DRAFT REPORT
GEOTECHNICAL ENGINEERING SERVICES
PORT OF WILLAPA SHIP FABRICATION AND LAUNCHING FACILITY
RAYMOND, WASHINGTON

APRIL 9, 2004

FOR
SANTA MARIA SHIPPING

April 9, 2004

Santa Maria Shipping
1510 – 10th Street
Bremerton, Washington 98337

Attention: Captain Virgil S. Elkinton

Subject: Draft Report
Geotechnical Engineering Services
Port of Willapa Ship Fabrication and Launching Facility
Raymond, Washington
File No. 9538-002-00

INTRODUCTION

GeoEngineers, Inc. is pleased to present the results of our geotechnical engineering services in support of design of a proposed ship fabrication and launching facility at the Port of Willapa Harbor property in Raymond, Washington. The site is located approximately as shown on the Vicinity Map, Figure 1. Our project understanding is based on information you provided and review of subsurface data in our files for nearby projects.

We understand the proposed facility will include a high bay, steel frame fabrication building. A launch rail apparatus will be situated within an excavated slipway that will extend north from the fabrication building into the waterway. The building will have plan dimensions of about 300 feet by 115 feet and will enclose a ship fabrication operation for vessels up to about 360 feet (110 meters) long and up to about 4,500 gross tons. The building will include supports for two, 80-ton capacity cranes. A second building is planned on the west side of the fabrication building and will have plan dimensions of about 50 feet by 200 feet. The proposed improvements are shown on the Site Plan, Figure 2.

The vessels will be launched rearward into the harbor on a dual-rail system within the slipway. The estimated load per rail is expected to range between about 10 tons per foot and 15 tons per foot. The total length of the rail system will be about 800 feet. The rails will extend from inside the building to about 50 feet into the adjacent waterway. The top of rail inside the fabrication building (Station 0+00) will be at about Elevation 13-1/2 feet (NGVD 1929). The top of rail at the outboard end of the launch rail (Station 8+00) will be at about Elevation -6.2 feet. Cuts to establish the slipway are expected to range up to about 10 feet.

In view of the relatively large loads, and soft surficial soil conditions, it will be necessary to support the proposed buildings and launch rails on pile foundations. We understand the floor system inside the fabrication building will not be pile supported and will comprise a series of individual, precast concrete panels. Each panel will have dimensions of about 8 feet by 8 feet. Individual panels will be used to

support portions of the vessel during fabrication. The support panels will typically be about 30 feet apart and be subjected to a continuous 1,200 pounds per square foot (psf) load for at least one to two months. Settlement of individual panels under the planned loads is expected and provisions will be made to relevel the panels.

PURPOSE AND SCOPE

The purpose of our services is to perform subsurface exploration and testing as a basis for developing geotechnical recommendations for pile foundations to support the building and launch rails, and to develop recommendations for other proposed improvements, as appropriate. Our scope of services is described in our proposal dated December 23, 2003, and a supplemental scope letter dated February 11, 2004. Specifically, our scope of services includes:

1. Reviewing readily available geologic, and geotechnical reports for the site and site vicinity.
2. Performing three shallow auger borings to evaluate soil conditions in the slipway area.
3. Performing subsurface exploration by advancing three Cone Penetrometer Tests (CPTs) along the alignment of the launch rail between the shoreline and the proposed ship fabrication building.
4. Performing laboratory testing to assist in our evaluation of pertinent engineering properties of the encountered materials.
5. Developing recommendations for pile foundations to support the building, crane rails and launch rails. We include allowable downward, upward and lateral load capacity criteria for selected piles. We also include construction recommendations.
6. Developing recommendations for support of the precast concrete floor slab panels.
7. Developing recommendations for design and installation of sheet pile walls, including deadman anchor criteria, if required for additional lateral restraint.
8. Discussing options for constructing the slipway using either cut slopes or sheet pile walls.

SITE CONDITIONS

GENERAL

We explored shallow subsurface conditions by drilling three hollow stem auger borings to depths ranging between about 19 and 20 feet below existing site grade at the approximate locations shown on Figure 2. We investigated subsurface conditions at depth by advancing three CPT soundings to practical refusal which occurred at depths ranging between about 74 feet and 83 feet below existing site grade. The approximate locations of the CPT soundings are also shown on Figure 2. The CPT soundings were performed on December 30, 2003. The auger borings were performed on March 10, 2004. More detailed descriptions of our site exploration and laboratory testing programs are included in Attachment A.

DATA REVIEW

We reviewed our prior report dated June 28, 2002 for information on subsurface conditions in the site vicinity. The study area for our prior report was generally to the south of the proposed ship fabrication

and launching facility. Our prior study included exploratory test pits and a hollow stem auger exploratory boring. The subsurface conditions disclosed in the test pits, typically comprises a variable thickness of fill overlying alluvially deposited silt with organics. The fill comprises a mixture of inorganic soil (sand and gravel) and wood waste and extends to depths ranging between about 2 feet and 8 feet below grade at the test pit locations. Some of the fill may be dredge fill. The native soil beneath the fill in the test pits is described as very soft organic silt. The exploratory boring penetrated the fill and encountered very soft silt with occasional very loose silty sand layers to a depth of about 59 feet.

SURFACE CONDITIONS

The proposed ship fabrication and launching facility is located on a relatively flat-lying site on the south side of the Willapa River at the Port of Willapa Harbor. Based on prior studies we have performed at the Port of Willapa Harbor, we understand up to four lumber mills have occupied portions of the property since the early 1900s. Existing grades at the site have been established by filling. The ground surface in the vicinity of the proposed fabrication building currently ranges between about Elevation 10 feet and Elevation 12 feet, except for an existing ditch near the south limit of the proposed building. The bottom of the ditch appears to be at about Elevation 8 feet in the proposed building area.

SUBSURFACE CONDITIONS

We encountered a variable thickness of existing fill at the locations of the exploratory borings performed for this study. The fill extends to depths ranging between about 6-1/2 feet and 12 feet below existing site grade at the boring locations and comprises very soft to medium stiff sandy silt with layers of very loose silty sand with wood debris. The native soil beneath the fill in the shallow borings comprises very soft silt with occasional organic material and sand. We encountered a very loose silty sand layer at a depth of about 19-1/2 feet in Boring 1.

Groundwater was encountered at depths ranging between about 1 foot and 17-1/2 feet during drilling. We expect that depth to groundwater will vary depending on seasonal precipitation and with tidal fluctuation in the Willapa River channel.

We advanced CPT soundings to evaluate subsurface conditions at depth beneath the site. The CPT logs are presented in Attachment A. Soil samples are not obtained with CPT soundings. Soil types are interpreted based on tip and side friction resistance correlations.

In CPT-1, the soft native silt layer is interpreted to extend to a depth of about 73 feet. Medium dense silty sand is interpreted from 73 feet to about 78 feet. Practical refusal was encountered at a depth of about 80.2 feet. Based on our understanding of local geologic conditions, we judge that refusal likely occurred in weathered basalt rock.

Similar conditions to those interpreted at the location of CPT-1 were interpreted at the location of CPT-2 to a depth of about 72 feet. Medium dense to dense sand layers are interpreted from 72 feet to a depth of about 76 feet. Practical refusal occurred in what we interpret is weathered basalt rock at a depth of about 83.2 feet.

The soft native silt extends to a depth of about 52 feet based on the results at CPT-3. Interlayered loose to medium dense sand and medium stiff silt extends from 52 feet to a depth of about 66 feet. Dense silty sand is inferred below 66-foot depth to a depth of about 74 feet, where we encountered practical refusal.

DISCUSSION AND RECOMMENDATIONS

GENERAL

The proposed facility is located in an area that is underlain by relatively deep and soft alluvial deposits. These deposits have poor supporting capacity and will consolidate (settle) when subjected to additional loads such as area loads induced by filling. We recommend pile foundations for heavily loaded or settlement sensitive structures. We provide recommendations for several pile sizes or pile types. We also provide recommendations for imported fill subbase material to support the planned segmental concrete floor panels in the fabrication building. We expect settlement of the panels will occur when subjected to sustained structural loads as discussed below. Accordingly, the panel design should include provisions for releveling panels that settle.

In addition, the site soils are highly susceptible to excavation disturbance and to moisture. It will be difficult, if not impossible to work the site soils during wet weather conditions. Therefore, we suggest that the work be performed during the normally dry time of the year (summer months). Specific recommendations concerning geotechnical elements of the project are contained in the following sections of this report.

SEISMIC CONSIDERATIONS

The 1997 Uniform Building Code (UBC) recommends that the design earthquake for this site be based on a 10 percent probability of exceedance in 50 years. The design earthquake that corresponds to this probability of exceedance has a Richter magnitude of 7.5 and a peak horizontal ground acceleration of about 0.30 g, where g is the gravitational constant. Based on UBC Table 16-J, the soil profile for the project site is best characterized as Type S_E.

Liquefaction is a loss of strength that certain soils experience when subjected to cyclic stresses, such as would occur during an earthquake. In general, soils which are susceptible to liquefaction include loose to medium dense sands and silty sands that are below the water table.

Based on our site exploration and evaluation loose silty sand layers within the native alluvial soils could liquefy during an earthquake. Although the subsurface silt soils might not liquefy, they are soft to very

soft and highly compressible. These soils will lose strength and will likely deform or settle as a result of ground shaking during and after the design earthquake. Because of this, we recommend that flexible connections be considered where utility pipes connect to rigid structures such as pile-supported buildings.

PILE FOUNDATIONS

Axial Capacity

We evaluated 16-inch- and 24-inch-diameter steel pipe piles and 24-inch-octagonal precast concrete piles for support of heavily loaded structures. Based on our analysis, piles will need to be driven to practical refusal in the dense sand or weathered basalt rock beneath the site in order to develop their structural capacity. Estimated allowable downward and uplift capacities with embedment depth, based on typical conditions disclosed in the CPT soundings are presented in Figures 3 through 8.

Lateral Capacity

We evaluated pile lateral load capacity using the LPILE computer program. The analysis was performed for a single pile, which we assumed was fixed at the top to prevent rotation. We assumed 1/2-inch-thick walls for the 16-inch-diameter pipe piles and 1-inch-thick walls for the 24-inch-diameter pipe piles. We conducted the analysis to estimate lateral load required for 1/2 inch and 1 inch of horizontal deflection at the top of the pile. The analysis results are presented in Attachment B. Reduced lateral load capacities may be appropriate if pile spacings are less than 5 pile diameters or if the top of the pile is not fully fixed. Specifically, we should be consulted if pile spacings in the direction of loading are less than 5 pile diameters.

Pile Installation

For planning purposes, piles should be driven to the estimated embedments indicated above. Actual pile embedments should be based on driving criteria, which are dependent upon the driving hammer characteristics, capacity required, group action and other factors. We are available as consultants to develop driving criteria after the pile installation contractor submits information regarding equipment that will be used for pile driving.

Actual pile embedment lengths also will depend on the subsurface conditions encountered at the location of each pile during driving. We expect pile lengths will vary and the contractor should be prepared to use longer piles in the event actual pile penetration exceeds estimates provided in this report. The contractor also should be prepared to cut and remove sections of piles that meet practical refusal at depths above the estimated pile tip embedments.

The allowable capacities are based on the strength of the supporting soils for the penetrations indicated and include a factor of safety of at least 2 for side friction and 3 for end bearing. Structural characteristics of the pile materials or foundation connections may impose more stringent limitations and should be evaluated during design.

We recommend that piles installed for support of the proposed improvements be furnished and prepared in accordance with Section 6-05 of the Washington State Department of Transportation (WSDOT) Standard Specifications. When selecting a pile hammer, the contractor should also consider wall thickness of the pipe piles and potential for overstressing piles during driving. The selected hammer must be capable of driving the piles to bearing and have sufficient energy to verify the required pile capacity. If precast concrete piles will be used, we recommend a test pile program be conducted before ordering final pile lengths. A Wave Equation Analysis for Piles (WEAP) also should be performed to assess potential for developing excessive tensile forces in the piles during driving. GeoEngineers is available to develop pile refusal criteria once a pile type and hammer has been selected, as discussed previously.

GeoEngineers is available to be present during pile installation. Our representative would monitor pile installation operations, review the driving records and be available to provide consultation, as needed.

We estimate that settlement of pile foundations designed and installed as recommended will be on the order of 1/2 inch or less. Most of the settlement should occur as the load is applied. Post construction differential settlement between piles is expected to be negligible.

SLIPWAY EXCAVATION

We evaluated stability of cut slopes up to about 10 feet high that extend through the existing fill and into the underlying soft native silt deposit. Based on our analysis, we recommend a maximum cut slope inclination of 3.5H:1V (Horizontal to Vertical) for design. The results of our stability analysis are presented in Figure 9. Our analysis does not include consideration of surcharge loads near the slope crest. We should be consulted if surcharge loads are anticipated within a horizontal distance from the slope crest equal to the height of the slope. We recommend a riprap facing be used to protect the slipway cut slopes within the tidal fluctuation zone. We anticipate the safety factor against sliding should increase after the riprap facing is installed.

SHEET PILE WALLS

We understand sheet pile walls may be used to accomplish excavation for portions of the slipway. Our recommended lateral soil pressures for design are presented in Figure 10. Hydrostatic pressures should be included when evaluating wall stability, when appropriate. The lateral earth pressures provided do not include surcharge loads. We should be consulted if surcharge loads are anticipated.

Based on our experience, the soft native silt can be sensitive to disturbance and lose strength. We recommend a minimum 2-foot-thick layer of quarry spalls be placed on the slipway subgrade in front of the sheet pile wall in order to provide improved stability against overturning, and to help limit outward rotation of the sheet pile wall.

If additional lateral restraint is needed for stability, a deadman anchor can be used. We recommend the anchor be placed not less than 12 feet from the face of the wall for a deadman no deeper than 3 feet. Anchor capacity can be estimated based on an equivalent fluid density of 240 pounds per cubic foot (pcf) against the face of the anchor block, for embedments up to 3 feet below grade. Short batter piles also could be used as an alternative to a deadman anchor to provide lateral restraint for the sheet pile wall.

A soldier pile and lagging wall could be used instead of sheet piles. In this case, the active earth pressures should be applied over the pile spacing above the cut line, and over the pile diameter below the cut. Passive pressure on the embeded portion of the pile may be assumed to act to cut over 2.5 pile diameters.

SEGMENTAL FLOOR PANELS

We understand you intend to use segmental floor panels in the fabrication building. Because of the variable density and quality of the existing fill, we recommend the upper 2 feet below subgrade be removed and replaced with sand and gravel subbase fill to provide more uniform support beneath the panels. We recommend placing a geogrid layer in the excavation base to help bridge possible soft/loose fill or native soil below the recommended overexcavation depth. We suggest Tensar 5T or equal be used.

We understand certain individual panels forming the fabrication building floor will be used for temporary support during ship fabrication. Based on an average panel load of 1,200 psf, we anticipate these panels will settle as a result of consolidation of the underlying soft native silt. The settlement will occur differentially with respect to adjacent panels that are not loaded. We calculate total settlement on the order of 2 to 4 inches over the anticipated loading period. Additional settlement will occur if the support load is left in place for several months. Accordingly, periodic re-leveling of temporary support panels should be anticipated.

