HARTCROWSER

Earth and Environmental Technologies

Geotechnical Engineering Design Report
Providence Hospital — East Tower
Seattle, Washington

Prepared for
NBBJ and
Providence Medical Center

February 5, 1988
J-2071

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INTRODUCTION

This report presents the results of our subsurface explorations and geotechnical engineering study for the proposed east tower addition to the Providence Medical Center located at the corner of E. Cherry Street and 18th Avenue in Seattle, Washington. The purpose of this study was to assess subsurface site conditions, assist the structural engineering in establishing foundation design criteria, and to provide geotechnical recommendations related to design and construction.

The scope of the field explorations for this study consisted of three hollow-stem auger borings drilled to depths ranging from 60 to 70 feet below the street level. Following completion of the field explorations, laboratory tests consisting of visual classification, water contents, and grain size analysis were performed on samples retrieved from the borings to aid in classification of the site soils and to establish the geotechnical index and general engineering properties of the materials. Engineering studies and analyses were then undertaken to develop recommendations for design and construction.

The exploration procedures are discussed and the exploration logs are presented in Appendix A. The locations of the explorations are shown on the Site and Exploration Plan, Figure 1. In Appendix B are presented a discussion and results of all laboratory tests completed for this study. Subsurface conditions interpreted from the explorations and the soil properties inferred from field and laboratory tests formed the basis for the engineering studies.

In addition to the work completed for this scope our studies utilized the results of previous work at the Providence Hospital site completed by Hart Crowser, Inc. for the linear accelerator/physicians office building in 1983 and by Shannon and Wilson for an earlier addition to the hospital completed in 1963. A single boring from the 1963 study, DH-1, has been reproduced in this report and is shown in Appendix A and located on Figure 1. The seismic response study completed in the 1983 study by Hart Crowser has been referenced in this report.

This study has been performed in general accordance with our proposal dated October 5, 1987. This report has been prepared for the exclusive use of Providence Medical Center and their design consultants for specific application to the subject project and site. This study has been performed in accordance with generally accepted geotechnical practices. No other warranty, express or implied, is made.

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

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

- Perimeter site grades range from about elevation 346 feet at the northwest corner to elevation 353 feet at the northeast corner to elevation 343 feet at the southeast corner. The existing structure has a basement slab elevation assumed to be 338 feet. The slab elevations and foundation elevations of the surrounding buildings are not known at the time of writing.

- The borings disclosed somewhat variable soil conditions across the project site. Borings B-2 and DH-1 encountered about 30 feet of glacial till overlying silty, fine sand. Boring B-3 had a small cap of recent fill overlying till to the bottom of the boring. Boring B-1 encountered silty, fine sand through the entire drilled length. These results suggest that a till cap overlies the project site and dips to the east and south. All materials encountered except the near-surface fill were very dense and assumed to be heavily overconsolidated.

- Groundwater was encountered in each of the borings at an elevation of about 300 to 305 feet.

- Most of the area can be open cut to the elevations planned. Based on an assumed excavation level across most of the site of 320 feet, we expect cut slopes to average between 3/4H:1V and 1:1. Erosion protection of the exposed slopes will be required.

- Along the east side, the location of the new building with respect to the property line may require installation of shoring. In this area we would expect such shoring to be designed as a cantilever or single support system. Recommendations for design are given in this report.

- In the southwest portion, near the existing building an elevator bank will be installed. This will require excavations to a lower level, on the order of 300 to 305 feet. Shoring with multiple supports may be necessary in this area, and underpinning or consideration of the surcharge loads from adjacent footings may also be required. The general design of the shoring is given in this report. Specific discussion of underpinning techniques and design are not provided herein, as the required information on the existing building is not yet available.

- Foundation support may consist of spread footings such as isolated column footings or strip footings founded on the dense natural soils. The maximum allowable bearing pressure is 10 kips per square foot. Foundation settlements are estimated to be on the order of 1 inch or less.

- The presence of groundwater at an elevation approximately equal to the excavation level in the southwest corner could require that local dewatering be accomplished. The permeability of the materials encountered at that depth is estimated to be about 10^{-3} cm/sec. Installation of well points or sumps could be necessary to provide for a dry excavation to permit construction of footings at that level.

- o A permanent subslab and behind the wall drainage system will be required for the structure. This system can consist of a network of underslab perimeter and cross drains with sheet drain installed behind the walls and tied to the underslab system.

- o An existing well located in the southwest portion of the site is intended to be maintained and saved throughout construction and after completion of the project. No information on the design of that well has been available to Hart Crowser, and therefore no recommendations for the continued use of the well can be made at this time.

- o The characteristic site period and smooth dual level response spectra were developed for the linear accelerator project and are referenced for use in the seismic design of this project.

SITE AND PROJECT DESCRIPTION

The project site, which is approximately 200 by 160 feet in plan dimension, is bordered by E. Cherry Street to the north, 18th Avenue to the east, the existing hospital building to the south, and the existing surgery addition to the west. The proposed east tower will consist of an 8-story structure (only 6 stories being constructed at this time) with below grade levels extending generally to elevation 324 feet. The lowest level of the east tower will be elevation 306 feet, and will be commensurate with the lowest level of the surgery addition to the west. The lower level limits of the project are generally 25 to 35 feet from the property line on the north side and range from 10 to about 22 feet from the property line on the east side.

The existing east tower which covers much of the project area consists of a 6-story brick building with one underground basement level. The slab level of that existing structure is apparently elevation 338 feet. Street grades decrease from a high at about elevation 354 feet at the corner of E. Cherry and 18th to grades 5 to 7 feet lower to the west and south adjacent to the east tower location. An interior court yard between the surgery addition and the existing east tower is at an elevation of approximately 338 feet, the basement level of the existing structure.

Structural loads on the proposed facility are not yet known. In addition, the footing elevations and locations of the adjacent existing buildings have not been available to Hart Crowser. As a result specific consideration of the effects of the existing buildings on the new construction are not addressed in this report. When the required information becomes available Hart Crowser will issue a supplemental design letter to address these considerations.

