP-SM)	21						10

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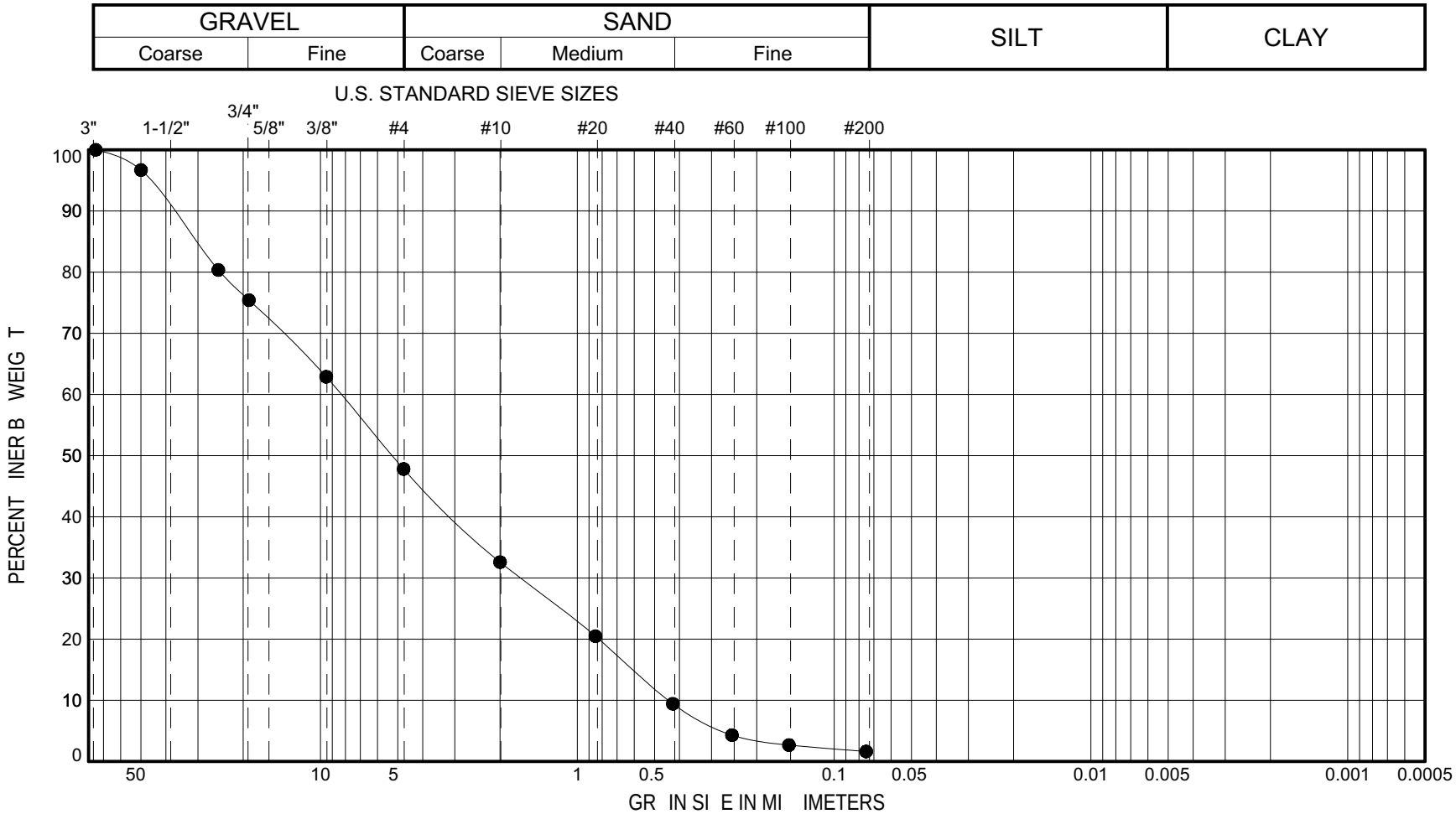
Project No. 092-004-001 **FIGURE B-2**



S MBO	S MP E		DEPT (ft)	C SSI IC TION	% MC		P	PI	% Gravel	% Sand	% ines
●	TP-1	S3	0.0 - 2.0	Brown, GRAVEL with sand, trace silt (GP)	5				52	46	2

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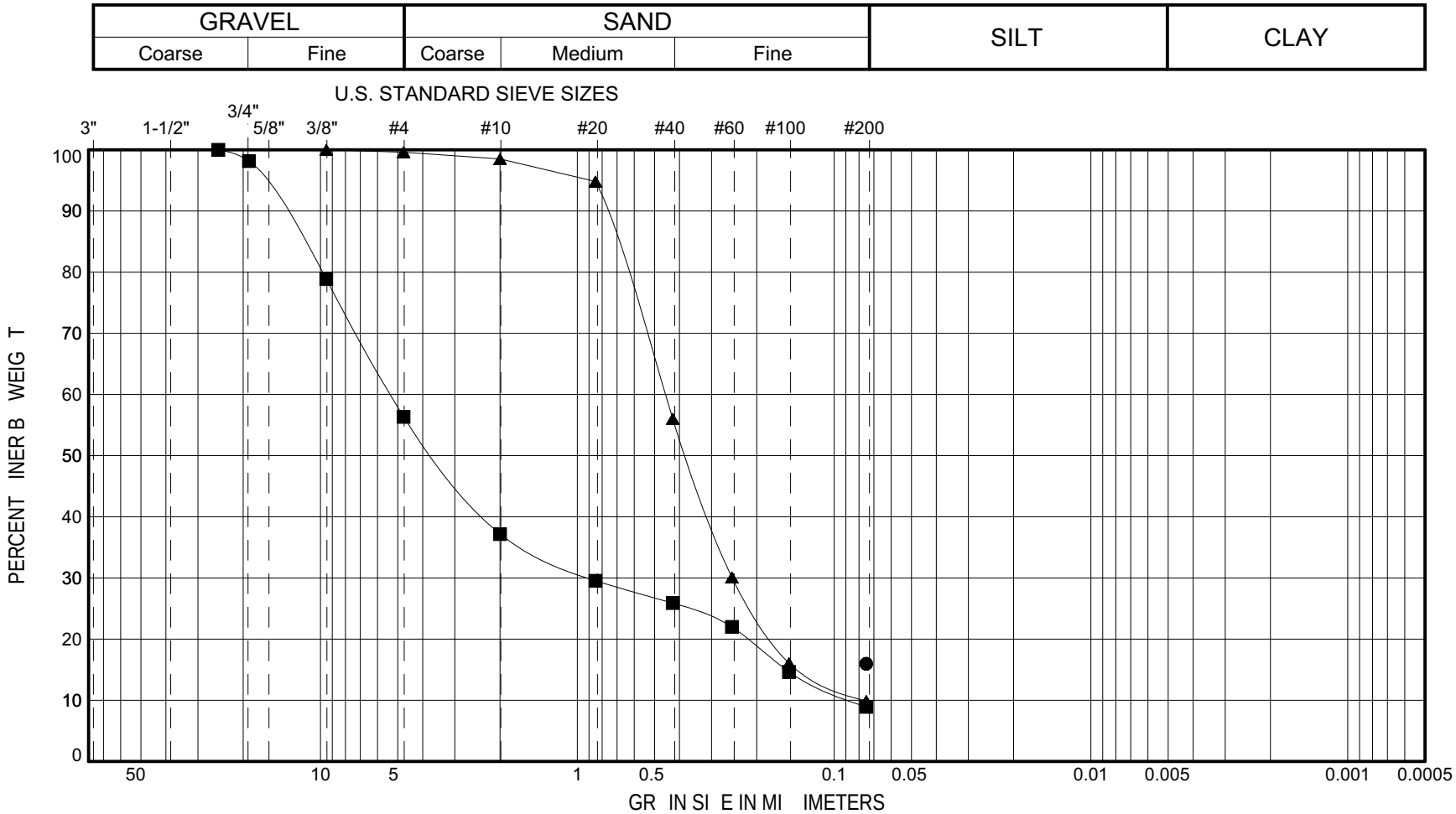
R L

Project No. 092-004-002 **FIGURE B-3**



S MBO	S MP E		DEPT (ft)	C SSI IC TION	% MC		P	PI	% Gravel	% Sand	% ines
●	B-1	8	27.5 - 29.0	Gray, silty SAND with gravel (SM)	12						16
■	B-2	3	7.5 - 8.9	Brown, SAND with gravel and silt (SW-SM)	12				44	47	9
▲	B-2	6	22.5 - 24.0	Brown, fine to medium SAND with silt (SP-SM)	22				0	90	10

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

Algona Transfer Station
Algona, Washington

ABKJ, Inc.

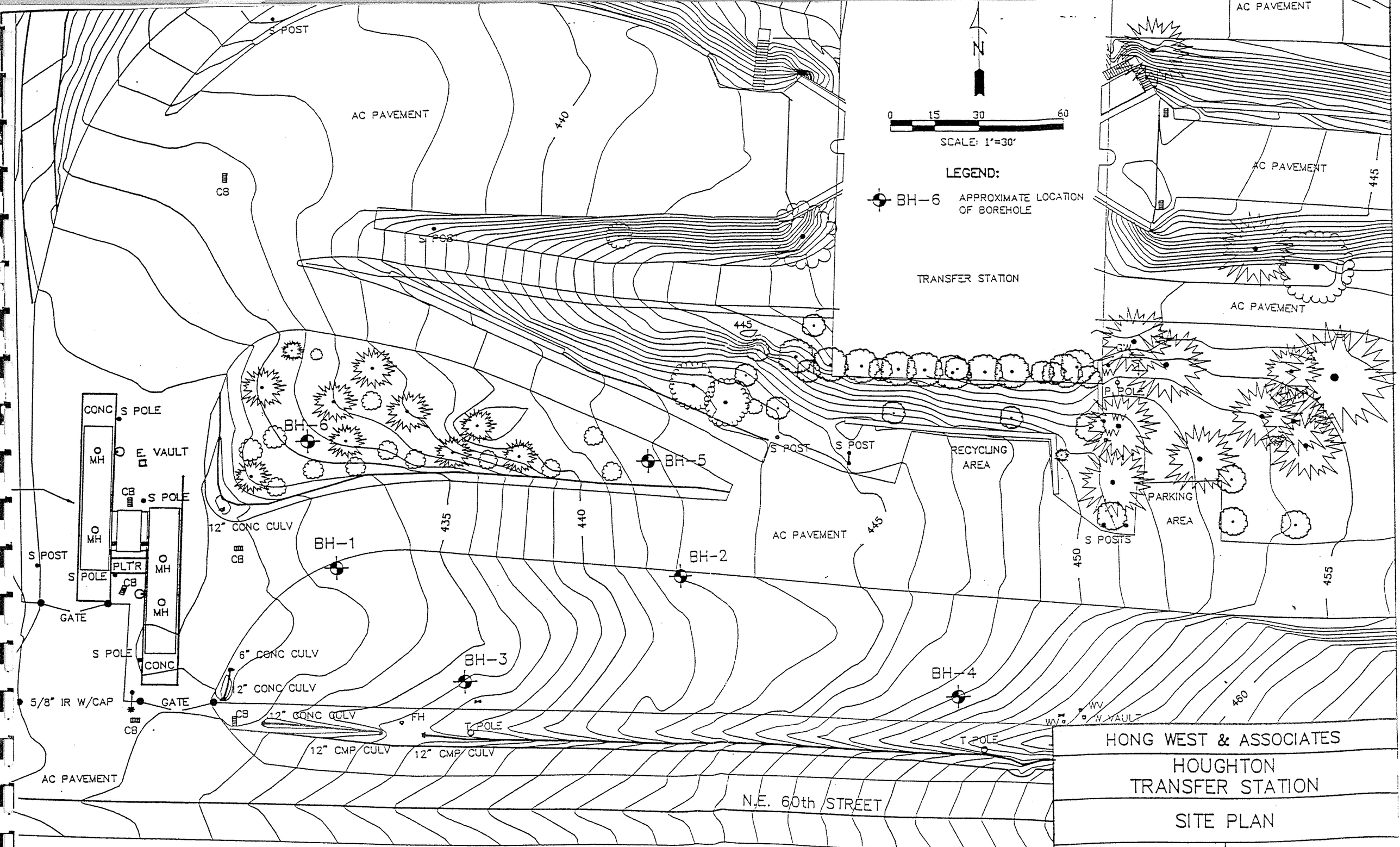
GR IN I N L I
R L

Project No. 092-004-003

FIGURE B-4

APPENDIX C

SUBSURFACE DATA BY OTHERS



MAP BASED ON HOUGHTON TRANSFER STATION
TOPOGRAPHIC MAP PREPARED BY THOMAS WRIGHT, INC.

175 / 175 down

131-B-001.1

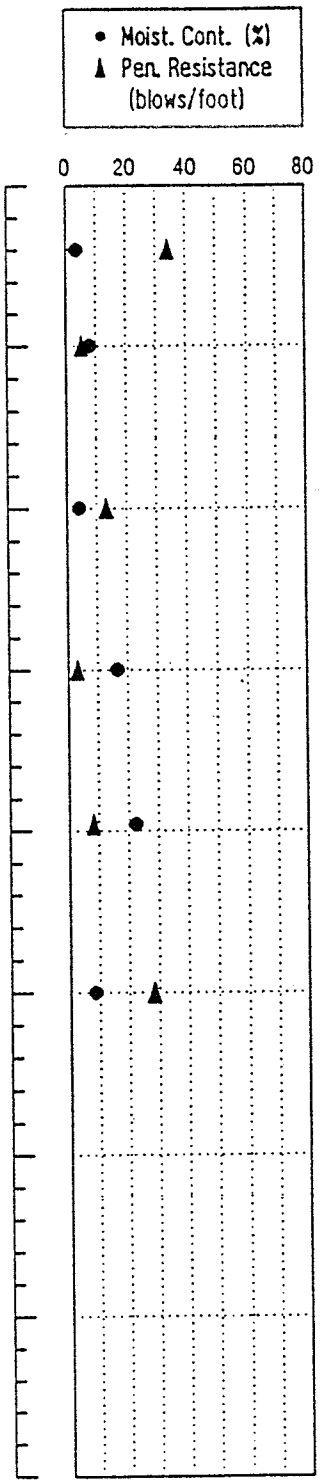
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 26.5 Feet
SURFACE ELEVATION: 433 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0						SM	Loose to Medium Dense, grey brown Silty SAND, with Gravel, Dry. (Fill)
		12/22/12	34	3.5			
5		10/3/2	5	7.9			
							Aluminum, and steel cans, plastic, and paper, mixed with Silty SAND, Loose, Damp. (Landfill)
10		7/8/5	13	4.0			
15		4/2/1	3	16.5			
20		2/2/8	8	22.2			Loose, dark brown Silty SAND, with trace of landfill refuse (odoriferous), Moist.
						SM	Loose, dark brown Silty SAND, with trace of landfill refuse (odoriferous), Moist. (Landfill)
25		10/15/13	28	8.5			Medium Dense, brown to grey Silty SAND, with trace of Clay and fine Gravel (odoriferous), Moist. (Advance Outwash)
							End Of Hole
30							
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: Houghton Transfer Station

BORING: BH-1

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1

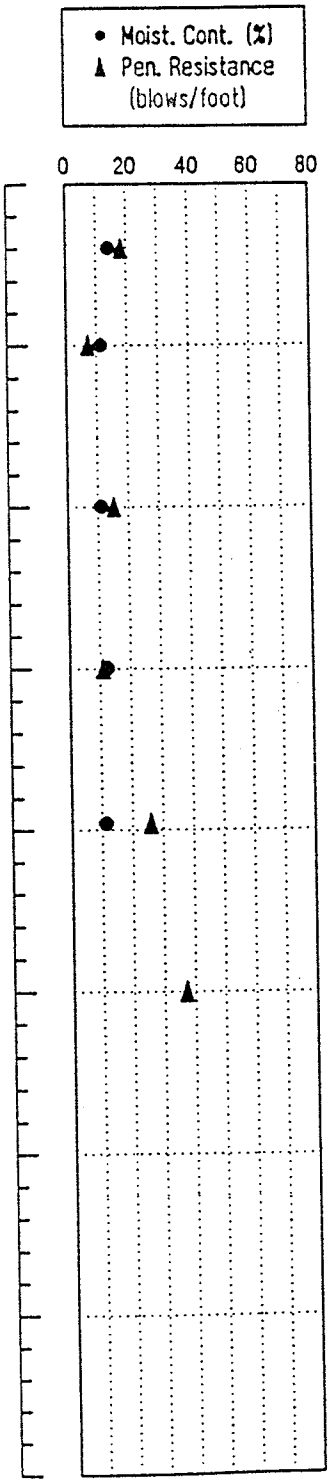
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 26.5 Feet
SURFACE ELEVATION: 443 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0						SM	Loose to Medium Dense, dark brown to grey Silty SAND, Moist. (Fill)
		6/6/12	18	13.9			
						SW	Loose, grey fine to medium grained SAND, with Clay and Gravel, Moist. (Fill)
5		3/4/3	7	11.2			
10		5/7/8	15	11.0			
15		6/5/6	11	12.2		SC	Loose, grey brown Clayey SAND, Damp. (Fill)
						SP/SM	Medium Dense, grey fine to medium grained slightly Silty SAND, Moist. (Advance Outwash)
20		8/10/16	26	11.2			
							Medium Dense, grey fine to medium grained SAND, Moist. (Advance Outwash)
25		8/12/25	37				
							End Of Hole
30							
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: Houghton Transfer Station

