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Earth and Environmental Technologies

Geotechnical Engineering Design Report
Snohomish County PUD Headquarters Building
Everett, Washington

Prepared for
TRA

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GEOTECHNICAL ENGINEERING DESIGN REPORT
SNOHOMISH COUNTY PUD HEADQUARTERS BUILDING
EVERETT, WASHINGTON

This report presents the results of our subsurface explorations and geotechnical engineering study for the proposed Snohomish County Public Utilities District (PUD) Headquarters Building in Everett, Washington.

We have organized this report into several distinct sections. In the first several pages we present a summary of our key conclusions and recommendations. The summary section should be used only as a reminder of the information discussed in the text. The main body of the report presents our design level results. Toward the end of the text, we summarize important construction considerations. The appendices present our field and laboratory test results.

PURPOSE, SCOPE, AND LIMITATION OF THIS STUDY

Purpose

The purposes of this study were to:

- Assess the site subsurface conditions;
- Assist the design engineers by developing criteria for site preparation and foundation design; and
- Provide geotechnical recommendations relevant to design and construction.

Scope

The scope of the study included the following:

- Reviewing the information in our files related to previous exploration in the project vicinity;
- Completing field explorations of the general site;

- Conducting laboratory soils tests;

- Identifying the geotechnical engineering considerations and performing analyses; and

- Preparing this report.

The field explorations consisted of six hollow-stem auger borings drilled to depths ranging from about 20 to 29 feet below the existing ground surface. Laboratory test procedures included visual classification, water contents, grain size analyses, Atterberg limits, and consolidation tests on selected samples retrieved from the explorations. We used the tests to classify the site soils and to estimate the geotechnical index and engineering properties of the materials. Finally, we performed engineering studies and analyses to develop recommendations for design and construction.

Limitation

We completed this work in general accordance with our proposal dated October 4, 1990. Our report is for the exclusive use of the Snohomish County Public Utilities District and their design consultants for specific application to the subject project and site. We completed this study in accordance with generally accepted geotechnical practices for the nature and conditions of the work completed in the same or similar localities, at the time the work was performed. We make no other warranty, express or implied.

As design level studies proceed, Hart Crowser should periodically review this report and update the recommendations, as needed, to respond to the developing structural design. Hart Crowser should also review the final plans and specifications. We recommend that Hart Crowser be retained to provide on-site review during construction.

SITE AND PROJECT DESCRIPTION

The project site, which is located in Everett, Washington, is bounded to the north by California Street, to the east by Virginia Avenue, to the south by Hewitt Avenue, and to the west by a vacant lot. A railroad track crosses the southwest corner of the site. The site has maximum plan dimensions of about 400 by 600 feet.

The new facilities will consist of a multi-story office building, an underground parking garage, a training center, and pavement areas. There are six existing structures located at the site. The existing office and warehouse buildings located in the northeast and northwest corners of the site will remain. All other structures will be removed. The proposed training center and office building will be located adjacent to the existing office building in the northeast corner of the site. The parking garage will be located to the south of the proposed office building. The maximum column loads are anticipated to range from 60 to 400 kips.

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The following is a summary of the principal conclusions and recommendations contained within this report. Refer to subsequent sections of the report for further discussion of each point, as well as for other recommendations.

Results of Field Explorations

A vicinity map and site and exploration plan are shown on Figures 1 and 2, respectively.

- Our elevations are based on a survey by Group Four provided by TRA and are actual elevations. Site grades vary from about elevations 163 to 172 feet.
- Subsurface soils consist of surficial fill materials over silt and clay over dense to very dense sand.
- Soils at shallow foundation levels vary from firm and compressible at the west side of the site to strong and relatively incompressible at the east side of the site.
- Soils at deep foundation levels are generally strong and relatively incompressible.

- Groundwater was encountered at a depth of about 6 feet in boring HC-1 at the time of exploration. A monitoring well was installed in HC-1 at the time of drilling. No other boring encountered groundwater.

Foundation and Slab Design

- Because of the weak and compressible nature of the upper subgrade soils, the new structure should be supported on a pile foundation.
- In order to reduce the potentially damaging vibrations associated with pile driving, cast-in-place (augercast) piling should be used.
- 25- to 30-foot-long, 12- to 18-inch-diameter augercast piles will be required for foundation support of the proposed column loads.
- 12- and 18-inch-diameter augercast piles have maximum vertical capacities ranging from 35 to 45 tons and 75 to 80 tons, respectively.
- Piles within 10 feet of new fill must have their capacities reduced by 15 to 25 tons for downdrag loads caused by settlement of the silt and clay.
- Settlements for both single piles and pile groups are estimated to be approximately 1/4 to 1/2 inches and 1/2 to 3/4 inches for 12- and 18-inch-diameter augercast piles loaded to 45 and 80 tons, respectively.
- As an alternative to a pile foundation, the structures may be supported using continuous and isolated spread footings with allowable net bearing pressures of about 3 to 4 kips per square foot (ksf), depending on footing depths, location, and allowable settlements. Shallow support is only possible following significant overexcavation of unsuitable bearing soil.
- Slabs may be constructed as slab-on-grade and should be underlain by at least 2 feet of compacted structural fill, including 6 inches of drainage fill.
- A slab-on-grade loaded to 200 psf will experience settlements less than 1/2 inch.

- An appropriate modulus of subgrade reaction for use in design of slabs-on-grade would be about 250 pci, as measured on a 1-foot-square plate.

Miscellaneous Recommendations

- Design subgrade walls backfilled on one side for pressures estimated using equivalent fluid weights of 35, 50, and 250 pounds per cubic foot (pcf) for active, at-rest, and allowable passive conditions, respectively. Add a surcharge component to represent loading from any adjacent building foundations founded above the base of the wall.

- Placement of about 4 feet of fill material around the training center building will result in about 1-1/4 inches of total settlement. The piles will be designed for the downdrag loads which will result from the settlement.

- Provide permanent underslab drainage in the parking garage area through a system of underslab perimeter drains leading to sumps. Drains are also needed at the base of any below grade walls.

- In many areas the excavated soils will be reusable as structural fill, depending on weather conditions at the time of placement.

- At least 6 inches of drainage fill should be placed below slabs and 18 inches adjacent to subgrade walls.

- Estimate sliding friction between the subgrade and the base of footings and slabs using an allowable coefficient of friction of about 0.30.

- The UBC soil profile type for use in seismic design is S_2 with a coefficient S of 1.2. Compressive pile capacities can be increased by up to one-third to resist seismic or wind loads. The potential for liquefaction is low due to the relatively deep groundwater level.

SUBSURFACE CONDITIONS

We interpreted the subsurface soil conditions from the explorations accomplished at the site by Hart Crowser and others. Soil properties inferred from the field and laboratory tests at this and other sites formed the basis for the foundation design and construction recommendations contained within this report. We noted some variation in the shallow conditions in the borings. The nature and extent of variations between the explorations may not become fully evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report. Details of the conditions observed at the exploration locations are shown on the logs included in Appendix A and should be referred to for specific information. Our interpretation of the soil conditions is given below, and shown graphically on Cross Sections, Figures 3 through 6.

Soils Conditions: Sand with Upper Silt and Clay

Soils encountered in our explorations generally consist of about 1-1/2 to 8 feet of silty, gravelly sand fill over 7-1/2 to 12 feet of silt and clay over sand with a varying silt and gravel content. The soil deposits vary uniformly across the site with the silt and clay thickness being greater at the west side of the site. No silt or clay was encountered in boring HC-6, in the northeast corner. The subsurface can be broadly grouped into three zones:

- **Fill Material.** Medium dense to dense, slightly silty to silty, gravelly sand extends from the ground surface to depths of about 1-1/2 to 8 feet. The fill contains occasional cobbles, organics, and wood fragments.

- **Silt and Clay.** The surface fill is underlain by medium stiff to hard, clayey silt and silty clay with varying amounts of sand extending to depths ranging from about 11 feet to the maximum depth explored in HC-1 of 20 feet. These soils are compressible and will settle under an increase in load.

- **Sand.** The silt and clay zone is underlain by medium dense to very dense, slightly silty to silty, slightly gravelly to gravelly sand extending to the maximum depth explored of about 29 feet. This material will provide good pile support with low resulting settlements.

Perched Groundwater

Groundwater was encountered in boring HC-1 at an elevation of about 156 feet at the time of exploration. Groundwater was encountered in no other boring. Therefore, we expect that any groundwater is perched above the silt unit. Water level readings have been made in the borings at the times and under conditions stated on the boring logs. These data have been reviewed and interpretations made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater occur due to variations in rainfall, temperature, seasons, and other factors at the time measurements are made.

GEOTECHNICAL ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

The following section includes three main subheadings addressing excavation, foundations, and other miscellaneous design components. Minor subheadings introduce specific topics. The recommendations given will be incorporated by the structural engineer into the excavation and foundation plans for the building.

As the design proceeds it may be necessary to analyze portions of the design recommendations in more detail. We expect to have continued interaction with the design team as the excavation and foundation plans are developed.

General Foundation Considerations

Foundations

Based on the subsurface conditions encountered, it is our opinion that the structural column loads would result in excessive settlements if the proposed facilities were founded on shallow spread footings founded directly on the near-surface soils. We therefore recommend the use of pile foundations. We expect that many piles will be installed within 10 feet of the existing office building in the northeast corner of the site. Considering the desired high pile capacities and therefore associated high pile driving hammer energy required for installation, intense and potentially damaging vibrations could result if driven piles are used. Hart Crowser met with the design engineers to discuss this recommendation. The parties concluded that where piles are recommended, only augercast would be used. Other types of piles are therefore not addressed in this report, even though they were considered.

As an alternative to a pile foundation, the structures may be supported using continuous and isolated spread footings with allowable net bearing pressures of about 3 to 4 kips per square foot (ksf), depending on footing depths, location, and allowable settlements. In order to obtain suitable bearing pressures and to minimize damage to the structures due to total and differential settlements, compressible material below each footing should be partially overexcavated and replaced with compacted structural fill material. The depth of the required overexcavation would vary, but an average depth of about 5 feet would be needed for the structures located at the west side of the site. Based on discussions with the design engineers (TRA) concerning our recommendations and this report, it is our understanding that a shallow foundation system will not be used. Therefore, the rest of this report focuses on pile support.

Settlement of Existing Foundations

The existing office building located in the northeast corner of the site could settle under increased surface loading due to the construction of the new office building and training center and placement of fill around the training center. We estimate that, if floor slabs and fill are placed within 5 and 10 feet of the existing office, building settlements of about 1/4 to 1/2 inch and less than 1/4 inch, respectively, could result.

Excavations

A below-grade parking garage will be located adjacent to and south of the proposed office building. We expect that the elevation of the base of the excavation will not be below 156 feet and will therefore not encounter the groundwater level (groundwater was not encountered in our borings within the plan area of the proposed excavation and at elevation 156 feet in HC-1).

Foundation Design

Augercast piles will support the columns for the proposed Snohomish County PUD headquarters building, training center, and the parking garage.

Vertical Pile Capacity

We analyzed three different pile sizes including 12-, 16-, and 18-inch-diameter augercast piles. We present design recommendations for all of these pile sizes at specific building areas.

The plan area of the parking garage has been roughly divided in half; the western half is designated as garage west and the eastern half designated as garage east. Piles within the plan area of the training center and new office building are designated as buildings.

Table 1 outlines our augercast pile design recommendations. Required pile lengths will be at least 30 feet in the western half of the garage and 25 feet in all other areas. The compressive capacities may be increased by one-third when computing resistance to transient loads such as wind and seismic loading. No such increase should be allowed for the uplift capacity.

Table 1 - Design Recommendations 12-, 16-, and 18-inch-diameter Augercast Piles

Pile Diameter in Inches	Compressive Capacity in Tons			Uplift Capacity in Tons		
	Garage West	Garage East	Buildings	Garage West	Garage East	Buildings
12	35	45	40	20	35	25
16	60	70	65	25	50	35
18	75	80	75	30	60	40

Note that piles within 10 feet of the edge of any fill thicker than 3 feet will need to be designed for downdrag loads caused by settlement of soil around the pile. The compressive capacity of the 12-, 16-, and 18-inch piles must be reduced to 25, 45, and 50 tons, respectively.

Pile Settlements

Single and group pile settlements for 12- and 18-inch-diameter piles are estimated to be about 1/4 to 1/2 inch and 1/2 to 3/4 inch, respectively. These settlement estimates are based on the application of the largest allowable static loads given in Table 1. Settlements of piles in the downdrag areas could be about ¼ inch more. There are no significant compressible layers beneath the pile tips at this site, so there is no long-term settlement component.

Lateral Foundation Capacity

Piles. Lateral forces developed during an earthquake or as a result of wind or other forces can be resisted by batter piles or the passive resistance of soil surrounding vertical piles. We understand that batter piling will not be used for this project. Therefore, all lateral forces would be resisted by vertical piling. This section presents a recommended method of analysis and appropriate soil parameters for laterally loaded vertical piles.

Lateral resistance and deflections of vertical pile foundations are governed primarily by the lateral capacity of near-surface soils and the strength of the pile itself. The design lateral capacity of the vertical piles will depend, to a large extent, on the allowable lateral deflections of the piles. Use of the procedure discussed below, incorporating the design charts on Figures 7 and 8 will allow the structural engineer to estimate the pile deflection and moments within the pile at any point at or below the pile cap for a given loading.

Development of lateral pile criteria requires an assumption of the degree of fixity at the pile head by the structural engineer. A pile is considered free-headed if the top is free to rotate. If the top of the pile is fixed against rotation by embedment in a pile cap that is sufficient to develop a fixed-end moment, the pile is considered restrained and fixed-headed. We expect that the piling would be structurally connected to the basement structure and therefore fixed to a great degree against rotation. We recommend that the structural engineer evaluate the degree of fixity and then linearly interpolate between results outlined on

Figure 7 (true fixity at head) and on Figure 8 (true free-headed condition).

In addition to the pile head fixity condition, the following information is required to determine lateral pile deflections and moments:

Moment and Deflection Equations

Free-Headed Condition

$$Y = \frac{A_y P_{xx} T^3}{EI} + \frac{B_y M_{xx} T^2}{EI}$$

$$M = A_m P_{xx} T + B_m M_{xx}$$

Fixed-Headed Condition

$$Y = \frac{A_y P_{xx} T^3}{EI}$$

$$M = A_m P_{xx} T$$

Where:

Y	=	Deflection at any point at or below the pile cap,
M	=	Moment at any point at or below the pile cap,
P_{xx}	=	Shear applied to the pile at pile cap (x-x plane),
M_{xx}	=	Moment applied to the pile at pile cap (x-x plane),
A_y, B_y	=	Deflection coefficients from Figure 7 or 8,
A_m, B_m	=	Moment coefficients from Figure 7 or 8,
EI	=	Flexural stiffness of the pile,
T	=	Relative stiffness factor = $(EI/n_h)^{1/5}$,
n_h	=	Coefficient of variation of horizontal subgrade reaction, in pounds per cubic inch,
$2T$	=	Assumed depth to point of zero deflection.