SUBSURFACE CONDITIONS

Subsurface conditions across the site have been evaluated on the basis of the three borings completed for this study together with the boring DH-1 completed in the 1963 study. The borings disclosed three distinct soil types.

The uppermost soil consisted of recently placed fill. This material was encountered only in B-3 and was in a loose condition. The fill was generally granular, but included a high percentage of fine-grained soil.

The second major soil type is glacial till. This material was encountered to depths of 30 feet in borings B-2 and DH-1, and was located below the fill to depths of at least 60 feet in boring B-3. In all three locations the till was very dense, with blow counts commonly greater than 100. The material was cemented with variable contents of silt, sand, and gravel. Generally the fines content was about 30 to 40 percent.

The third soil encountered in the borings was classified as slightly silty to silty, fine to medium sand. The sand was located below the till to the bottom of the borings in B-2 and DH-1, and was encountered exclusively in boring B-1. The sand is also very dense, with blow counts generally ranging from 70 to more than 100. The fines content within the sand was generally less than 20 percent, and in many cases appeared to be less than 5 percent.

Groundwater was encountered at elevations ranging from 299 feet to 305 feet in the borings. An observation well located in boring B-1 identified the groundwater level at elevation 299 feet. At the other locations the groundwater level was observed at the time of drilling. In borings B-2, B-1, and DH-1 the water was clearly within the sand unit. However, in B-3 water was also noted at this consistent elevation during drilling, even though the drilling was within glacial till at that elevation.

In general, the encountered soils will provide good support for the proposed development. Although both the till and sand were encountered in a very dense condition, the till will tend to exhibit more stability on cut slopes because it is cemented. The sand will have a tendency to ravel and erode if the slopes are cut too steep and if the material is exposed to moisture. The extent of the fill materials across the site is unknown, as it was encountered only in boring B-3. It is likely that some filling has occurred adjacent to the existing buildings. Demolition of these buildings will probably reveal more fill than suggested by the exploration results. We would expect the cut slopes in fill to require some protection and may need to be flatter than cut slopes in the natural soils. The materials expected at the foundation level should possess relatively high strength and have low compressibility characteristics.

In the areas where deeper excavations and lower footings are required, groundwater could be encountered. The water may have a tendency to loosen or disturb the soils at the base of the excavation. It will be necessary to complete the excavation and subgrade preparation in such a way that a firm surface can be maintained for support of the footings. This may require localized dewatering and restriction of passage of construction equipment.

It should be noted that the nature and extent of subsurface variations between the borings may not become evident until construction. Should significant variations appear evident, it would be necessary to reevaluate the recommendations of this report.

GEOTECHNICAL ENGINEERING RECOMMENDATIONS

The recommendations and considerations presented in this report are based on the data obtained from the field explorations accomplished and previous studies. In addition, our recommendations are sensitive to the project description and assumptions outlined in various sections of the report. Should these design criteria change prior to construction it would be necessary for Hart Crowser to review and reevaluate the recommendations of the report.

Excavation Consideration

The area of the new facility is currently occupied by a brick structure with one basement level extending to about elevation 338 feet. The limits of the building extend to within about 10 feet of the property line on the north side and about 15 feet on the east side. Demolition of this structure and removal of the construction debris will be required prior to construction of the proposed building. At the time of writing we do not know if the limits for excavation and construction of the new facility are defined by the existing property line, or by the street. The report addresses various excavation support alternatives on the assumption that the excavation can not extend beyond the property line. If the additional distance between the property line and street line can be utilized for excavation, some of the recommendations will need to be modified.

It is our recommendation that the perimeter walls of the existing facility be completely removed in conjunction with the general site excavation. If, for example, the existing exterior walls are left in place without the internal support, they may be overstressed and fail in an uncontrolled manner. If it is desired to leave these walls in place without support, the structural engineer should evaluate the conditions to determine that an adequate factor of safety exist for either overturning or sliding.

It is strongly recommended that Hart Crowser meet with the structural engineer and the contractor to identify the stability conditions as a function of the excavation sequence.

The general excavation to the foundation level for the new facility should occur predominantly in glacial till soil on the southeast half of the site with increasing thicknesses of sand encountered to the northwest. In the northwest corner, the excavation should encounter nothing but the silty sand described in the SUBSURFACE CONDITIONS section of this report. We expect the excavation levels to extend some three to four feet below the basement slab elevation of 324 feet across most of the site, and three to four feet below the elevator bank slab elevation of 306 feet in the extreme southwest corner.

Excavation in the glacial till will likely require use of conventional heavy equipment such as large bulldozers and front end loaders. Due to the very dense nature and partial cementing of the till, ripping will probably be required for much of the excavation in that material. Cobbles and/or boulders are common in this soil and are expected to be scattered throughout the excavation. The sandy soils can probably be excavated with less effort. Although this material is also very dense, it lacks the cementing which bonds the till soil. As a result cuts into the silty sand can be made more easily. The silty sand is expected to be in a generally moist to wet condition, as indicated by the borings. However, as the excavation deepens and approaches the groundwater level, the moisture content of the silty sand will increase and the material may be more difficult to handle. Within the till, zones of seepage are frequently encountered, but are typically isolated.

Excavation of on-site soils required to reach the basement level of the proposed structure can be completed as open cut wherever sufficient room allows this construction technique. The excavation slope and support requirements will be generally addressed in the context of three distinct areas of the site.

-North Building Wall

Along the north side of the building the planned excavation level of about 320 feet is located some 20 feet from the existing basement wall. The depth to which the wall footings extend is unknown, but the old basement slab level is elevation 338 feet. Ten feet away from the existing basement wall is the property line. The street line lies about 15 feet north of the property line. Along this side of the project, glacial till should be encountered to about elevation 325 feet at the east side and to about 345 feet at the western one-third point of the wall. Below the till is the dense silty sand unit. It appears that sufficient clearance exists along this area for the entire cut to be made open cut.