SITE PREPARATION

General

We understand some limited cuts and fills are planned to establish the fabrication building subgrade. We also anticipate cuts for the slipway will extend up to about 10 feet. The native soil and some of the fill soils encountered in our explorations have high fines (material passing the U.S. No. 200 sieve) contents and are extremely moisture sensitive. These materials will be difficult, if not impossible, to operate equipment on when wet, or to compact during wet weather. Operation of heavy equipment at the site under wet conditions can be expected to result in considerable disturbance to the exposed subgrade soils. Once disturbed, the softened soils will likely have to be removed and replaced. As a consequence, the earthwork will be much more economical during periods of dry weather.

Clearing and Site Preparation

The building areas should be stripped of all deleterious matter and any subsurface debris or rubble that would be in the way. We understand only limited site grading (less than 2 feet of cut or fill) will occur in

the building area. Prior to placement of additional fills or subbase fill, we recommend that a member of our staff evaluate the exposed subgrade conditions. The exposed subgrade soil should be proofrolled with heavy rubber-tired equipment during dry weather or probed with a 1/2-inch-diameter steel rod during wet weather. Any soft, loose or otherwise unsuitable areas delineated during proofrolling or probing should be recompacted, if practical, or overexcavated to a depth of at least 2 feet and replaced with structural fill, based on the recommendations of our site representative. If deep pockets of soft or pumping soils are encountered, it may be possible to limit the depth of overexcavation by placing a woven geotextile fabric on the overexcavated subgrade and covering the geotextile with structural fill. The geotextile will provide additional support by bridging over the soft material, and will help reduce fines contamination into the structural fill.

STRUCTURAL FILL

General

All new fill used within the proposed building areas should be placed as structural fill. Structural fill material should be free of debris, organic contaminants or rock fragments larger than 6 inches. Particle sizes larger than 3 inches should be excluded from the top 1 foot of fill.

The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil at the time of placement. Adequate compaction cannot be achieved when the soil moisture content is more than a few percent above or below the optimum level, particularly in soils with high fines contents. As the amount of fines increases, soil becomes increasingly more sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve.

Portions of the existing fill material may be suitable for re-use for structural fill during dry weather conditions, provided the material is at a moisture content suitable for proper compaction. The existing native soil at the site can be used for structural fill only if the material can be properly moisture conditioned to achieve compaction. The material will likely need to be aerated to achieve a suitable moisture content even during dry weather conditions. The existing site soils are not suitable for use as subbase fill beneath the planned segmental floor panels. We recommend subbase fill comprise well graded imported sand and gravel with a fines content not exceeding 5 percent.

Fill Placement

All fill must be placed in thin lifts so that uniform compaction can be achieved throughout the entire lift thickness. In general, granular soils with less than 5 percent fines can be placed in lifts of about 12 inches or less (loose thickness). A soil with a higher fines content may necessitate thinner lifts. Each lift must be compacted prior to placing the subsequent lift.

All structural fill should be uniformly compacted in horizontal lifts to the recommended percent of the maximum dry density (MDD) as determined in accordance with ASTM D 1557 (modified Proctor).

Building subgrade soils and utility trench backfill should be compacted to at least 90 percent of the MDD when placed at depths greater than 2 feet below subgrade; the upper 2 feet of fill should comprise select subbase material and should be compacted to at least 95 percent of MDD. Compaction must be achieved by mechanical means. During fill and placement, a suitable number of in-place density tests should be performed concurrently with the fill operations to check that the required compaction is being achieved.

CONSTRUCTION DEWATERING

We anticipate that dewatering may be necessary to construct some of the planned improvements at the site. Groundwater levels encountered in our explorations ranged from 1 to 17-1/2 feet bgs. Groundwater levels will vary at the site according to season, tidal conditions and precipitation. In addition, the amount of groundwater seepage into excavations should be expected to vary across the site. Granular fill and possible wood waste fill are more permeable than the underlying native silt deposit and will produce higher water flows.

The level of effort required for dewatering will depend to a great extent on the time of year during which construction is accomplished. Based on our experience in the area, groundwater levels are likely higher during the wet season. More extensive dewatering measures will be necessary during periods of high groundwater levels. If dewatering during construction is used, we recommend the water levels be maintained a minimum of 1 foot below the bottom of the excavations.

It is the contractors' responsibility to review the data presented in this report, perform any additional investigations and tests deemed necessary to characterize the soil and groundwater conditions at the site, and to develop independent conclusions regarding the design, construction and operation of the dewatering system.

LIMITATIONS

We have prepared this report for use by Santa Maria Shipping for the Port of Willapa Harbor Ship Fabrication/Launch Facility.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Attachment C titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

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We appreciate the opportunity to be of service to Santa Maria Shipping on this project. Please call if you have questions or require additional information.

Yours very truly

GeoEngineers, Inc.

Mark A. Rohrbach
Geotechnical Engineer

Garry H. Squires, PE, LG, LEG
Associate

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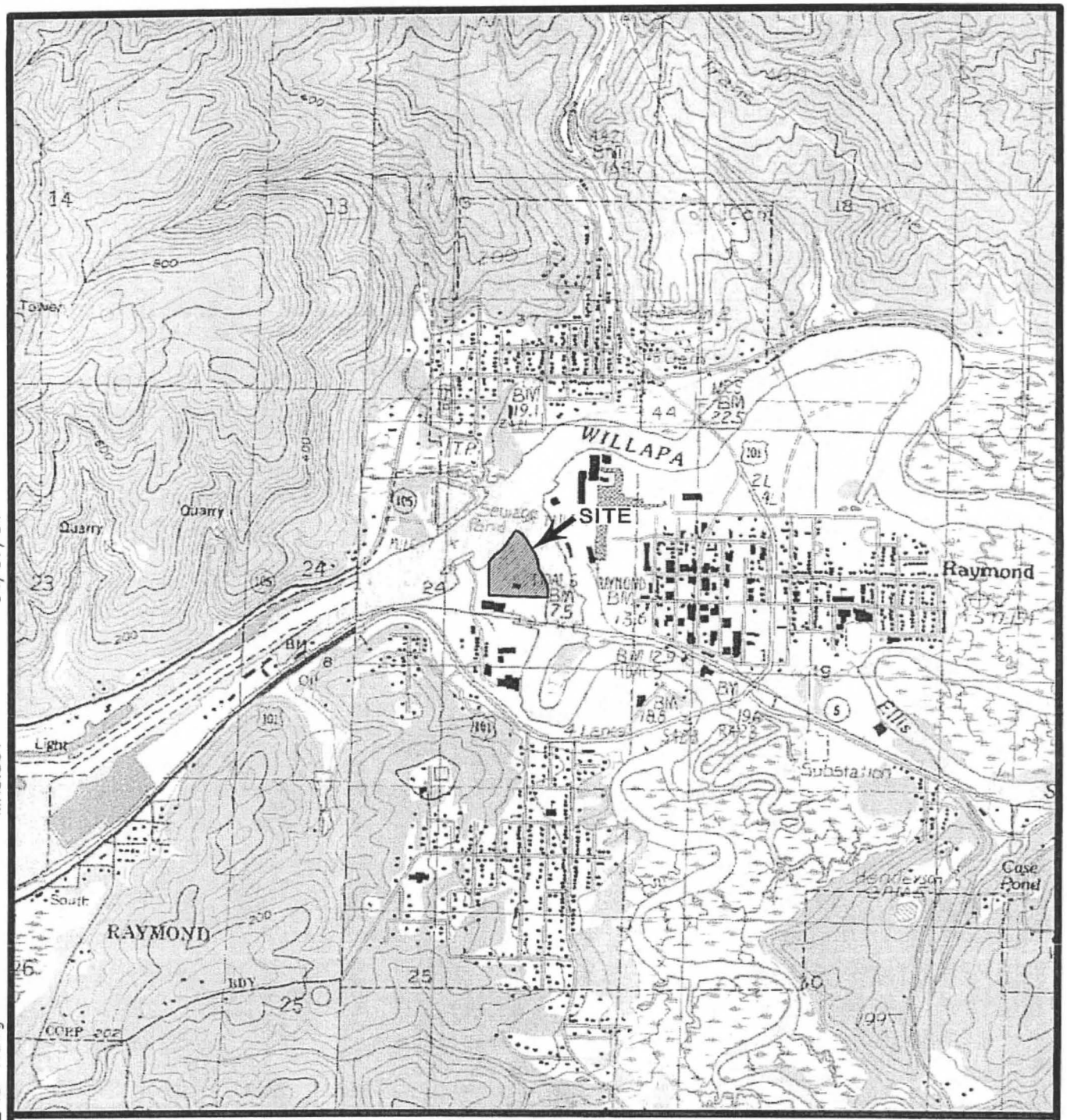
Disclaimer: This document (email, text, table, and/or figure) and any attachments are only a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Enclosures:

- Figure 1 – Vicinity Map
- Figure 2 – Site Plan
- Figures 3 through 8 – Pile Capacity Charts
- Figure 9 – Slope Stability Analysis
- Figure 10 – Sheet Pile Lateral Earth Pressure Criteria
- Attachment A – Site Explorations and Laboratory Testing
  - Figure A-1 – Soil Classification System
  - Figure A-2 – Key to Boring Log Symbols
  - Figures A-3 through A-5 – Log of Borings 1 through 3
  - Figures A-6 through A-8 – Log of Cone Penetrometer Tests
  - Figure A-9 – Atterberg Limit Test Results
- Attachment B – LPILE Analysis Results
- Attachment C – Report Limitations and Guidelines for Use



0 2000 4000

SCALE IN FEET

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Notes: 1. The locations of all features shown are approximate.

2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document.

Reference: Sure Maps! Raster Maps– USGS based quadrangles.

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VICINITY MAP

FIGURE 1

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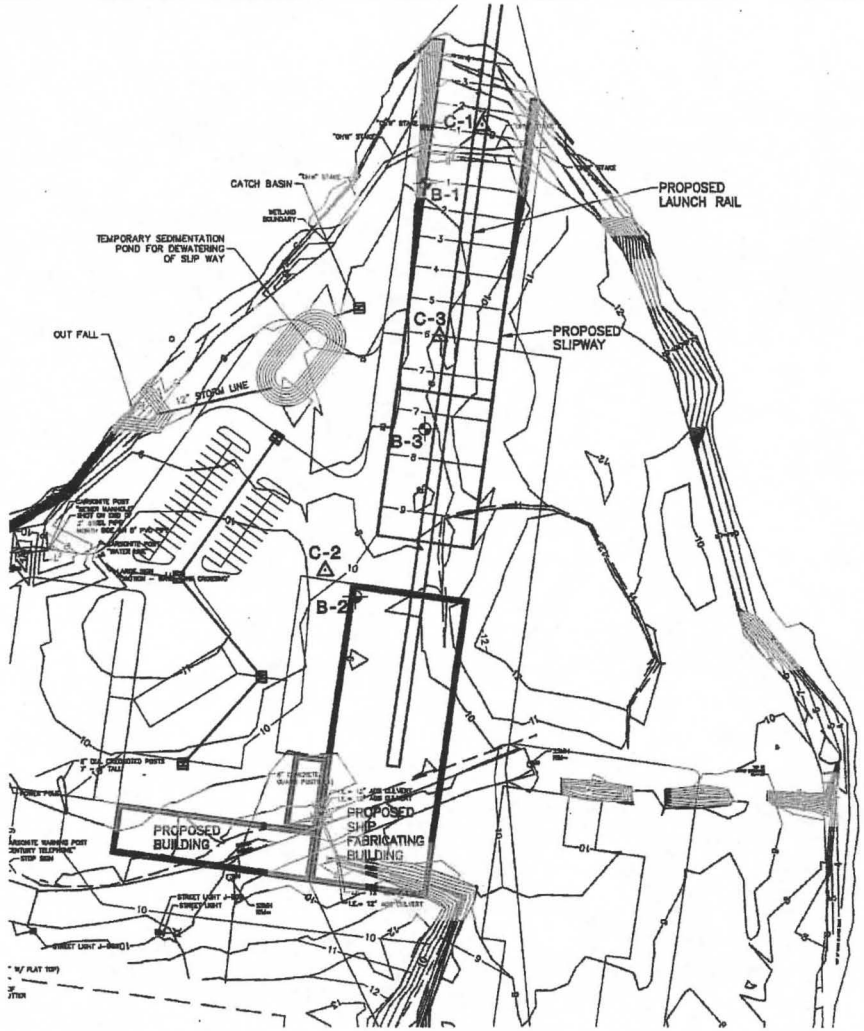

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EXPLANATION:

C-1△ CONE PENETROMETER TEST NUMBER AND APPROXIMATE LOCATION

B-1 BORING NUMBER AND APPROXIMATE LOCATION

0 120 240

APPROXIMATE SCALE IN FEET

Notes: 1. The locations of all features shown are approximate.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Reference: Drawing provided by MC Squared Inc.