The rate of increase of horizontal subgrade reaction, n_h, is related to the stiffness and density of the soil. The soil above about 2T (2 times the relative stiffness factor) usually controls the lateral capacity of the pile. Most of the lateral resistance will be mobilized by the soil surrounding the upper 10 feet of the pile. We therefore recommend using an average value over this depth interval. For the sand fill materials and silt and clay encountered in this zone, an appropriate value of lateral modulus of subgrade reaction is estimated to be 20 pounds per cubic inch (pci).

The coefficient of variation of horizontal subgrade reaction should also be modified for pile group effects. Table 2 outlines the recommended reduction factors depending on pile spacing.

Table 2 - Pile Group Reduction Factors for Coefficient of Variation of Horizontal Subgrade Reaction

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reaction Reduction Factor R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

In order for the pile to develop fixity, it is generally considered that a total embedment of at least 4T (four times the relative stiffness factor) must be attained. With 4T or greater embedment the ultimate resistance to an applied lateral load is governed primarily by the strength characteristics of the pile and not the strength of the soil. In contrast, should the pile be embedded to a depth of only 2T or less, the ultimate resistance to lateral loads would be governed primarily by the strength of the soil with the pile acting as a rigid member (or pole). An embedment of between 2T and 4T would be considered an intermediate case, i.e., the ultimate lateral loading is dependent on both the soil and the pile strength.

For the typical case at this site, we anticipate that an embedment greater than 4T will be realized, given the embedment requirements for vertical capacity.

The moment formulations calculated using the procedures do not contain a factor of safety. The structural engineer should incorporate a suitable factor of safety in the lateral load design, and should verify the strength of the pile to resist the applied lateral loads.

Augercast Pile Construction Recommendations

Many of the recommendations in this section are appropriate for inclusion in the project specifications.

We make the following general recommendations:

- All piles should be spaced no closer than 3 pile diameters on center.
- Allow at least one day between installation of piles within 5 pile diameters of each other.

A contractor constructs an augercast pile following these steps:

- Drill a hole with a hollow-stem auger.
- Pump concrete (grout) under pressure through the auger as it is slowly withdrawn.
- Push reinforcing steel into the wet concrete.

Once the concrete has set up, the contractor constructs the foundation in the typical manner for a pile foundation.

During withdrawal of the auger, soil may collapse into the hole if the pile is not constructed properly, displacing grout from the pile. This can also happen if a pile is installed near (within 5 pile diameters) a recently installed pile.

We make the following recommendations to help with quality control during installation:

- Require the contractor to provide a pressure gage in the grout line.
- Minimum pressures should be those required to maintain a steady flow of grout to the auger. A typical value of 100 psi should be used for this purpose.
- Rapid drops in the grout pressure of 50 psi or more occurring when otherwise accepted procedures are used should be specified as a possible cause for reconstructing the pile.

- The rate of grout injection and rate of auger withdrawal from the soils should be balanced to maintain a positive grout head of at least 15 feet above the bottom of the auger.

- Require contractor to provide a means of monitoring quantity of grout used per pile, for example, a stroke counter on the grout pump.

- Require contractor to rotate the auger after initial grout pumping (about 2 cubic feet) prior to beginning withdrawal of the auger.

Augercast pile installation requires an experienced contractor and careful observation. We strongly recommend a geotechnical engineer or experienced technician observe the pile installation. Also, we suggest that all contractors considered for the project be required to demonstrate previous satisfactory performance on augercast pile installation projects.

Floor Slab Design

The garage and office basement floor slabs will be constructed as slab-on-grade. The training center floor slab will be supported structurally, with a crawl space below. The training center floor elevations are several feet above existing grade. Due to the soil conditions, the design configuration, and the anticipated time of construction, the design engineers have indicated that a structural floor rather than a slab on preloaded fill will be used.

In order to support the garage and office floor, at least 2 feet of the unsuitable fill or silt and clay must be excavated below the bottom of the slab and replaced with compacted structural fill. Even with this overexcavation, there is some potential for uneven support of the floor, and thus, uneven settlement. Average settlements of about 1/2 inch for typical slab loads of about 200 psf should be expected.

The slabs should also be underlain by a 6-inch thickness of clean, well-graded sand and gravel with a fines content of less than 3 percent based on the minus ¾-inch fraction. This layer serves as a capillary break and drainage layer and is intended to reduce the potential build-up of hydrostatic pressures beneath the slab and, in the garage slab, envelop the recommended subslab drains (see ***Drainage Considerations*** section). This 6-inch layer can be included in the 2-foot structural fill thickness.

Any disturbed or loose areas should be recompacted or removed to provide a dense, non-yielding surface for the placement of drainage fill or for slab construction.

Excavations and Filling

Open Cut Excavations

We expect that all excavations can be open cut. Cut slopes within the silt and clay zone should stand at a maximum slope of 1-1/2 Horizontal to 1 Vertical (1-1/2H:1V), with proper protection. Steeper cut slopes of 1H:1V should stand in the lower dense sand zone. Flatter slopes of 2H:1V will probably be required through the surface fill materials. The actual temporary slopes are the contractor's responsibility. We expect that plastic sheeting will provide the necessary protection from precipitation and excessive drying. Depending on the steepness of the slopes, more substantial protection may be needed. Construction equipment, workers, and materials should be kept at least 5 feet back from the top of any slopes.

The excavation can be completed using conventional heavy equipment such as a large bulldozer and backhoe. Most of the cut will be within medium dense to dense sandy fill materials and medium stiff to hard silt and clay.

Perched groundwater was encountered in HC-1 at an elevation of 156 feet. Groundwater was encountered in no other boring. However, we expect to find seepage in isolated seams, particularly during wet weather periods. The granular soils are susceptible to loosening and caving when exposed to moisture or when unconfined in a very wet condition.

Because of probable precipitation runoff during excavation, the contractor should be prepared to provide temporary drainage, such as sumps, to maintain the excavation in a workable condition.

Shoring

If shoring is needed, temporary lateral support may be provided by a cantilever soldier pile wall. Design the wall for pressures estimated using equivalent fluid weights of 35 and 250 pounds per cubic foot (pcf) for active and allowable passive conditions, respectively. If more detailed design information is needed, Hart Crowser can provide a complete design.

Structural Fill

We make the following recommendations:

- Place all fill beneath slabs-on-grade or behind basement walls as structural fill.
- Place structural fill in lifts not exceeding 10 inches loose thickness and compact to a minimum of 95 percent of the modified Proctor maximum dry density (as determined by ASTM D 1557) below slabs, and 92 percent within 4 feet of walls.
- We recommend using a clean, well-graded sand or sand and gravel with less than 5 percent by weight (based on the minus 3/4-inch fraction) passing the No. 200 mesh sieve. Soils with more than 5 percent fine-grained material will be hard to compact properly in wet weather.
- Use well-graded fill with a fines content of less than 3 percent for fill within 6 inches of slabs-on-grade (drainage layer) and pavement sections, 18 inches of backfilled subgrade walls, and around all drains.
- All structural fill should have a gravel content (material coarser than a U.S. No. 4 sieve) of at least 30 percent.
- Prior to structural fill placement, remove, recompact, or dry all disturbed, loose, or wet subgrade areas.

Before fill control can begin, the compaction characteristics must be determined from representative samples of the structural and drainage fill. Samples should be obtained from the borrow area as soon as work begins. A study of the compaction characteristics should include determination of optimum and natural moisture contents of these soils at the time of placement.

We performed grain size analyses on samples of the surface soils obtained from borings HC-1 and HC-4. The soils tested have a fines content of about 16 to 26 percent and a moisture content of about 8 to 20 percent. The existing surface fill materials may be suitable for reuse as structural fill provided these soils are reworked and placed during extended dry weather periods. The silt and clay soils are not suitable for reuse as structural fill. We expect that most of the excavated material in the garage area will be silt.

Underground Utilities

In order to provide suitable support and to minimize damage due to settlements, underground utility lines should be founded on at least 2 feet of compacted structural fill. We make the following recommendations about utility trench backfill, and reference the previous recommendations for structural fill.

- Place all fill in pipeline or utility trenches as structural fill in lifts not exceeding 8 inches loose thickness.
- The upper 2 feet of trench backfill in pavement areas should be compacted to at least 95 percent of the modified Proctor maximum dry density as determined by ASTM D 1557.
- Below a depth of 2 feet, 92 percent compaction can be used.

Trench excavations can be accomplished by open cut excavations (see ***Open Cut Excavations*** section) or the use of trench boxes to provide lateral support. Selection of an appropriate trench box or temporary slopes are the contractors responsibility.

Miscellaneous Recommendations and Conclusions

Lateral Loads and Resistances of Pile Caps and Grade Beams

The pile cap and grade beams, upon deflection, will contribute some lateral resistance to the foundation system. The allowable equivalent passive fluid weight above the groundwater table is 250 pounds per cubic foot (pcf) for compacted structural fill. Lateral forces can also be resisted by friction, with an allowable coefficient of friction of about 0.3, appropriate for mass concrete against sand. Active and at-rest pressures can be estimated using equivalent fluid weights of 35 and 50 pcf, respectively.

The ultimate passive values are only obtained with 2% strain. Usually this much strain is considered failure so the allowable equivalent passive fluid weight should be used. There is a balance between deflections of piles, passive pressure on pile caps and grade beams, and friction developed. When the allowable pile cap deflection is finalized, passive resistance can be calculated.

Drainage Considerations

Based on observations made during drilling, we do not expect unmanageable groundwater conditions. Wet zones may be encountered in excavations during wet weather periods, but we expect no temporary dewatering other than ditches and sumps to be needed.

Regardless of whether significant water is encountered during the construction phase, it is necessary to provide some type of permanent drainage system and pressure relief behind all subgrade walls and beneath slabs-on-grade and pavement sections.

Floor Slab and Pavement Section Drainage. An underslab and pavement section drainage system should be installed to prevent a buildup of hydrostatic pressures. We therefore recommend:

- Provide subslab drainage to the garage slab by using a perimeter drain beneath slabs-on-grade, together with cross drains on about a 50-foot spacing.

- All slabs and pavement sections should be underlain directly, everywhere, by a 6-inch drainage layer and capillary break.

The drains (with cleanouts) should consist of 4-inch-diameter perforated pipe wrapped in filter fabric and placed on a bed of, and surrounded by, 6 inches of clean (less than 3 percent fines), free-draining sand and gravel. The drains should be sloped to carry the water to a sump or other suitable discharge location.

Backfilled Wall Drainage. Walls with soil backfilled on only one side will require drainage, or must be designed for full hydrostatic pressures. To provide drainage, we recommend:

- Backfill within 18 inches of any backfilled retaining or subgrade walls with clean, free-draining sand and gravel.

- Install drains (as described above) behind and at the base of any backfilled subgrade walls.

This backfill should be continuous and envelop the drains behind the wall. The backfill should also provide a hydraulic connection to the subslab drainage layer.

Waterproofing. Note that the described subslab and wall drainage system is designed to prevent a damaging build-up of hydrostatic pressure. The recommended systems may not result in a totally dry subgrade wall or slab.

If waterproofing is required below grade, we recommend placement of a heavy plastic liner against the walls coupled with full coverage of a drainage medium such as miradrain. Alternatively, Volclay panels may be used. Beneath the slabs, we recommend the thickness of drainage fill be increased to 12 inches and that cross drains be installed on about 30-foot centers.

Slope pavement and sidewalks to drain away from the building and provide adequate runoff disposal. Do not tie the roof drains to any of the subgrade wall or slab drainage pipes.

Lateral Pressures on Permanent Basement Walls

For retaining walls backfilled on one side only, the structural engineer can estimate the lateral load and resistance on the walls using an equivalent fluid to represent the soil. We make the following design recommendations for a wall backfilled with structural fill and above the groundwater level:

- For a yielding wall with level backfill, the equivalent active fluid weight of the soil is 35 pcf. We define the yielding wall as one where the top moves, when loaded, at least 0.1 percent of its height.

- For a non-yielding wall with level backfill, the equivalent at-rest fluid weight is 50 pcf.

- Add a surcharge component to the lateral pressures to represent loading from other foundation elements, if applicable. Simple area surcharges can be estimated by taking the applied load within the influence area and multiplying it by 30 percent or 50 percent for the lateral pressure increase under active and at-rest conditions, respectively.

- An appropriate passive resistance would be based on an equivalent fluid weight of 250 pcf (including a factor of safety of at least 1.5). Note that the use of passive pressure is appropriate if the subgrade wall is allowed to yield a minimum 0.1 percent of its height. For a non-yielding wall, at-rest conditions should be used.

- Include all surcharge loads in the wall design.

Soil Modulus and Friction

An appropriate modulus of subgrade reaction for use in the design of slabs-on-grade would be 250 pci for the medium dense sands and gravels. Sliding friction between the footings and slabs and subgrade may be determined using an allowable (FS = 1.5) coefficient of friction of about 0.3.

Pavement Design

A pavement section consisting of 2 inches of asphalt over 4 inches of aggregate base rock should be suitable for automobile parking and access areas. In heavier traffic areas, 3 inches of asphalt over 5 inches of aggregate base rock would be appropriate. The pavement section should be underlain by at least 1-1/2 feet of compacted structural fill, or recompacted natural material, including about 6 inches of drainage fill directly beneath the asphalt. Loose or unstable areas should be stabilized by recompaction or excavation and replacement of materials.

To assist in pavement design we recommend the following California Bearing Ratio (CBR) be used:

- 25 percent for pavement on at least 1 to 2 feet of compacted structural fill.

All pavement areas should be proof rolled and observed by an experienced geotechnical engineer or geologist for determination of overexcavation. If the proofrolling shows the area to be firm and non-yielding, no additional preparation is needed before the crushed rock and asphalt are placed. Unstable areas should be overexcavated and replaced with clean, free-draining structural fill or crushed rock before the asphalt section is placed.

Seismic Site Coefficient

The UBC soil profile type for use in seismic design is S_2 with a coefficient S of 1.2. Compressive capacities of piles can be increased by up to one-third to resist seismic or wind loads. The potential for liquefaction is low due to the relatively deep groundwater level.

RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

Before construction begins, we recommend that Hart Crowser:

- Review the final foundation and excavation design plans and specifications in order to see that the geotechnical engineering recommendations are properly interpreted and implemented into the design.

During the construction phase of the project, we recommend that Hart Crowser observe the following activities:

- Excavation;
- Verification of the foundation subgrade conditions;
- Placement of foundations and slabs-on-grade;
- Augercast pile installation;

- Placement and testing of fill;
- Installation of subslab and wall drainage; and
- Other geotechnical considerations which may arise during the course of construction.