By our estimation, slopes of about 3/4H:1V to 1:1 would be required in the sand and 1/2H:1V within the till. We believe the 1/2:1 slopes in the till soils are obtainable and maintainable without significant slope protection. It would be necessary to protect the slopes from precipitation and surface runoff. This could probably be accomplished using plastic sheeting. In addition if gravel or cobbles are encountered along the cut face, and appear loose or susceptible to spalling, some additional protection such as wire mesh or chain link fencing over the area may be necessary.

In the sand, the steepness of the slope which can be safely attained depends on several factors. These factors include the moisture conditions of the material, the presence of seepage zones, the percentage of fine-grain soil, the grain size of the sand itself, and several other factors. It is possible therefore only to estimate the steepness of the slopes which can be maintained through the sand. For planning purposes, we would expect slopes with steepness of about 1:1 to require protection only as given for till slopes, mainly, plastic sheeting to protect against the change in moisture conditions. If the slopes are steeper than 1:1 and approach what we consider to be the limit of about 3/4H:1V, it is recommended that they be protected with a wire mesh and shotcrete facing. While a wire mesh/shotcrete protection does not add significant strength to the slope, it does reduce the likelihood that minor erosion and sloughing would lead to a progressive larger scale problem.

-East Side

Along the east side the location of the new wall is close to the location of the existing wall. The ground surface elevation along that wall averages about 350 feet. Therefore cuts on the order of 25 to 30 feet will be necessary to construct that wall. The wall location is within 10 feet of the property line on the northern half of the wall and 22 feet of the property line on the southern half of the wall. The street is located some 20 feet east of the property line. As a result, if the excavation can extend all the way to the street line, or close to it, an open cut can be made through this area. If, however, the excavation can only extend to the property line, some form of shoring will probably be required. Even if shoring is necessary, it would be possible to limit the height by completing an open cut in the upper portions of the excavation. It is likely that the shoring wall along this side could be either cantilevered or require only a single level of internal or external support.

Soil conditions across this side of the property are expected to consist of fill near the ground surface, possibly extending as deep as 10 feet, with till located below. Again, cuts made in glacial till could be as steep as 1/2H:1V with proper protection. Cuts in the fill would need to be flatter, and for planning purposes may be estimated to be about 1:1. Depending on the nature of the fill soils that are exposed upon initial excavation, wire mesh and shotcrete slope protection could be required.

The specific recommendations for design of the shoring system discussed here are presented in subsequent sections of the report.

The decision on whether an open cut excavation, shoring support, or combination of the two will be used depends on a number of factors. These include the relative cost of each alternative, the construction time frame, the demolition plans for the existing buildings, and the area available for excavation. The decision on whether or not to use shoring should be one which is arrived at through consultation with the geotechnical and structural engineers, and the contractor. This consultation is particularly important in those portions of the site where no alternative is clearly preferable to another.

-Southwest Corner

In the area of the southwest corner where the excavation will extend to about elevation 300 to 305 feet, it appears that some shoring will be necessary. The depth of the required excavation support could necessitate installation of a multiple braced or supported system. Recommendations for design and support of this system are given in the Temporary Shoring section of this report. The soils in this corner will consist of glacial till over the silty sand. A design factor which could have a significant effect on the shoring system is the location, depth, and loads of existing footings for the adjacent structures. If underpinning is required, Hart Crowser will address potential alternatives and design considerations in a future letter. If the footings are so close to the shoring wall that surcharge loads will be applied to the wall, the magnitude of those loads can be estimated using the information in this report, although it is recommended that Hart Crowser provide consultation with the structural engineer to review the design approach and results.

Because of the many variables involved, the actual limiting steepness to maintain stability of temporary cut slopes can only be approximately estimated prior to construction. Actual temporary sloping in soils should be made the responsibility of the contractor since he is continuously present at the job site to observe the nature and conditions of the subsurface materials encountered, including groundwater.

In general, it is recommended that surcharge loads such as those occurring from traffic, construction equipment, or stockpiled materials, be maintained a minimum of 4 feet behind the top of any till or natural sand soil slopes, and 6 feet behind the top of any cuts in existing fill. Surcharges should also be kept behind any shoring walls, a distance equal to the depth of cut, unless the surcharges are included as lateral loads in the shoring wall design.

Temporary Shoring

-General Considerations

In those areas where open cutting is not feasible, a temporary shoring wall will be required. Several types of shoring would be appropriate for this site, but we expect a conventional shoring system consisting of a soldier pile wall to be most likely. This report discusses considerations for such a wall. The design criteria given in the report may, however, be applied to other wall types.

The shoring recommendations presented in this and subsequent sections are intended to be applied to the design of an appropriate system. The recommendations should be used by the structural engineer and shoring subcontractor in conjunction with the other information in this report during the design and construction of the shoring. It is generally not the purpose of this report to provide specific criteria for construction methods, materials, or procedures. It should be the responsibility of the shoring subcontractor to verify actual ground conditions of the site and determine the construction methods and procedures needed for installation of an appropriate shoring system.

Soldier pile walls in this area typically consist of some type of steel beam installed in a predrilled vertical hole. Pressures of the soil are resisted through cantilever action of the soldier pile or by internal or external anchors or supports. Based on the anticipated wall heights, it is possible that either a cantilever, single support, or multiple support wall or series of walls will be appropriate.

-Lateral Pressures

Design of temporary shoring could be based on either active or at-rest lateral earth pressures depending on the degree of deformation of the shoring which can be tolerated. Shoring which is free to deform at the top on the order of 0.001 to 0.002 times the height of the shoring is considered to be capable of mobilizing active earth pressures. This lateral deformation is likely to be accompanied by vertical settlement at the top of the shored face up to roughly 0.005 times the height of the shoring, which gradually reduces to zero over a horizontal distance equal to roughly the height of the cut (i.e., within a zone defined by an imaginary plane extending up and back from the base of the shored cut at roughly 1:1). A greater amount of lateral deformation could allow greater vertical settlement.

If no structural elements, such as existing footings, are located within this zone or if the structural elements within this zone are not considered to be sensitive to this degree of settlement, then it would be appropriate to design using active earth pressures. If, on the other hand, structural elements or other facilities such as utilities are located within this zone it would be more appropriate to design for an at-rest earth pressure conditions. Adjacent to streets, it is generally accepted that active design criteria can be employed.