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SITE PLAN

FIGURE 2

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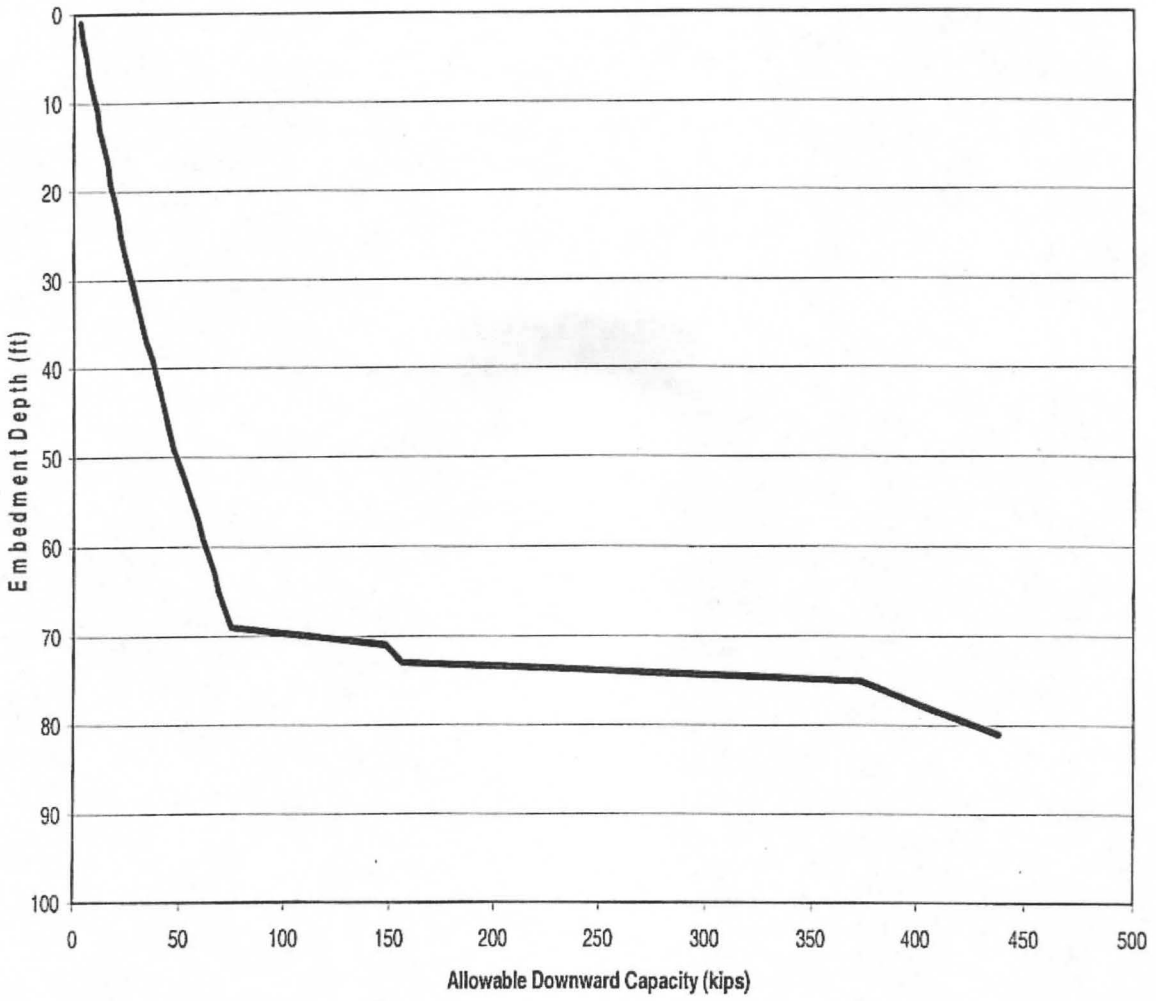
Ship Fabrication/Launch Facility



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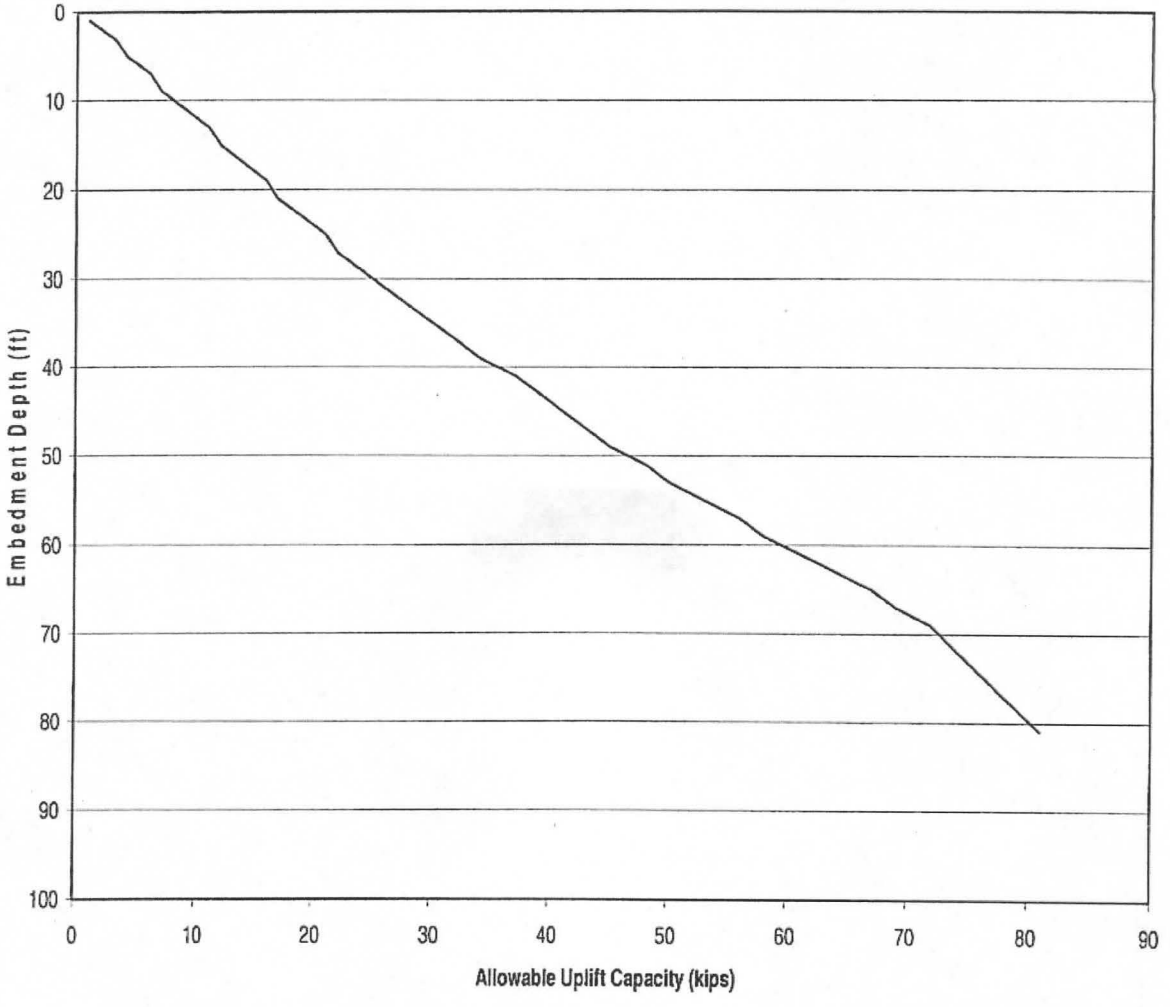
Pile Downward Capacity (16-in steel pipe)

FIGURE 3

Ship Fabrication/Launch Facility



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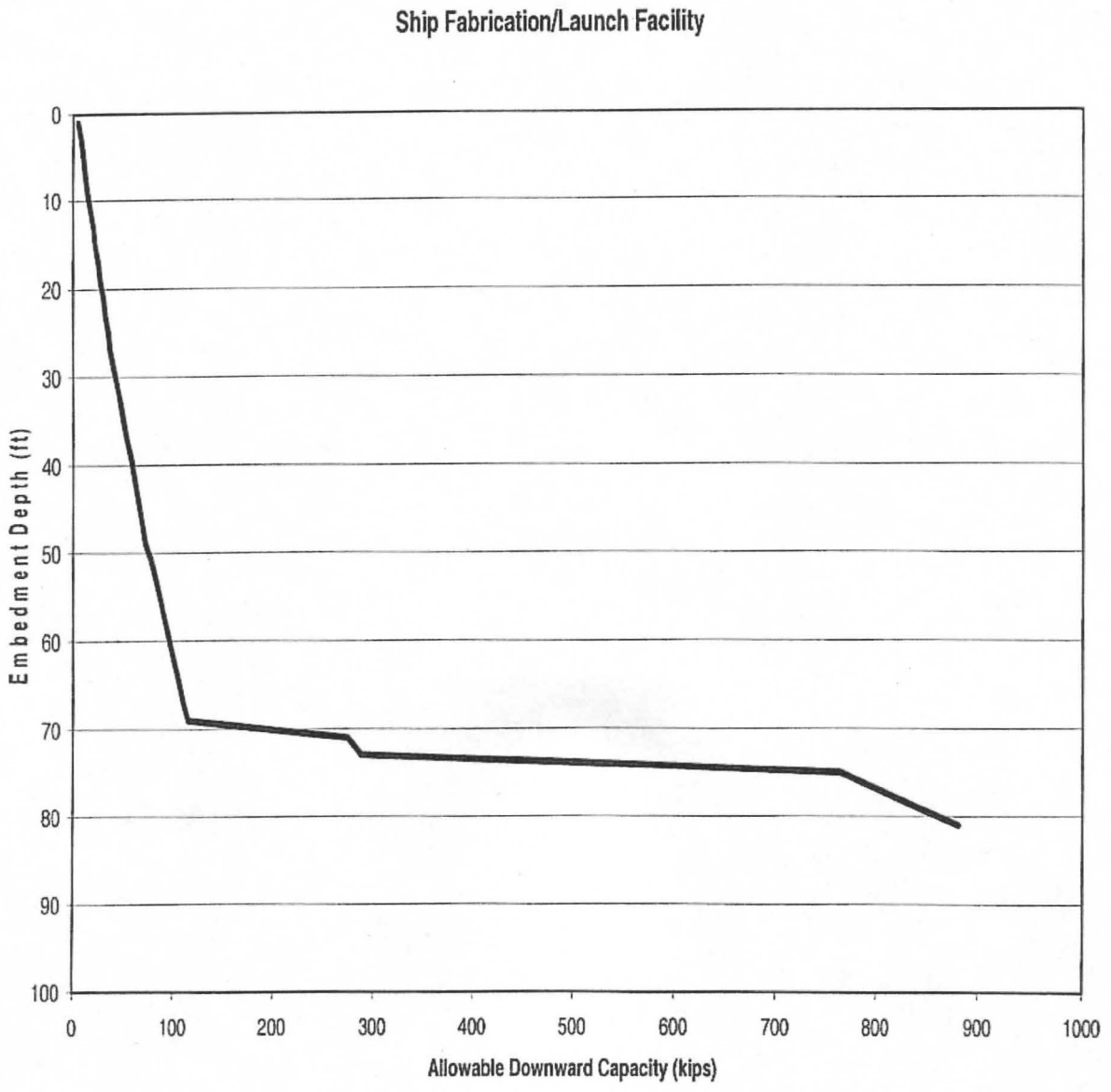
Pile Uplift Capacity (16-in steel pipe)

FIGURE 4



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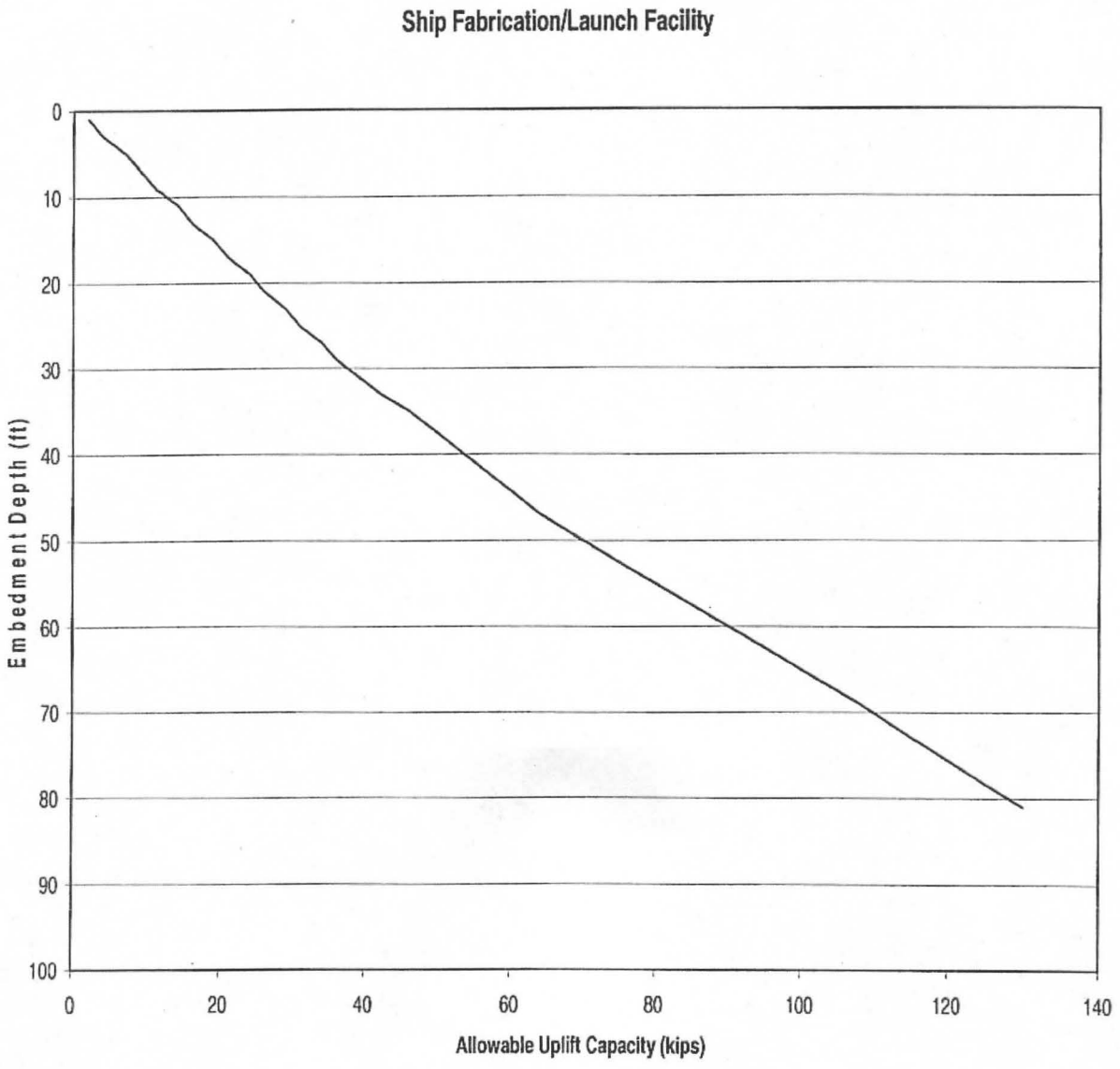
Pile Downward Capacity (24-in steel pipe)