BORING: BH-2

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1

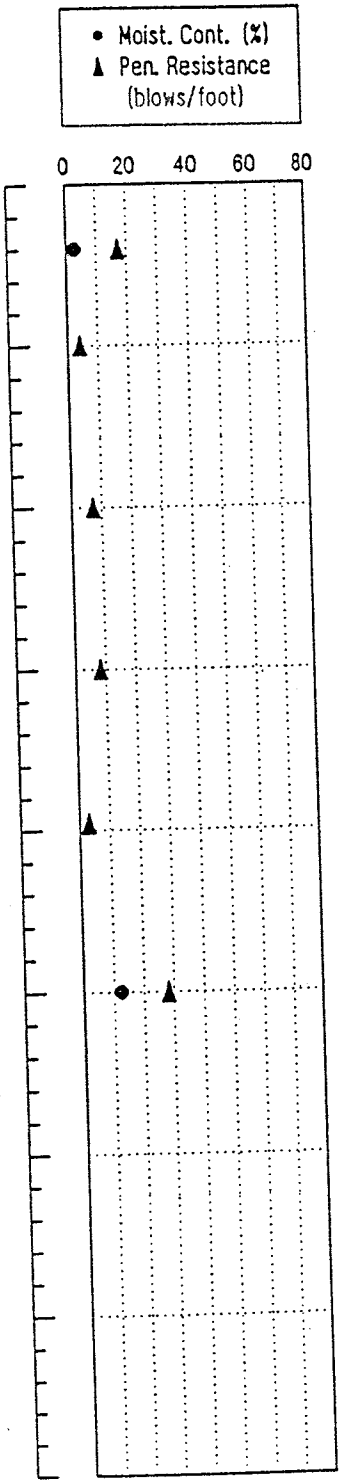
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 26.5 Feet
SURFACE ELEVATION: 436 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0						SM	Loose to Medium Dense, grey to brown Silty Gravelly SAND, Dry.
	X	12/12/5	17	2.7			(Fill)
5	X	10/2/2	4				Aluminum and steel cans, plastic, paper, mixed with Loose, grey Silty SAND, Moist.
							(Landfill)
10	X	1/2/5	7				
15	X	4/4/4	8				Lower, concentration of refuse in SAND.
							(Landfill Refuse)
20	X	3/2/1	3			SP SM	Loose, grey fine to medium grained Slightly Silty SAND (odoriferrous), Moist.
							(Fill)
							Firmer drilling
25	X	14/14/14	28	12.2			Medium Dense, grey fine to medium grained slightly Silty SAND, Moist.
							(Advance Outwash)
							End Of Hole
30							
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: Houghton Transfer Station

BORING: BH-3

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1

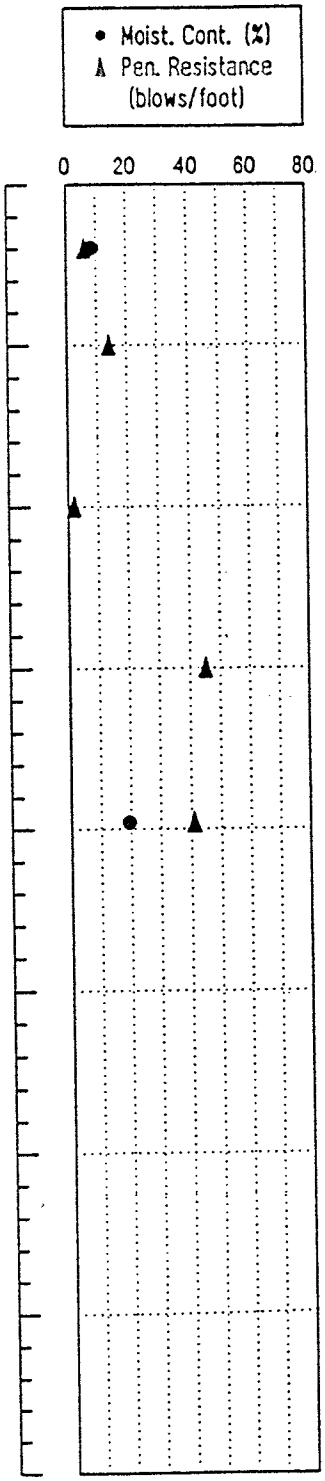
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 21.5 Feet
SURFACE ELEVATION: 449 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0						SM	Loose, brown Silty fine to medium grained SAND, Dry.
	X	12/3/3	6	8.4		SW	(Fill)
							Loose, grey fine to medium grained SAND, Moist.
5	X	4/6/8	14				(Fill)
							Aluminum and steel fragments, plastic mixed with Sandy Soil, Very Loose, Damp.
10	X	2/1/1	2				(Landfill)
						SW	Dense, grey fine to medium grained SAND, with slightly Clayey zones, Moist.
15	X	15/20/25	45				(Advance Outwash)
20	X	15/19/22	41	19.1			Dense, grey to grey brown fine to medium grained SAND, with slightly Clayey zones, Moist.
							(Advance Outwash)
							End Of Hole
25							
30							
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: Houghton Transfer Station

BORING: BH-4

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1

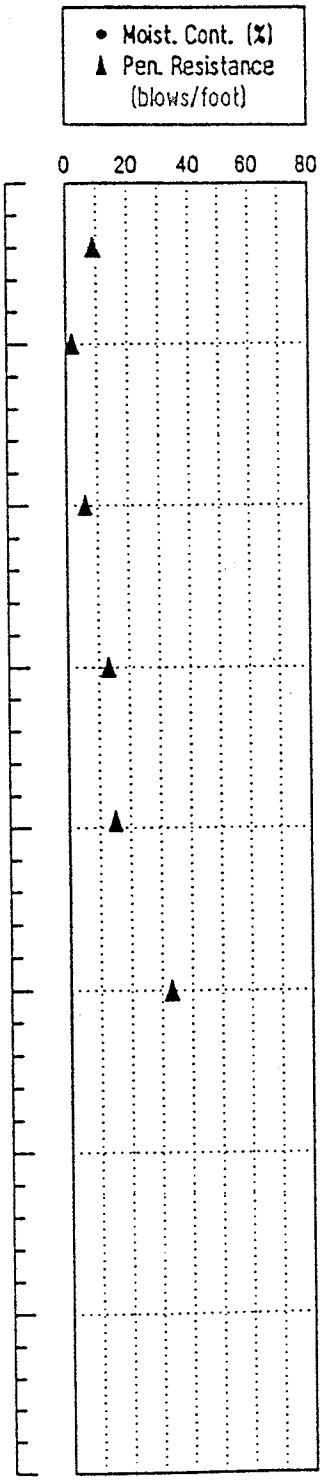
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 26.5 Feet
SURFACE ELEVATION: 442 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0						SC	Loose, dark brown Clayey SAND, with Gravel. Grading to a grey slightly Clayey SAND, Moist. (Fill)
	X	7/4/5	9				
5	X	3/1/1	2				Metal fragments, plastic, paper, mixed with Sandy Soil, Very Loose, Damp. (Landfill)
10	X	5/3/3	8				
15	X	8/7/8	13				Lower concentration of refuse in Sandy Soil.
20	X	8/7/8	15				Metal fragments, plastic, paper, mixed with Sandy Soil. (Landfill)
						SW	Dense, grey fine to coarse SAND, with fine Gravel, Moist. (Advance Outwash)
25	X	14/14/19	33				
							End Of Hole
30							
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: Houghton Transfer Station

BORING: BH-5

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1

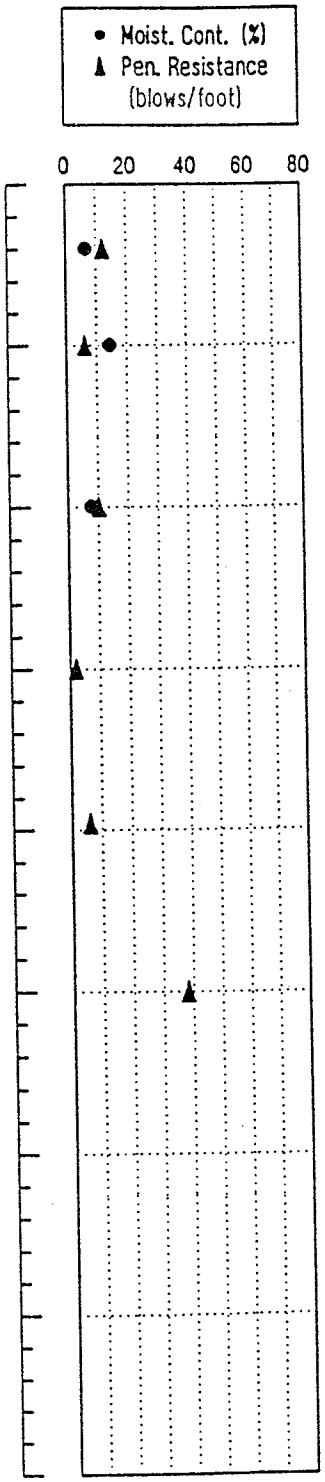
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY: Gregory Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 26.5 Feet
SURFACE ELEVATION: 434 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0	X	11/6/6	12	8.5		SW	Loose, grey brown slightly Clayey SAND, with Gravel, Moist. (Fill)
5	X	4/3/3	8	14.3		SM	Loose, dark grey very Silty SAND, fine to medium grained, very Moist. (Fill)
10	X	4/6/4	10	7.8			
15	X	2/1/1	2				Relatively low concentration of metal, glass, plastic, and paper in dark grey SAND, Very Loose, Moist. (Landfill)
20	X	4/3/3	6				Small amounts of metal, glass, plastic, and paper in dark grey SAND, Moist.
25	X	27/20/18	38			SM	Dense, olive Silty SAND, fine to medium grained, very Moist. (Advance Outwash)
30							End Of Hole
35							
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

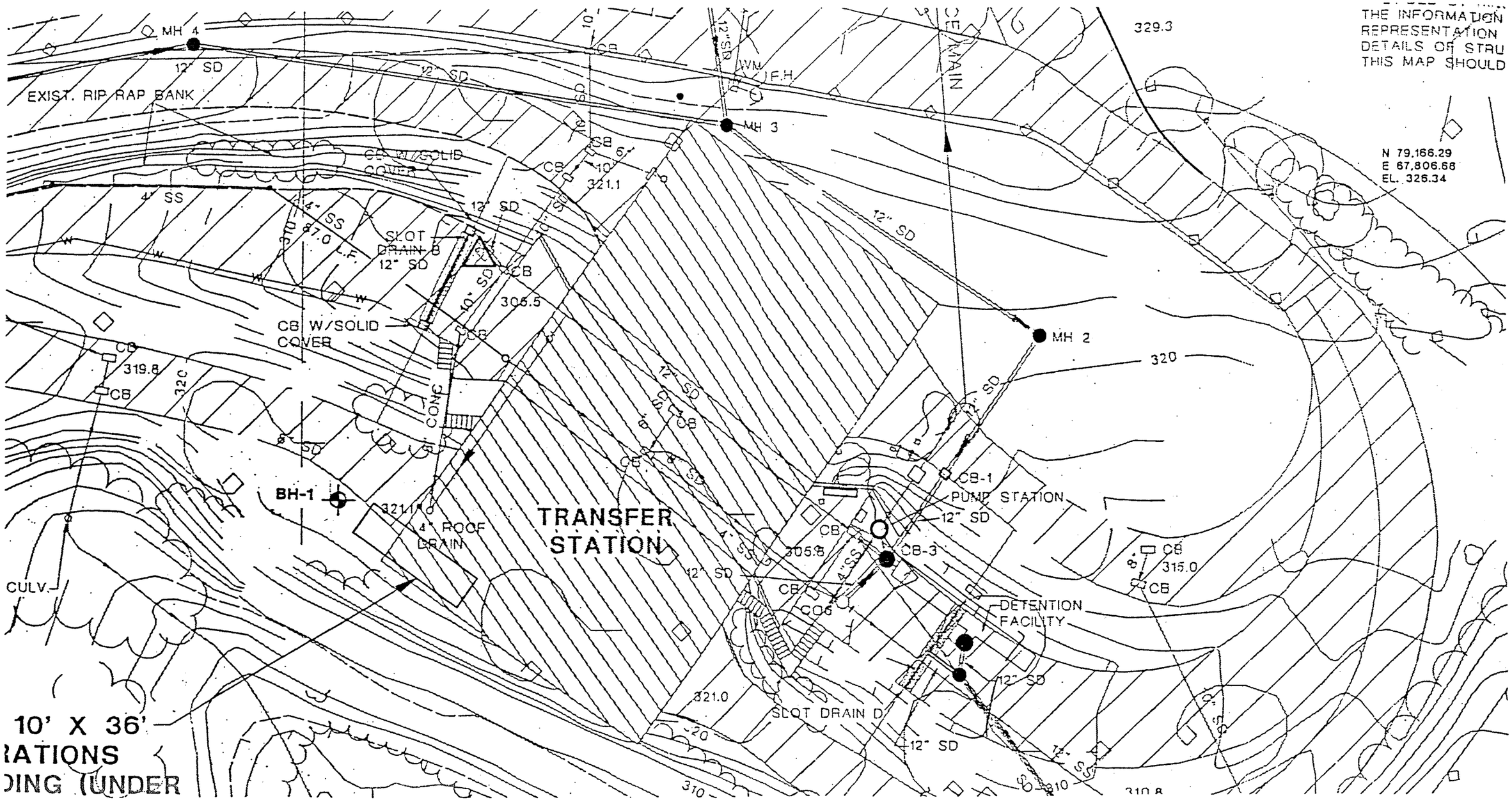
PROJECT: Houghton Transfer Station

BORING: BH-6

LOCATION: NE 60th Street, Kirkland WA.
DATE COMPLETED: 6/8/92
LOGGED BY: RM

PROJECT NUMBER: 91131

PAGE: 1 OF 1



LEGEND

BH-1 BORING DESIGNATION AND APPROXIMATE LOCATION (HWA, 1996)

0' 10' 20' 30' 60'

SCALE: 1"=30'

HWA HONG WEST & ASSOCIATES, INC.

RENTON TRANSFER STATION

SITE AND EXPLORATION PLAN

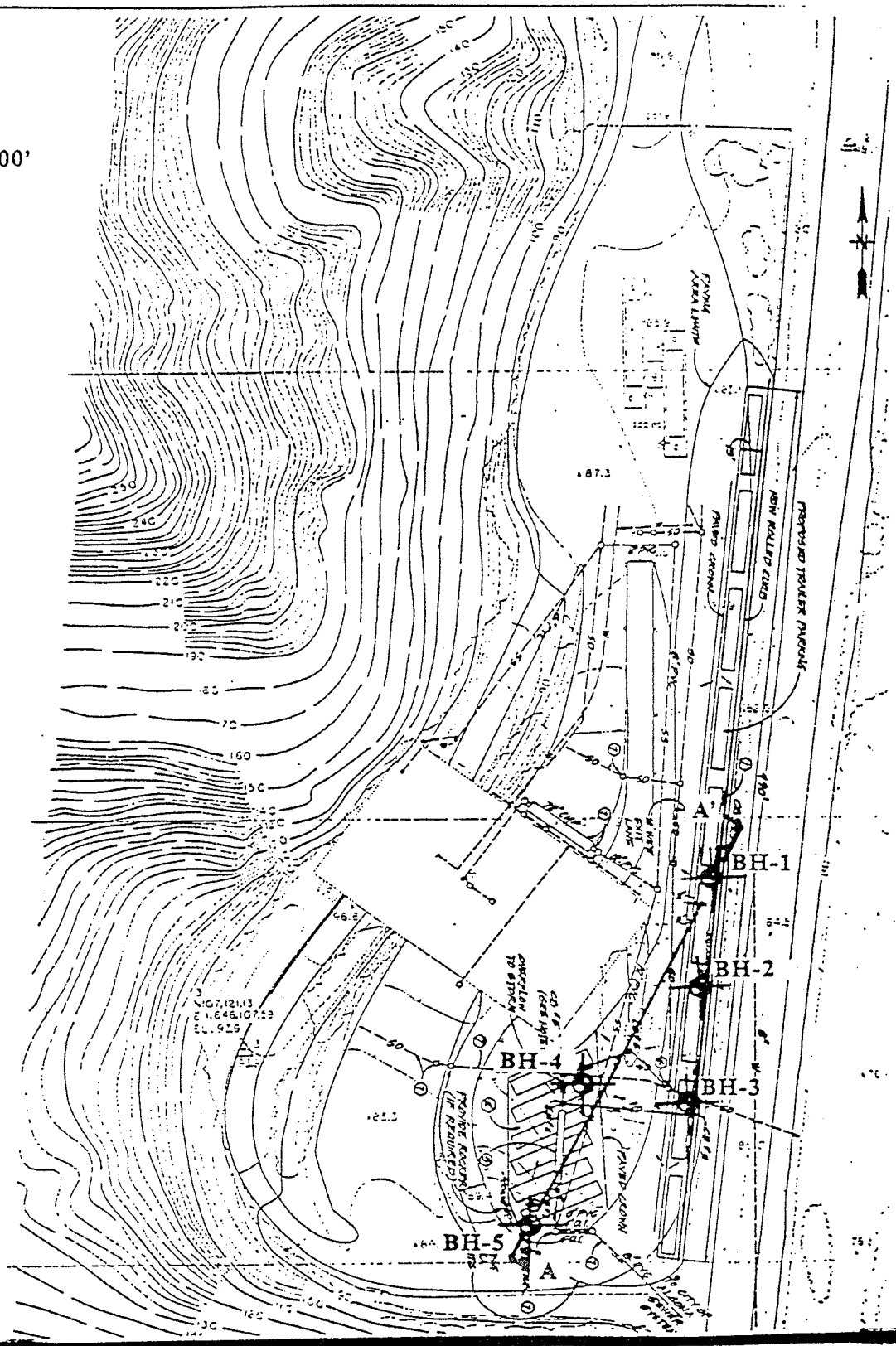
PROJECT NO.: 96116

FIGURE: 2

REFERENCE: Base map provided by ABKJ.

D:\JOBS\96116\96116002.DWG

SCALE 1" = 100'



SITE PLAN MAP

ALGONA TRANSFER STATION

TRANSFER STATION IMPROVEMENT PROJECT

HONG CONSULTING

ENGINEERS, INC.

FIGURE 1a

HONG WEST & ASSOCIATES, INC.