The distribution of lateral earth pressure for the various shoring walls expected at this site are depicted on Figures 2 and 3 and discussed in the following paragraphs. The expected conditions along the east wall are represented on Figure 2. We anticipate some open cutting at an average slope of about 1:1 in conjunction with a cantilever or single support system to be used along much of that wall. As this area borders a sidewalk and street, we have assumed that active conditions are appropriate. The active pressure on the wall is represented by an equivalent fluid weight of 30 pounds per cubic foot (pcf). The open cut slope above the top of the wall is treated essentially as a surcharge, with a larger equivalent fluid weight being applied through a distance of two-thirds the slope height. The pressures on the unexcavated side of the wall extend to the bottom of the soldier pile. These pressures are resisted on the excavated side of the wall below the excavation by passive earth pressures. Those pressures are estimated using an equivalent fluid weight of 400 pcf, including a factor of safety of about 1.5. Resistance in the upper 2 feet below the base of the excavation is neglected due to potential soil disturbance.

The required depth of soldier piles is not known. However it is possible that the soldier piles could extend to depths below the groundwater table, estimated at elevation 305 feet. If the shoring extends below the water table a reduction in the equivalent fluid pressures to account for submerged unit weights must be made. Those reductions are shown on Figure 2. Together with the reduction in the pressure however is an increase in applied pressure on both sides of the wall due to hydrostatic forces. Except below the water table, the pressure shown on Figure 2 are based on an assumption of no buildup of hydrostatic pressure behind the wall. In other words, it is assumed that water which may occur above the excavation level behind the wall is free to drain through the wall.

It is recommended that the soldier piles be extended a distance of 1 to 2 feet above the bottom of the slope if an open cut exists above the wall. This stick up area should be lagged with 2-inch wood lagging. The purpose of this stick up is to prevent materials from sloughing off the open cut slope and falling into the excavation below.

In the southwest corner of the site it may be necessary to install shoring with multiple supports. The design of a multiple supported system is illustrated on Figure 3. On Figure 3 we have identified a pressure distribution for both active conditions and at-rest conditions.

The shoring pressure shown on Figure 3 assume a horizontal back slope, with no surcharge component. If either of these conditions are not prevalent, Hart Crowser should be consulted to provide additional recommendations. The design of a multiple supported system is based on the assumption that the pressures are distributed among the various levels of support. The distributions are derived from actual field instrumentation of deep excavations. The distributions represent an envelope of the maximum pressure which may potentially occur across the shoring walls. The actual average pressures across the wall would probably be less than its envelope value. The distribution assumes a simple area contribution of stress to each support and is intended to predict the maximum load that could reasonably be expected on any support for a given depth of cut at this site.

We have estimated that the appropriate design pressure for assumed active conditions is about 18 times the height of the excavation (in feet) in pounds per square foot. For at-rest conditions, the assumed pressure is 24 times the height of the excavation. The at-rest distribution is rectangular and assumes that little or no movement will occur. The active distribution assumes that some yielding will occur near the ground surface, resulting in a reduction in the applied pressures. As a result, there is a linear increase from zero to the maximum value at a depth of H/5. The distributed pressures are applied, in both the active and at-rest conditions, only over that portion of the wall above the excavation level. Those pressures are assumed to act over the entire wall.

The active or at-rest pressures are resisted by earth pressures below the ground surface as well as the internal or external support. If active conditions are assumed, then passive pressures may be used to provide resistance below the base of the excavation. The values of passive resistance are the same as shown on Figure 2. However, it is assumed that where multiple supports will be needed, the base of the excavation will be at or below elevation 305 feet, the level of the assumed groundwater table. Therefore, submerged unit weights must be used in estimating the equivalent fluid pressures available to resist wall kick-out. A value of 200 pcf would be appropriate for passive resistance below the excavation level below elevation 305 feet. If at-rest pressure conditions are assumed, the value of only 23 pcf would be available to resist the kick-out pressures. The factor of safety of 1.5 has been incorporated into these values. Again support in the upper 2 feet below the base of the excavation should be ignored.

Shoring walls should be designed for additional lateral soil pressure due to any vertical loads likely to occur within a horizontal distance of the wall equal to the depth of the wall below the adjacent ground surface. Determination of at-rest lateral earth pressures due to surcharge loadings should be calculated in accordance with the methods presented on Figure 4. Again is is recommended that Hart Crowser review any surcharge loading conditions, as they were not specifically available at the time of writing. Surcharge loads include not only those applied from building foundations, but also those associated with material stockpiles, construction equipment, or streets. Street loads are typically modeled as a uniform surcharge with an equivalent magnitude of 250 psf. They are applied to the wall as horizontal pressures with a magnitude of about one-half of the vertical value, with the point of application beginning at a depth equal to the distance of the nearest point of the load behind the wall.

-Soldier Piles

Soldier piles in the Puget Sound area are typically installed in predrilled vertical holes. Using the predrilling procedure, steel reinforcement such as an H beam, channels, or the equivalent are placed in cased or uncased holes, typically 24 to 36 inches in diameter. Concrete is dumped into the hole around the soldier pile (steel reinforcement) to the approximate level of the base of the excavation. This concrete should be strong enough to transfer load from the pile to the soil through both end-bearing and friction mechanisms. If vertical load transfer is not required, a weaker grout may be used. The backfill within the length of the soldier pile hole above the bottom of the excavation may also consist of a weaker grout, if desired. The material above the base of the excavation may be chipped away to facilitate placement of lagging, as required. If a casing is used during drilling, it should be pulled as the concrete is placed.

Structurally, soldier piles must be designed to carry the bending stresses between tiebacks or struts, and the vertical load resulting from any down-angle tieback anchors. The stresses can be calculated from the apparent earth pressure diagrams for the appropriate conditions. From a soil standpoint, the soldier pile must be capable of lateral stability below the lowest tieback or strut level and possess adequate vertical capacity.

Soldier piles may be designed to resist vertical loads using an allowable friction on the concreted surface of 1.0 kips per square foot (ksf) and a maximum allowable end-bearing resistance of 30 ksf. These values include a factor of safety of at least 2.0. A minimum soldier pile embedment of 6 feet is recommended.