The purpose of these observations is to observe compliance with the design concepts, specifications, or recommendations and to allow design changes or evaluation of appropriate construction measures in the event that subsurface conditions differ from those anticipated prior to the start of construction.

We trust that this report meets your needs. If you have questions or if we can be of further assistance, please call at your earliest convenience.

HART CROWSER, INC.

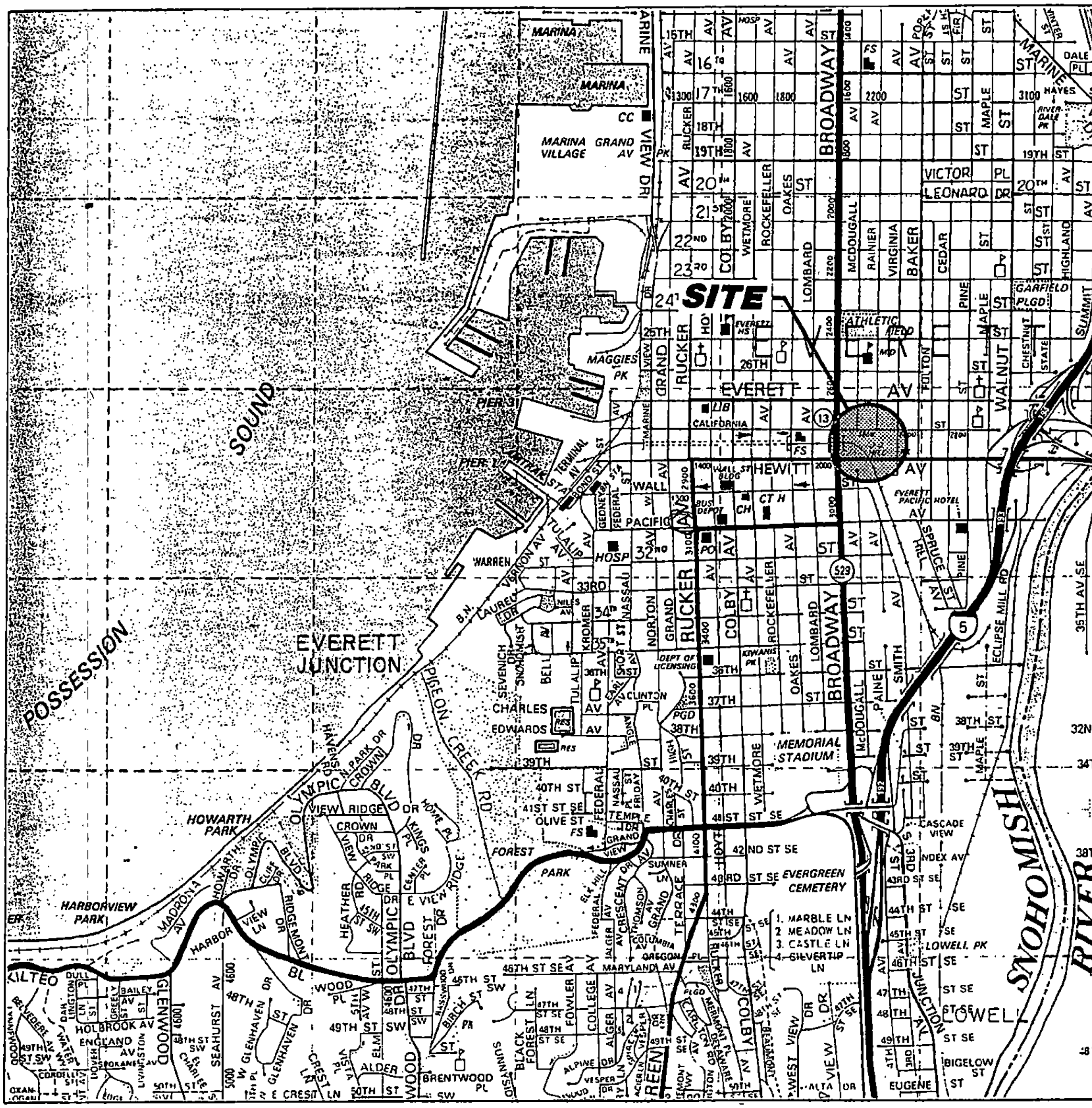
John F. Lemke (jjr)

DAVID G. WINTER, P.E.
Project Manager
Senior Associate

JOHN F. LEMKE
Staff Geotechnical Engineer

DGW/JFL:alm
sno.fr

Vicinity Map



0 1/2 1

Scale in Miles

N

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Figure 1

Site and Exploration Plan

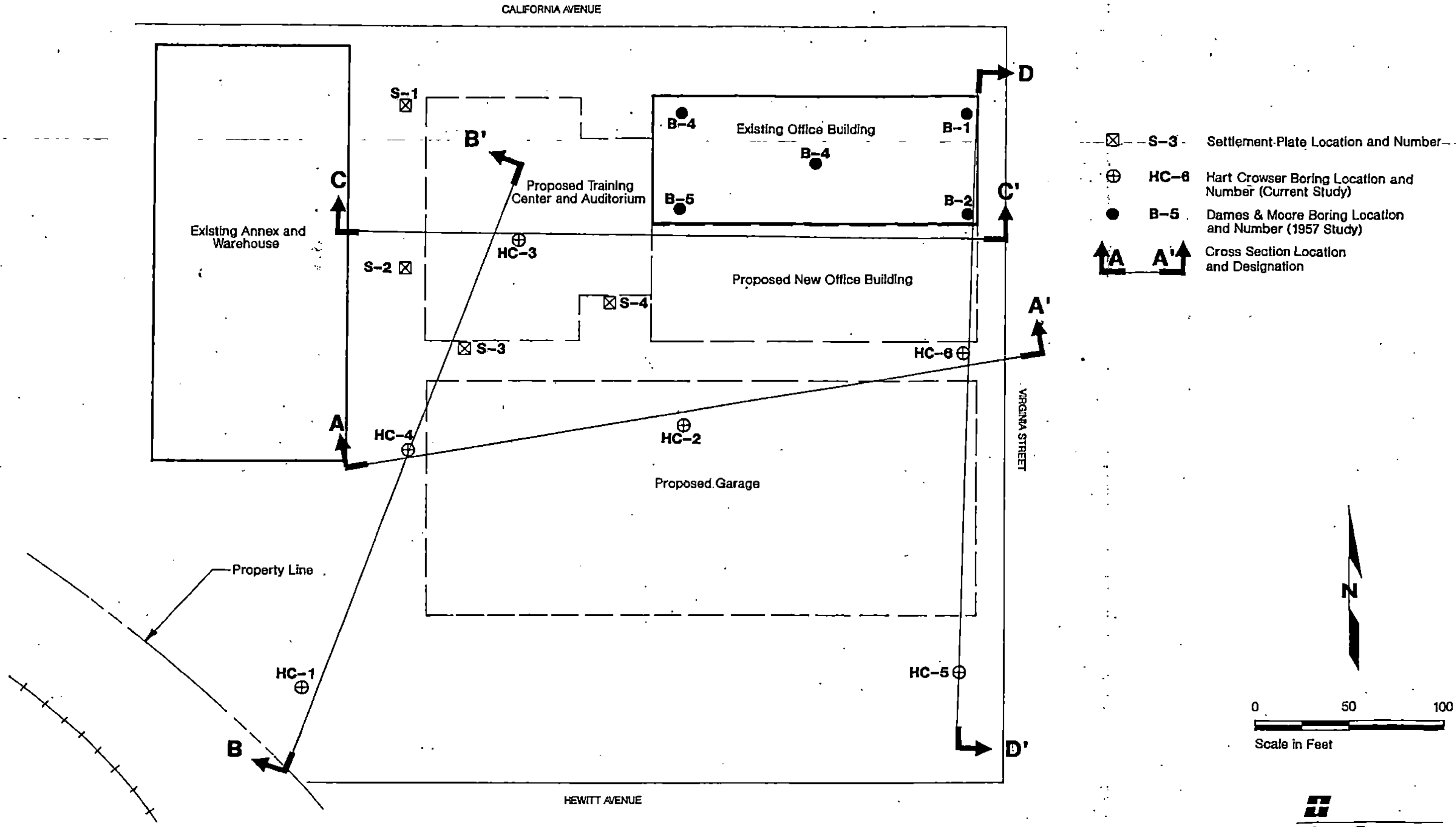

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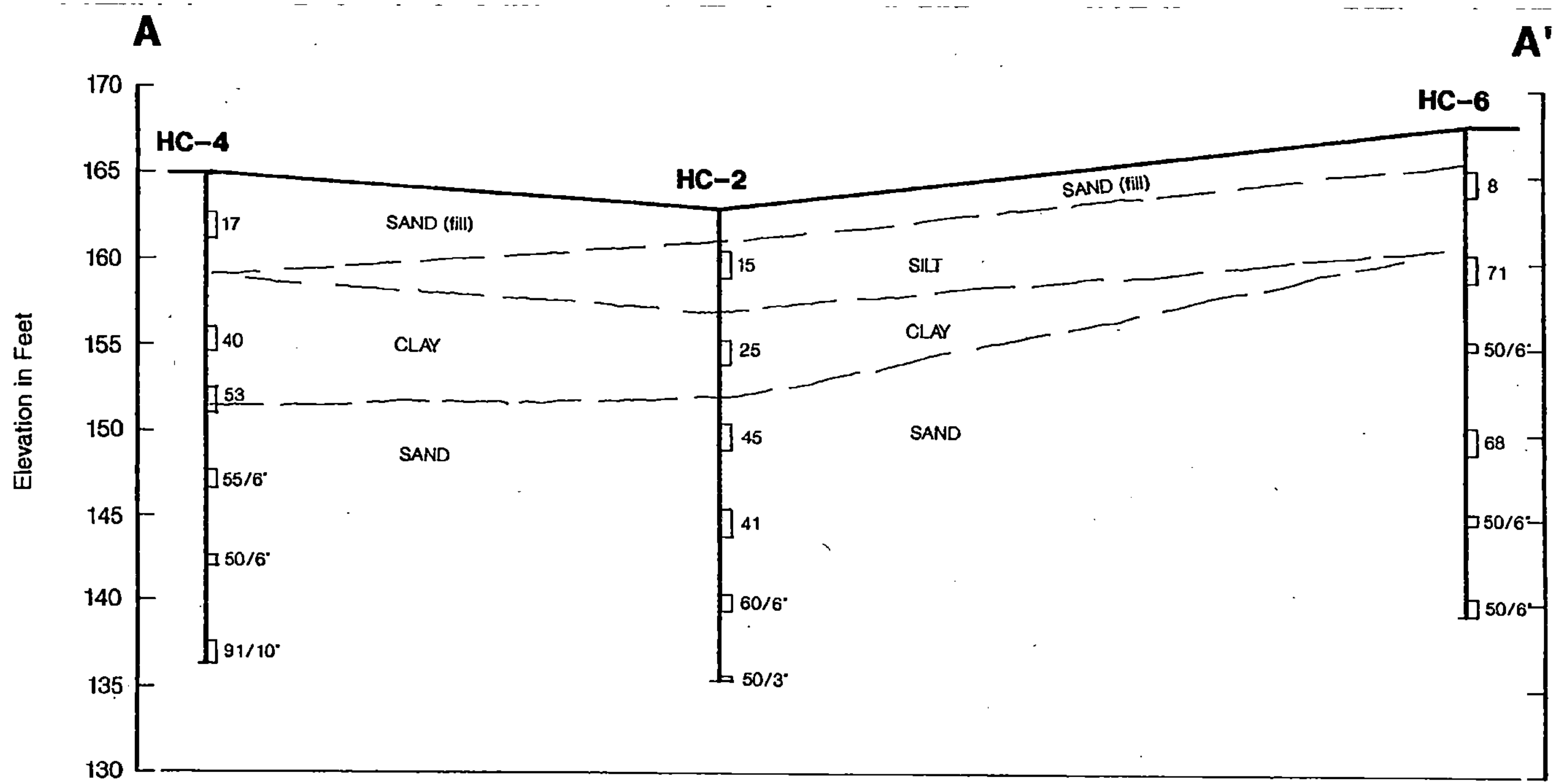
Figure 2

Generalized Subsurface Cross Section A – A'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

HC–6 Boring Number

Boring Location

10 Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet

0 40 80

0 10 20

Vertical Scale in Feet

Vertical Exaggeration x 4

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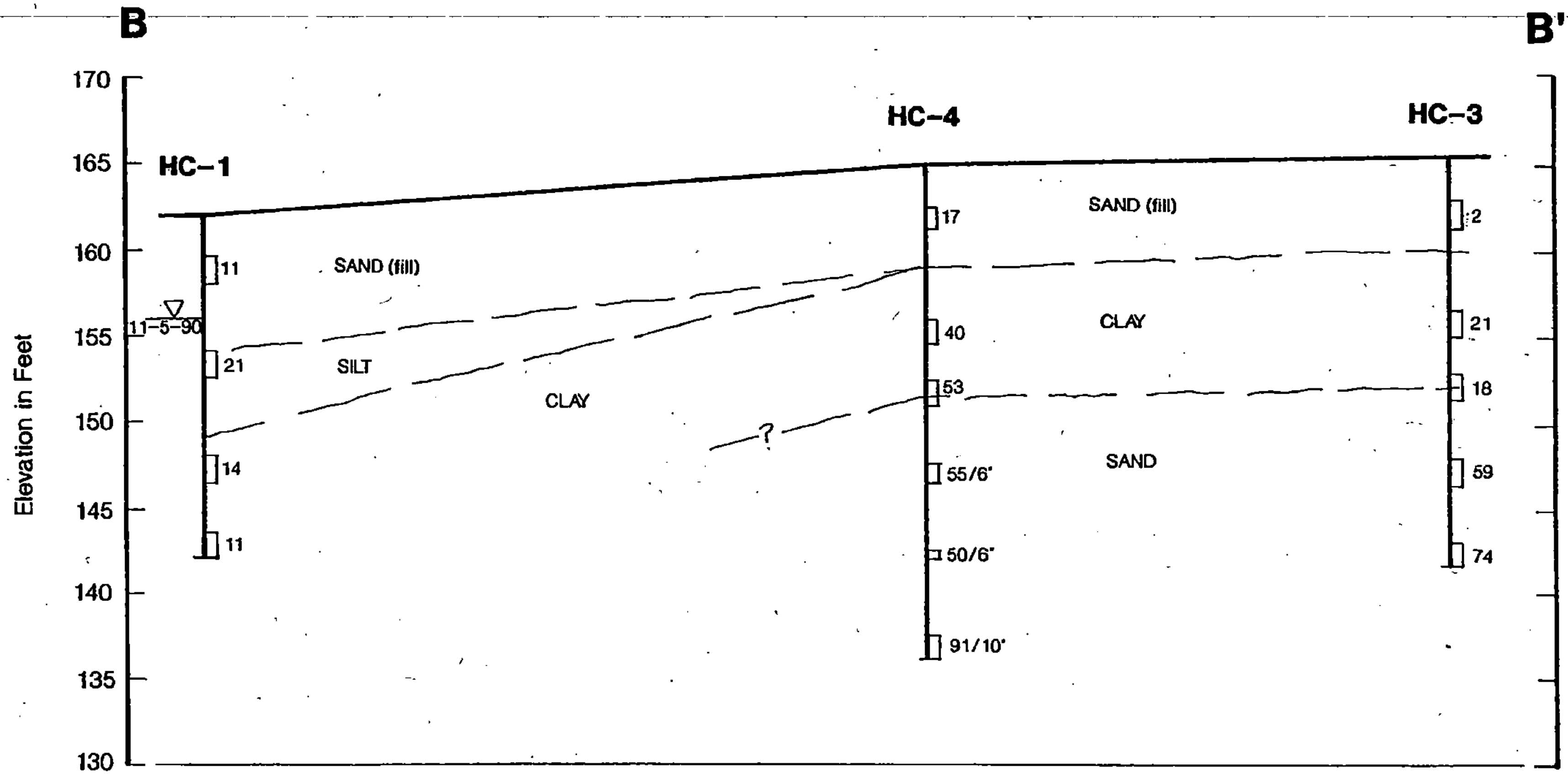
Figure 3