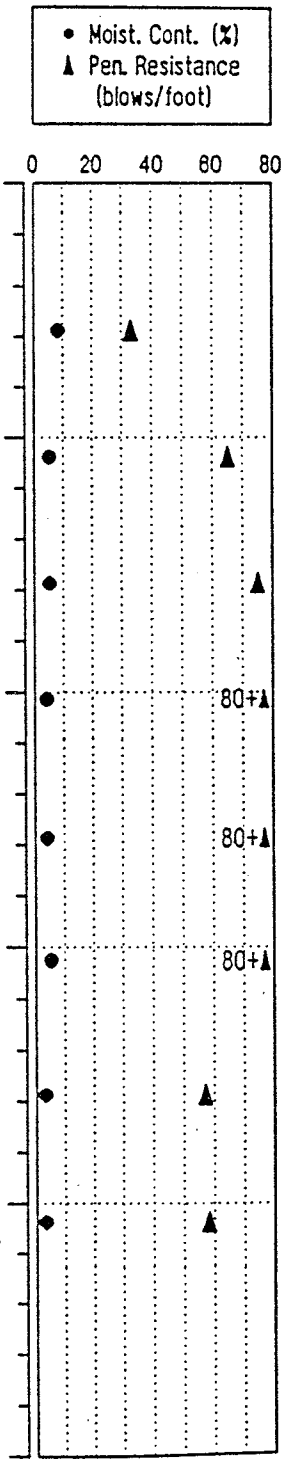
BORING LOG

DRILLING COMPANY: Associated Drilling, Inc.
DRILLING METHOD: 4¼" Hollow Stem Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 41.5 Feet
SURFACE ELEVATION: ±320 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	OTHER TESTS	SYMBOL	SOIL CLASS. (ASTM D-2487 or 2488)	DESCRIPTION
0								SM	3 inches of gravel surfacing over dense to very dense, dark brown to olive brown, silty SAND with gravel, moist. Subrounded to angular 1 inch minus gravel.
	☒	1	10/16/17	33	8	GSA			(ADVANCE OUTWASH)
10	☒	2	19/30/35	65	5				
	☒	3	19/30/45	75	5				
								GP	Very dense, dark brown to olive brown, poorly graded GRAVEL with sand, moist. Sand fine to coarse grained; gravel subangular to rounded.
20	☒	4	58-6"	50+	4				(ADVANCE OUTWASH)
	☒	5	27/37/55	50+	4				
30	☒	6	17/60-6"	50+	5				
	☒	7	15/24/33	57	3	GSA			
40	☒	8	20/28/30	58	3				
50									Boring terminated at a depth of 41½ feet.

No groundwater was observed at time of drilling.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated, and therefore, may not necessarily be indicative of other times and/or locations.

PROJECT: King County Solid Waste

BORING: BH-1

LOCATION: Renton Transfer Station
DATE COMPLETED: July 11, 1996
LOGGED BY: D. Sowers

PROJECT NUMBER: 96116

PAGE: 1 OF 1

Figure A-2

PROJECT #88112
BORE HOLE LOG

DEPTH (feet)	SOIL DESCRIPTION	SAMPLE	GROUND WATER CONDITION
	6" asphalt/crushed rock		
2	Dense to medium dense, brown, silty very gravelly, fine to medium SAND: fines slightly plastic; moist; SW-SM (Fill)	X	
4			
6		X	
8		X	
10	Loose, gray, gravelly fine to medium SAND: peat bed at base; some silt; wet; SW/SM (Alluvial Fan Deposits)	X	
12			
14	Medium dense, dark gray, fine SAND: saturated; SP	X	13:25 10-25-88 ▽
16			
18	Medium stiff to stiff, interbedded gray clayey SILT: PEAT and sandy fine GRAVEL: laminated in places; mixed slide debris at base (Alluvial Fan Deposits)	X	
20			
22		X	
24			
26			
28		X	
30	END OF HOLE		

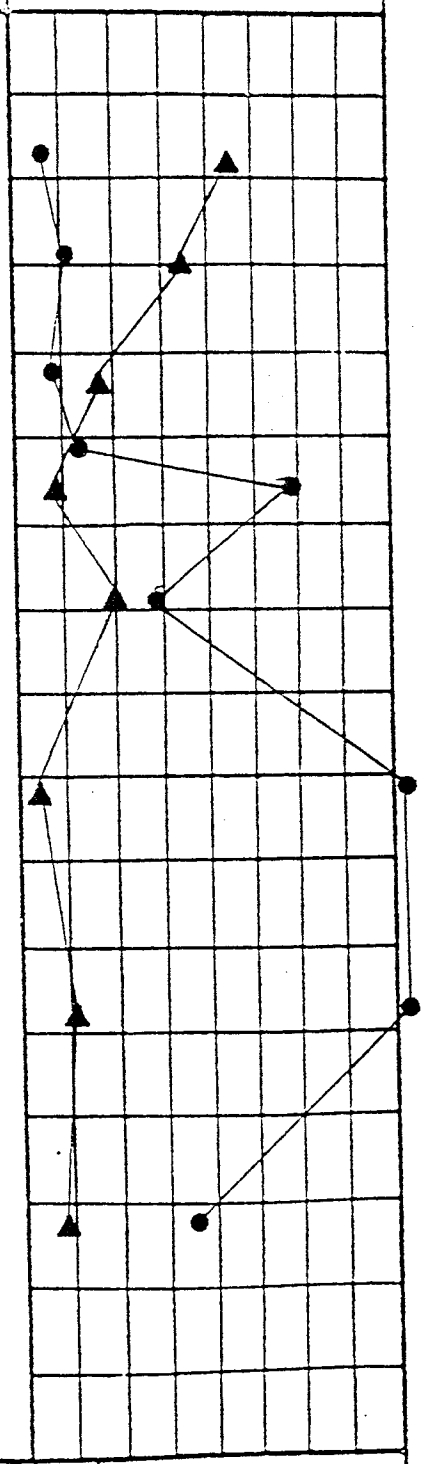


Figure 2	PROJECT Trans. Stn. Improvements Algona Trans. Stn. Algona, WA.	DATE 10-25-88 LOGGED BY SHE ELEVATION 85' DEPTH 29'	HOLE NO. BH-1 SHEET 1 of 1

HONG CONSULTING ENGINEERS, INC.

PROJECT #88112
BORE HOLE LOG

DEPTH (feet)	SOIL DESCRIPTION	SAMPLE	GROUND WATER CONDITION	MOISTURE CONTENT % ● SPT RESISTANCE ▲ 10 20 30 40 50 60 70
	6" asphalt/crushed rock			
2	Dense to medium dense, brown, silty gravelly fine to medium SAND: wet; fines slightly plastic; SW/SM (Fill)			
4			15:00 10-25-88 ▽	
6				
8				
10	Interbedded, medium stiff, gray, clayey SILT, PEAT, and medium dense, gray to brown gravelly fine SAND (Alluvial Fan Deposits)			
12				
14	Medium dense, dark gray, fine SAND: occasional peaty laminates; silty laminates; occasional gravel; SP			
16				
	END OF HOLE			

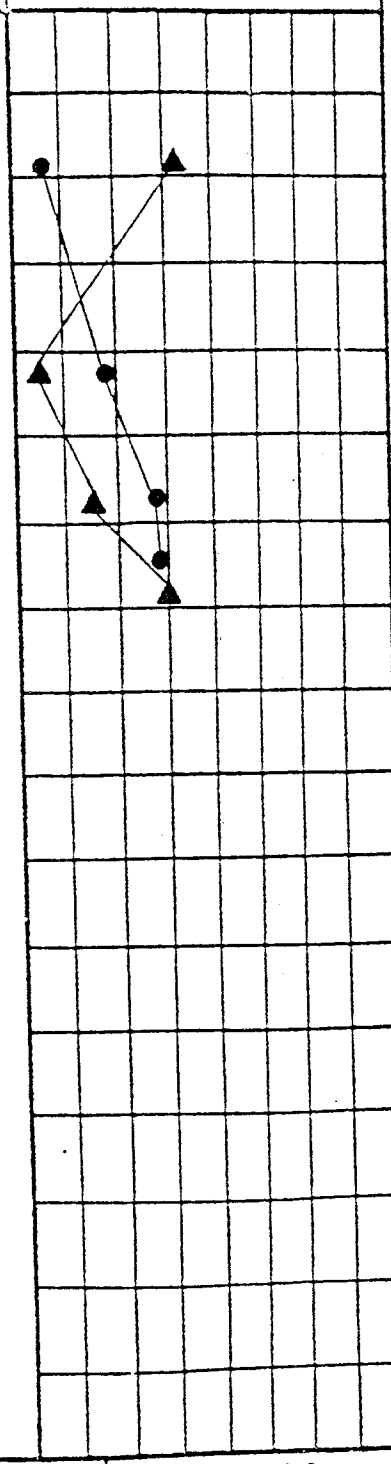


Figure 3	PROJECT Trans. Stn. Improvements Algona Trans. Stn. Algona, WA.	DATE 10-25-88 LOGGED BY SHE ELEVATION 85' DEPTH 14'	HOLE NO. BH-2 SHEET 1 of 1

HONG CONSULTING ENGINEERS, INC.

PROJECT #88112
BORE HOLE LOG

DEPTH (feet)	SOIL DESCRIPTION	SAMPLE	GROUND WATER CONDITION
	6" asphalt/crushed rock		
2	Medium dense, brown, very gravelly fine to medium SAND: some silt ; wet to saturated SW/SM (Fill)	X	
4			15:20 10-25-88 ▽
6		X	
8		X	
10		X	
12	Medium dense, dark gray, fine SAND: laminated; occasional plant debris and wood fiber; SP	X	
14			
16	END OF HOLE		

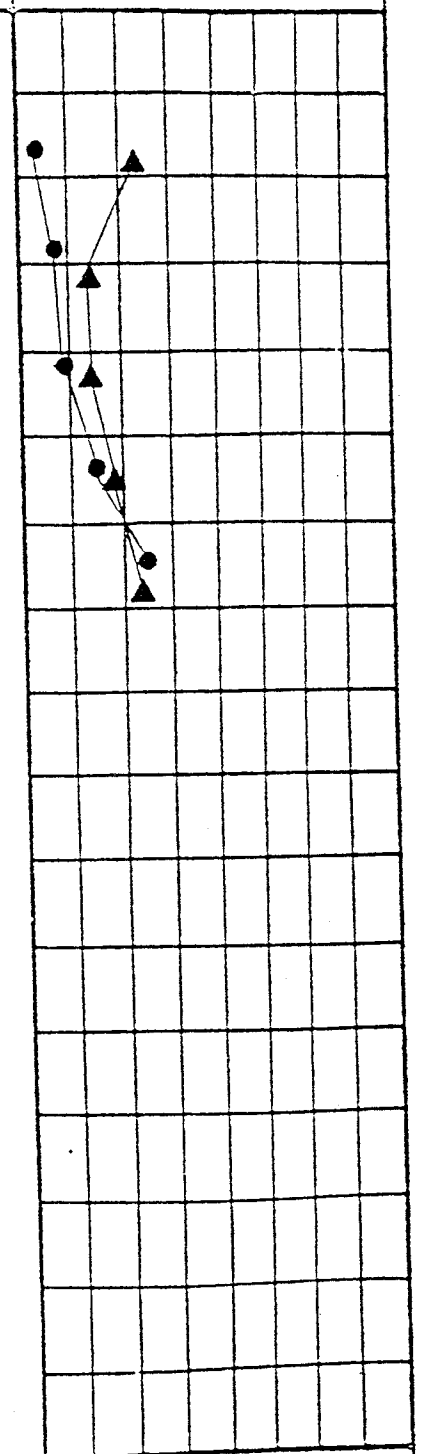


Figure 4	PROJECT Trans. Stn. Improvements Algona Trans. Stn. Algona, WA.	DATE 10-25-88 LOGGED BY SHE ELEVATION 85' DEPTH 14'	HOLE NO. BH-3 SHEET 1 of 1

PROJECT #88112
BORE HOLE LOG

DEPTH (feet)	SOIL DESCRIPTION	SAMPLE	GROUND WATER CONDITION
2	Dense to medium dense, brown to gray, silty gravelly SAND: SW/SM (Fill)		
4	Soft, brown PEAT: PE		0930 10-26-88
6			
8			
10			
12			
14	Interbedded, brown PEAT: medium stiff, gray, silty CLAY and medium dense, brown silty sandy fine to coarse GRAVEL: wood debris (Alluvial Fan Deposits)		
16			
18			
20			
22			
24	END OF HOLE		

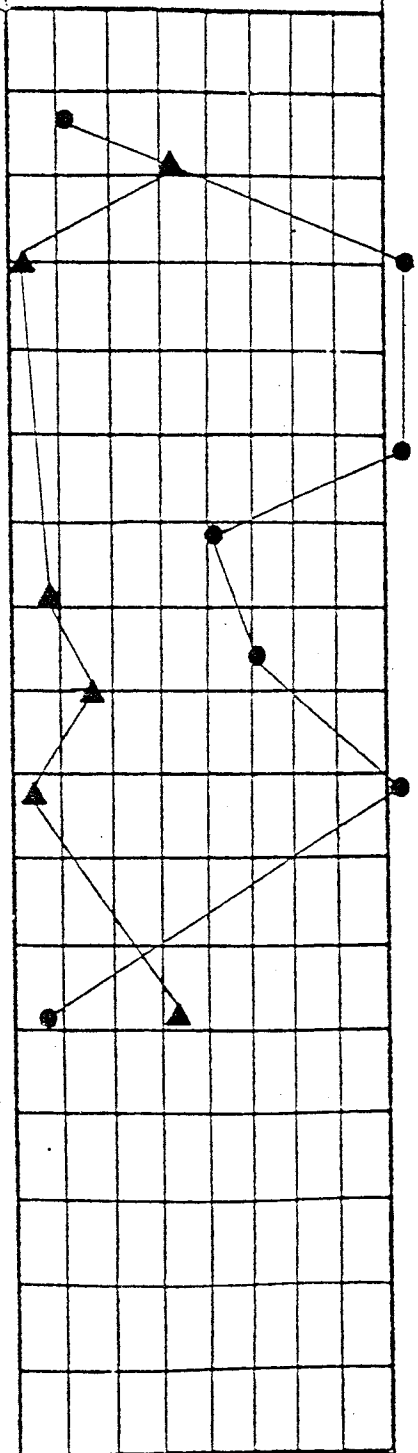


Figure 5	PROJECT Trans. Stn. Improvements Algona Trans. Stn. Algona, WA.	DATE 10-26-88 LOGGED BY SHE ELEVATION 82' DEPTH 24'	HOLE NO. BH-4 SHEET 1 of 1

PROJECT #88112
BORE HOLE LOG

DEPTH (feet)	SOIL DESCRIPTION	SAMPLE	GROUND WATER CONDITION
2	Medium dense, brown, sandy, very gravelly SILT: moist; ML/GM (Fill)	X	
4	Medium dense, gray, gravelly, fine to medium sand: some silt, SM (Alluvial Fan Debris)	X	
6	Brown Peat: PE	X	▽
8		X	
10		X	
12	Interbedded, loose, brown to gray, silty, gravelly, fine to medium SAND, brown PEAT: 6" beds; wet; SM/PE (Alluvial Fan Deposits)	X	
14		X	
16	Loose to medium dense, gray, silty, gravelly, fine to medium SAND: wet; SW-SM (Alluvial Fan Deposits)	X	
18		X	
20		X	
22	END OF HOLE		