Embedment depth of soldier pile below final excavation level must be designed to provide adequate lateral or "kick-out" resistance to horizontal loads below the lowest strut or tieback level, or the entire wall for a cantilever design. For design, the lateral resistance may be computed on the basis of the passive resistance computed using an equivalent fluid weight of 400 pcf above elevation 305 feet and 200 pcf below acting over twice the diameter of the concreted soldier pile section or the pile spacing, whichever is less. The passive pressure value has been derived using a factor of safety of 1.5.

The installation of soldier piles may be complicated by the presence of wet sandy zones within the upper soils and any portions below the water table. These materials could cave during installation. The contractor should be prepared to case the holes if necessary. It is likely that casing will be required below elevation 305 feet. The actual conditions and necessity for casing the holes should be determined in the field. The soldier pile holes should be as free as possible from water during placement of concrete. If concrete placement is required under water, the concrete mix should be properly designed for such conditions and should be tremied into place.

Cobbles and occasional boulders are often present in the glacial till soils. Large cobbles or boulders may present difficulties for installation of soldier piles or tiebacks. If drilling obstructions are encountered it may be necessary to relocate the shaft or break up and remove the obstruction.

-Lagging

The necessity of lagging between the soldier piles depends on the nature of the soil, the presence of groundwater, and the size and spacing of the soldier piles. Installation of lagging may also be subject to government regulations. It has been our experience that some form of lagging or other substantial protection of the excavation face will be required by the City adjacent to city streets, regardless of soil conditions. At this site, lagging will be required in the fill and silty sand soils. The need for lagging in the till can be determined at the time of excavation. Prompt and careful installation of lagging will reduce potential spalling or caving in loose and gravelly areas. The requirements for lagging should be made the responsibility of the shoring subcontractor to prevent soil failure, sloughing, and loss of ground, and to provide safe working conditions. We recommend voids between the lagging and soil be backfilled. However, the backfill should not allow potential hydrostatic pressure buildup behind the wall. Drainage behind the wall must be maintained. A permeable grout or sand slurry may be considered.

Because of soil arching between soldier piles, a reduced lateral pressure could be used for design of lagging. Experience suggests that about 30 percent of the lateral soil pressure uniformly distributed over the width of the lagging may be appropriate where the free space between the concreted soldier pile section is three diameters of less. If the clear space is greater than three soldier pile diameters, the lagging should be designed for about 50 percent of the design lateral pressures. Lagging 4 inches thick or less should be sufficient to provide the necessary support.

In areas where soil will be open cut above the top of the wall, it is recommended that lagging or some other type of protection be extended at least one foot above the top of the wall. This is to prevent loose material or slough from the cut slope from rolling into the excavation.

-Criteria for Bracing or Supports

Lateral loads on shoring walls may be resisted by installing internal struts or braces or external tieback anchors. Struts are commonly installed at angles to transfer the wall loads to small footings at the base of the excavation. The footings may be designed either as V-shaped wedges or with a typical horizontal bearing surface. Construction and design of these footings is similar to the conventional building foundations.

For a horizontal bearing surface, the maximum allowable soil pressure is 6,000 psf for footings a minimum of 2 feet wide and 2 feet deep. In addition, passive resistance on the side of the footing may be used with values as discussed in the previous sections. A third component, friction on the base of the footing may be estimated using an allowable coefficient of friction of 0.35.

For V-shaped footings the maximum allowable soil pressure is a function of the depth of the bearing surface and the width. If the ratio of depth to width is about zero (little or no embedment) the allowable pressure is estimated to be 400 times the footing width (in feet) in pounds per square foot (psf). If the depth of embedment is about the same as the footing width a value of 1,300 times the footing width (in feet) in psf could be assumed. A friction component on the other side of the V may also be included.

Tieback anchors must be fully located a sufficient distance behind the wall to develop resistance within a stable soil mass. This "no load" zone is approximately defined by a line inclined at 60^{o} up from the horizontal extending from the base of the excavation to the ground surface. To provide a safety factor, the line is set back behind the wall a distance of the excavation height divided by 4.

The anchor loads are transferred to the soil through friction along the shaft. The length of the anchor section required may be approximated using an allowable soil-anchor friction of 1.5 ksf above elevation 305 feet and 1.0 ksf below. We recommend all tieback anchors have a minimum length of 8 feet beyond the no load zone.

These recommended anchor design values include a factor of safety of at least 2.0, considered standard geotechnical practice. This factor of safety would provide for a reasonable additional load capacity should an unforeseen increase in unit soil load develop because of irregularities that can occur during installation of the anchor. The ultimate frictional resistance should be verified through field tests as subsequently discussed.

In order to allow for latitude in methods of installation, we recommend that selection of the materials and the installation technique be left to the shoring subcontractor. The anchor holes should be drilled in a manner which will minimize loss of ground and not disturb previously installed anchors. During the drilling, wet or saturated zones may be encountered, and caving could occur. Drilling with a continuous-flight auger or a casing (under extreme conditions) would reduce the potential for loss of ground. The shoring subcontractor should particularly note the presence of existing facilities adjacent to the project site, including buried utilities and foundations, as these may affect the location or extent of the anchor holes. The design for anchor locations, capacities, and related criteria, should be reviewed by Hart Crowser prior to implementation. In addition, the selected tieback anchor installation should be subject to performance testing by field anchor tests and proof loading.

We recommend that concrete be placed in the drilled tieback anchor hole by tremie methods such as pumping through a hose placed in the bottom of the hole or pumping through the center of a continuous-flight auger. In this way, the grout is forced up through the anchor zone under some pressure, with the resulting anchor more likely to be continuous. The grout should not be placed into the anchor zone by simple gravity methods such as flowing down a chute.

All tieback holes within the non-stable soil zone (no load zone) should be backfilled. The sole purpose of the backfill is to prevent possible collapse of the holes, loss of ground, and surface subsidence. We recommend that the backfill consist of sand, a sand-pozzolan-water mixture, or equivalent non-cohesive mixture. A sand-cement grout could be utilized only if some acceptable form of bond breaker (such as plastic sheathing) is applied to the tie rods within the no load zone.