Generlized Subsurface Cross Section B – B'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

HC–4 Boring Number (Hart Cowser)

Boring Location

10 Standard Penetration Resistance in Blows per Foot

11-5-90 Water Level at Time of Drilling for Date Indicated

Horizontal Scale in Feet

0 40 80

0 10 20

Vertical Scale in Feet

Vertical Exaggeration x 4

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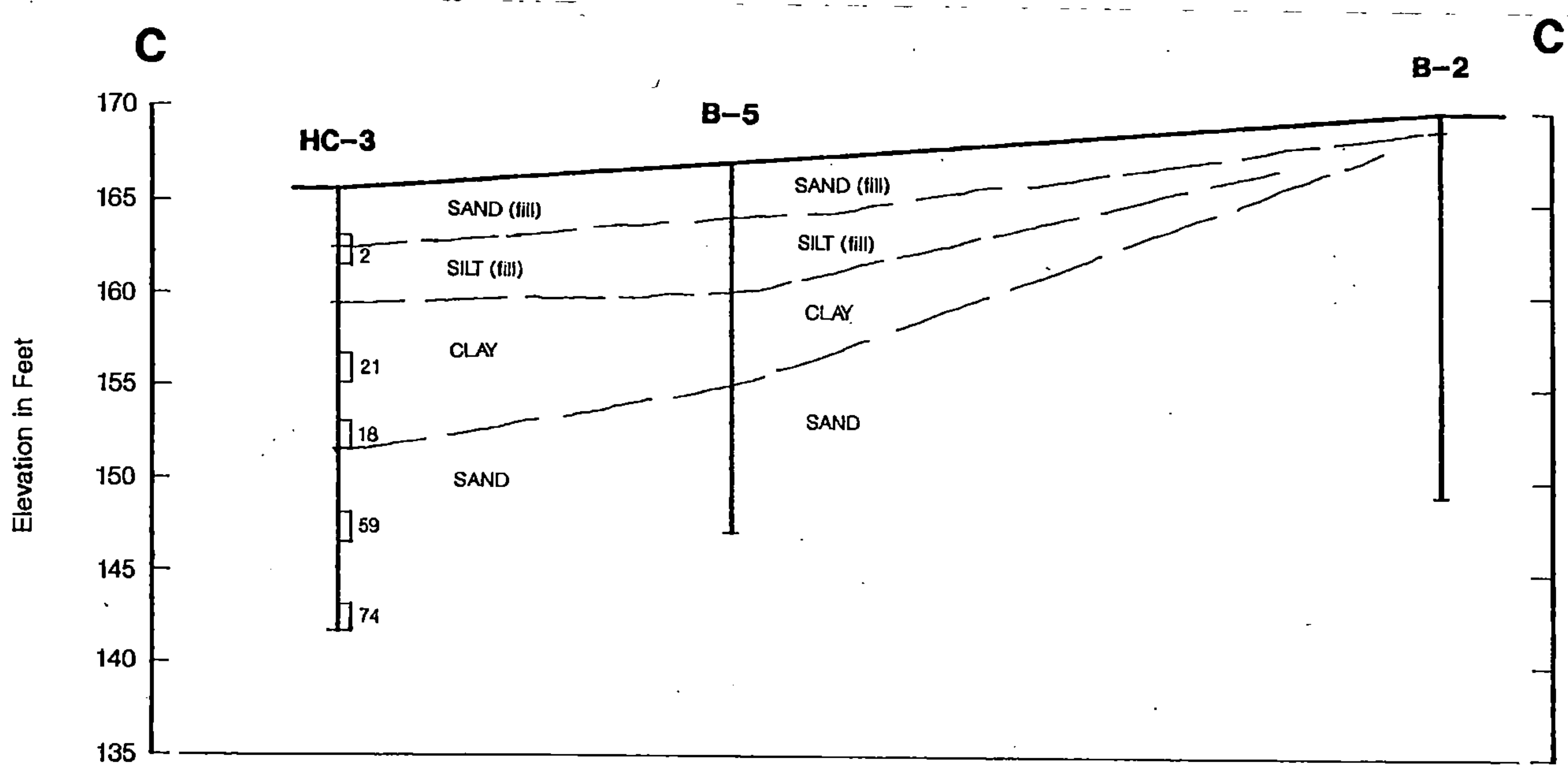
Figure 4

Generalized Subsurface Cross Section C – C'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

HC–3 Boring Number (Hart Crowser)

B–5 Boring Number (Others)

Boring Location

10 Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet

0 40 80

0 10 20

Vertical Scale in Feet

Vertical Exaggeration x 4

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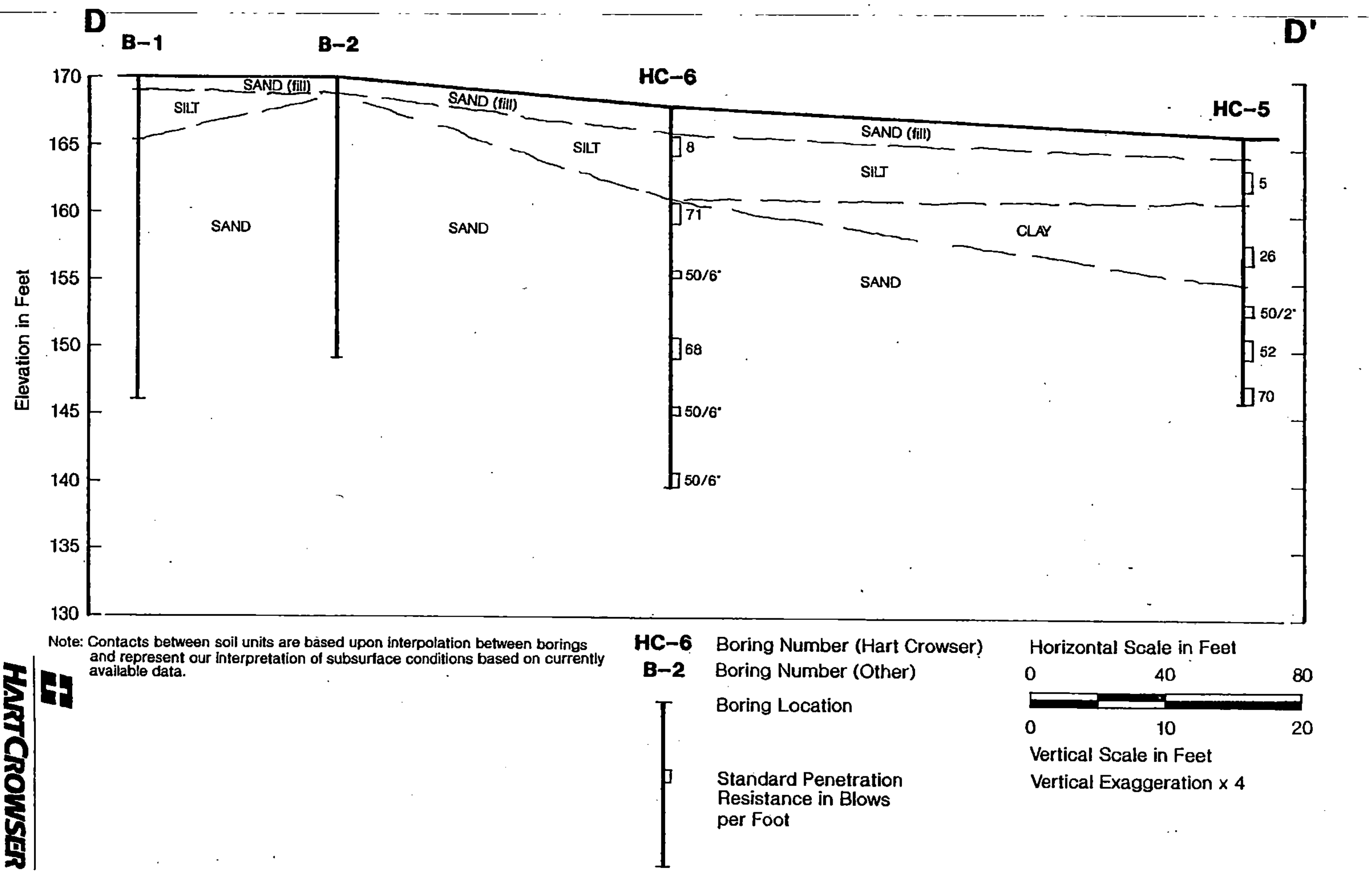
Figure 5

Generalized Subsurface Cross Section D – D'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

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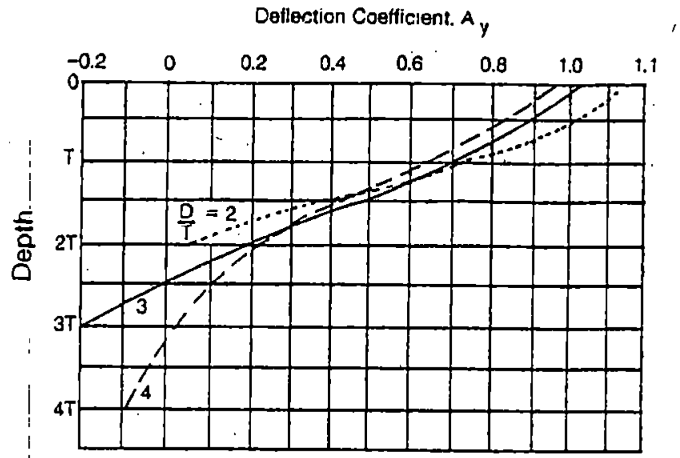
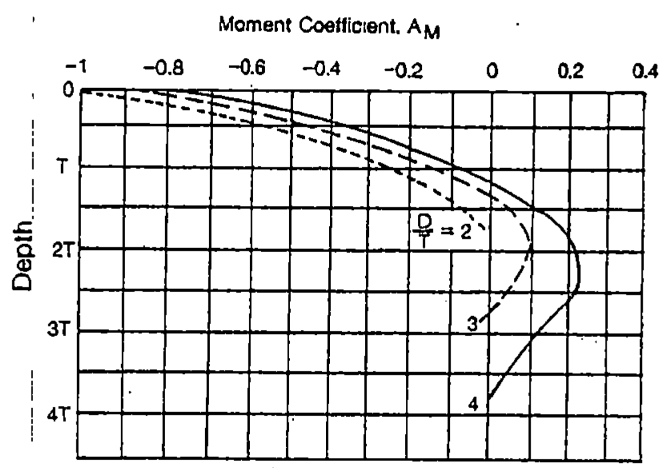
Figure 6

Laterally Loaded Piles in Elastic Subgrade Deflection and Moment Criteria

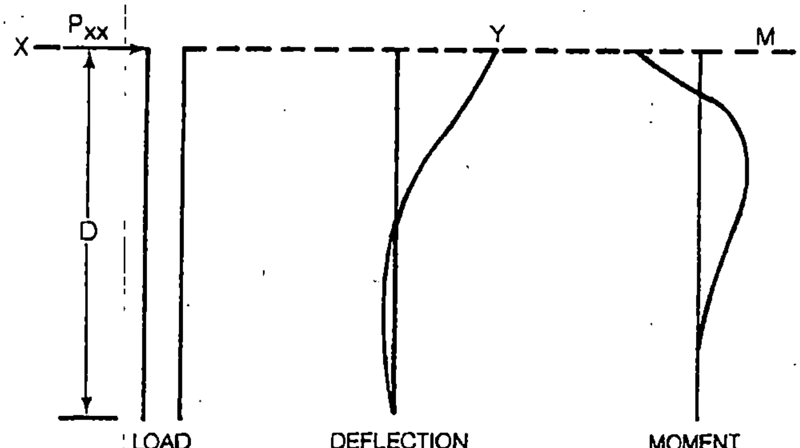
Fixed–Headed Pile Condition

(a) Deflection and Moment Coefficients





(b) Typical Deflection and Moment Curves



P_{XX} Pile Shear at Ground Surface

T Relative Stiffness Factor

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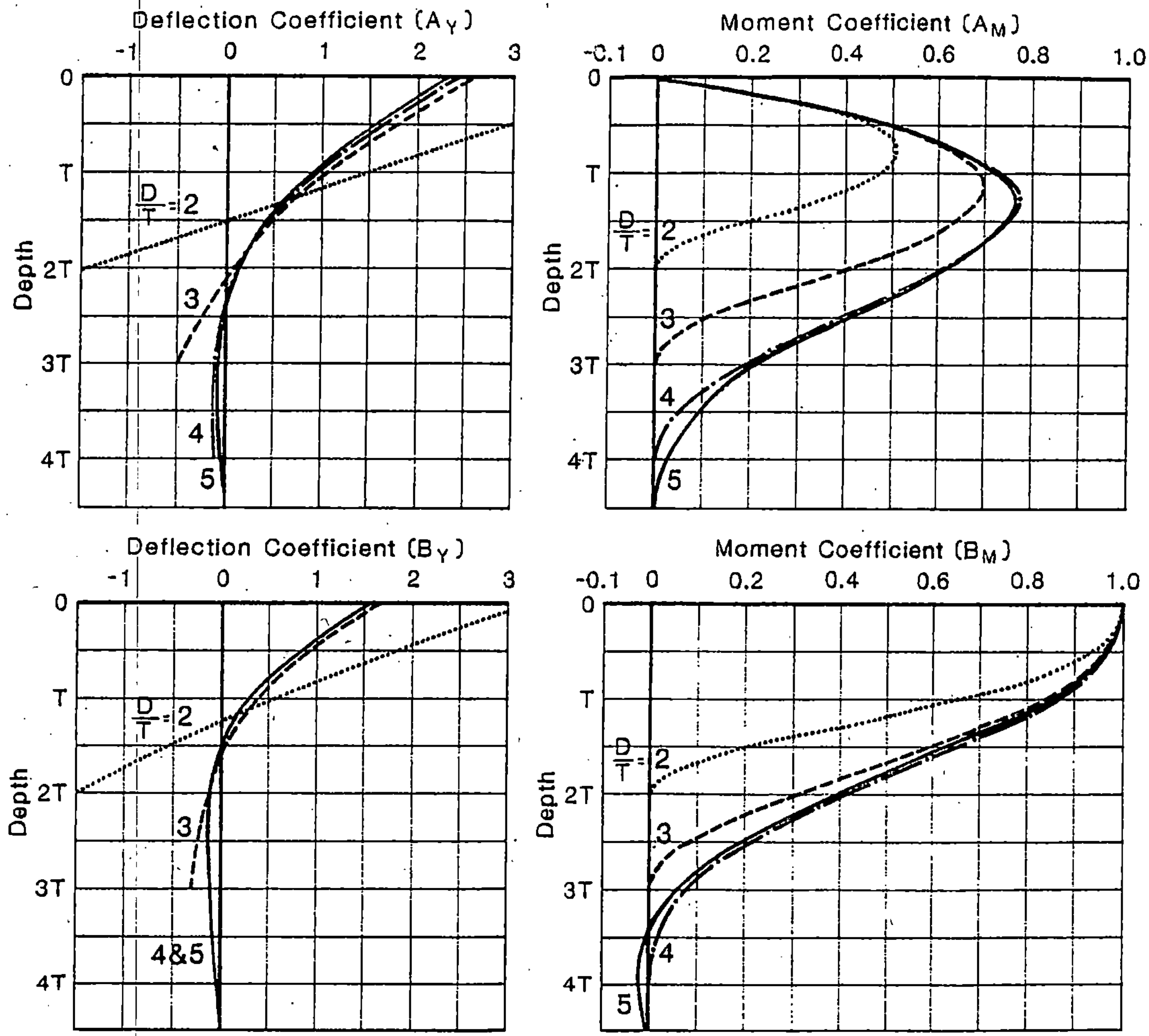
Figure 7