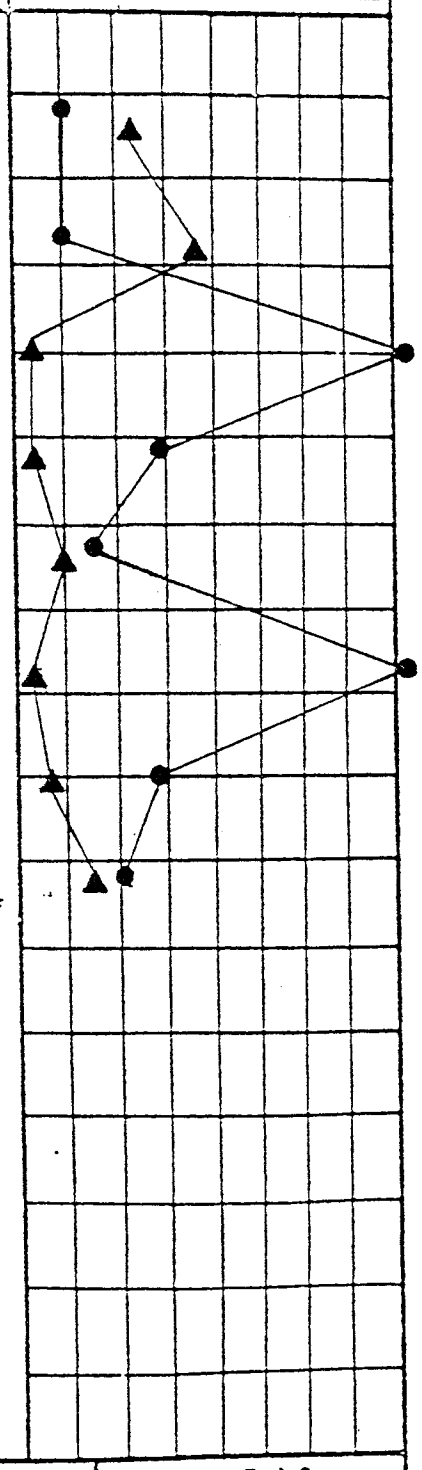


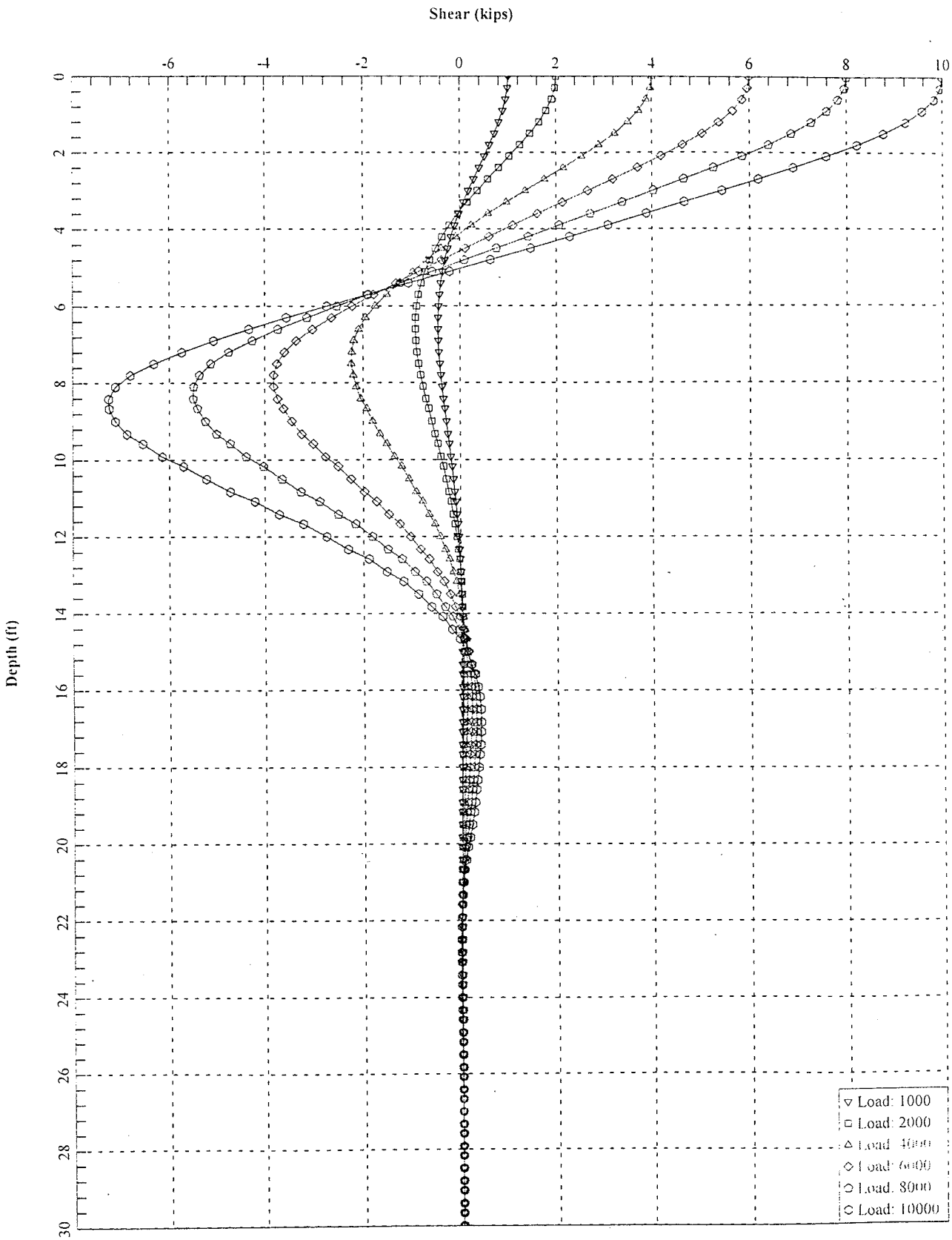
Figure 6	PROJECT Trans. Stn. Improvements Algona Trans. Stn. Algona, WA.	DATE 10-26-88 LOGGED BY SHE ELEVATION 85' DEPTH 21'	HOLE NO. BH-5 SHEET 1 of 1

HONG CONSULTING ENGINEERS, INC.

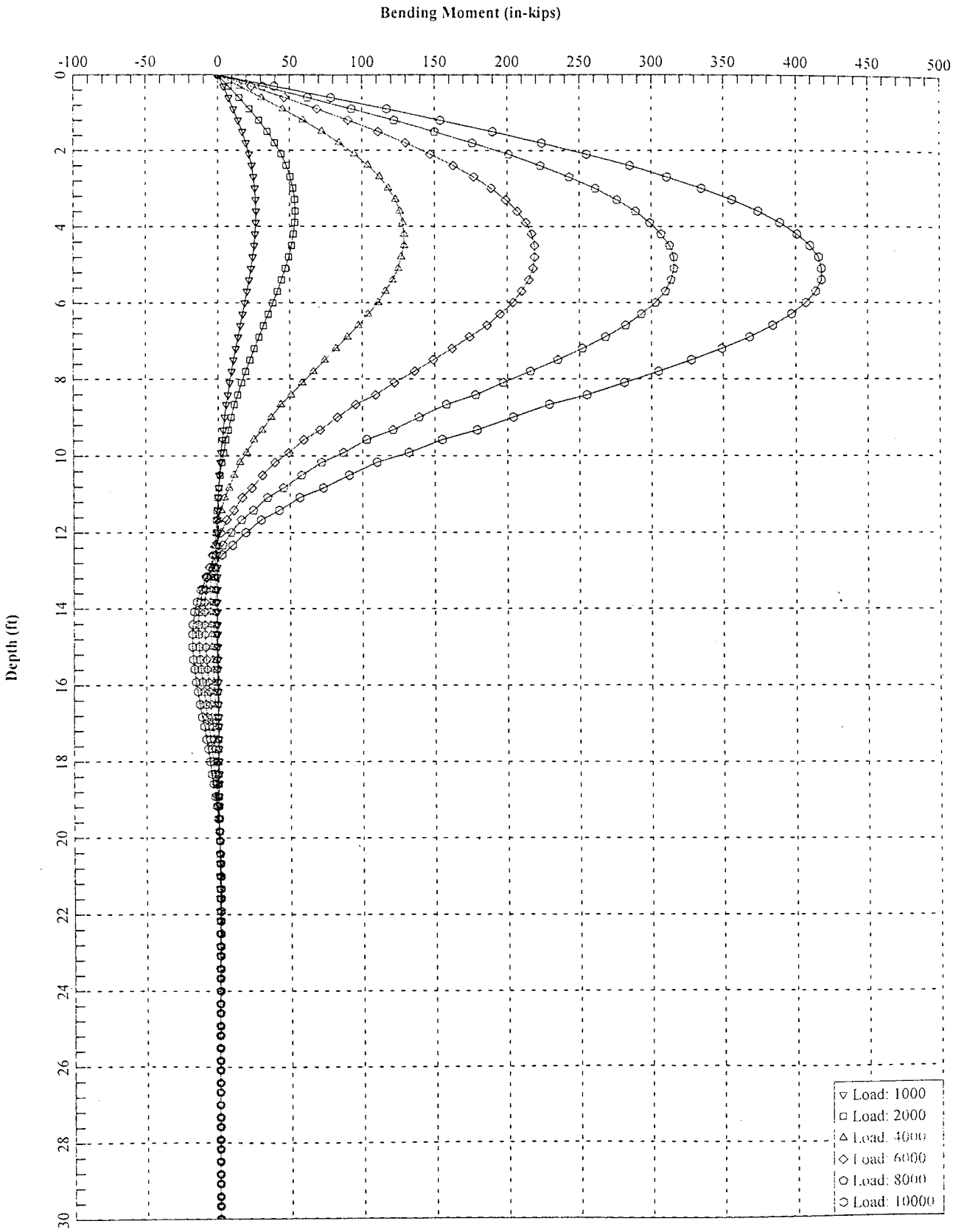
APPENDIX D

LPILE ANALYSIS RESULTS

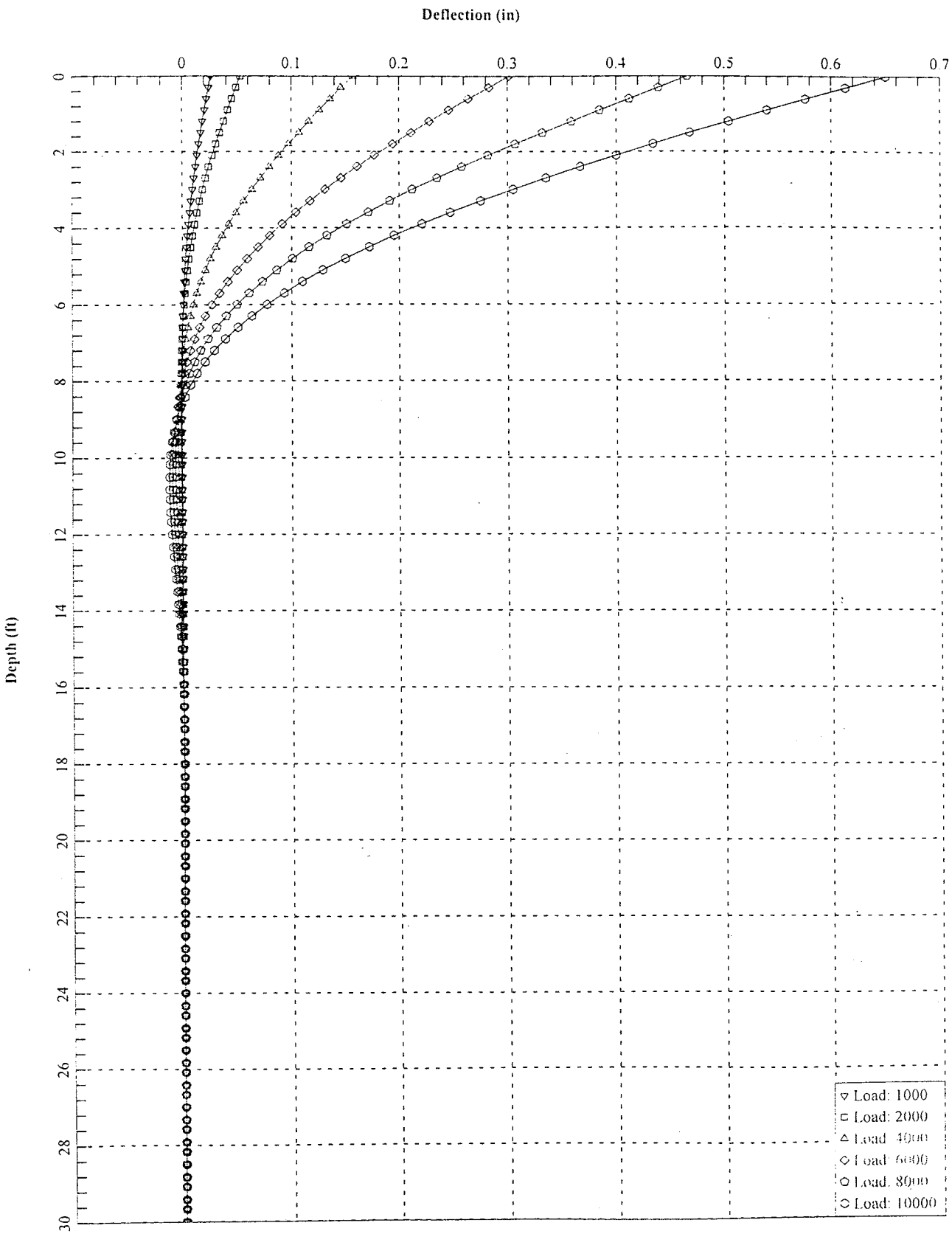
(SHEAR, MOMENT AND DEFLECTION DIAGRAMS)



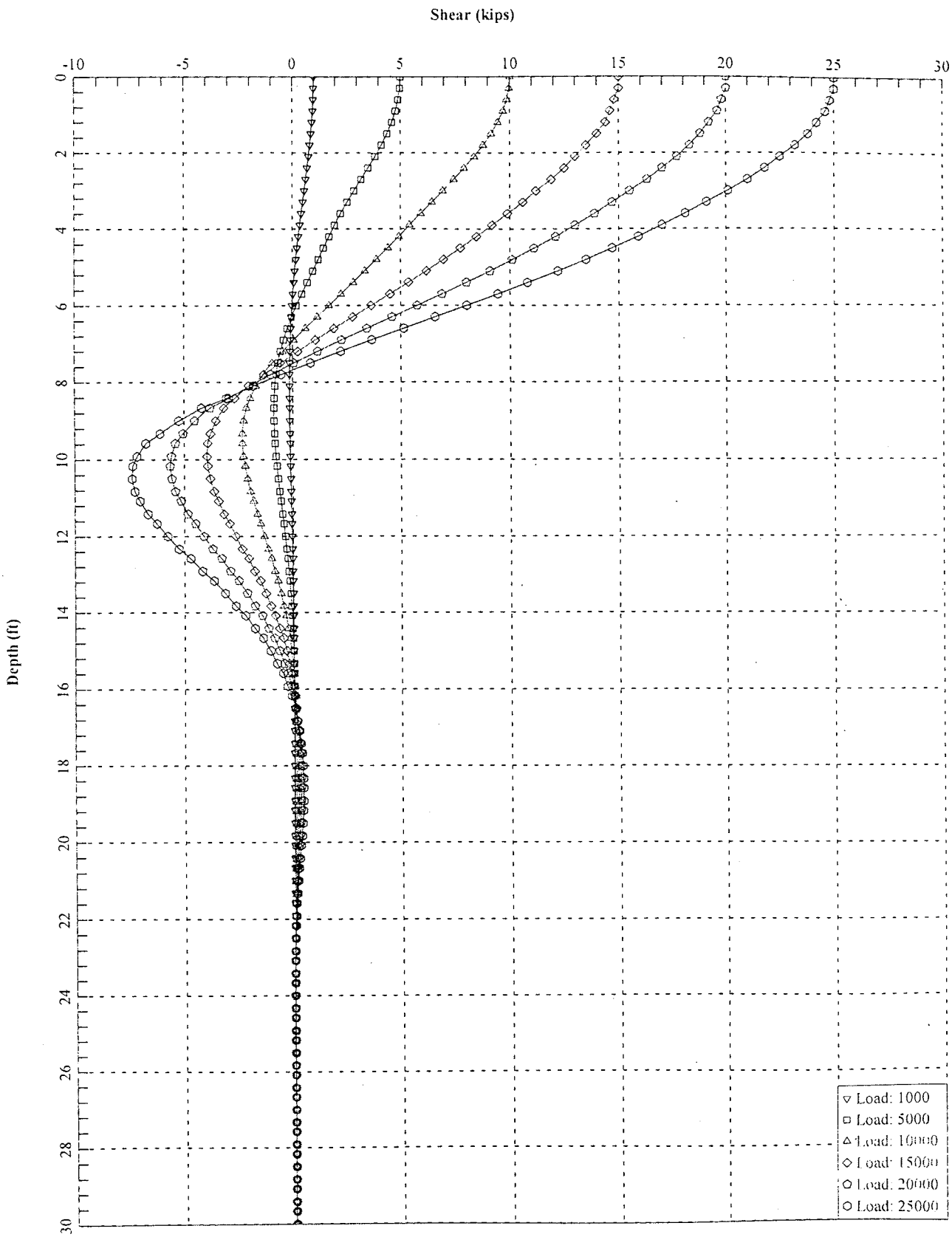
Houghton T.S. 12-inch dia. Augercast Piles Free Head



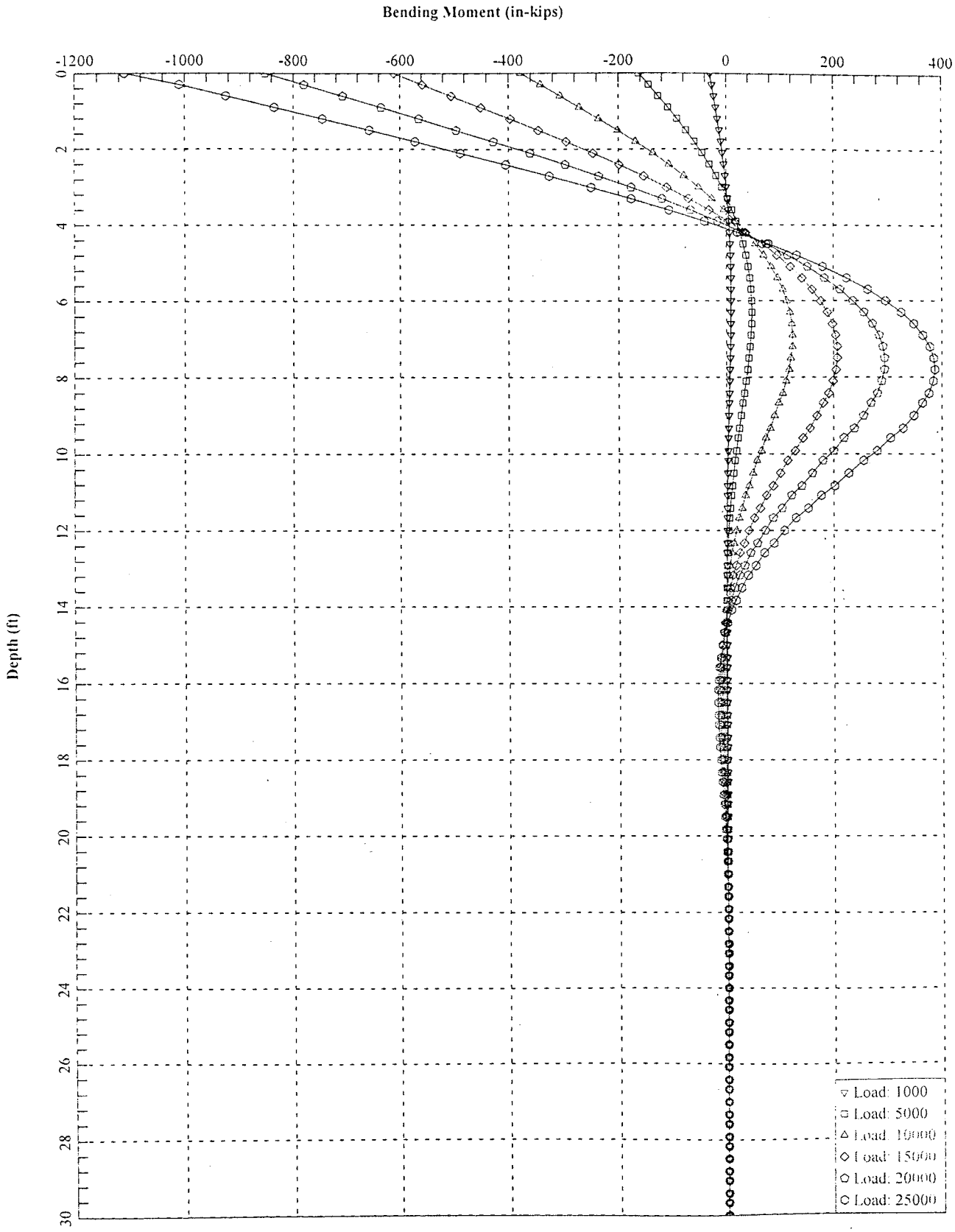
Houghton T.S. 12-inch dia. Augercast Piles Free Head