-Testing of Supports

Tieback anchor design and installation can be tested using production and performance tests outlined in this section. Such testing is not convenient for struts. It is recommended, however, that struts be jacked into place to load magnitudes approximately equal to the design load. A key to successful performance of a strut will be the proper preparation and construction of the supporting footing on firm, undisturbed soil.

The unit friction resistance to be used in design of tieback anchors shall be verified under controlled test conditions. The shoring subcontractor shall be required to complete verification tests of 200 percent of design stress prior to installation of any production anchors in a soil type. At least two successful tests must be completed for anchors in a particular soil type with successful tests completed on a total of 5 percent of the anchors through the course of the project.

The verification tests shall measure anchor stress and displacement incrementally to a value of unit skin friction of 200 percent of the design stress. The anchor shall be loaded in increments of about 20 percent of the design load with each increment held without significant movement. During the holding period, the displacement will be read at approximately 1, 2, 4, 8, 15 and 30 minutes. All measurements of movement shall be obtained with a transit and a scale accurate to within 0.01 inch.

Tests shall be performed without backfill ahead of the anchor to avoid any resistance contributed by the backfill. If it is necessary to place backfill to prevent caving, the subcontractor shall take necessary steps so that load transfer does not occur within the backfill. For proper interpretation of the test results, the length of the anchor zone must be verified through direct measurement, rather than estimated during grouting.

An anchor performance verification test shall be deemed successful if the anchor exhibits neither excessive movement nor excessive creep. The relationship between the anchor stress and movement over the entire 200 percent stress range shall be approximately linear. If movements greater than about 0.04 inch are observed under constant load during the incremental holding period the holding should continue. The test should not be discontinued until all incremental and final movements are within acceptable limits. In addition, the rate of movement during the thirty minute hold period at the 200 percent stress level shall not exceed 0.08 inch per log cycle of time. If this rate is exceeded, a new thirty minute hold period should begin.

Following verification of the anchor design, each production anchor shall be proof-loaded to 130 percent of the design load. The anchor shall be loaded in increments of approximately 25 percent of the design load, with each increment held for a sufficient period to obtain a stable displacement reading. The 130 percent proof load shall be maintained for at least five minutes with displacements noted at 0.5, 1, 2, and 5 minutes. Measurements of movement shall be obtained with a transit and a scale accurate to 0.01 inch. Following proof loading, the anchor shall be locked off at 80 to 100 percent of design loading.

A production anchor shall be considered acceptable if total movements are less than 3 inches and movements during the 5-minute hold period do not exceed 0.08 inch per log cycle of time (i.e., from 0.5 to 5 minutes). Movements in excess of 12 inches are considered indicative of failure and require anchor replacement. Movements between 3 and 12 inches are signs of deficiencies in the installation. Acceptance or rejection of these anchors is at the discretion of the engineer.

-Shoring Monitoring

Any time an excavation is made below the level of existing streets, utilities, or other structures, there is a risk of damage even if a well-designed shoring system has been planned. We recommend, therefore, that a systematic program of observations be conducted during the project construction to delineate the effects of construction on adjacent facilities and structures. We believe that such a program is necessary for two reasons. First, if excessive movement is detected sufficiently early, it may be possible to undertake remedial measures which could prevent serious damage to existing facilities or structures. Second, the responsibility for damage may be established more equitably if the cause and extent of the damage are better defined.

The monitoring program should include measurements of the horizontal and vertical movements of: 1) the surface of the adjacent streets, 2) the adjacent structures on the south and west side of the excavation, and 3) the shoring system itself. A reference line should be established adjacent to the excavation at a horizontal distance back from the excavation face of about H, where H is the final excavation height. Monitoring of the shoring system should include measurements of vertical and horizontal movements at the top of each soldier pile or at intermittent intervals as considered appropriate by the geotechnical engineer.

The measuring system used for shoring monitoring should have an accuracy of at least 0.01 foot. All reference points on the existing ground surface should be installed and read prior to commencing the excavation.

Subsequent points at depth along the shoring wall should be installed and read as soon as possible during excavation. All reference points should be read prior to and during critical stages of construction. The frequency of readings will depend on the results of previous readings and the rate of construction. As a minimum, readings should be taken about once a week throughout construction until the basement walls are completed. More frequent readings may be required at critical times during construction or if significant movement is indicated. All readings should be reviewed by the geotechnical engineer.

In addition to the monitoring program described above, we recommend that the owner or his representative make a complete inspection of all pavements and structures and other facilities adjacent to the project site. This inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures taken where necessary. Likewise, the contractor and the shoring subcontractor should be familiar with the existing site conditions. They should be allowed to review the data obtained by the owner and may also choose to complete a survey. Regardless, the contract should clearly define the responsibilities of the owner, contractor, and shoring subcontractor in making inspections, reviewing data, and repairing possible damage.

Foundation Design

Spread foundation support may be used for design of all areas of the building. In general, dense to very dense till or silty sand is expected at the proposed footing elevations. The footings must bear on undisturbed soil surfaces. Should disturbance occur during excavation, the footing excavation should be extended until dense, undisturbed material is reached. The overexcavated area could then be backfilled with either structural concrete or lean mix concrete. In those areas where glacial till is expected to be encountered at the bearing level, disturbance is less likely. However in the areas of the silty sand, movement of construction equipment, or excessive foot traffic could result in some loosening of the exposed soil. All footing excavations should be observed by the geotechnical engineer prior to placement of reinforcing steel and concrete.

We recommend the footings be designed using a maximum allowable soil bearing pressure of 10,000 psf. This bearing pressure would be appropriate for either the dense undisturbed silty sand or the glacial till soils. If the structural design requires that a higher value be used, one could be provided for the till soils. The use of 10,000 psf is appropriate for footings with a minimum width of 4 feet. For footings smaller than 4 feet wide, a maximum allowable pressure of 6,000 psf should be used.