FIGURE 5



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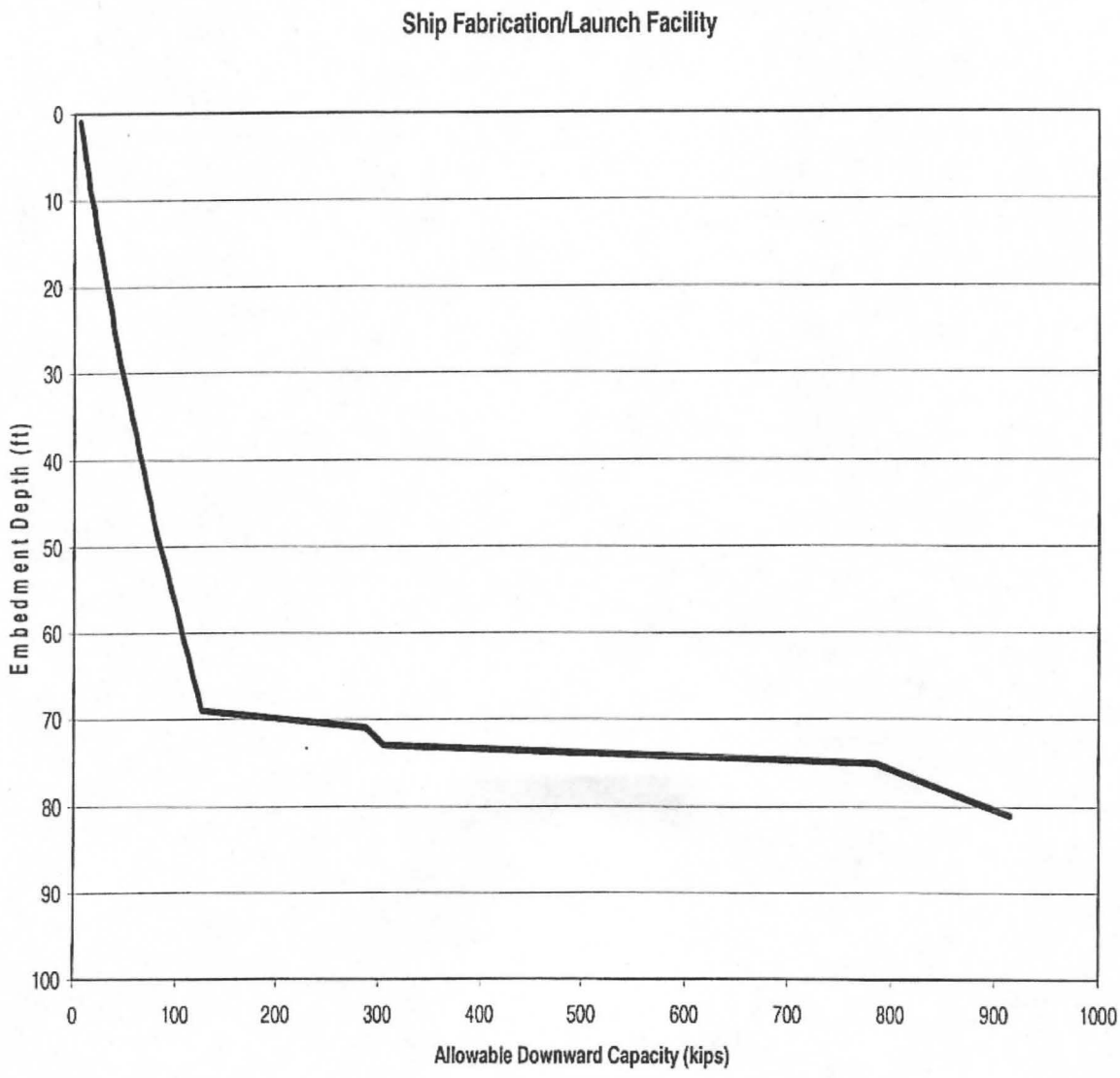
Pile Uplift Capacity (24-in steel pipe)

FIGURE 6



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Pile Downward Capacity (24-in octagonal concrete)

FIGURE 7

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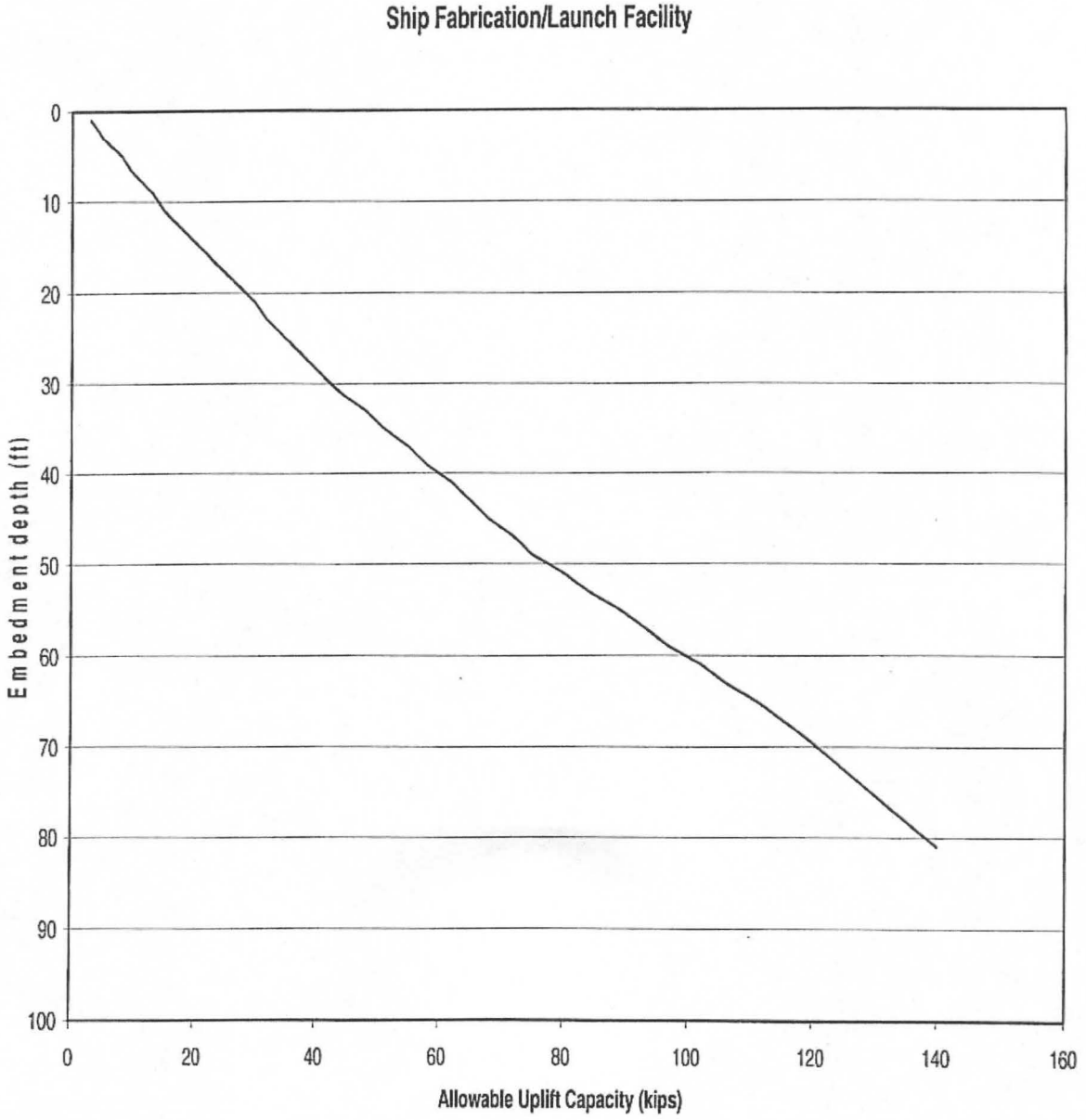

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Pile Uplift Capacity (24-in octagonal concrete)

FIGURE 8

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Santa Maria Ship Building Facility
Willapa Bay, Washington
9538-002-00

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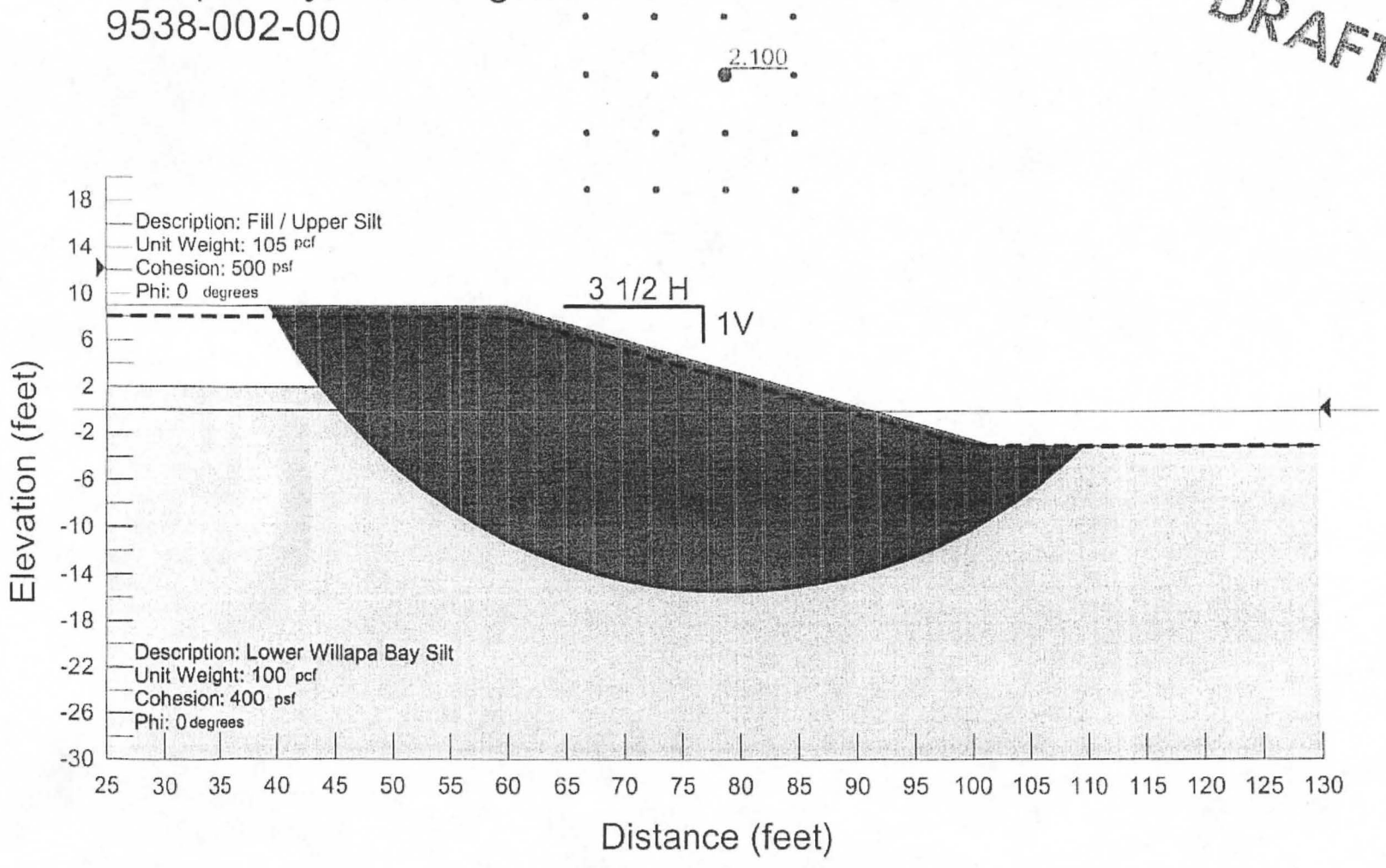

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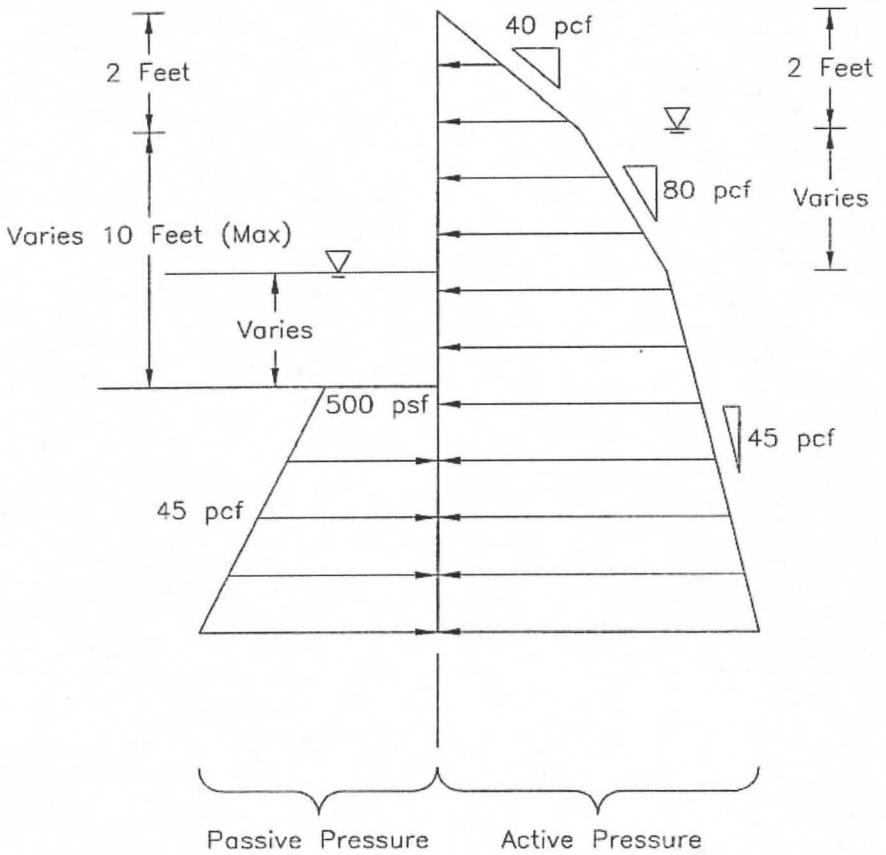
SLOPE STABILITY ANALYSIS

FIGURE 9

SHEET PILE LATERAL EARTH PRESSURE CRITERIA



Note: 1. See text of report for additional discussion.

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SHEET PILE LATERAL EARTH PRESSURE CRITERIA

FIGURE 10

04/09/04

GHS:SCY

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ATTACHMENT A

SITE EXPLORATIONS AND LABORATORY TESTING

ATTACHMENT A

SITE EXPLORATIONS AND LABORATORY TESTING

SITE EXPLORATIONS

General

Subsurface conditions at the proposed ship fabrication and launching facility were explored by drilling three borings and advancing three Cone Penetrometer Tests (CPTs) at the approximate locations shown in Figure 2. The drilling was accomplished on March 10, 2004. The CPTs were conducted on December 30, 2003.

Borings

The borings were advanced to depths ranging between about 19-1/2 feet and 20 feet bgs using a trailer-mounted, continuous flight, hollow stem auger drill rig. Relatively undisturbed soil samples were obtained from the borings using a 2.4-inch, inside-diameter split barrel. The sampler was driven into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler the last 12-inches, or other indicated distance, is recorded on the boring logs. The estimated equivalent Standard Penetration Test (ASTM D 1586) 'N' value is indicated in the notes on the boring log.

Borings were continuously monitored by an engineer from our firm who examined and classified the soil encountered, obtained representative soil samples, observed groundwater conditions, and maintained a detailed log of the explorations. The borings were located in the field by taping from the existing buildings. Boring elevations were estimated by referencing topographic information shown on a site plan provided by MC Squared Engineers (elevation datum is NGVD 1929). Locations and elevations should be considered accurate to the degree implied by the method used.

Soil encountered in the explorations was classified in general accordance with ASTM D 2488, the Standard Practice for Classification of Soils, Visual-Manual Procedure, which is summarized in Figure A-1. A key to the boring log symbols is presented in Figure A-2. Logs of the borings are presented in Figures A-3 through A-5. These logs are based on our interpretation of the field and laboratory data and indicate the depths at which the subsurface materials or their characteristics change, although the change might actually be gradual. If the change occurred between sampling attempts in the borings, it was interpolated. Observations of groundwater conditions were made as the explorations were performed and are noted on the boring logs.

Cone Penetrometer Tests

CPTs were performed at the approximate locations shown in Figure 2 in general conformance with ASTM Test Method D 5778. No samples are collected from CPT soundings. Figures A-6 through A-8 show the CPTs logs with an interpreted soil profile.