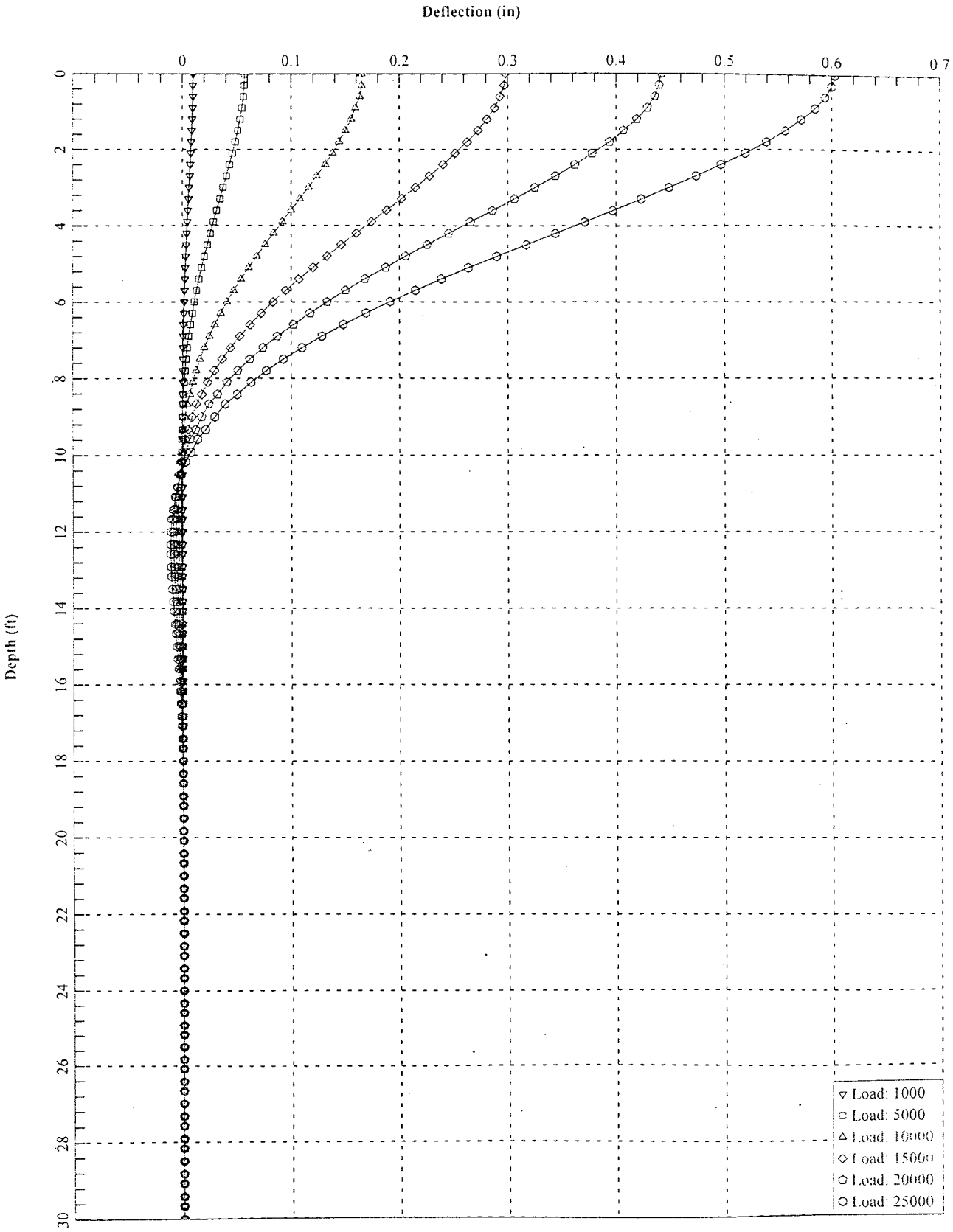
Houghton T.S. 12-inch dia. Augercast Piles Free Head



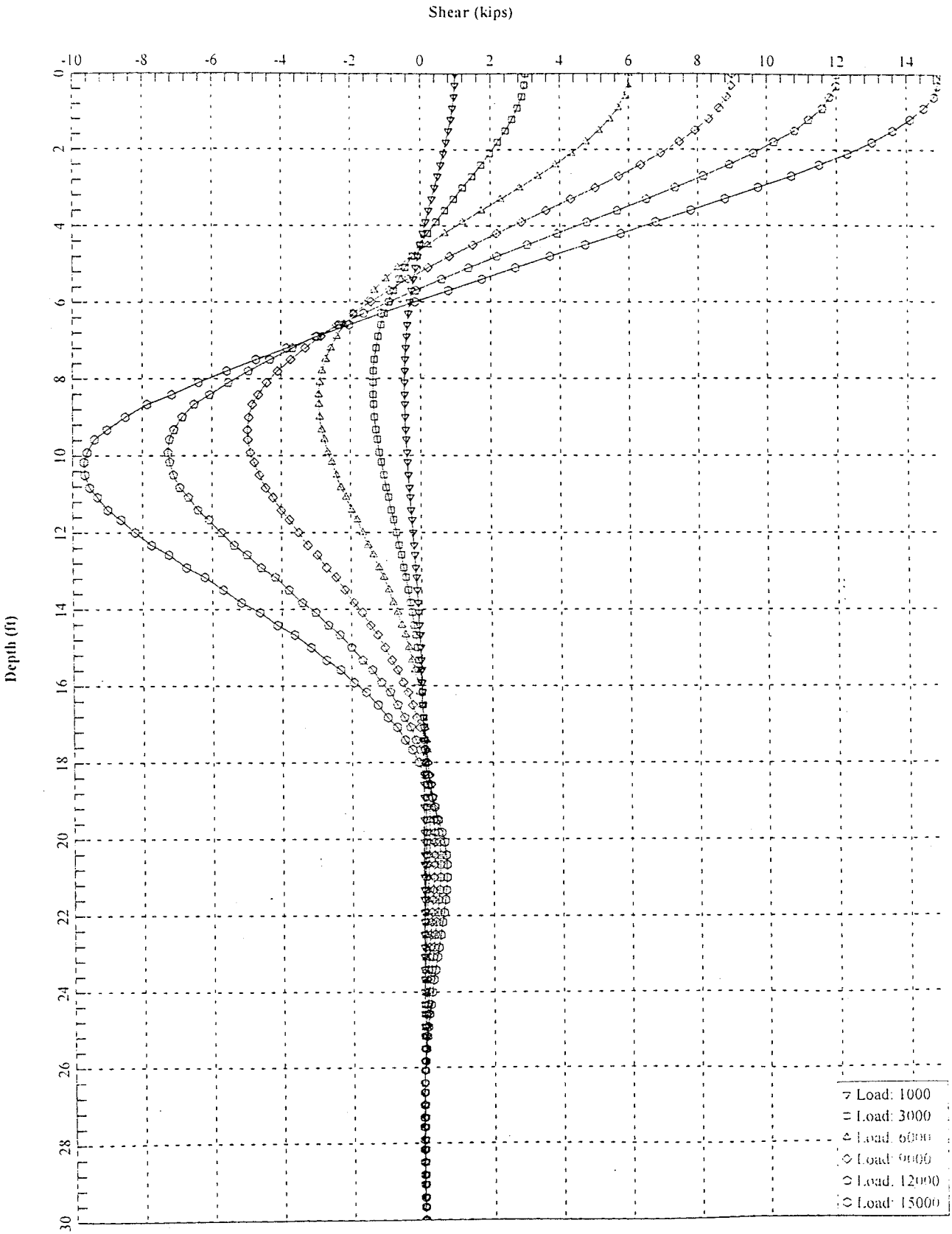
Houghton T.S. 12-inch dia. Augercast Piles Fixed Head



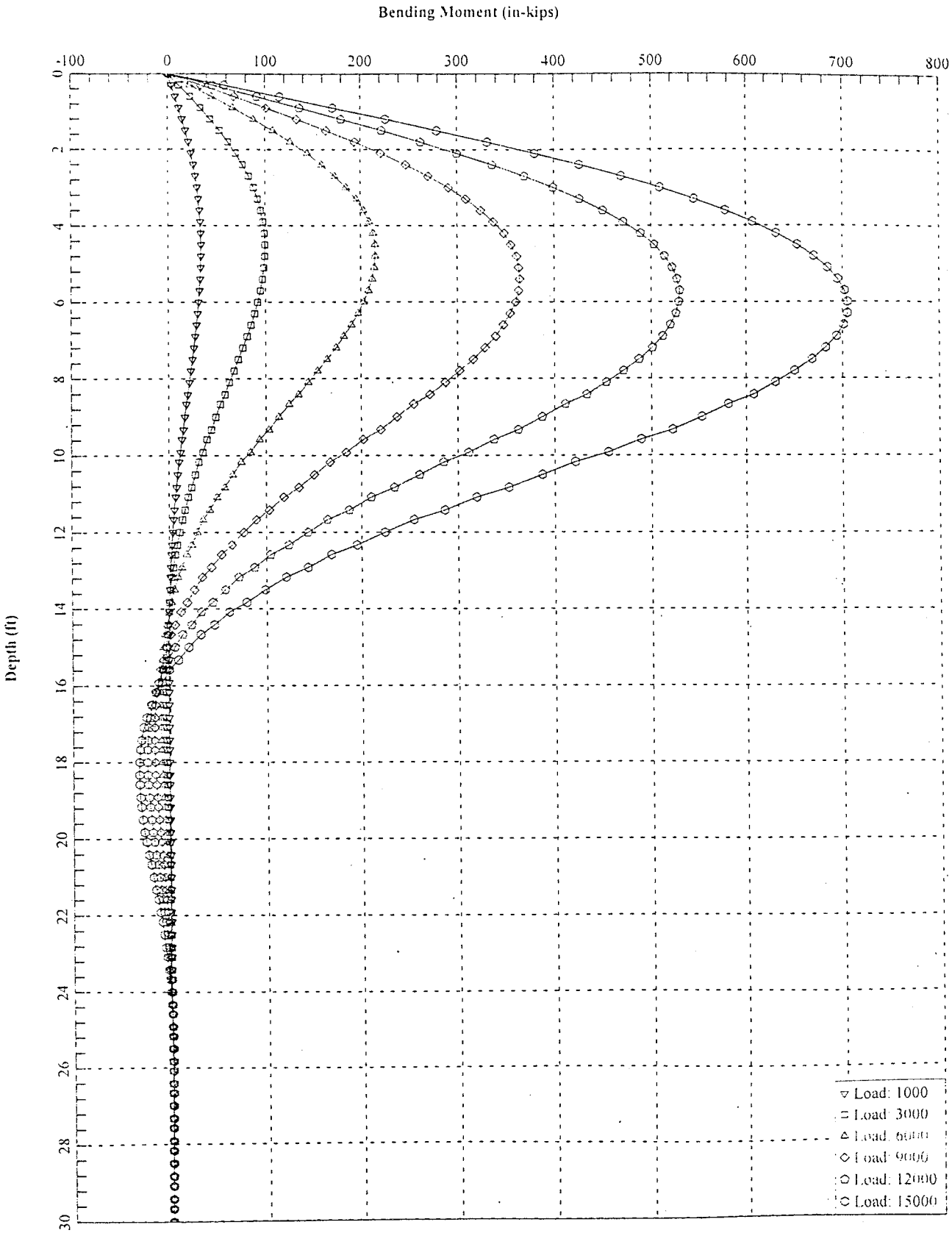
Houghton T.S. 12-inch dia. Augercast Piles Fixed Head



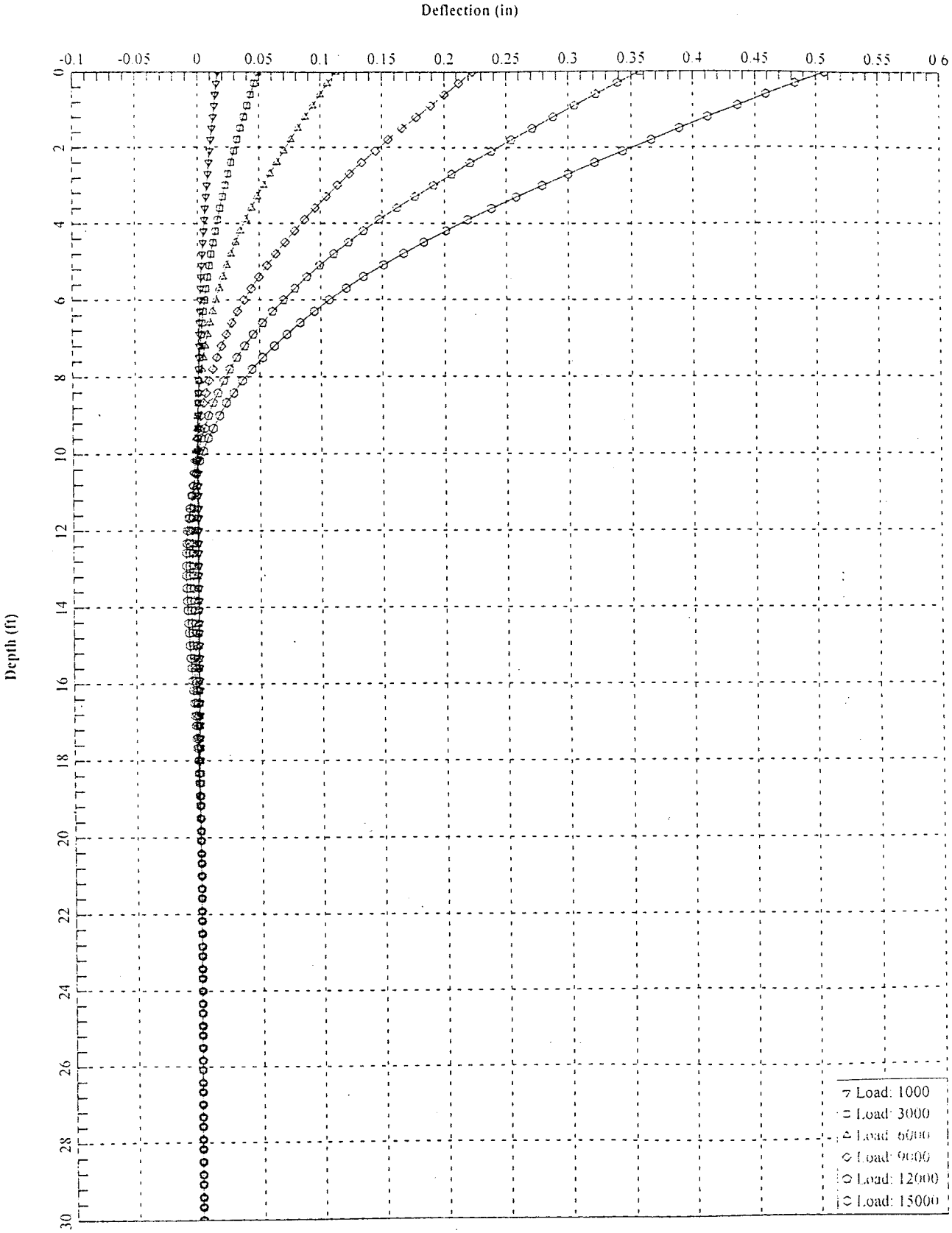
Houghton T.S. 12-inch dia. Augercast Piles Fixed Head



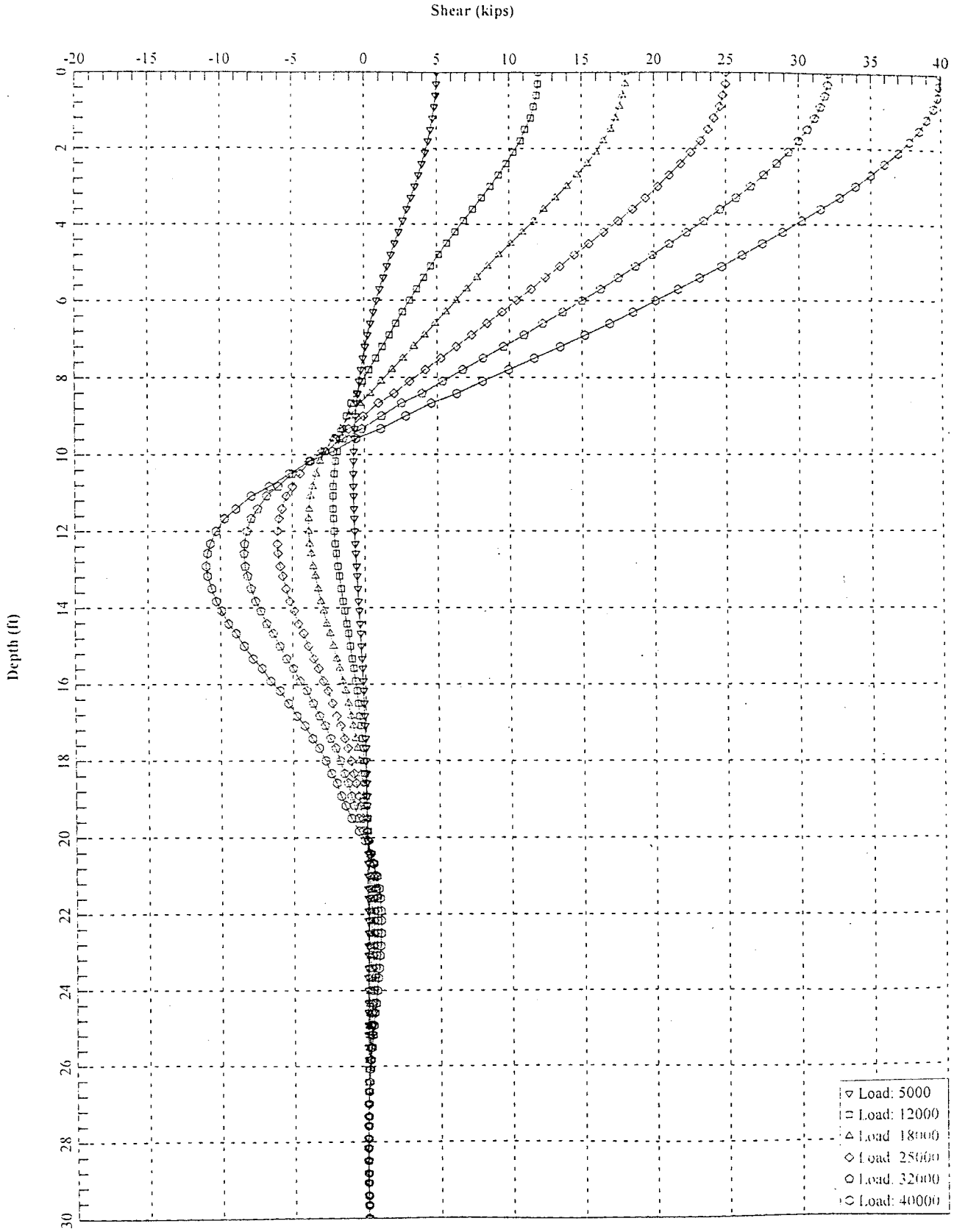
Houghton T.S. 16-inch dia. Augercast Piles Free Head