These recommended allowable soil bearing pressures are appropriate for a combination of dead loads plus frequently applied live loads. For short duration and infrequently applied live loads, such as seismic loads, the allowable soil bearing pressure may be increased by one-third. For individual and continuous spread footings we recommend a minimum width of 24 inches. The bottom of all footings should be embedded a minimum of 18 inches below the lowest adjacent finished grade.

Lateral loads applied to the footings and foundation walls could be resisted by a combination of passive resistance and frictional sliding resistance. The appropriate design values for estimation of these resisting forces are the same as given in the shoring section of this report. The allowable friction on the base of footings can be estimated using a friction factor of 0.35. Passive resistance can be estimated using equivalent fluid weights of 400 pounds per cubic foot (pcf) above elevation 305 feet and 200 pcf below. A factor of safety of about 1.5 has been incorporated into these allowable values.

Total settlements are estimated to be on the order of 3/4 to 1-1/4 inches. Differential settlements between adjacent footings or across the building area will range from 1/2 to 3/4 inch. Because the site soils are elastic in nature, we expect most of the settlement to occur during and shortly after construction. Disturbance of the foundation base during excavation or subgrade preparation could result in larger settlements of the shallow foundation due to the loosening effect on the soil. We therefore recommend that areas loosened or disturbed during subgrade preparation or excavation be hand cleaned, if necessary, to remove the loosened soil prior to concrete placement.

Lateral Earth Pressures on Permanent Basement Walls

For backfilled basement walls or foundation walls backfilled on one side only, lateral soil pressures will be a function of the active or at-rest conditions depending on the amount of lateral movement permitted at the top of the wall during backfilling operations. If backfilled walls are free to yield at the top a distance of at least 0.001 times the height of the wall during backfilling, soil pressures would be in an active condition. If the movement is limited by stiffness or by construction of a structural floor network prior to backfilling, an at-rest conditions should be assumed.

Based on soil backfill and compaction criteria given in a subsequent section, we recommend that an equivalent fluid pressure of 35 pcf or 55 pcf be used for yielding (active) or non-yielding (at-rest) walls, respectively. To reduce the potential for buildup of lateral earth pressures in excess of the above design pressures, overcompaction of the fill behind the wall should be avoided. This can be accomplished by placing the backfill within 18 inches of the wall in lifts not exceeding 6 inches in loose thickness and compacting with hand operated or small self propelled equipment.

For basement or foundation walls poured flush against the in-place shored walls, the lateral soil pressures will be a function of the final backfilled height of the soil adjacent to the wall. Equivalent fluid pressures of 35 pcf or 55 pcf for active or at-rest conditions, respectively, should be used for the portion of the wall which is backfilled, such as that area which has been previously open cut. The portion below the open cut area, may be designed using reduced equivalent fluid pressures to represent the intact soil conditions. Values of 30 pcf or 45 pcf may be used for active or at-rest conditions, respectively, in the previously shored portions of the wall.

For a multiple supported system, the applied pressures for the permanent design of the wall may be the same as those assumed during shoring, if the pattern of floor spacing is similar to the spacing used for the shoring support. If significant differences exist between the spacing of the shoring support and the floor network, a triangular distribution of pressure should be assumed for design of this wall, using the criteria given in the preceding paragraph.

The preceding lateral earth pressure recommendations are based on horizontal backfill, utilizing granular soil for backfill, and no buildup of hydrostatic pressures behind the wall. The effects of surcharge, such as traffic or floor loads, should also be included. For a uniformly distributed load behind the wall, a corresponding uniformly distributed pressure equal to 35 percent or 50 percent of the surcharge should be added to the lateral soil pressure for yielding or non-yielding walls, respectively.

Slab-on-Grade

The lowest basement level may be constructed as slab-on-grade above a drainage layer placed on the dense or hard natural soils. Following excavation and footing construction it is likely that some loosening of the soil near the surface will have occurred. Loose areas should be recompacted to provide a dense, non-yielding surface. If structural fill is required to bring the surface to the desired final grade, it should be placed in accordance with the provisions given in the structural fill section of this report.

All slabs-on-grade should be underlain directly by a drainage layer at least 6 inches thick. This layer should consist of well graded sand and gravel with a fines (soils smaller then the U.S. No. 200 sieve) content of less than 3 percent by weight. This layer serves as a capillary break and drainage layer and is intended to reduce the potential for buildup of hydrostatic pressures beneath the slab and envelop the recommended subslab drains as discussed in a subsequent section.

Structural Fill

All fill placed beneath slab-on-grade, or behind basement walls should be placed as structural fill. The structural fill should be placed in lifts not exceeding 8 inches loose thickness and should be thoroughly compacted to at least 95 percent of the modified Proctor maximum dry density as determined by the ASTM D1557 test procedure. The moisture content during compaction should be controlled within 2 percent of optimum moisture. Optimum moisture is the water content which results in the highest compacted dry density. It is recommended that a representative of our firm be present during placement to monitor filling and perform field density tests.

Prerequisite to fill control is the determination of the compaction characteristics from representative samples. Samples should be obtained from either the excavated on site natural soils or a borrow area as soon as work begins. The study of the compaction characteristics should include determination of optimum and natural moisture contents of these soils at the time of placement. The suitability of excavated site soils or imported soils for compacted structural fill would depend on the gradation and moisture content of the soil when it is placed. We recommend all structural fill material consist of well graded sand or sand and gravel with a low fines content. As the amount of fines increases, the soil become increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soils containing more than about 5 percent fines cannot be consistently compacted to a non-yielding conditions when the water content is significantly above or below optimum. The fines content should be limited to less then 5 percent (based by weight on the minus 3/4-inch fraction using the wet sieve analysis) if placed during periods of wet weather. Fill within 6 inches of slab-on-grade, 18 inches of backfilled subgrade walls, and around all drains must be well graded with a fines content of less then 3 percent. This material is referred to as drainage fill.