LABORATORY TESTING

General

Soil samples obtained from the borings were returned to our laboratory for further examination and testing. Selected soil samples were tested to assist in our evaluation of pertinent engineering properties. Laboratory test descriptions are presented below.

Moisture Content and Dry Unit Weight

The moisture content and/or dry unit weight of selected samples were determined in general accordance with ASTM Test Methods D 2216 and D 2937, respectively. The test results are used to aid in soil classification and correlation with other pertinent engineering soil properties. The test results are presented on the exploration logs at the respective sample depths.

Atterberg Limits

Atterberg limit tests were completed on one soil sample. The test was used to classify the soil as well as to aid in evaluating index properties and consolidation characteristics of the fine-grained soil deposits. The liquid limit and the plastic limit were obtained in general accordance with ASTM D 4318. The results of the Atterberg limit test are summarized in Figure A-9.

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More Than 50% Retained on No. 200 Sieve	GRAVEL More Than 50% of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More Than 50% of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More Than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less Than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or More	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is in general accordance with ASTM D2487-90.
3. Descriptions of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below water table

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SOIL CLASSIFICATION SYSTEM

FIGURE A-1

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LABORATORY TESTS:

AL	Atterberg Limits
CP	Compaction
CS	Consolidation
DS	Direct Shear
GS	Grain Size
%F	Percent Fines
HA	Hydrometer Analysis
SK	Permeability
SM	Moisture Content
MD	Moisture and Density
SP	Swelling Pressure
TX	Triaxial Compression
UC	Unconfined Compression
CA	Chemical Analysis

FIELD SCREENING TESTS:

PID/TLV	Headspace vapor concentration
PPM	Data given in parts per million

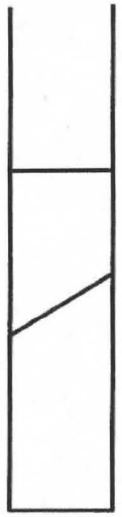
SHEEN CLASSIFICATION SYSTEM:

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

ODOR CLASSIFICATION SYSTEM:

NO	No Odor
SO	Slight Odor
MO	Moderate Odor
HO	Heavy Odor
NT	Not Tested

SOIL GRAPH:



SM Soil Group Symbol (See Note 2)

Distinct Contact Between Soil Strata

Gradual or Approximate Location of Change Between Soil Strata

Water Level

Bottom of Boring

BLOW COUNT/SAMPLE DATA:

Blows required to drive a 2.5-inch I.D. split-barrel sampler 12 inches or other indicated distances using a 140-pound hammer falling 30 inches.

Blows required to drive a 1.5-inch I.D. (SPT) split-barrel sampler 12 inches or other indicated distances using a 140-pound hammer falling 30 inches.

"P" indicates sampler pushed with weight of hammer or against weight of drill rig.

22 Location of relatively undisturbed sample

12 Location of disturbed sample

17 Location of sampling attempt with no recovery

10 Location of sample obtained in general accordance with Standard Penetration Test (ASTM D 1586) procedures

26 Location of SPT sampling attempt with no recovery

Location of grab sample

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NOTES:

1. The reader must refer to the discussion in the report text, the Key to Boring Log Symbols and the exploration logs for a proper understanding of subsurface conditions.

2. Soil classification system is summarized in Figure A-1.

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KEY TO BORING LOG SYMBOLS

FIGURE A-2

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Date(s) Drilled	03/10/04	Logged By	GRL	Checked By	MAR
Drilling Contractor	Geologic Drill	Drilling Method	Hollow Stem Auger	Sampling Methods	California Spoon Ring Sampler
Auger Data	6-inch HSA	Hammer Data	140 (lb) hammer/ 30 (in) drop	Drilling Equipment	Deep Rock XL Trailer Rig
Total Depth (ft)	20	Surface Elevation (ft)	9	Groundwater Level (ft. bgs)	17.5 ATD; 16.5
Datum/ System	Unknown				

Elevation feet	Depth feet	SAMPLES Interval	Number	Recovered (in)	Blows/6in	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							ML	Brown sandy silt with organics (medium stiff, moist) (fill)			
								SM	Brown silty fine to coarse sand with gravel (very loose, moist) (fill)			
-5	5		1	8	11							TV=0.18 TSF N_{eq}=3 possible fill
								ML	Brown sandy silt with organics and occasional gravel (very soft, moist)			
-0	10		2	13	4					23	84	TV=0.25 TSF N_{eq}=1
								MH	Brown silt with organics and trace sand (very soft, moist)			
-5	15		3	18	4							TV=0.12 TSF N_{eq}=1
						▽						Water level 1 hr after drilling
						▼						Water level at time of drilling
-10			4	18	4				Very soft, wet	65	58	TV=0.5 TSF N_{eq}=1 LL=69 PI=33
	20							SM	Brown fine silty sand with organics (very loose to loose, wet)			

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING 1

GEOENGINEERS

Project: Santa Maria Ship Fabrication & Launching Facility
Project Location: Raymond, Washington
Project Number: 9538-002-00

Figure: A-3
Sheet 1 of 1

9538-002-00 GEI_GTBORING_2.1.0 P:\9\9538002\00\FINALS\9538002008.GPJ GEIV2_2.GDT 4/9/04

Date(s) Drilled	03/10/04	Logged By	GRL	Checked By	MAR
Drilling Contractor	Geologic Drill	Drilling Method	Hollow Stem Auger	Sampling Methods	California Spoon Ring Sampler
Auger Data	6-inch HSA	Hammer Data	140 (lb) hammer/ 30 (in) drop	Drilling Equipment	Deep Rock XL Trailer Rig
Total Depth (ft)	20	Surface Elevation (ft)	9	Groundwater Level (ft. bgs)	7.10 ATD; 4.10; 2.15
Datum/ System	Unknown				

Elevation feet	Depth feet	SAMPLES Interval	Number	Recovered (in)	Blows/6in	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							SM	Brown silty fine to coarse sand with gravel (moist) (fill)			
						▽		ML	Brown silt with sand and trace organics (very soft, moist) (fill)			Water level 2 hrs after drilling
-5			1	18	13	▽		SM	Brown silty fine to medium sand with wood debris (very loose, wet) (fill)			Water level 1 hr after drilling TV=0 TSF N_{eq}=2
	5					▼						Water level at time of drilling
-0			2	18	5					45	75	TV=0.1 TSF N_{eq}=1
	10							MH	Brown silt with trace organics (very soft, moist)			
-5			3	18	5					62	62	TV=0.2 TSF N_{eq}=1
	15											
-10			4	18	5							TV=0.2 TSF N_{eq}=1
	20											

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING 2

GEOENGINEERS

Project: Santa Maria Ship Fabrication & Launching Facility
Project Location: Raymond, Washington
Project Number: 9538-002-00

Figure: A-4
Sheet 1 of 1

9538-002-00 GEI GTBORING 2.1.0 P:\9\9538002\00\FIN\ALS\9538002008.GPJ GEIV2_2.GDT 4/9/04

Date(s) Drilled	03/10/04	Logged By	GRL	Checked By	MAR
Drilling Contractor	Geologic Drill	Drilling Method	Hollow Stem Auger	Sampling Methods	California Spoon Ring Sampler
Auger Data	6-inch HSA	Hammer Data	140 (lb) hammer/ 30 (in) drop	Drilling Equipment	Deep Rock XL Trailer Rig
Total Depth (ft)	19	Surface Elevation (ft)	8	Groundwater Level (ft. bgs)	4.3 ATD; 1
Datum/ System	Unknown				

Elevation feet	Depth feet	SAMPLES Interval	Number	Recovered (in)	Blows/6in	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							ML	Brown sandy silt (medium stiff, moist) (fill)			Water level 1 hr after drilling
-5			1	18	11			SM	Brown silty fine to medium sand (very loose, wet) (fill)	30	96	TV=0.2 TSF N_{eq}=3 %F=25 Water level at time of drilling
	5											
-0			2	18	5			MH	Brown silt with organics (very soft, moist)			TV=0.3 TSF N_{eq}=2
	10											
-5			3	17	4				Brown silt (very soft, moist)	123	38	TV=0.2 TSF N_{eq}=2
	15											
-10			4	15	4					83	50	TV=0.2 TSF N_{eq}=2

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING 3

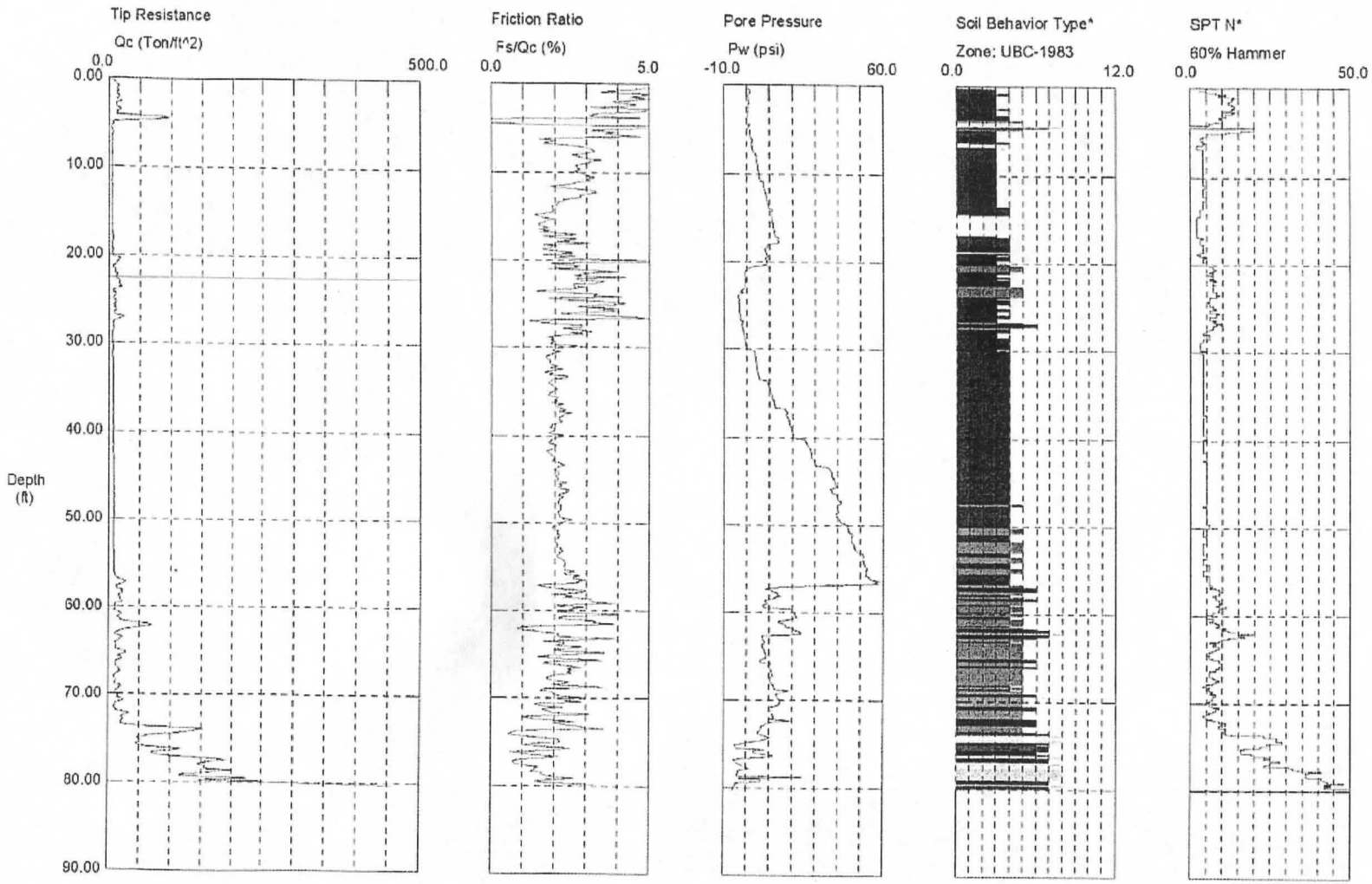
GEOENGINEERS

Project: Santa Maria Ship Fabrication & Launching Facility
Project Location: Raymond, Washington
Project Number: 9538-002-00

Figure: A-5
Sheet 1 of 1

9538-002-00 GEI GTBORING 2.1.0 P:\0\9538002\00\FINALS\9538002000B.GPJ GEIV2_2.GDT 4/9/04

TACO\9\9538002\00\CAD\953800200_FIG-A-6 TO 8.dwg GHS:SCY 04/09/04



Maximum Depth = 80.22 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Refused at 80.22 feet.

Northwest Cone Exploration

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NOT TO SCALE

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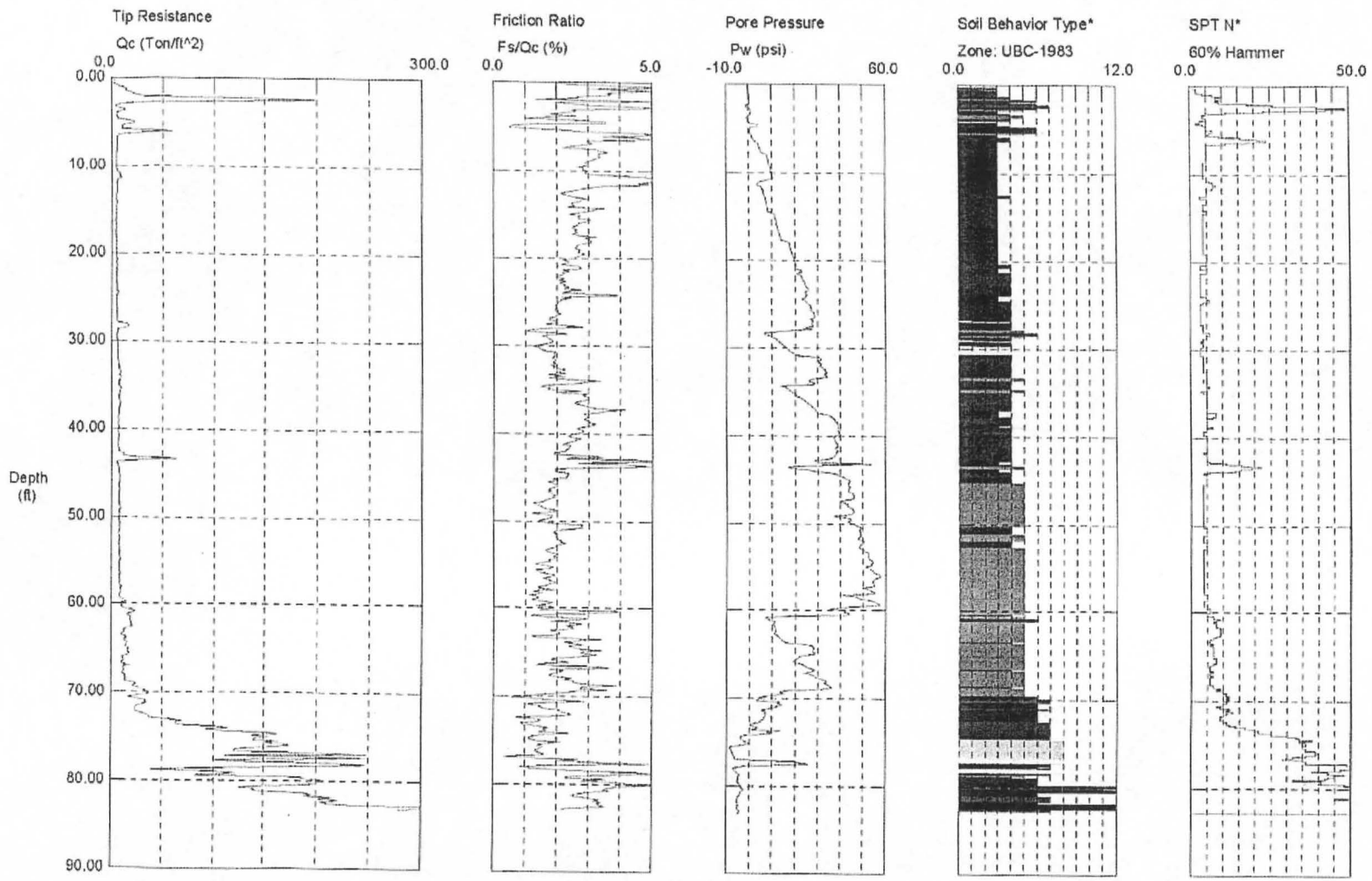
CPT-1