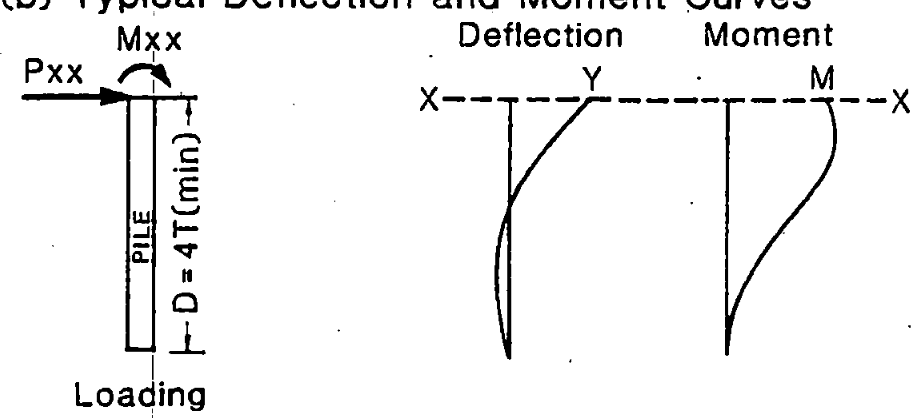
Laterally Loaded Piles in Elastic Subgrade Deflection and Moment Criteria

Free-Headed Pile Condition

(a) Deflection and Moment Coefficients



(b) Typical Deflection and Moment Curves



Pxx	Pile Shear at Ground Surface
Mxx	Pile Moment at Ground Surface
T	Relative Stiffness Factor

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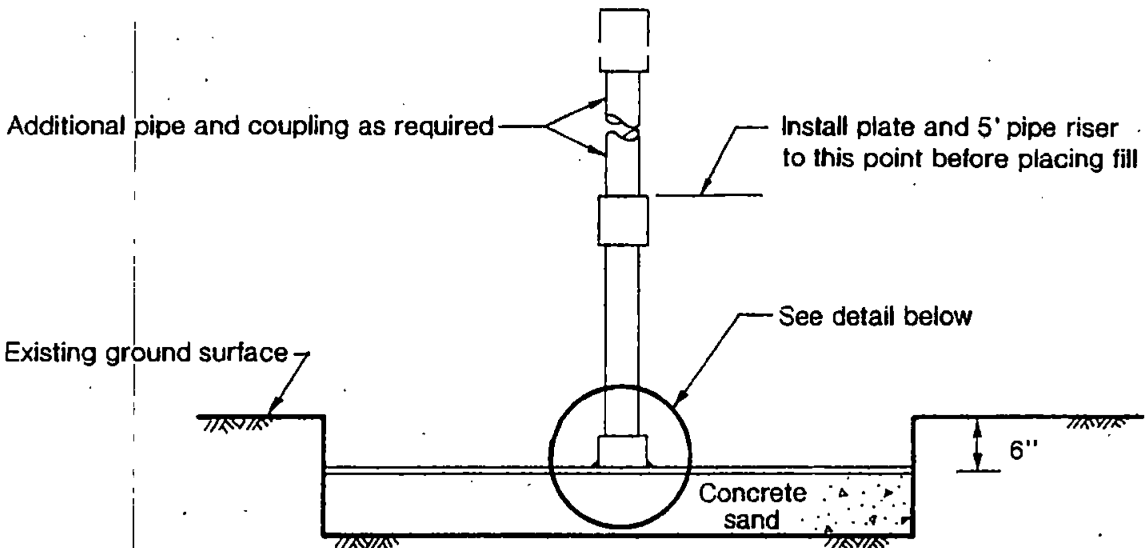
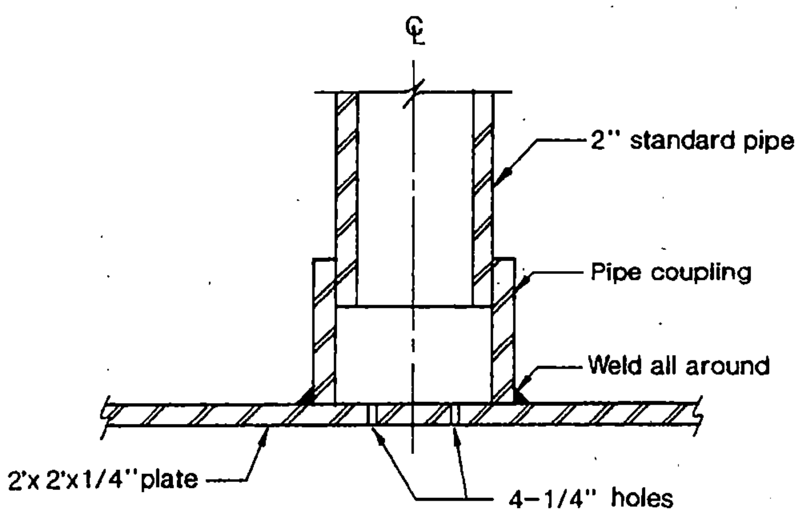
Figure 8

Settlement Plate Installation





HARTCROWSER
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Figure 9

APPENDIX A
FIELD EXPLORATIONS METHODS AND ANALYSIS

APPENDIX A
FIELD EXPLORATIONS METHODS AND ANALYSIS

This appendix documents the processes Hart Crowser uses in determining the nature of the soils underlying the project site addressed by this report. The discussion includes information on the following subjects:

- Explorations and Their Location
- The Use of Auger Borings
- Standard Penetration Test (SPT) Procedures
- Use of Shelby Tubes

Explorations and Their Location

Subsurface explorations for this project include six hollow-stem auger borings. The exploration logs within this appendix show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1 - Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

Location of Explorations. Figure 2 shows the location of explorations, located by hand taping or pacing from existing physical features. The ground surface elevations at these locations were interpreted from elevations shown on a site plan by TRA. The method used determines the accuracy of the location and elevation of the explorations.

The Use of Auger Borings

A subsurface investigation was performed using a truck- mounted, 3-3/8-inch inside diameter hollow-stem auger, subcontracted by Hart Crowser. Six borings, designated HC-1 through HC-6, were drilled on November 5 and 6, 1990, to a maximum depth of about 29 feet. The drilling was continuously observed by a geotechnical engineer from Hart Crowser. Detailed field logs were prepared of each boring. We obtained samples at 2-1/2- to 5-foot-depth intervals using the Standard

Penetration Test (SPT) with a split-barrel sampler and thin-walled Shelby tubes.

The borings logs are presented on Figures A-2 through A-7 at the end of this appendix.

Standard Penetration Test (SPT) Procedures

This test is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT (as described in ASTM D 1587) was used to obtain disturbed samples. This test employs a standard 2-inch outside diameter split-spoon sampler. Using a 140-pound hammer, free-falling 30 inches, the sampler is driven into the soil for 18 inches. The number of blows required to drive the sampler the last 12 inches only is the Standard Penetration Resistance. This resistance, or blow count, measures the relative density of granular soils and the consistency of cohesive soils. The blow counts are plotted on the boring logs at their respective sample depths.

Soil samples are recovered from the split-barrel sampler, field classified, and placed into water tight jars. They are then taken to Hart Crowser's laboratory for further testing.

In the Event of Hard Driving

Occasionally very dense materials preclude driving the total 18-inch sample. When this happens, the penetration resistance is entered on logs as follows:

Penetration less than six inches. The log indicates the total number of blows over the number of inches of penetration.

Penetration greater than six inches. The blow count noted on the log is the sum of the total number of blows completed after the first six inches of penetration. This sum is expressed over the number of inches driven that exceed the first 6 inches. The number of blows needed to drive the first six inches are not reported. For example, a blow count series of 12 blows for 6 inches, 30 blows for 6 inches, and 50 (the maximum number of blows counted within a 6-inch increment for SPT) for 3 inches would be recorded as 80/9.

Use of Shelby Tubes

To obtain a relatively undisturbed sample for classification and testing in fine-grain soils, a 3-inch-diameter thin-walled steel (Shelby) tube sampler was pushed hydraulically below the auger. The tubes were sealed in the field and taken to our laboratory for extrusion, classification, and testing.

Key to Exploration Logs

Sample Descriptions

Classification of soils in this report is 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.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

SAND or GRAVEL Density	Standard Penetration Resistance in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 - 4	Very soft	0 - 2	<0.125
Loose	4 - 10	Soft	2 - 4	0.125 - 0.25
Medium dense	10 - 30	Medium stiff	4 - 8	0.25 - 0.5
Dense	30 - 50	Stiff	8 - 15	0.5 - 1.0
Very dense	>50	Very stiff	15 - 30	1.0 - 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

Minor Constituents

	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Legends

Sampling

BORING SAMPLES

- ⊠ Split Spoon
- ⧅ Shelby Tube
- ▥ Cuttings
- ◫ Core Run
- * No Sample Recovery
- P Tube Pushed, Not Driven

TEST PIT SAMPLES

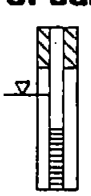
- ⊠ Grab (Jar)
- ⧄ Bag
- ⧅ Shelby Tube

Test Symbols

GS	Grain Size Classification
CN	Consolidation
TUU	Triaxial Unconsolidated Undrained
TCU	Triaxial Consolidated Undrained
TCD	Triaxial Consolidated Drained
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits

Water Content in Percent
Liquid Limit
Natural
Plastic Limit

Ground Water Observations



Surface Seal

Ground Water Level on Date (ATD) At Time of Drilling

Observation Well Tip or Slotted Section

Ground Water Seepage (Test Pits)

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Figure A-1

Boring Log HC-1

Soil Descriptions

Ground Surface Elevation in Feet 162.0

Depth in Feet

0 — 2 to 3 inches of asphalt over medium dense, moist, brown, slightly silty, slightly gravelly SAND. (FILL)

5 — Grading wet, clayey, with wood fragments.

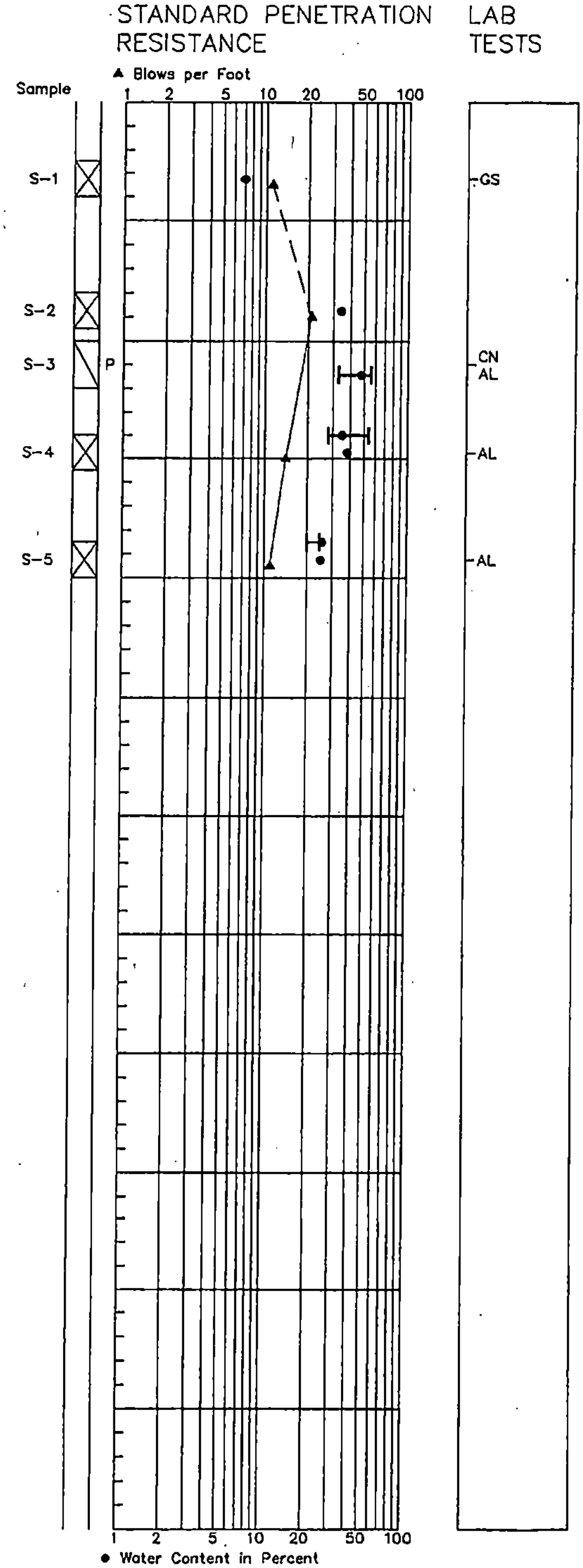
ATD

Very stiff, wet, brown, very silty CLAY.