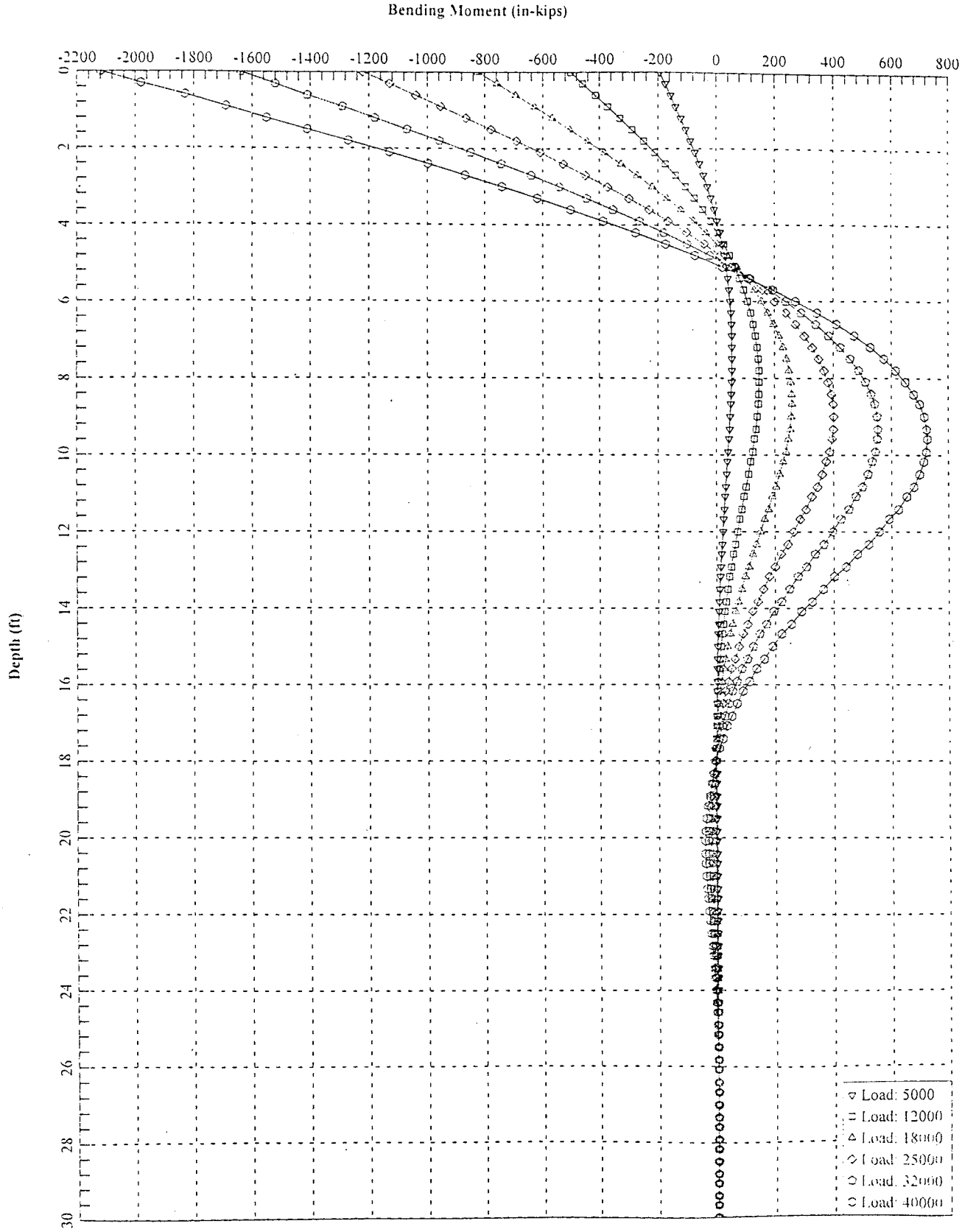
Houghton T.S. 16-inch dia. Augercast Piles Free Head



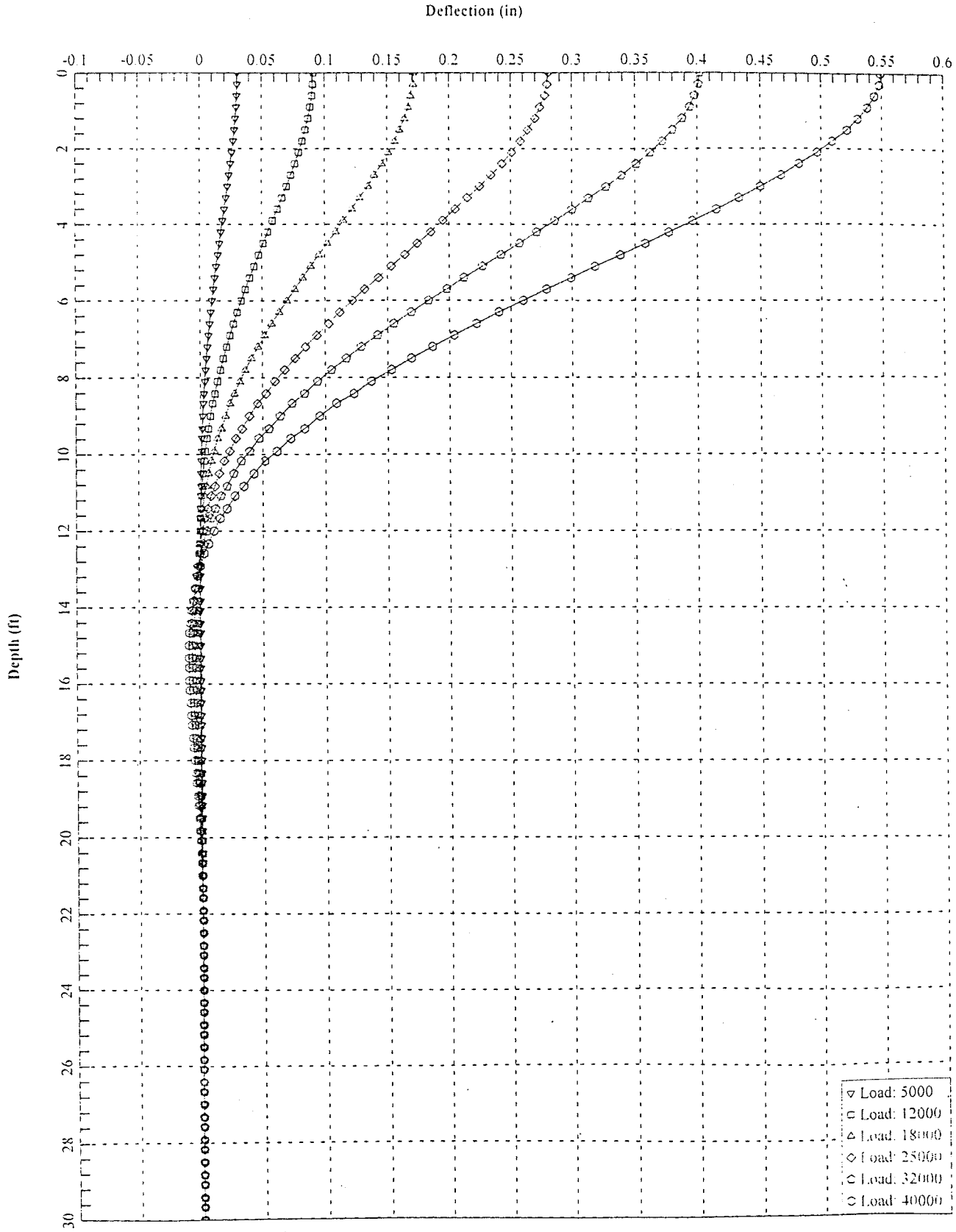
Houghton T.S. 16-inch dia. Augercast Piles Free Head



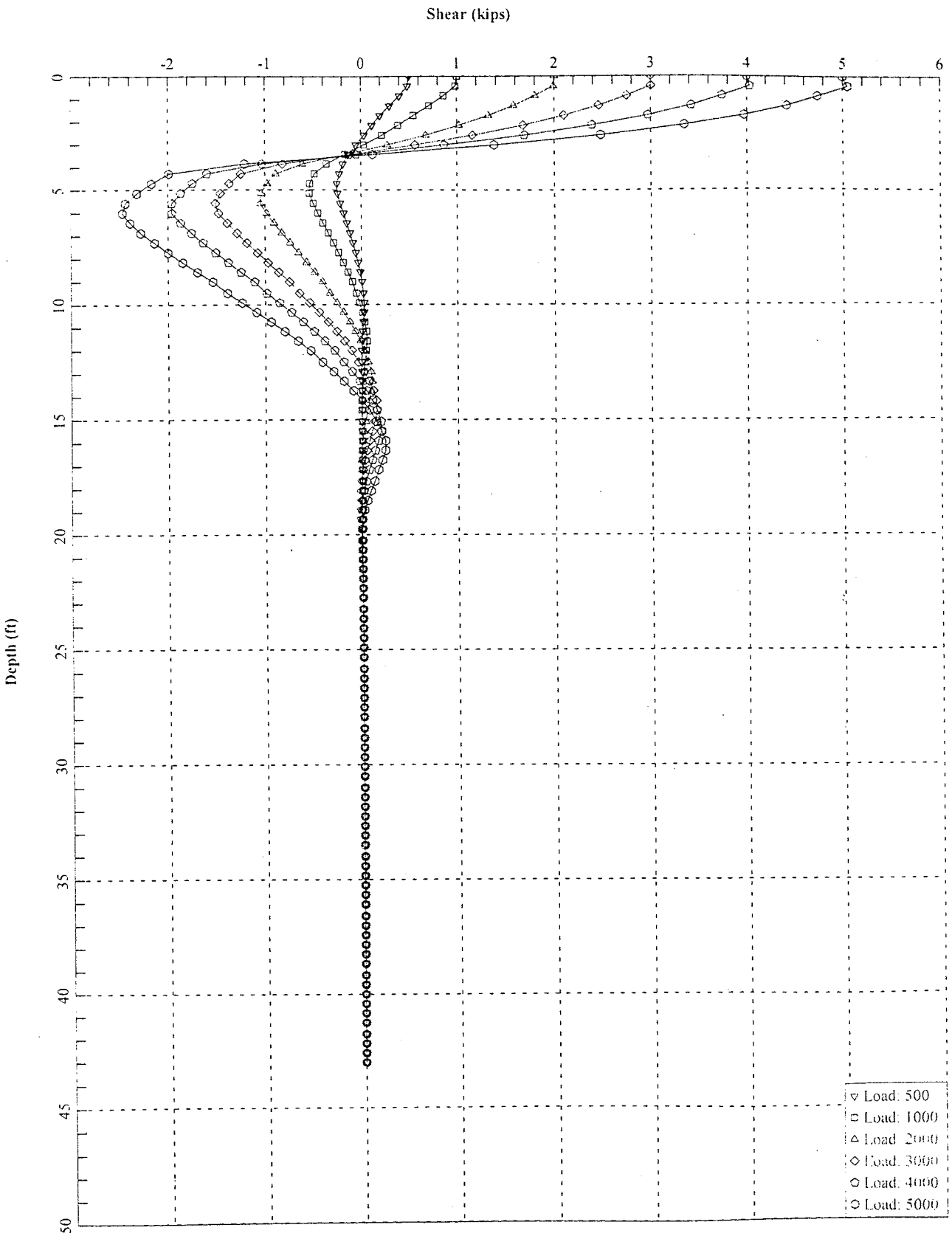
Houghton T.S. 16-inch dia. Augercast Piles Fixed Head



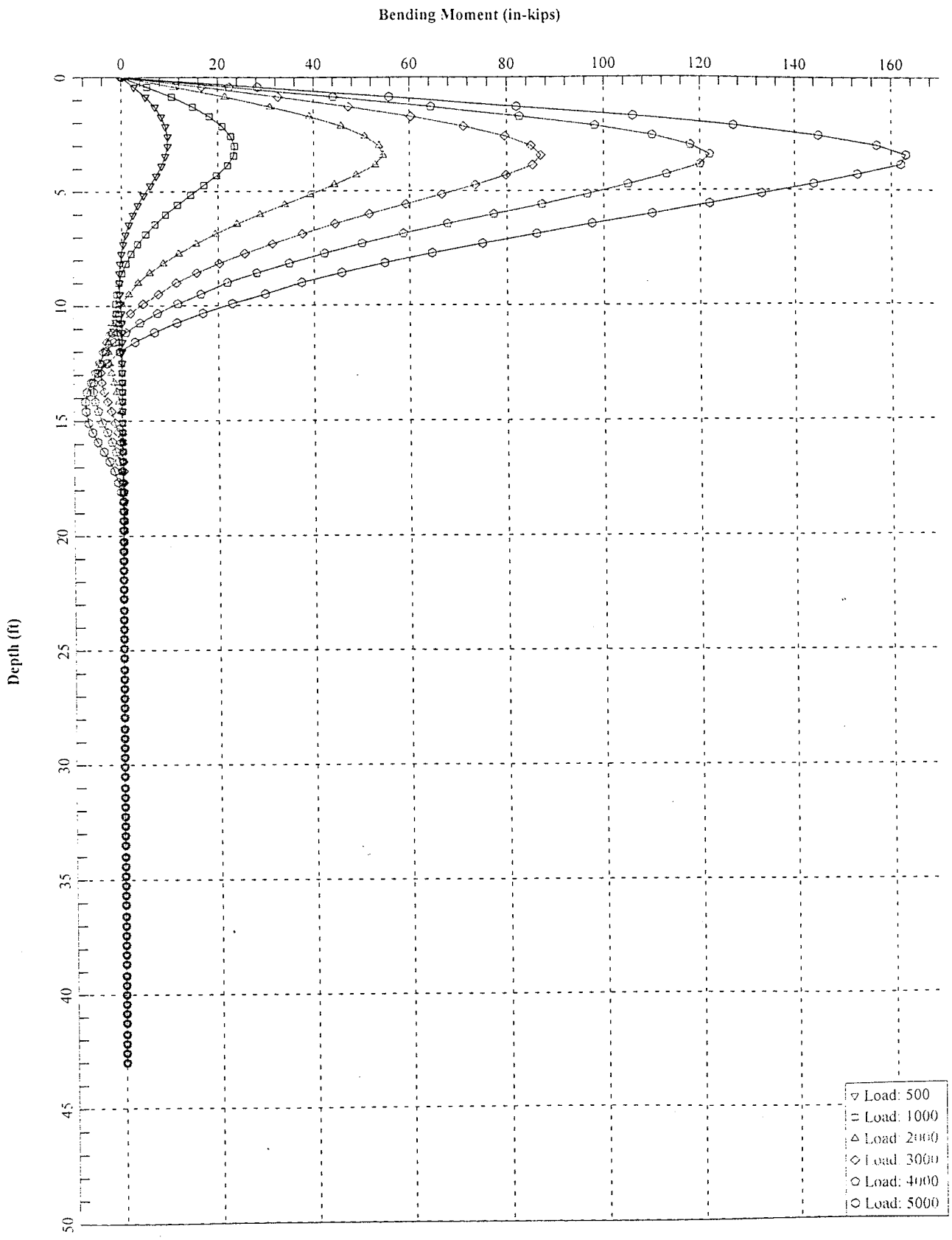
Houghton T.S. 16-inch dia. Augercast Piles Fixed Head



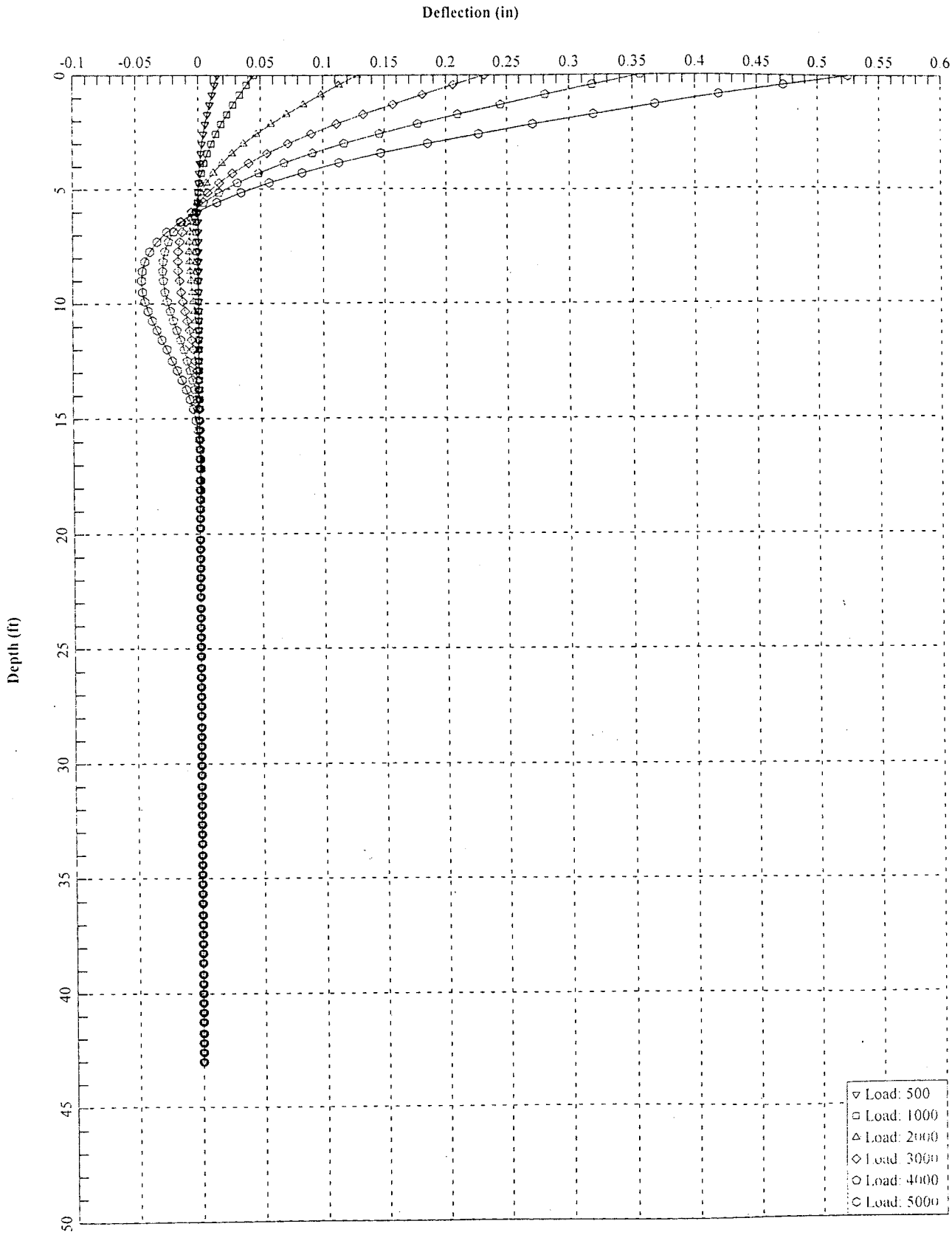
Houghton T.S. 16-inch dia. Augercast Piles Fixed Head