FIGURE A-6

TACO\9\9538002\00\CAD\953800200_FIG-A-6 TO 8.dwg GHS:SCY 04/09/04



Maximum Depth = 83.17 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Northwest Cone Exploration

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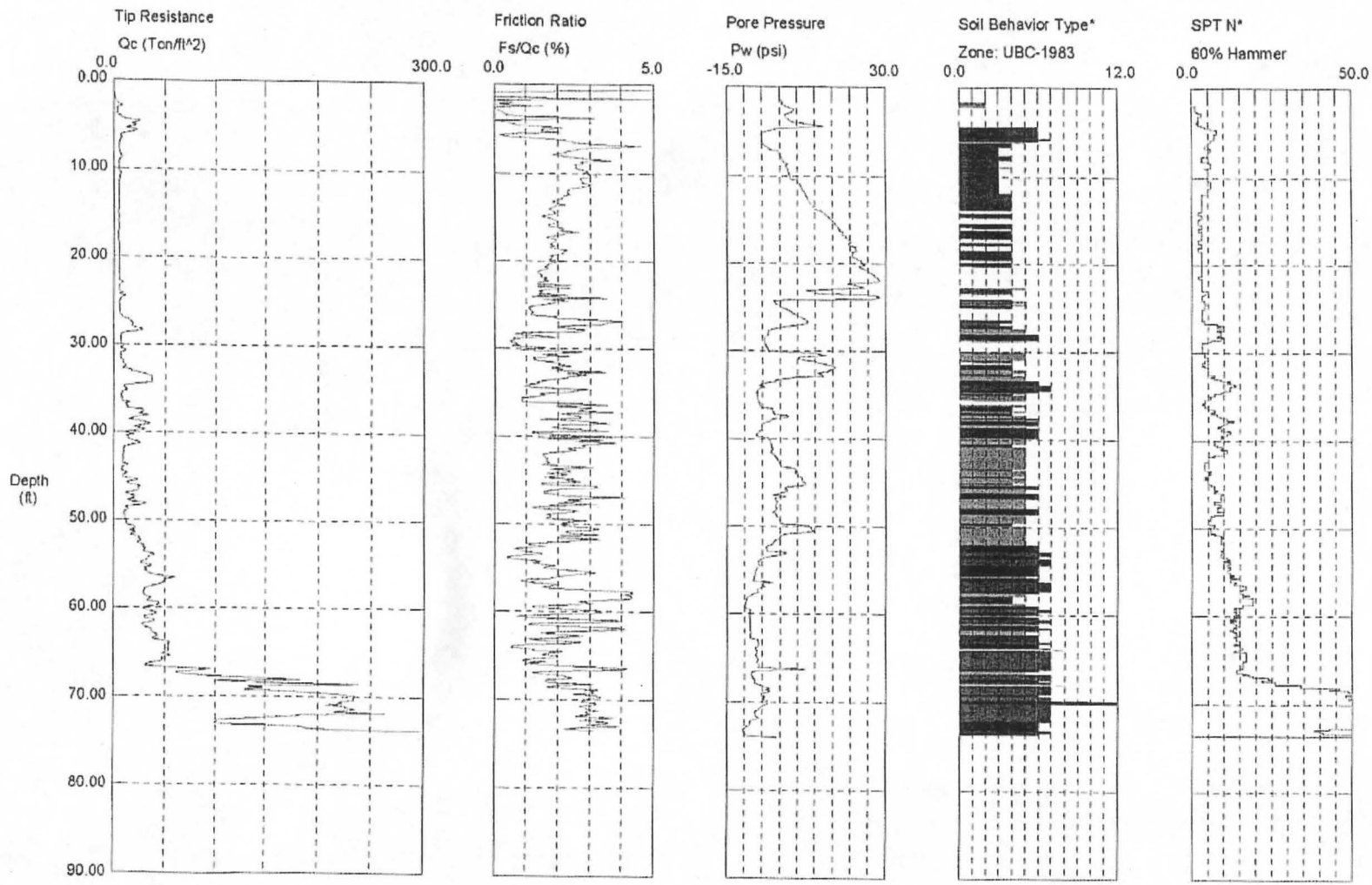
NOT TO SCALE

GEOENGINEERS

CPT-2

FIGURE A-7

TACO\9\9538002\00\CAD\953800200_FIG-A-6 TO 8.dwg GHS:SCY 04/09/04



Maximum Depth = 73.98 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

Northwest Cone Exploration

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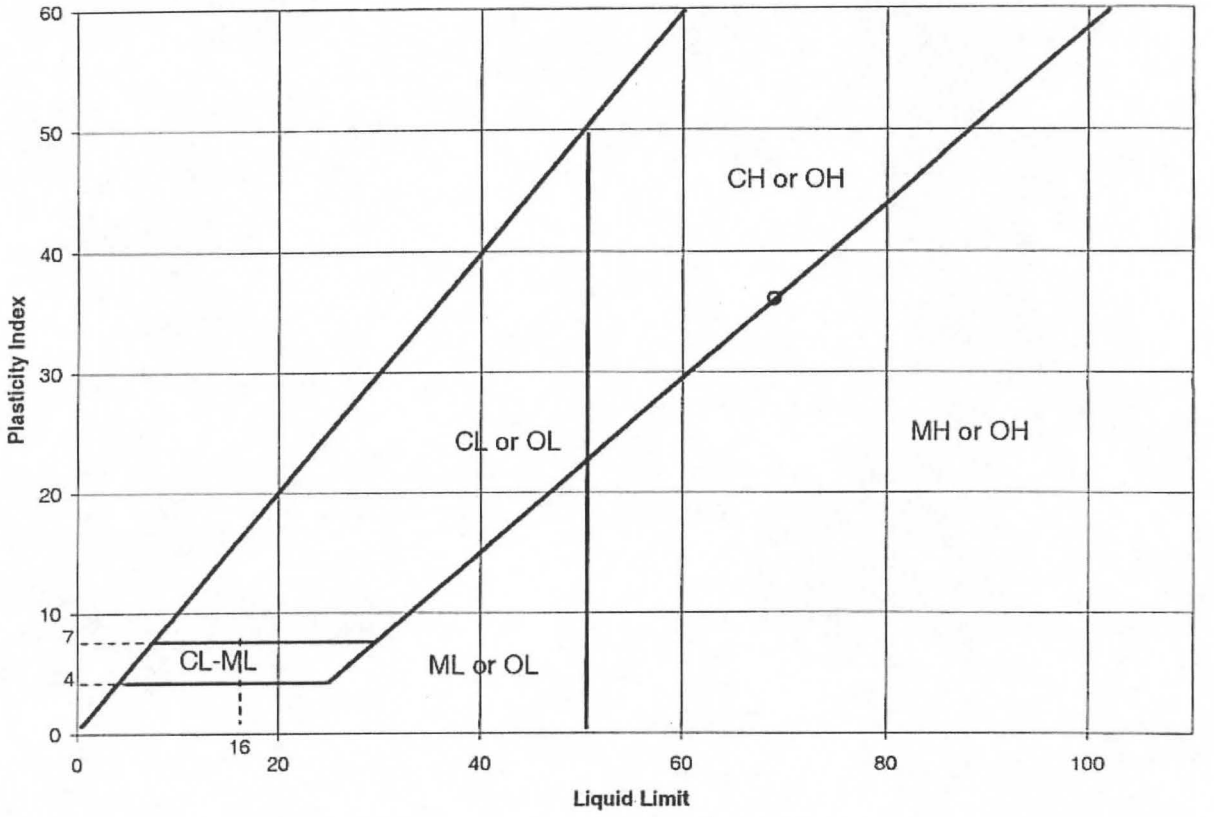
NOT TO SCALE

GEOENGINEERS

CPT-3

FIGURE A-8



	Job Number:		9538-002-00		
Exploration No.	Sample Depth	Moisture Content	Liquid Limit	Plasticity Index	Soil Description
1	18.5	64.7%	69	36	Brown silt of high plasticity (MH)

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ATTERBERG LIMIT TEST RESULTS

FIGURE A-9

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ATTACHMENT B

LPILE ANALYSIS RESULTS

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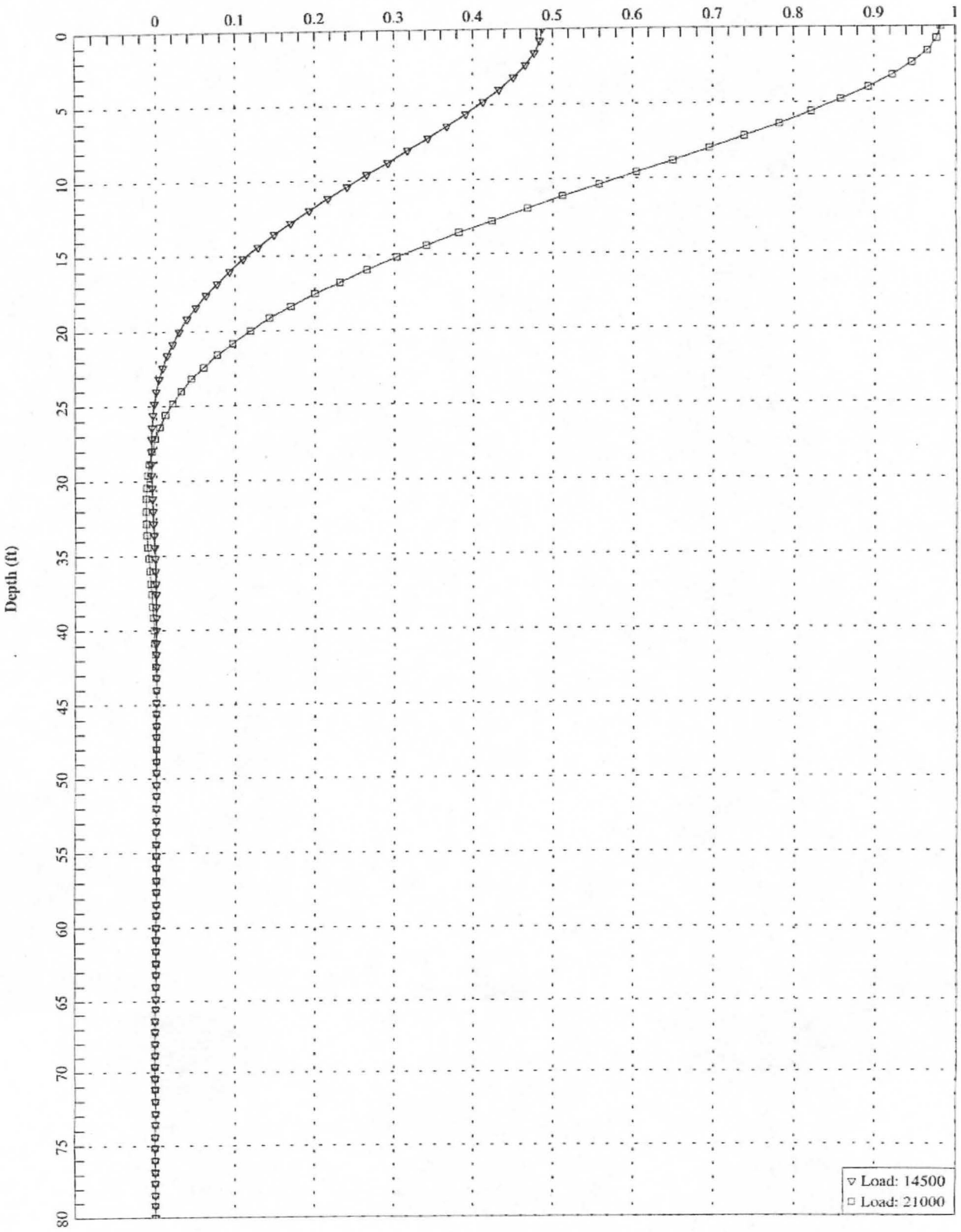
Santa Maria - 16 in Steel Pipe