15 — Grading stiff.

Stiff, moist, brown, slightly clayey SILT with interbedded laminations of sand.

20 — Bottom of Boring at 20.0 Feet. Completed 11/5/90.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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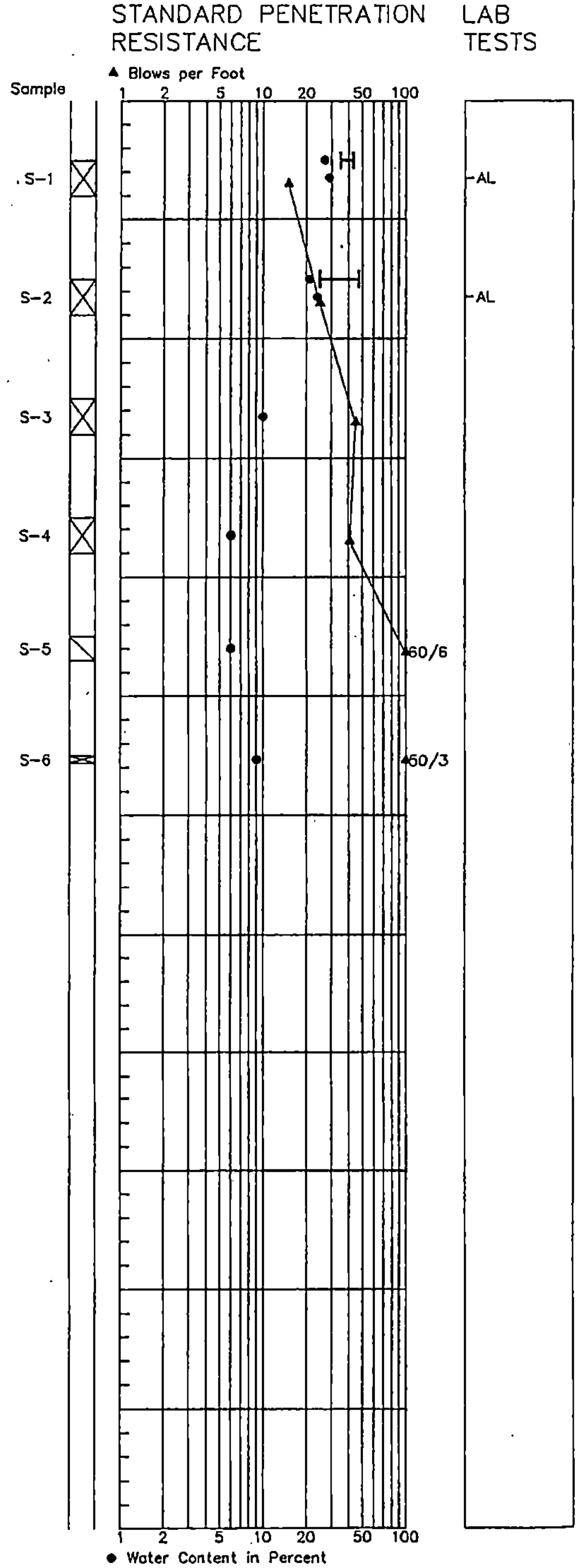
Figure A-2 1/1

Boring Log HC-2

Soil Descriptions

Ground Surface Elevation in Feet 163.0

Soil Descriptions	Depth in Feet
2 to 3 inches of asphalt over medium dense, moist, brown, very silty, gravelly SAND with trace cobbles. (FILL)	0
Stiff, moist, blue-gray, very clayey SILT.	
Very stiff, moist, brown, slightly sandy, very silty CLAY.	5
Dense, moist, brown, silty SAND.	10
Grading slightly silty.	15
Grading very gravelly and very dense.	20
Bottom of Boring at 27.9 Feet. Completed 11/5/90.	25



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-3 1/1

Boring Log HC-3

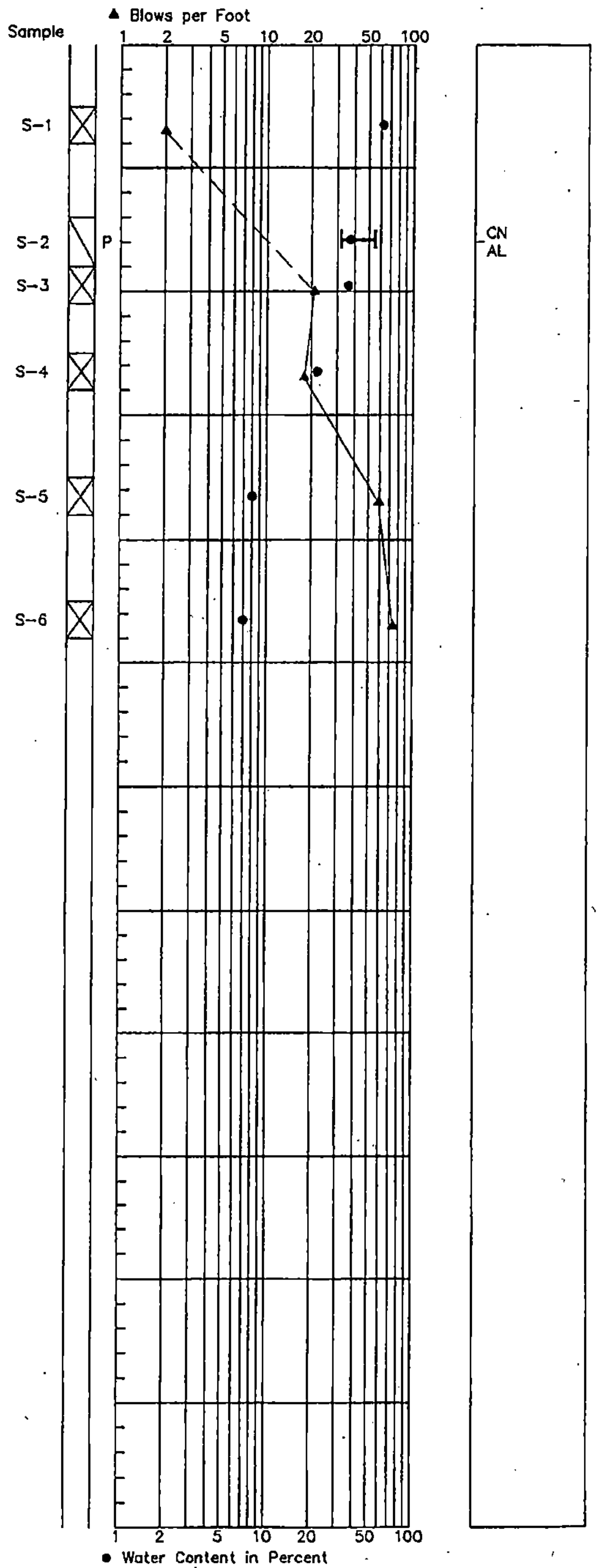
Soil Descriptions

Ground Surface Elevation in Feet 165.5

Soil Descriptions	Depth in Feet
2 to 3 inches of asphalt over medium dense, moist, brown, silty, gravelly SAND. (FILL)	0
Very soft, moist, brown, sandy SILT. (FILL)	
Very stiff, moist, brown, slightly sandy, silty CLAY.	5 – 10
Medium dense, moist, brown, silty SAND.	15
Grading gravelly and slightly silty.	
Grading very dense.	20
Bottom of Boring at 24.0 Feet. Completed 11/5/90.	25

STANDARD PENETRATION RESISTANCE

LAB TESTS



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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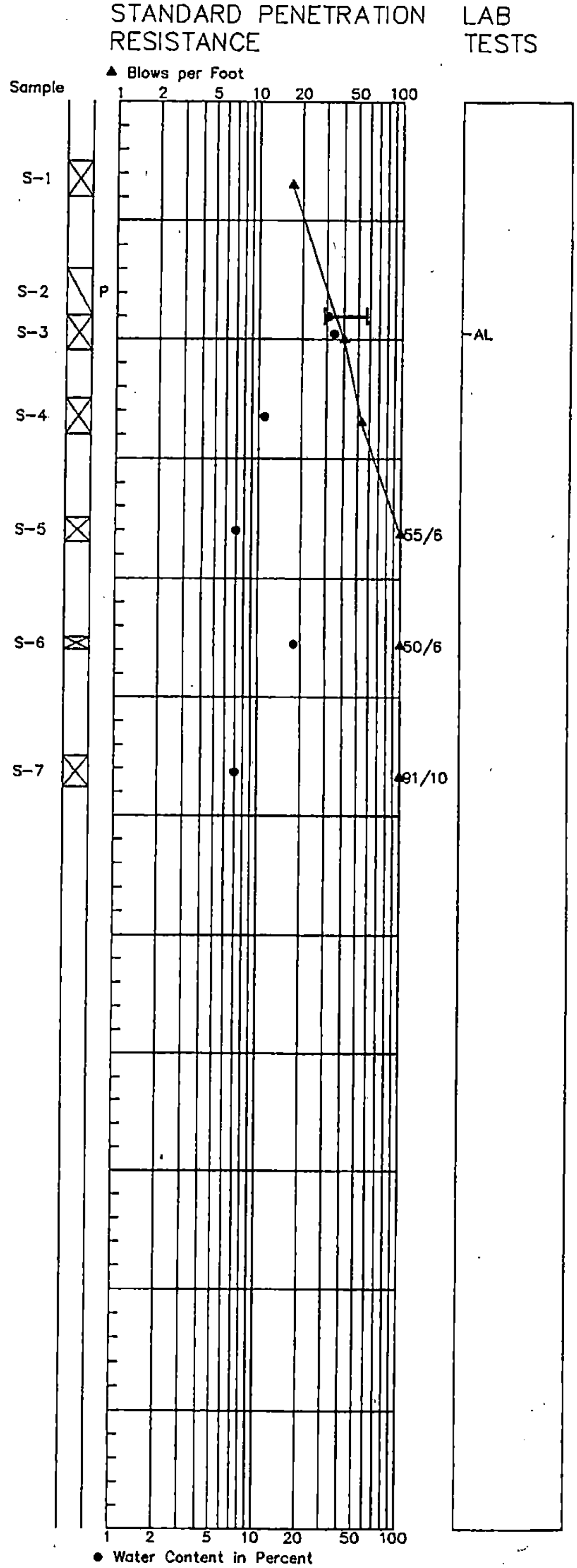
Figure A-4 1/1

Boring Log HC-4

Soil Descriptions

Ground Surface Elevation in Feet 165.0

Depth in Feet	Soil Descriptions
0	2 to 3 inches of asphalt over medium dense, moist, brown, very silty, gravelly SAND. (FILL)
~6	Hard, damp, brown, very silty CLAY.
~13.5	Very dense, moist, brown, slightly silty SAND.
	Grading gravelly.
	Grading silty.
	Grading slightly silty and slightly gravelly.
28.9	Bottom of Boring at 28.9 Feet. Completed 11/5/90.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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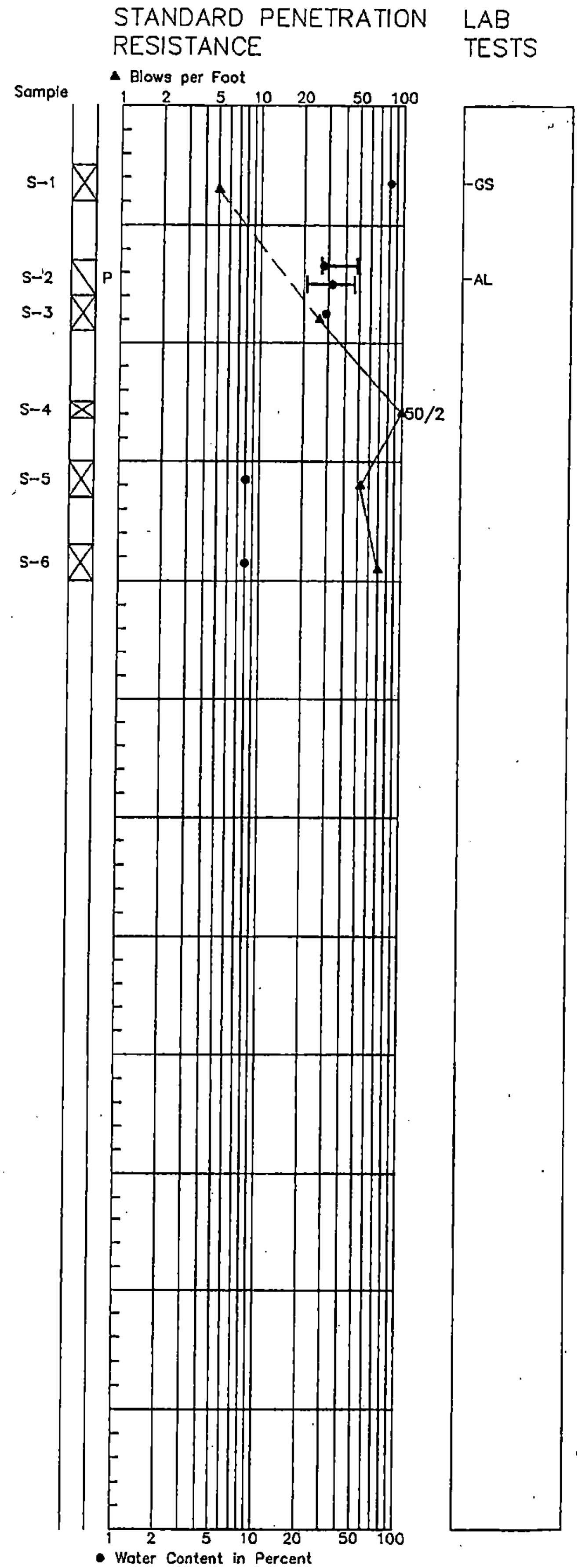
J-3149 11/90
Figure A-5 1/1

Boring Log HC-5

Soil Descriptions

Ground Surface Elevation in Feet 166.0

Soil Description	Depth in Feet
2 to 3 inches of asphalt over dense, damp, brown, very silty, gravelly SAND. (FILL)	0
Medium stiff, moist, dark brown, slightly sandy SILT. (FILL)	
Very stiff, moist, brown, slightly sandy, silty CLAY.	5
Very dense, moist, brown, silty, gravelly SAND.	10
Bottom of Boring at 20.0 Feet. Completed 11/6/90.	20