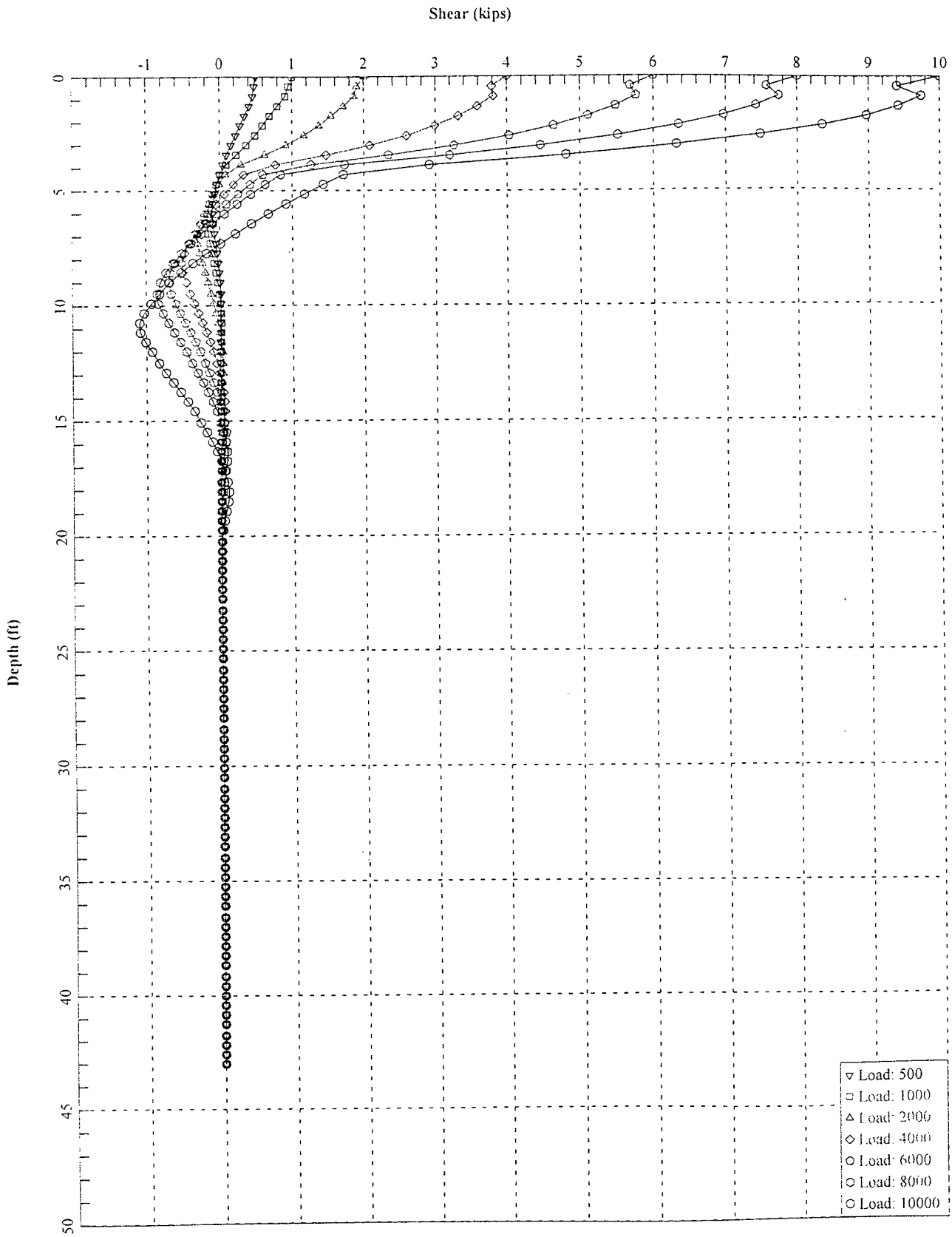
Algona T.S. 5.5-inch dia. Micropiles Free Head



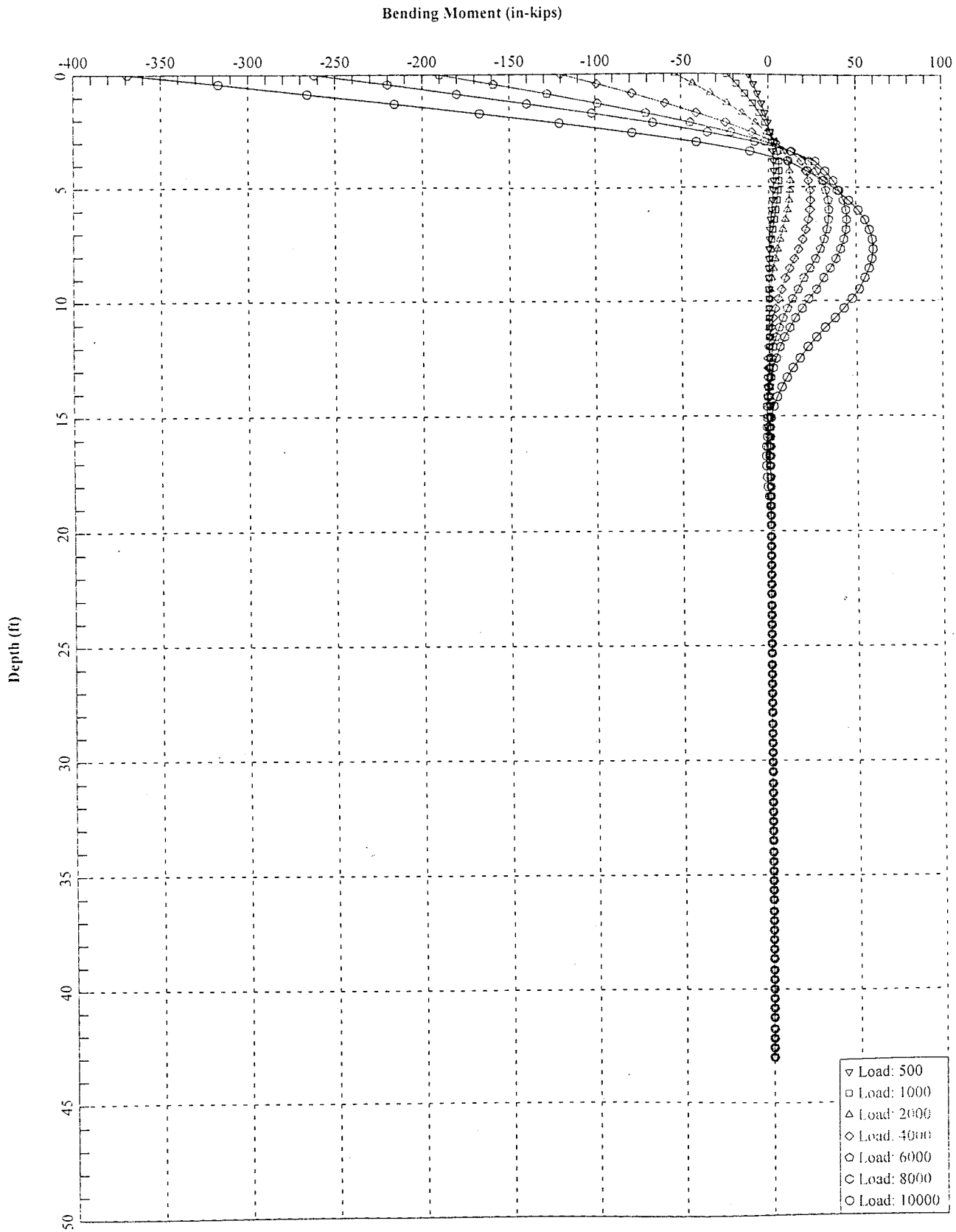
Algona T.S. 5.5-inch dia. Micropiles Free Head



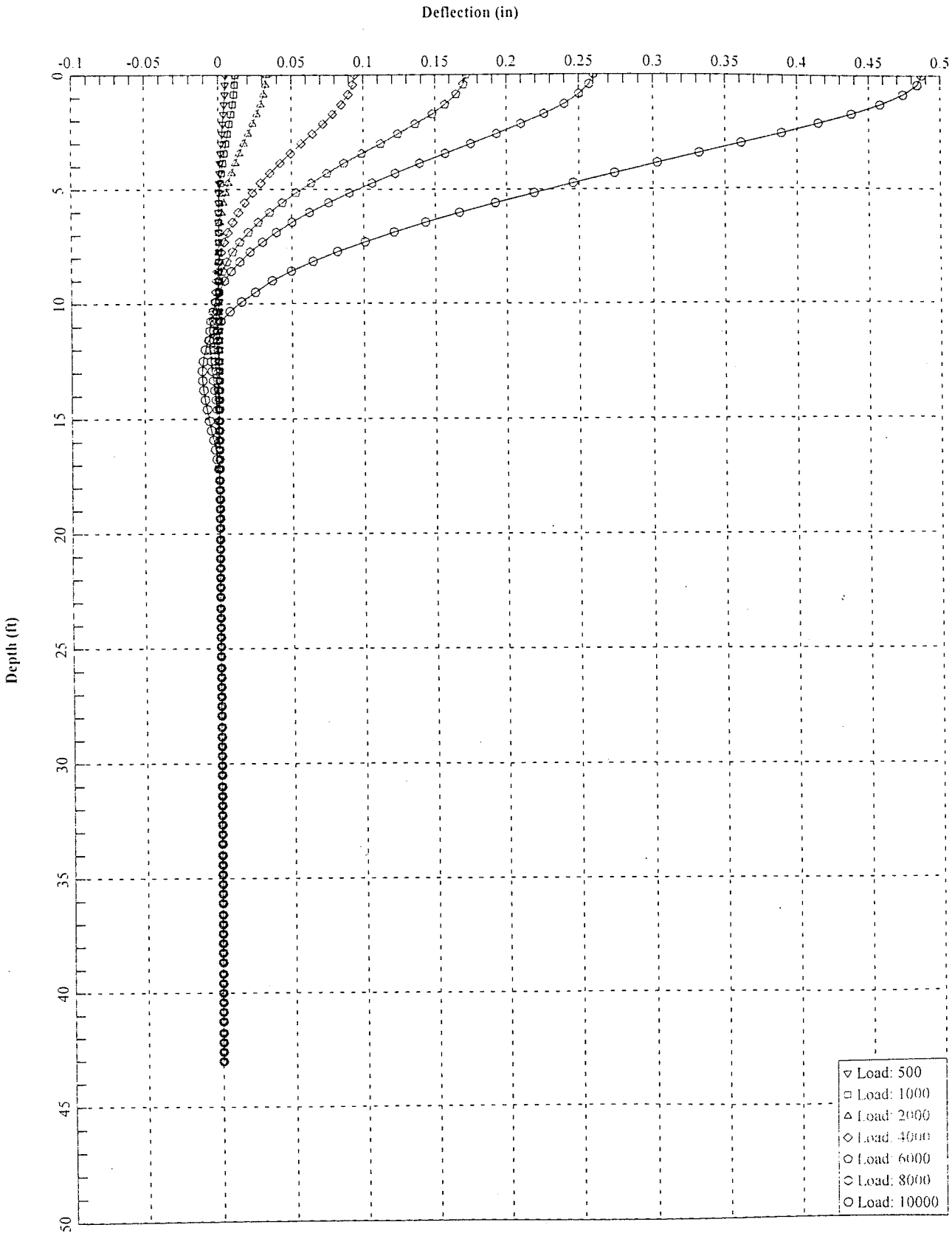
Algona T.S. 5.5-inch dia. Micropiles Free Head



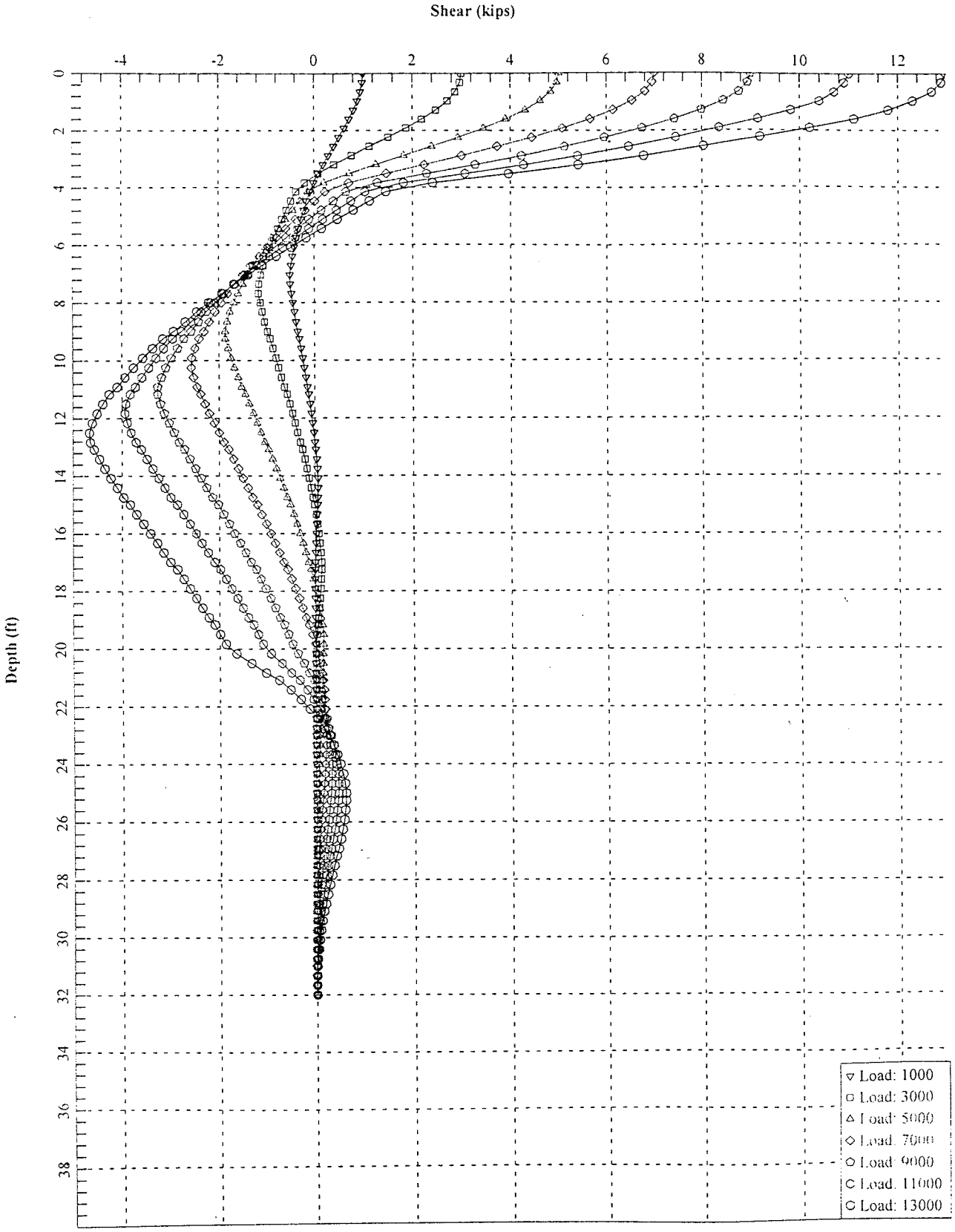
Algona T. S 5.5-inch Micropiles Fixed Head