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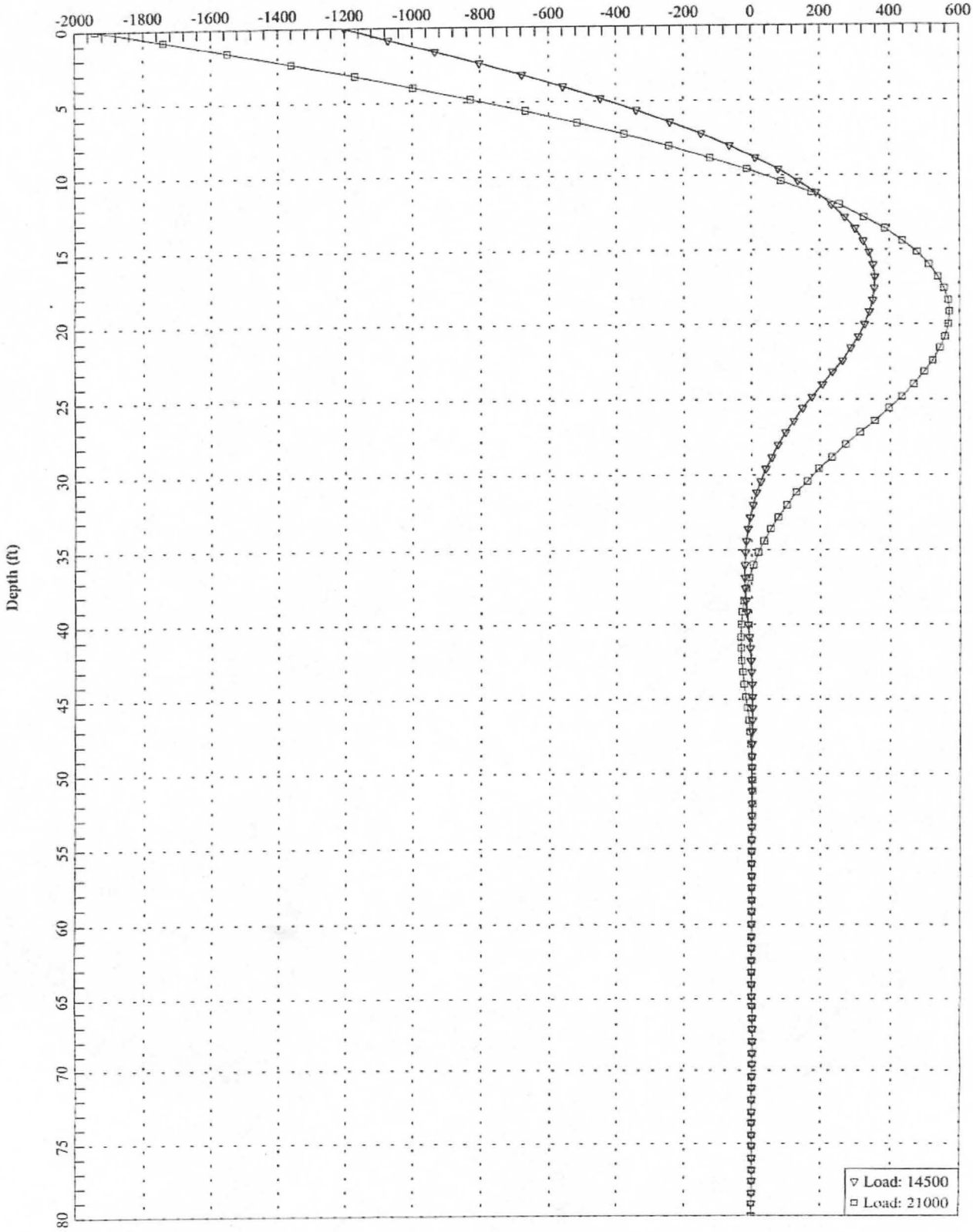
Santa Maria - 16 inch Steel Pipe

Bending Moment (in-kips)



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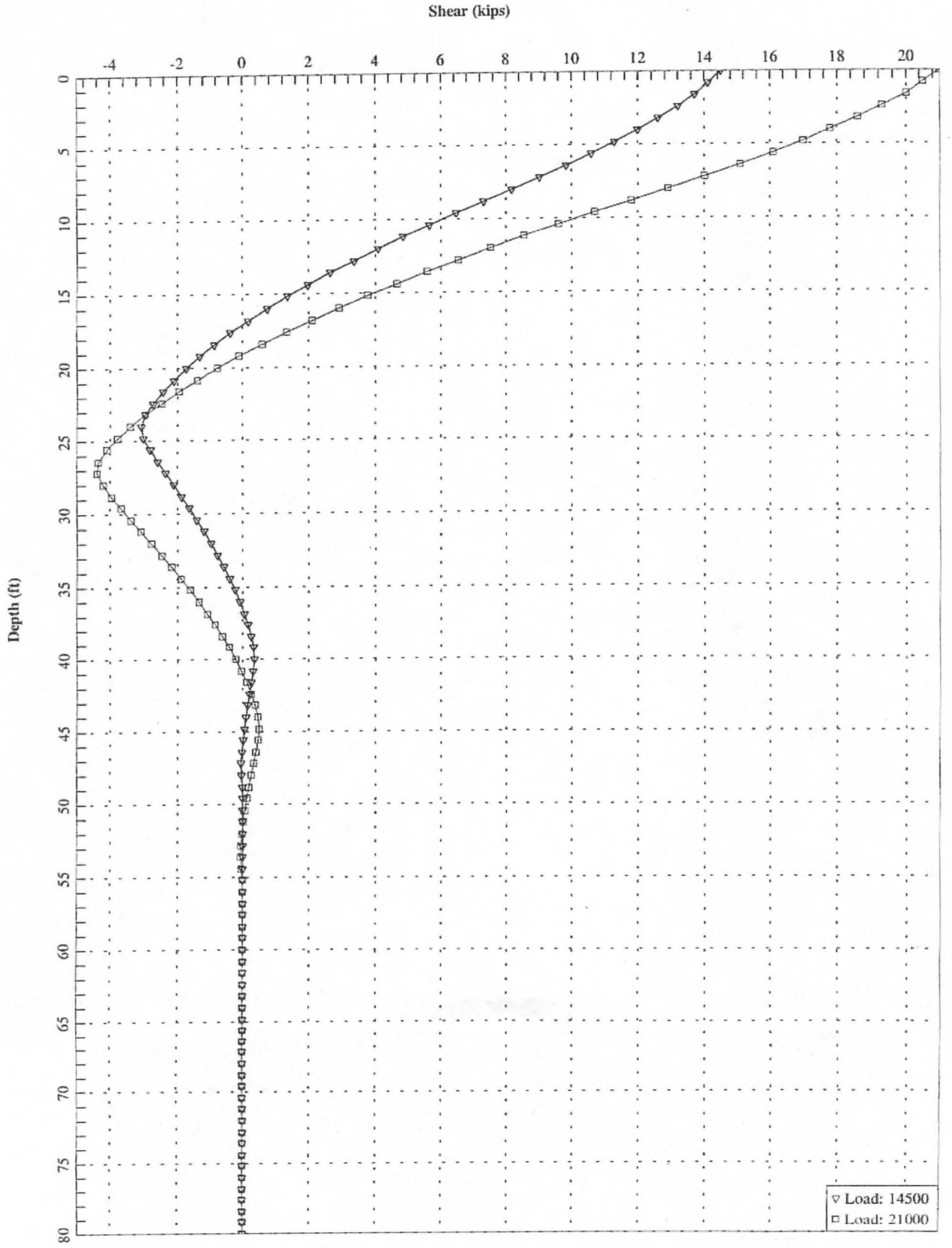
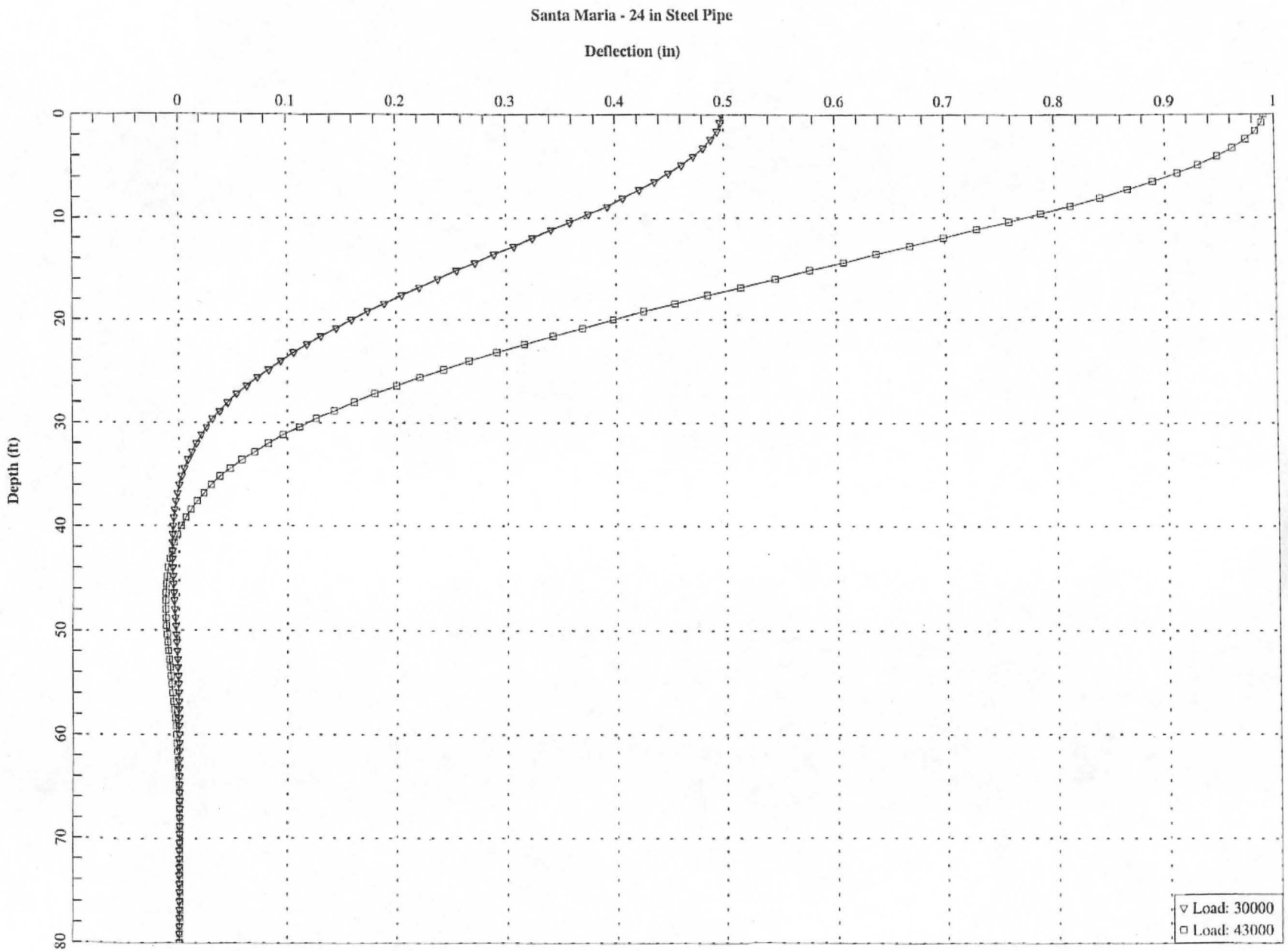
Santa Maria - 16 inch Steel Pipe




Santa Maria - 24 in Steel Pipe
Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Depth (ft)
0
10
20
30
40
50
60
70
80
Load: 30000
Load: 43000

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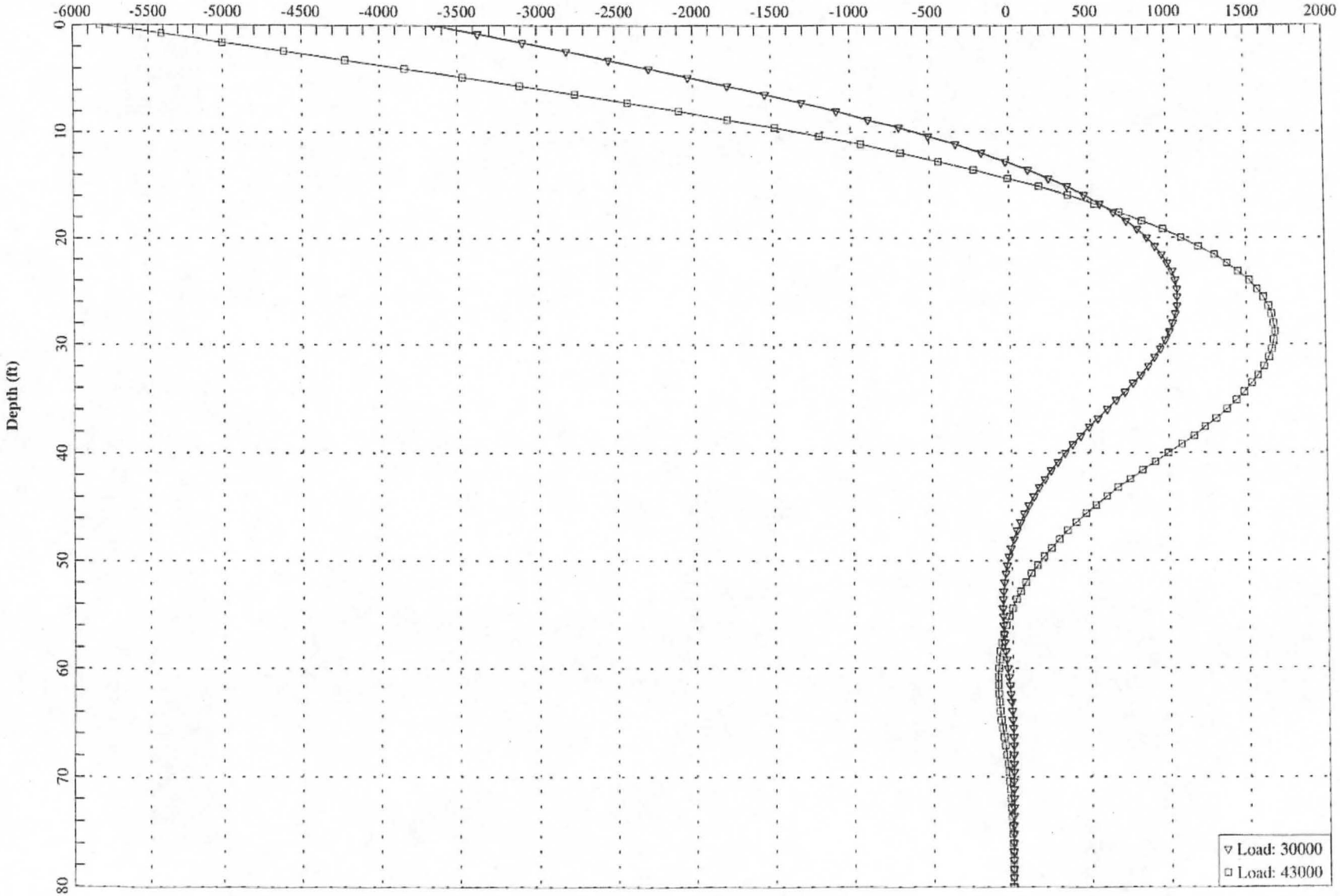
Santa Maria - 24 inch Steel Pipe

Bending Moment (in-kips)