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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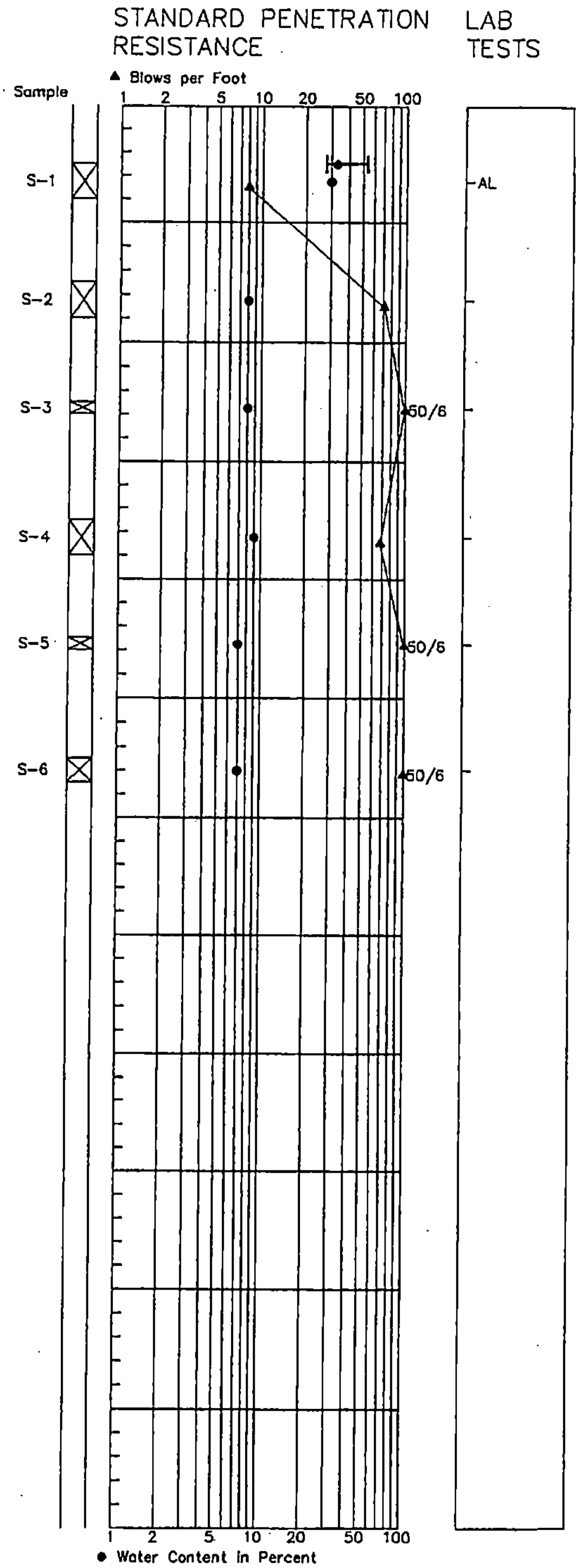
Figure A-6 1/1

Boring Log HC-6

Soil Descriptions

Ground Surface Elevation in Feet 168.0

Soil Descriptions	Depth in Feet
2 to 3 inches of asphalt over medium dense, moist, brown, silty, gravelly SAND. (FILL)	0
Medium stiff, moist, brown, sandy SILT.	
Grading gravel.	5
Very dense, moist, brown, silty, gravelly SAND.	
Grading slightly gravelly.	10
Grading slightly silty.	15–20
Grading gravelly.	20–25
Bottom of Boring at 28.5 Feet. Completed 11/6/90.	28.5



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-7 1/1

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

A laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils. Both disturbed and relatively undisturbed samples were tested. The tests performed and the procedures followed are outlined below.

Soil Classification

Field Observation and Laboratory Analysis. Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include density/consistency, moisture condition, and grain size and plasticity estimates.

The classifications of selected samples were checked by laboratory tests such as Atterberg limits determinations and grain size analyses. Classifications were made in general accordance with the Unified Soil Classification (USC) System, ASTM D 2487, as presented on Figure B-1.

Water Content Determinations

As soon as possible following their arrival in our laboratory, water contents were determined for most samples recovered in the explorations in general accordance with ASTM D 2216. The results of these tests are plotted at the respective sample depth on the exploration logs. In addition, water contents are routinely determined for samples subjected to other testing. These are also presented on the exploration logs.

Atterberg Limits (AL)

We determined Atterberg limits for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D 4318-84. The results of the Atterberg limits analyses and the plasticity characteristics are summarized in the Liquid and Plastic Limits Test Report, Figures B-2 and B-3. This relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit. The results of the Atterberg limits tests are shown graphically on the boring logs.

Grain Size Analysis (GS)

Grain size distribution was analyzed on representative samples in general accordance with ASTM D 422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figure B-4 plotting percent finer by weight versus grain size.

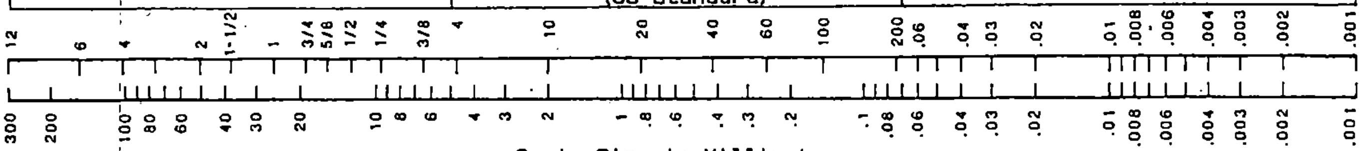
Consolidation Test (CN)

The one-dimensional consolidation test provides data for estimating settlement. The tests were performed in general accordance with ASTM D 2435. Relatively undisturbed, fine-grained samples were carefully trimmed and fit into a rigid ring with porous stones placed on the top and bottom of the sample to allow drainage. Vertical loads were then applied incrementally to the sample in such a way that the sample was allowed to consolidate under each load increment. Measurements were made of the compression of the sample (with time) under each load increment. Rebound was measured during the unloading phase. In general, each load was left in place until the completion of 100 percent primary consolidation, as computed using Taylor's square root of time method. The next load increment was applied soon after attaining 100 percent primary consolidation. The test results plotted in terms of axial strain and coefficient of consolidation versus applied load (stress) are presented on Figures B-5 and B-6.

Unified Soil Classification (USC) System

Soil Grain Size

Size of Opening in Inches	Number of Mesh per Inch (US Standard)	Grain Size in Millimetres



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines		* GRAVEL with >12% fines		Clean SAND <5% fines		* SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \le \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \le 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A Line with PI <4

G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

D_{10}, D_{30}, and D_{60} are the particle diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						

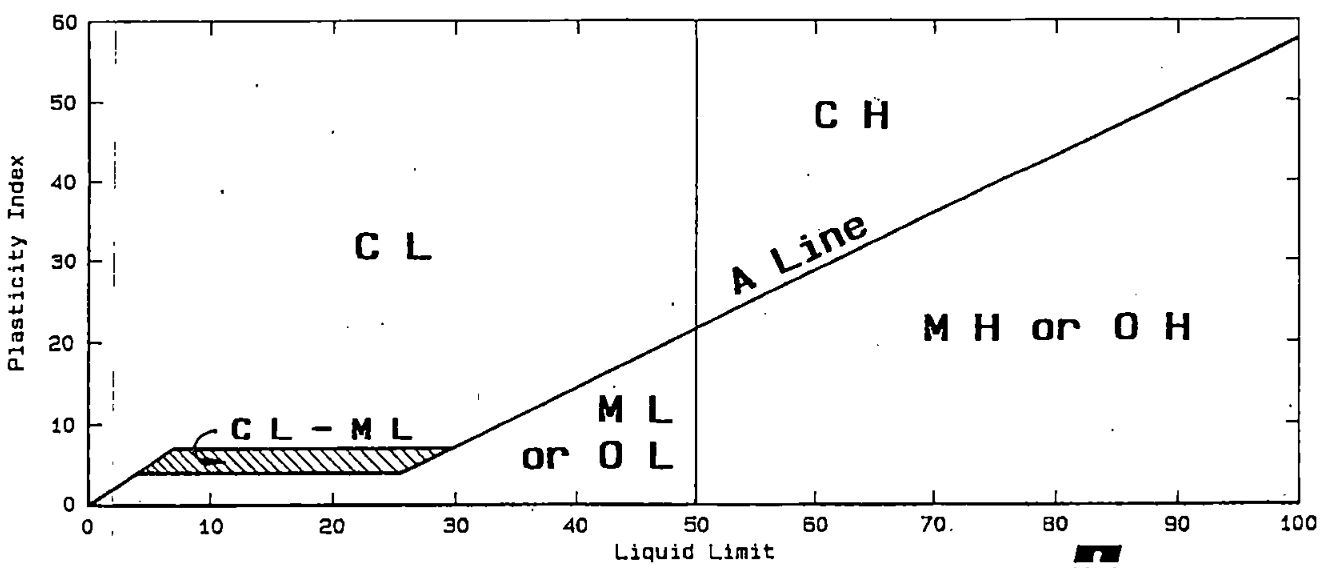


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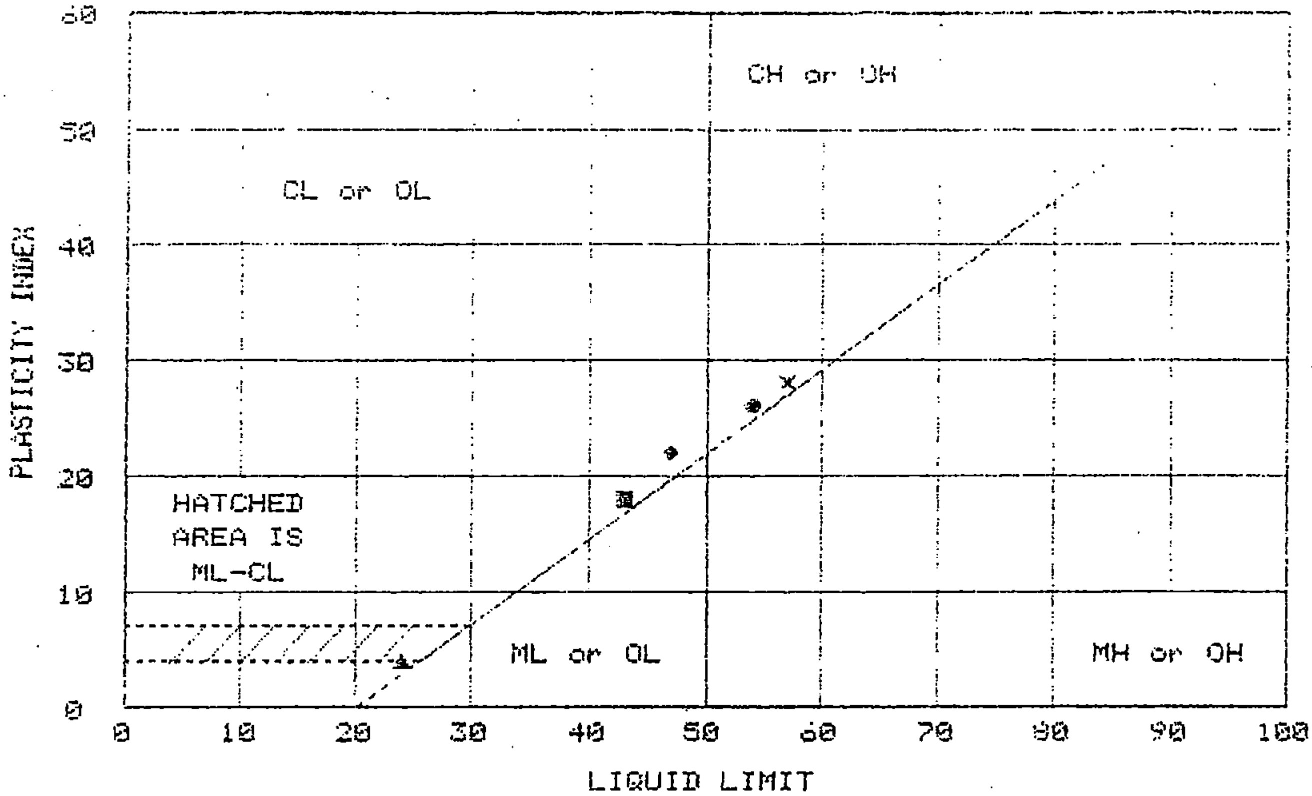
Figure B-1

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-85
●	HC-1 S-4 DEPTH 14' NATURAL W.C.= 35%	54	28	26		CH, Fat clay
▲	HC-1 S-5 DEPTH 18.5' NATURAL W.C.= 25%	24	20	4		CL-ML, Silty clay
▣	HC-2 S-1 DEPTH 2.5' NATURAL W.C.= 27%	43	25	18		CL, Lean clay
◆	HC-2 S-2 DEPTH 7.5' NATURAL W.C.= 21%	47	25	22		CL, Lean clay
X	HC-4 S-3 DEPTH 9.0' NATURAL W.C.= 31%	57	29	28		CH, Fat clay

Remarks:

Project: NEW FACILITIES

Client: SNOHOMISH COUNTY P.U.D.

Location: SNOHOMISH COUNTY WA.

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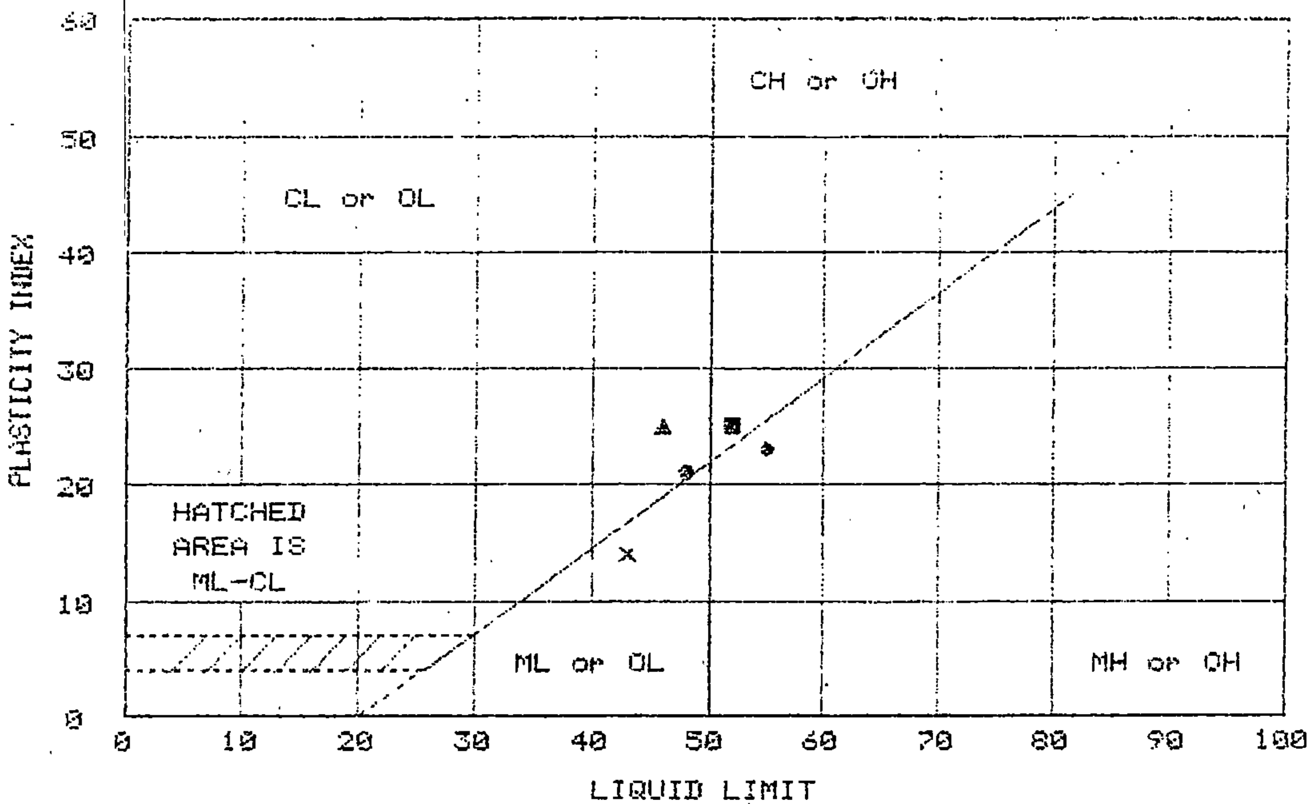
J-3149 11/18/92

Figure B-2

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-85
●	HC-5 S-2 DEPTH 6.6'-6.8' NATURAL W.C.= 28%	48	27	21		CL, Lean clay
▲	HC-5 S-2 DEPTH 7.5' NATURAL W.C.= 32%	46	21	25		CL, Lean clay
■	HC-6 S-1 DEPTH 2.5' NATURAL W.C.= 33%	52	27	25		CH, Fat clay
◆	HC-1 S-3 DEPTH 11.3'-11.5' NATURAL W.C.= 37%	55	32	23		MH, Elastic silt
x	HC-3 S-2 DEPTH 7.8'-8.0' NATURAL W.C.= 27%	43	29	14		ML, Silt

Remarks:

CONSOLIDATION TESTS ON

HC-1,S-3 AND HC-3, S-2

Project: NEW FACILITIES

Client: SNOHOMISH COUNTY P.U.D.