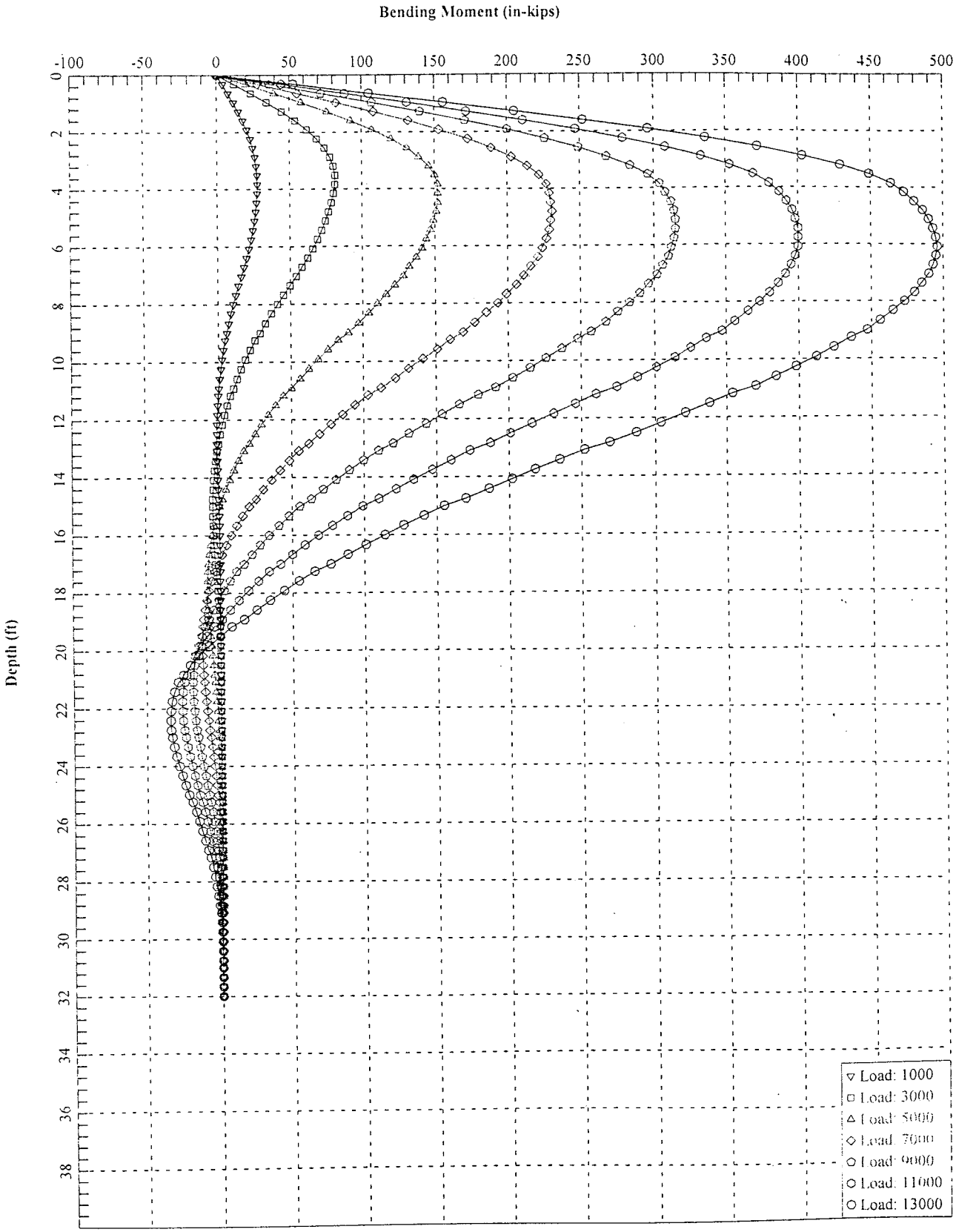
Algona T.S. 5.5-inch dia. Micropiles Fixed Head



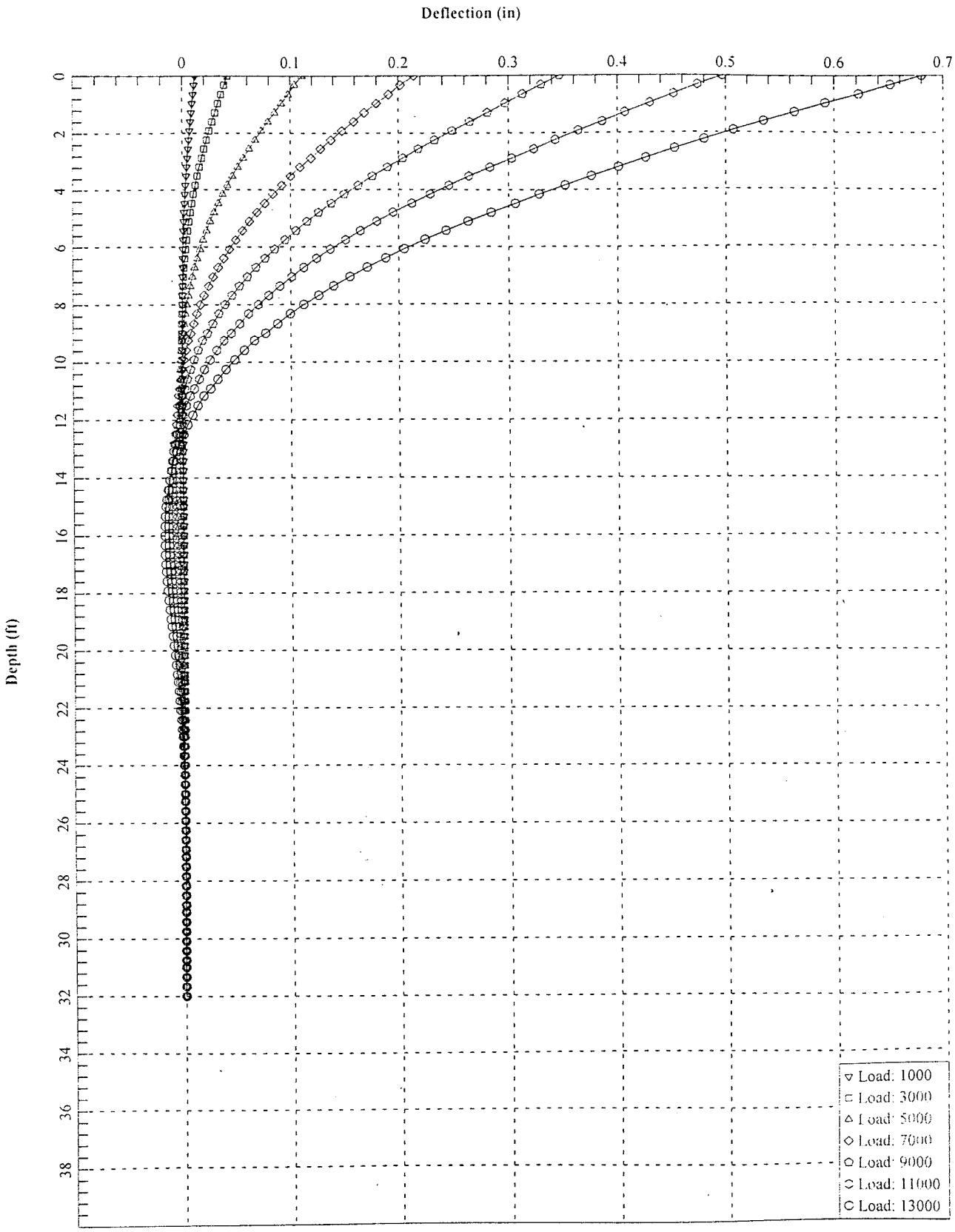
Algona T.S. 5.5-inch dia. Micropiles Fixed Head



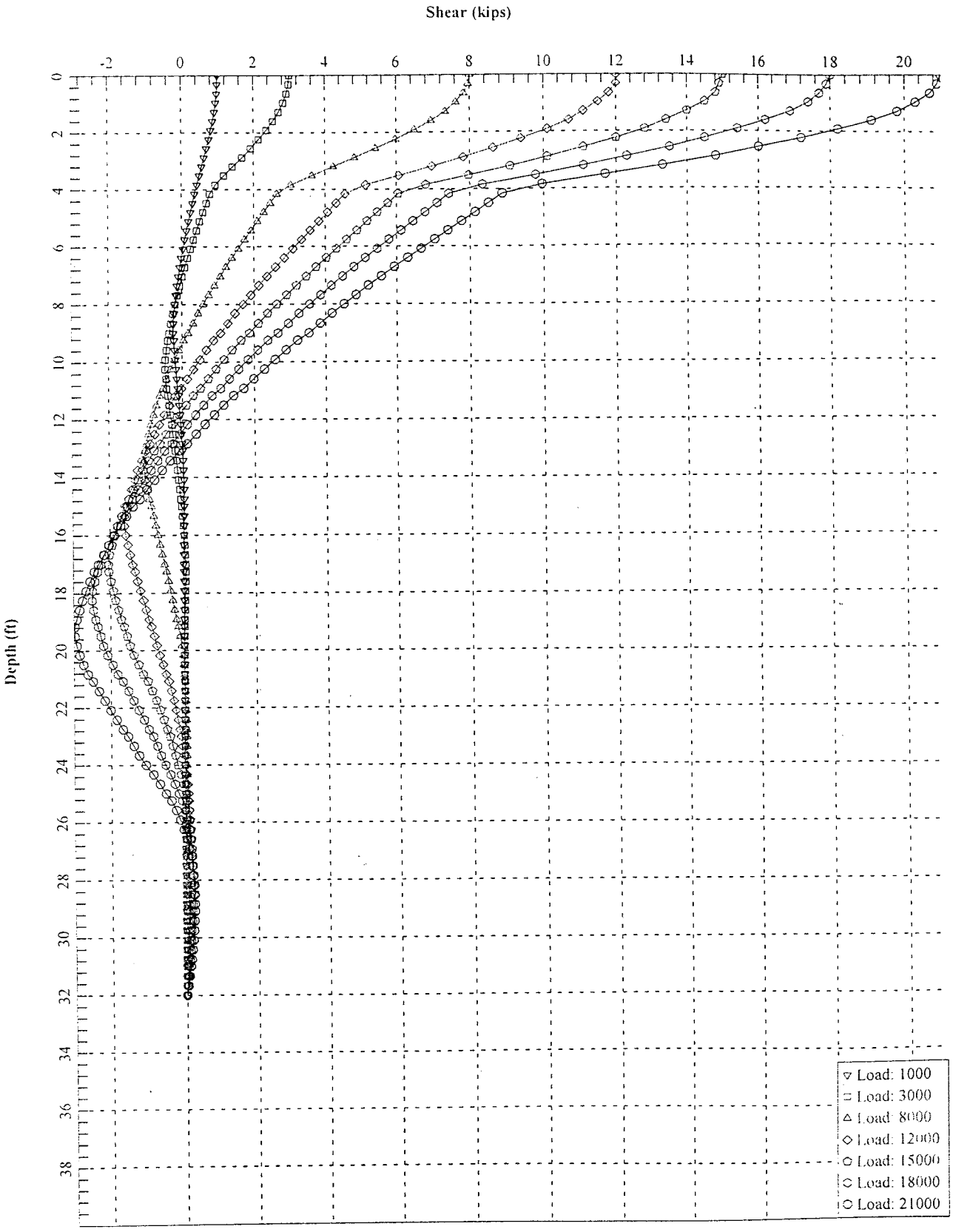
Algona T.S. 12-inch dia. Steel Pipe Piles Free Head



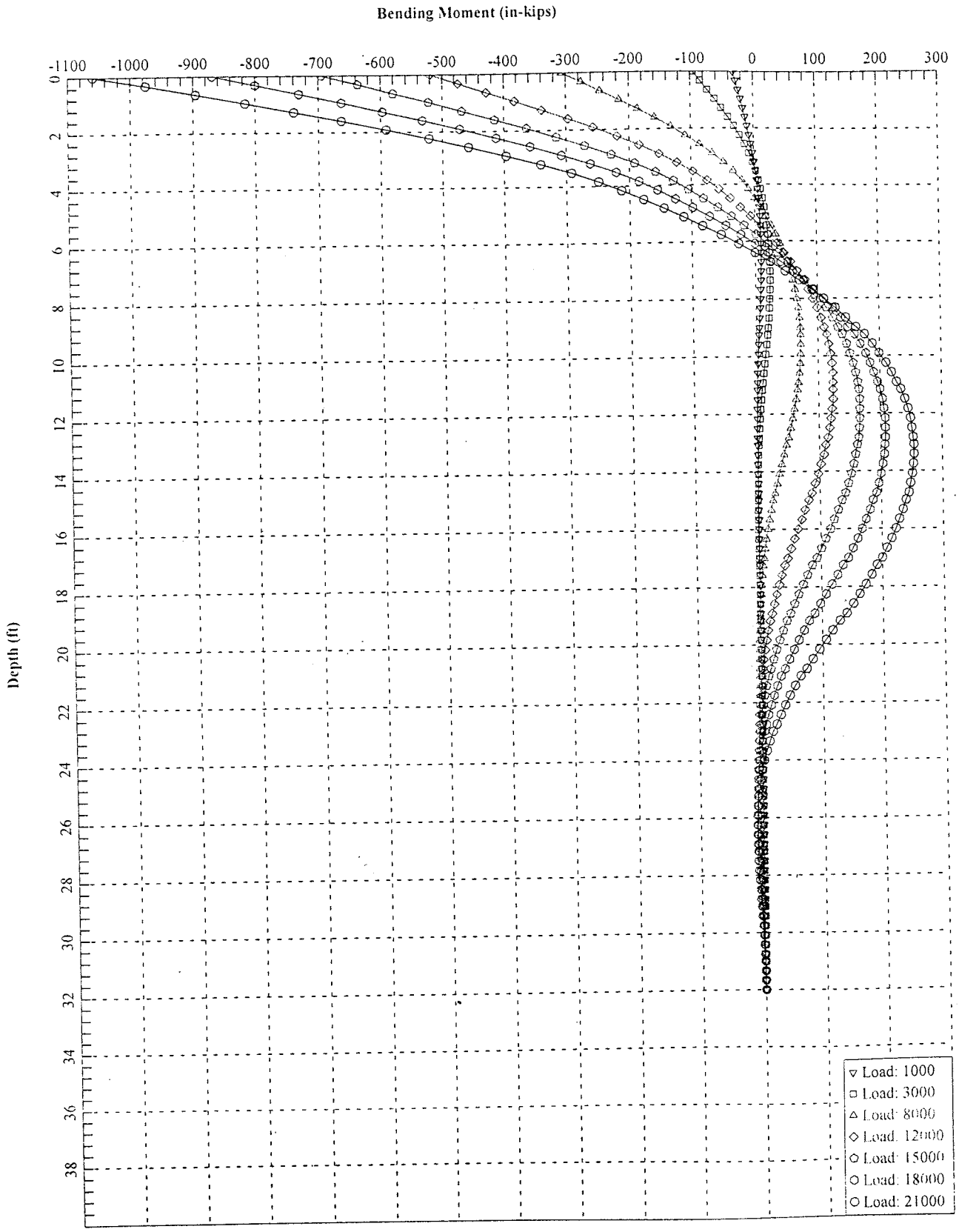
Algona T.S. 12-inch dia. Steel Pipe Piles Free Head



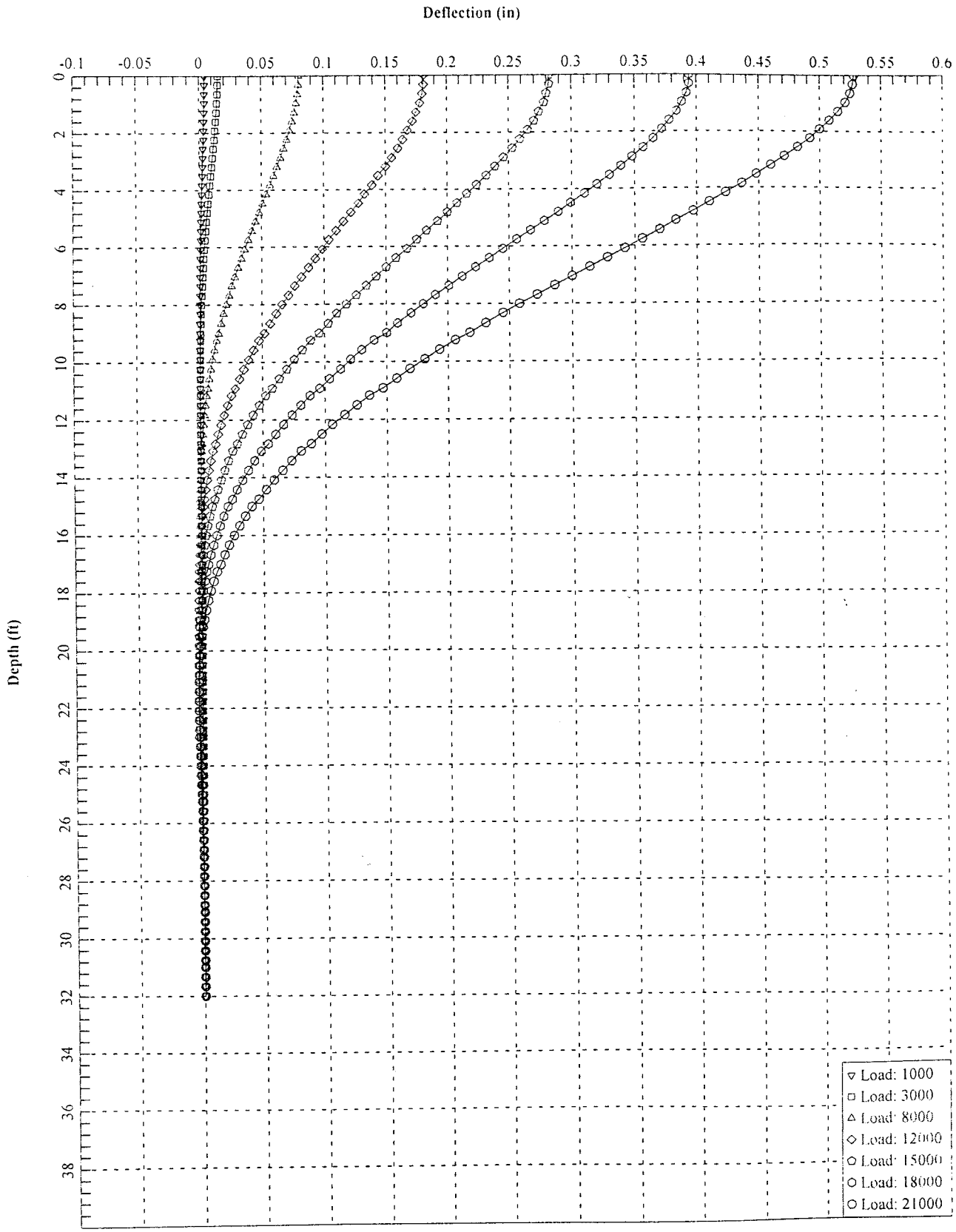
Algona T.S. 12-inch dia. Steel Pipe Piles Free Head



Algona T.S. 12-inch dia. Steel Pipe Piles Fixed Head



Algona T.S. 12-inch dia. Steel Pipe Piles Fixed Head



Algona T.S. 12-inch dia. Steel Pipe Piles Fixed Head