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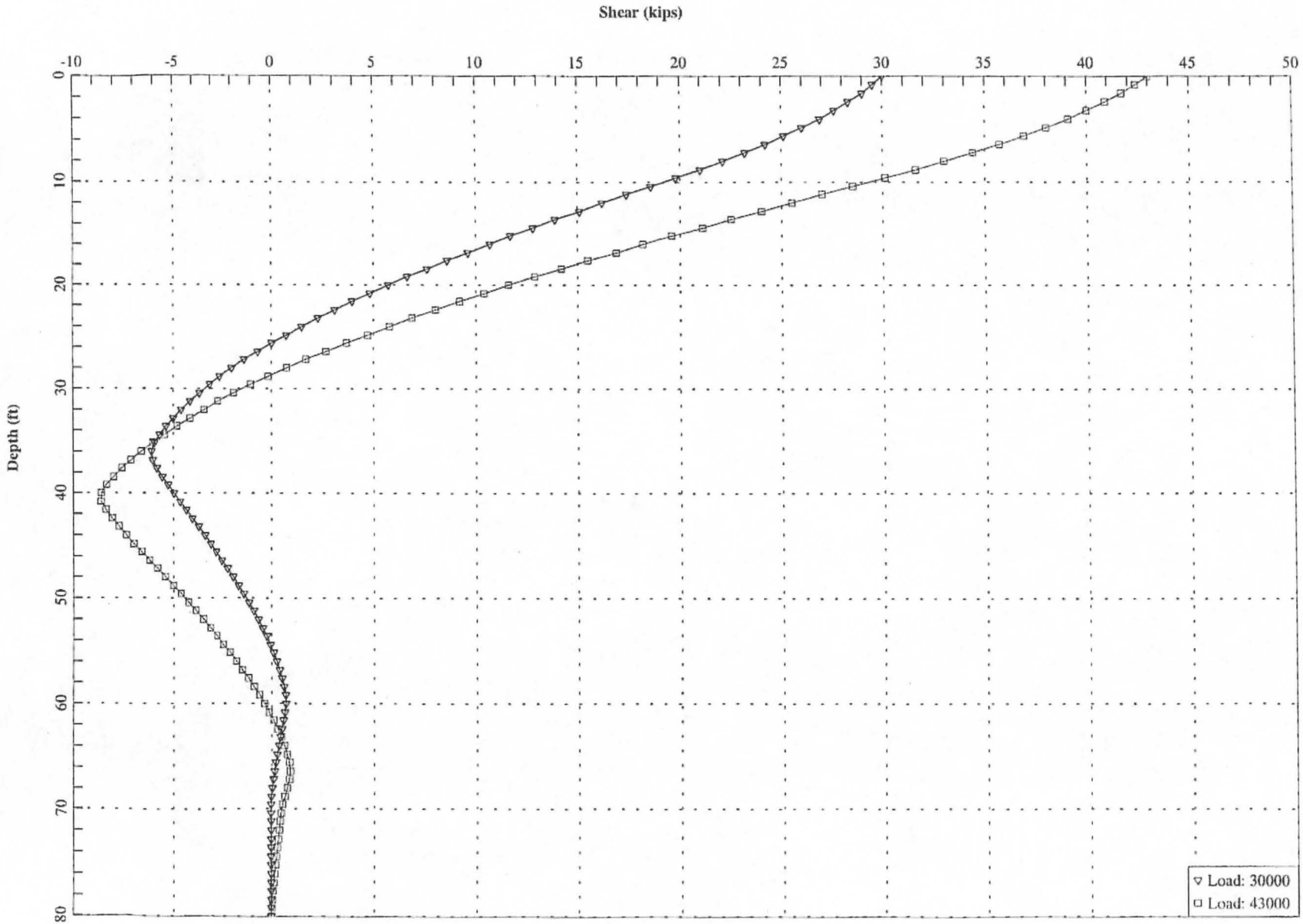
Santa Maria - 24 inch Steel Pipe



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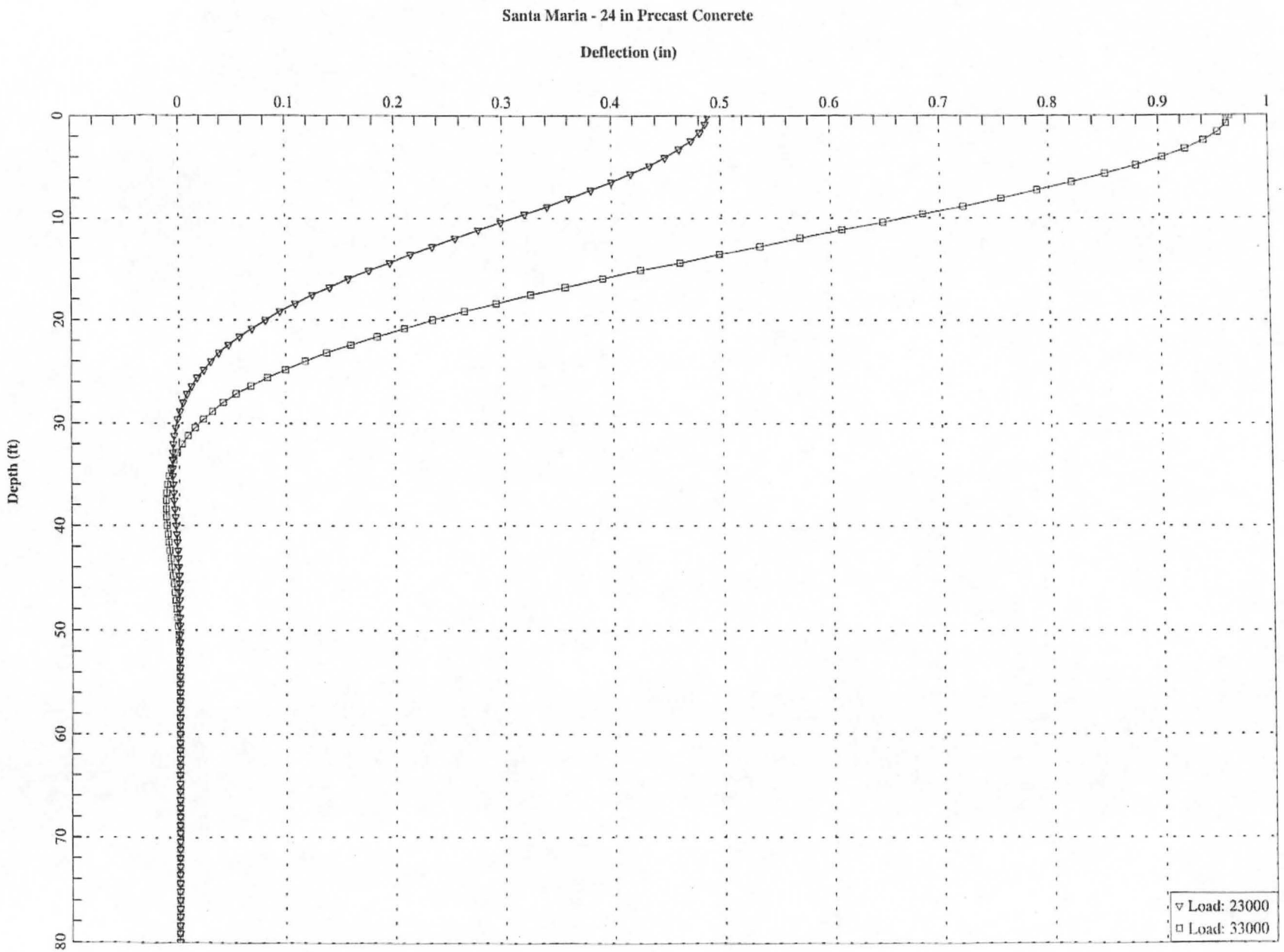
Santa Maria - 24 in Precast Concrete
Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Depth (ft)
0
10
20
30
40
50
60
70
80
Load: 23000
Load: 33000

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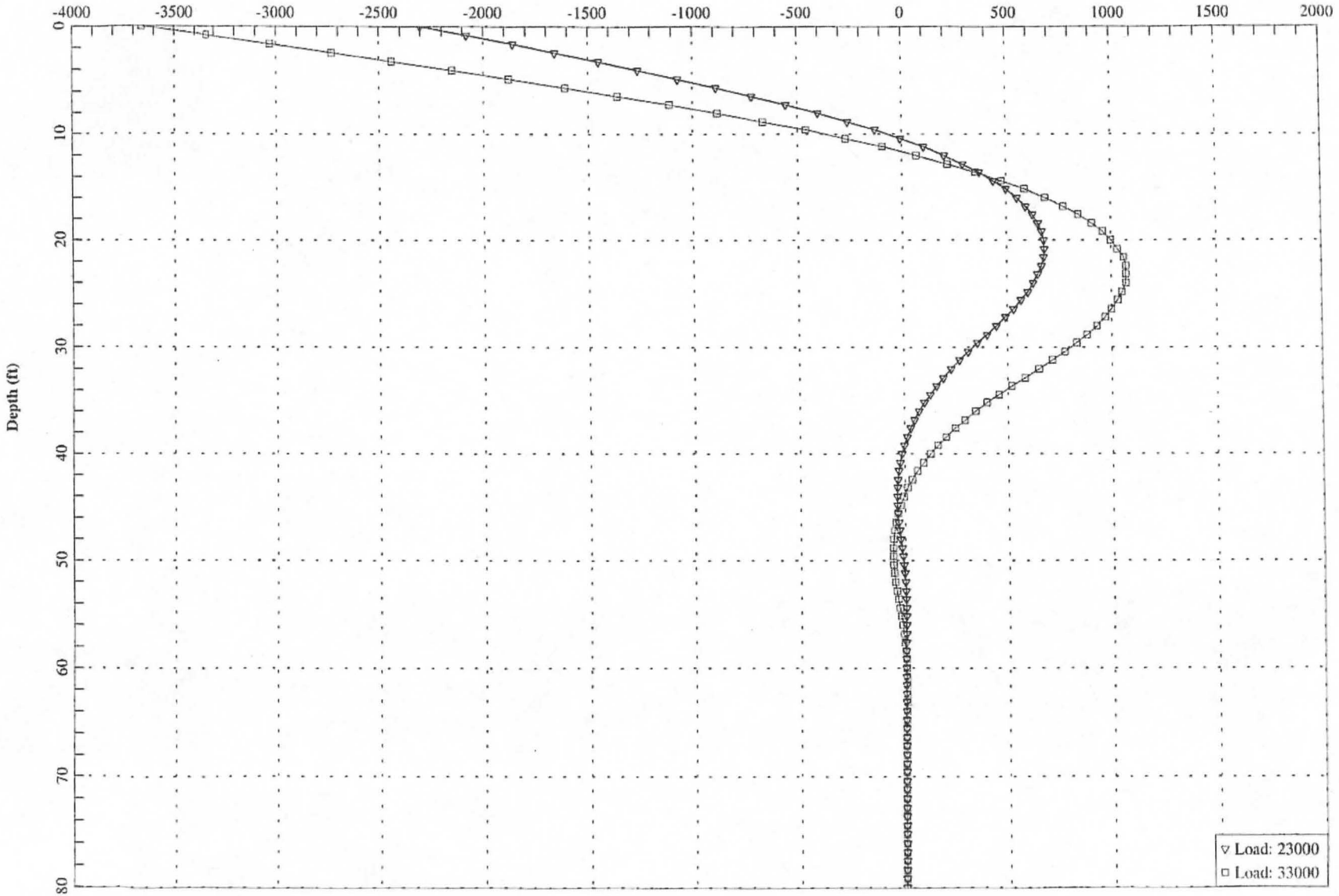
Santa Maria - 24 inch Precast Concrete

Bending Moment (in-kips)



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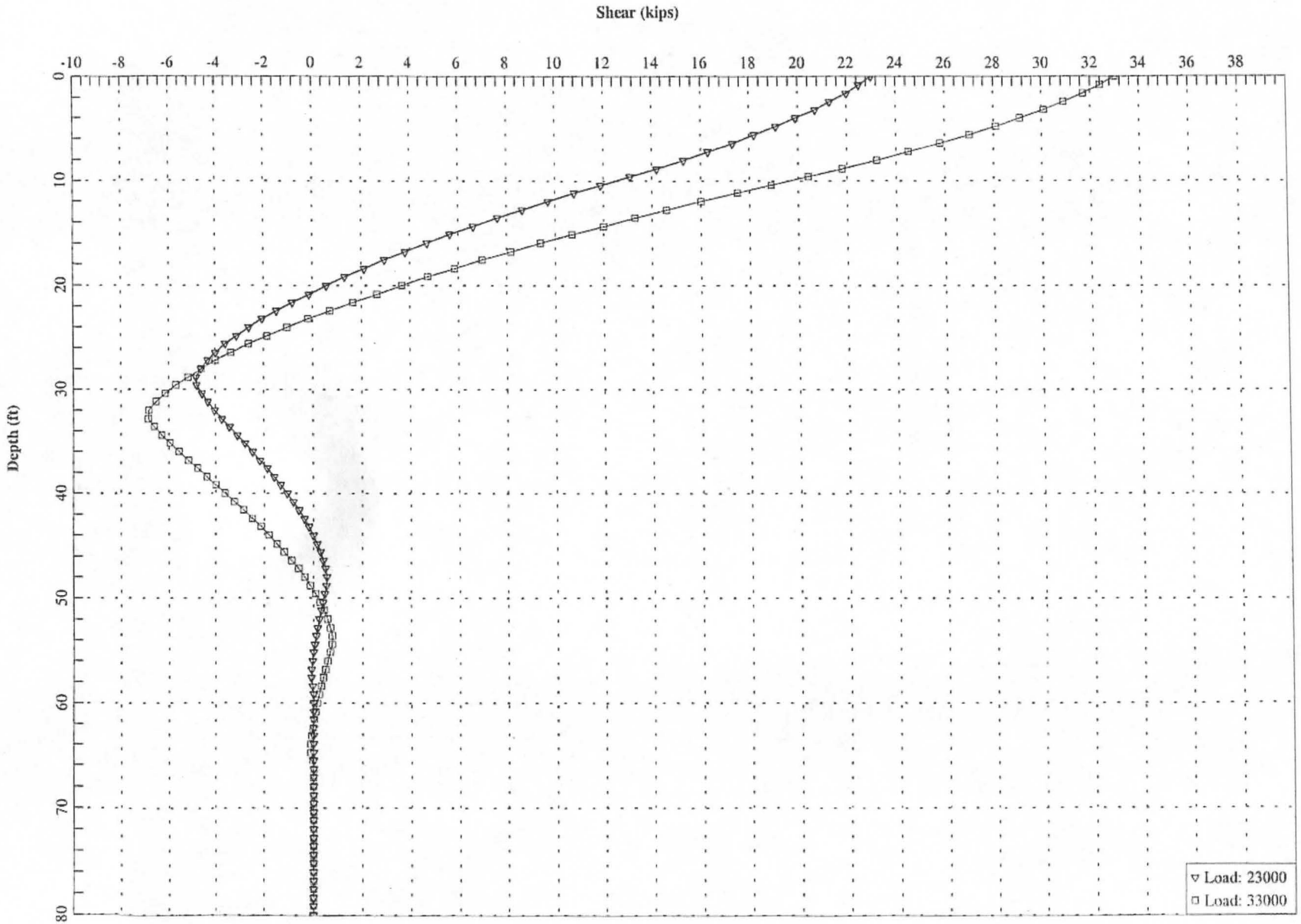
Santa Maria - 24 inch Precast Concrete



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ATTACHMENT C

REPORT LIMITATIONS AND GUIDELINES FOR USE

ATTACHMENT C

REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This attachment provides information to help you manage your risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for the exclusive use of Santa Maria Shipping and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the proposed Ship Fabrication/Launching Facility at the Port of Willapa Harbor. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or ground water fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

CONTRACTORS ARE RESPONSIBLE FOR SITE SAFETY ON THEIR OWN CONSTRUCTION PROJECTS

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

READ THESE PROVISIONS CLOSELY

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions

in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

BIOLOGICAL POLLUTANTS

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention, or assessment of the presence of Biological Pollutants in or around any structure. Accordingly, this report includes no interpretations, recommendations, findings, or conclusions for the purpose of detecting, preventing, assessing, or abating Biological Pollutants. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.