Location: SNOHOMISH COUNTY WA.

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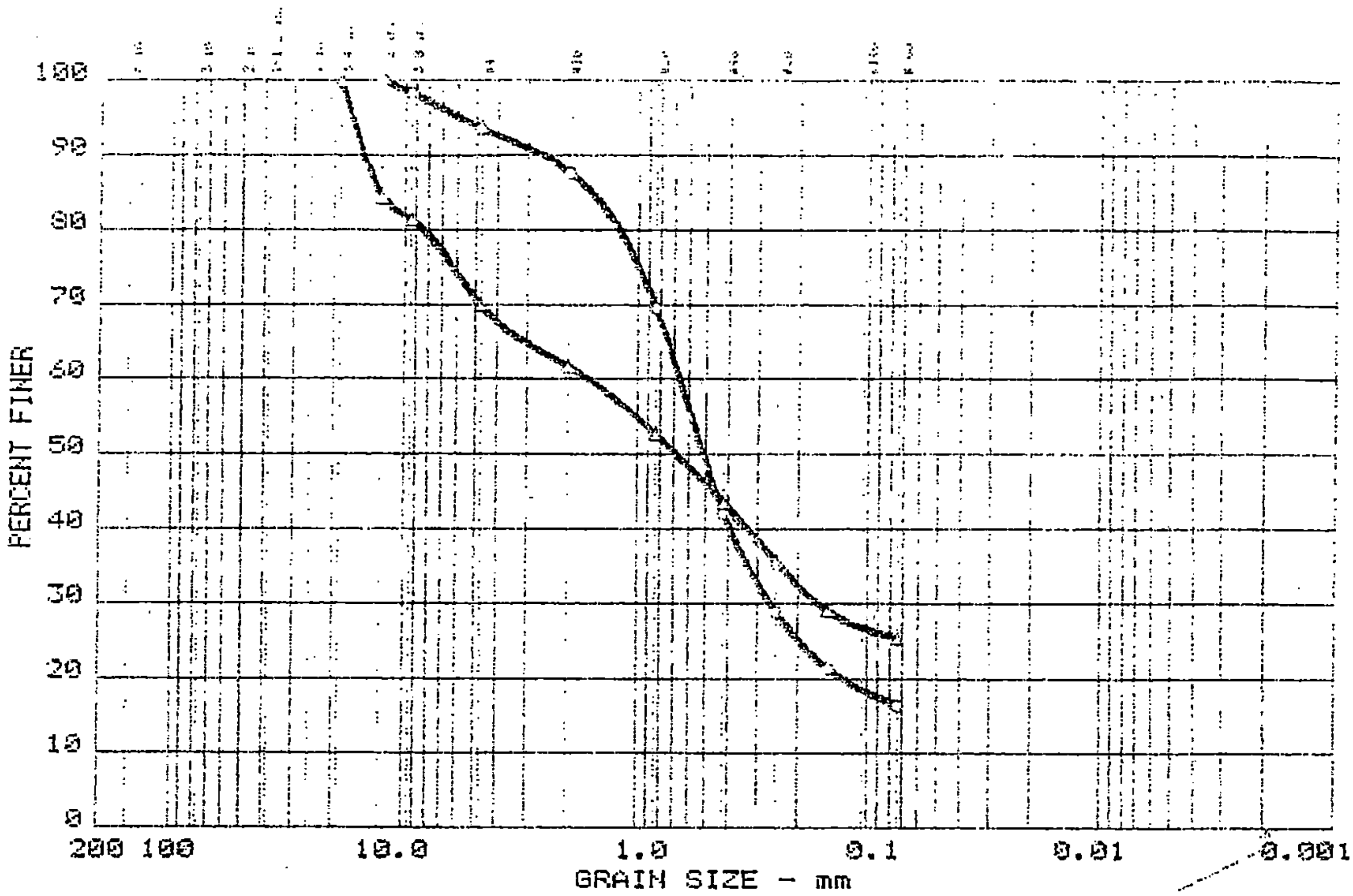
Figure B-3

GRAIN SIZE DISTRIBUTION TEST REPORT



	%+75mm	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	6.4	77.1	16.5	
△	0.0	30.0	44.5	25.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			1.62	0.66	0.52	0.264				
△			13.03	1.64	0.67	0.162				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Slightly gravelly,silty,SAND	SM	8%
△ Silty,very gravelly,SAND	SM	19%

Remarks:

Project: SNOPUD
○ Location: HC-1,S-1 DEPTH 2.5'
△ Location: HC-4,S-1 DEPTH 2.5'

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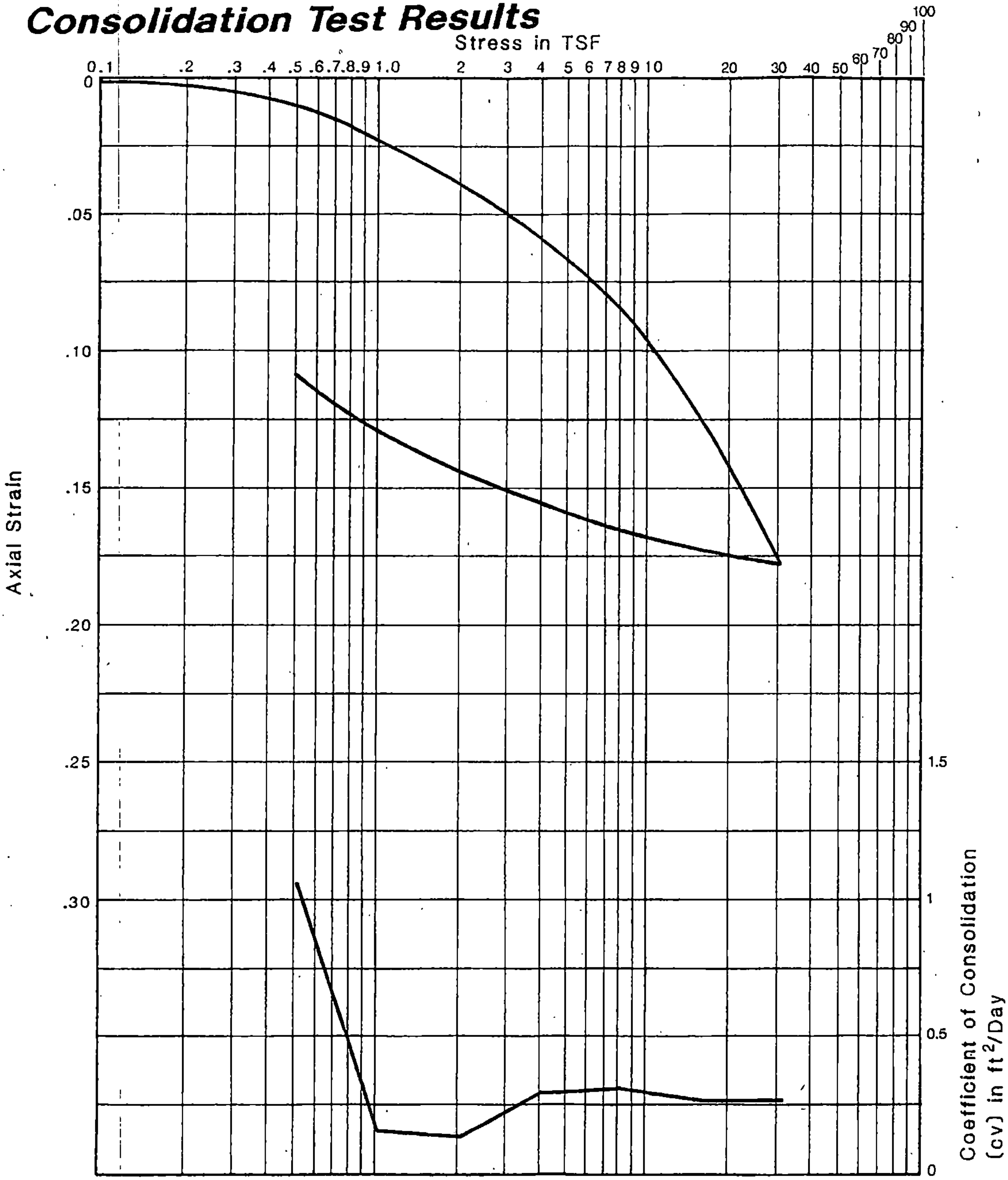
Figure B-4

Consolidation Test Results



SYMBOL	BORING NUMBER	SAMPLE NUMBER	DEPTH IN FEET	WATER CONTENT IN % NAT.	AFTER	LL	PL	PI	CLASSIFICATION	WET UNIT WEIGHT IN PCF
	HC-1	S-3	11.5 11.7	36.8	33.1	55	32	23	MH	113.9

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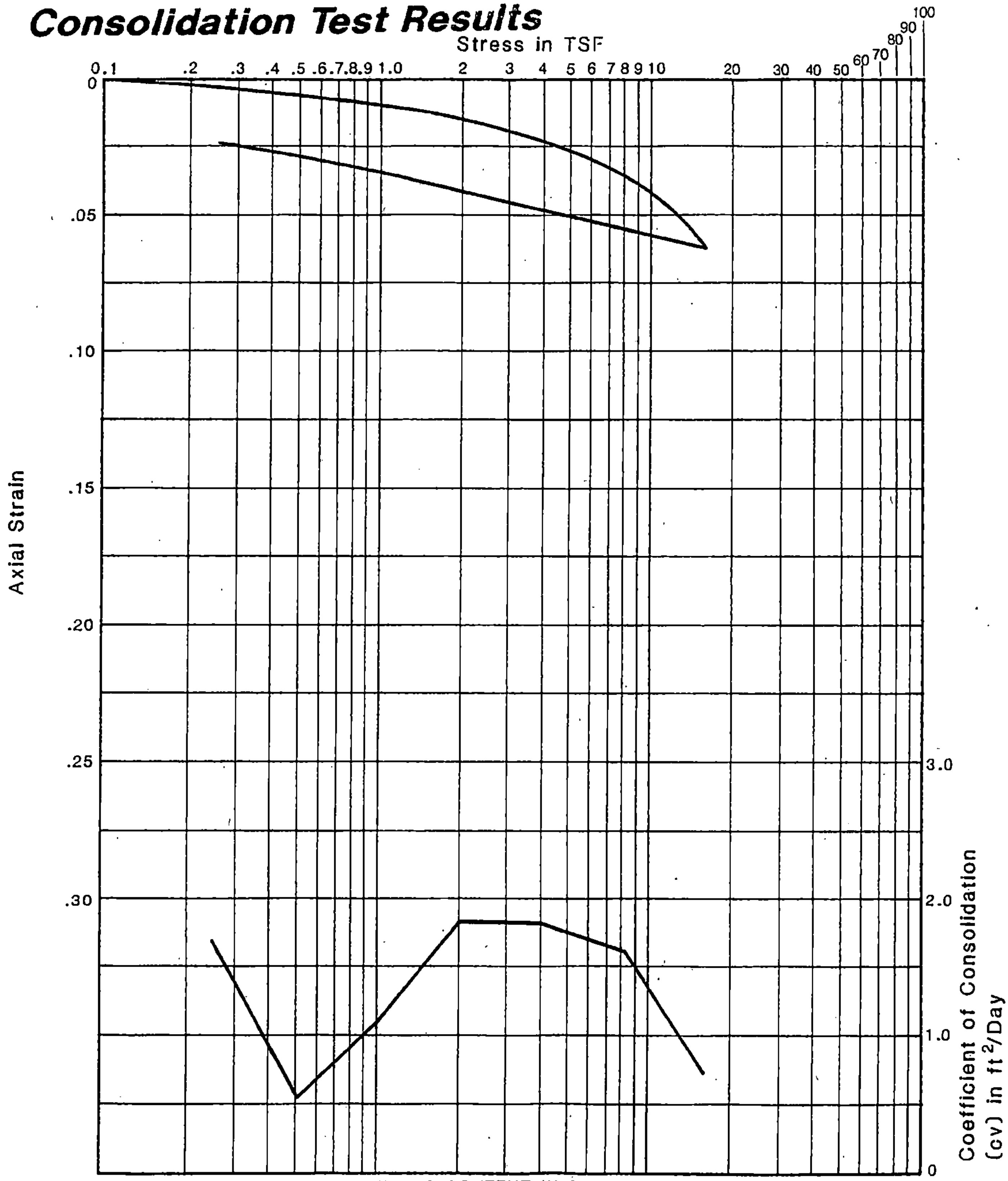
Figure B-5

Consolidation Test Results



SYMBOL	BORING NUMBER	SAMPLE NUMBER	DEPTH IN FEET	WATER CONTENT IN % NAT.	AFTER	LL	PL	PI	CLASSIFICATION	WET UNIT WEIGHT IN PCF
———	HC-3	S-2	8.0-8.2	28.5	28.6	43	29	14	ML	123.5

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Figure B-6