The existing site soils consist of silty, medium to fine sand, and very silty, gravelly sand. The fines content of the till soils is typically in the range of about 30 to 40 percent. The silty sand has less fines, typically less than 20 percent. These soils are considered moderately to highly moisture sensitive. The site soils appear suitable for use as fill only during extended dry weather periods that typically occur in the late summer months. It may be possible that some areas of the silty sand would have a smaller fines content and could be placed as fill. During wet weather construction, however, we recommend using imported fill consisting of free draining, well graded, sand and gravel containing less than 5 percent fines. Regardless of the source, material used as structural fill should be free of construction debris, organic material, and cobbles greater than 6 inches in size. The existing site soils are not suitable for use as drainage fill.

Construction Dewatering

In our opinion significant construction dewatering will not be required. Most of the area will be excavated to a level estimated to be some 15 feet above the groundwater table. However in the southwest corner of the project, excavation will extend to depths at or near the estimated groundwater level of elevation 305 feet. In those areas it may be necessary to provide some measure of dewatering to the general excavation, or to individual footing excavations. The type of dewatering system which would be most appropriate depends on the drainage characteristics of the material encountered in this area. We have found the silty sand soils to be variable, with silt contents estimated to be as high as about 20 percent or low as less then 5 percent. The permeability or drainage characteristics of the materials is therefore also expected to be variable.

It may be possible that installation of sumps and small pumps would be sufficient to allow the excavation to maintained in a workable condition.

Deeper excavations such as those for individual footings, may require installation of well points to drain a confined area. We have estimated the permeability of the materials based on limited grain size data to be about 10^{-3} cm/sec. We believe the contractor should be prepared to deal with water in all areas of the site, based on the presence of zones of water bearing material even within the fill soils.

It is recommended that the geotechnical engineer reevaluate the groundwater conditions in the excavation in terms of both temporary and permanent dewatering systems when the excavation has progressed to the necessary levels.

Permanent Drainage Considerations

In those areas where the slab-on-grade will be at or near elevation 324 feet, a conventional permanent drainage system is appropriate. In the areas where the lowest level would be nearer elevation 306 feet, a higher capacity system is required.

In general, subslab drainage should be provided using a combination of perimeter and cross drains beneath slabs-on-grade. In addition, we recommend drains be installed behind any backfilled subgrade walls. The drain (with clean-out) should consist of 4-inch-diameter perforated pipe placed on a bed of and surrounded by 6 inches of clean (less than 3 percent fines), well graded sand and gravel. The drain should be sloped to carry the water to a sump or other suitable discharge location. The cross drains should be installed on about 40- to 50-foot centers and also surrounded by sand and gravel. As previously mentioned all slabs should be underlain directly everywhere by a 6-inch drainage layer and capillary break.

Backfill within 18 inches of any backfilled retaining or subgrade walls should consist of clean, free draining sand and gravel. This backfill should be continuous with and envelop the drains behind the wall.

Permanent drainage behind basement walls constructed flush against shoring walls can be provided using a manufactured drainage medium such as miradrain. Such a material should be attached directly to the lagging or soil. A one-foot-wide strip of the drainage medium between each pair of soldier piles should be sufficient. The drainage material should have a filter fabric between the drain and the lagging and should be covered on the back face to prevent concrete contamination. The drainage medium should be hydraulically connected to the drainage fill which will be behind any backfilled wall areas above, and to the perimeter or subslab drainage system below.

We recommend perimeter drains be placed at the base of foundation walls constructed against shoring and that they be sloped to carry water away from the foundations to the collection system described previously. These drains should also consist of minimum 4-inch-diameter perforated pipe. The drains should be hydraulically connected to the drainage fill and to the manufactured drainage medium behind the walls.

In the deeper excavation area of the site, it may be necessary to increase to the size of the subslab pipes drainage pipes from 4 inches to 6 inches. In addition a special discharge point or sump will probably be required in this area with a permanent pump to provide for the additional water quantities expected. A backup sump or pump system should also be considered in this area.

Installation of the drainage system should be within any guidelines established by the structural engineer, architect, and owner, and should be reviewed by the geotechnical engineer. It should be noted that the subslab and wall drainage recommendations are designed to remove excess water and prevent a damaging buildup of hydrostatic pressure. The recommended systems may not result in a totally dry wall or slab.

Surface runoff and roof drainage should not be allowed to infiltrate adjacent to the foundation walls. Pavement and sidewalks should be sloped to drain away from the building and adequate runoff disposal should be provided.

Seismic Design

In 1983 Hart Crowser performed a study of the seismic design parameters for use at the linear accelerator facility located at the corner of 16th Avenue and E. Jefferson Street on the Providence campus. Soil conditions at that location are similar to those at the east tower site. While some advances in the state-of-the-practice have occurred since completion of that study, our review of the design assumptions and procedures suggests that significant changes in the conclusions and recommendations are not appropriate.

We therefore recommend that those previously developed results be used for this project, to the degree required by the structural design. The results of that study have been given to the structural engineer, and may be obtained by others through Hart Crowser.

RECOMMENDATIONS FOR ADDITIONAL SERVICES

As the design process continues, additional geotechnical services should be provided by Hart Crowser as needed. We would be available to address all aspects of the geotechnical design and construction. It is recommended that Hart Crowser be provided the opportunity for a general review of the final foundation design plans and specifications in order that the geotechnical engineering recommendations may be properly interpreted and implemented in the design and specifications.

As the design continues it is expected that Hart Crowser will need to provide additional input on the design of subgrade walls to resist surcharge pressures from adjacent facilities and footings, and design of underpinning, if needed. These requirements will be established by the structural engineer as the design process continues and more information becomes available. In addition, it is expected that geotechnical input will be required when more information becomes available on the existence, design, and plans for an existing well located near the southwest corner of the project. We will be available to meet with the other design consultants and with the owner to review the various requirements and changing design as they effect the geotechnical aspects of the project.

We also recommend that Hart Crowser continue to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This would include observations and review of 1) excavation and installation of shoring; 2) foundation construction to verify the nature of the bearing soils prior to placing concrete; 3) assessment of the suitability of on site or imported soils for use as structural backfill; 4) slab-on-grade areas including surface preparation and placement and compaction of structural fill; 5) subslab and wall drainage provisions; and 6) other geotechnical considerations which may arise during the course of construction.

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

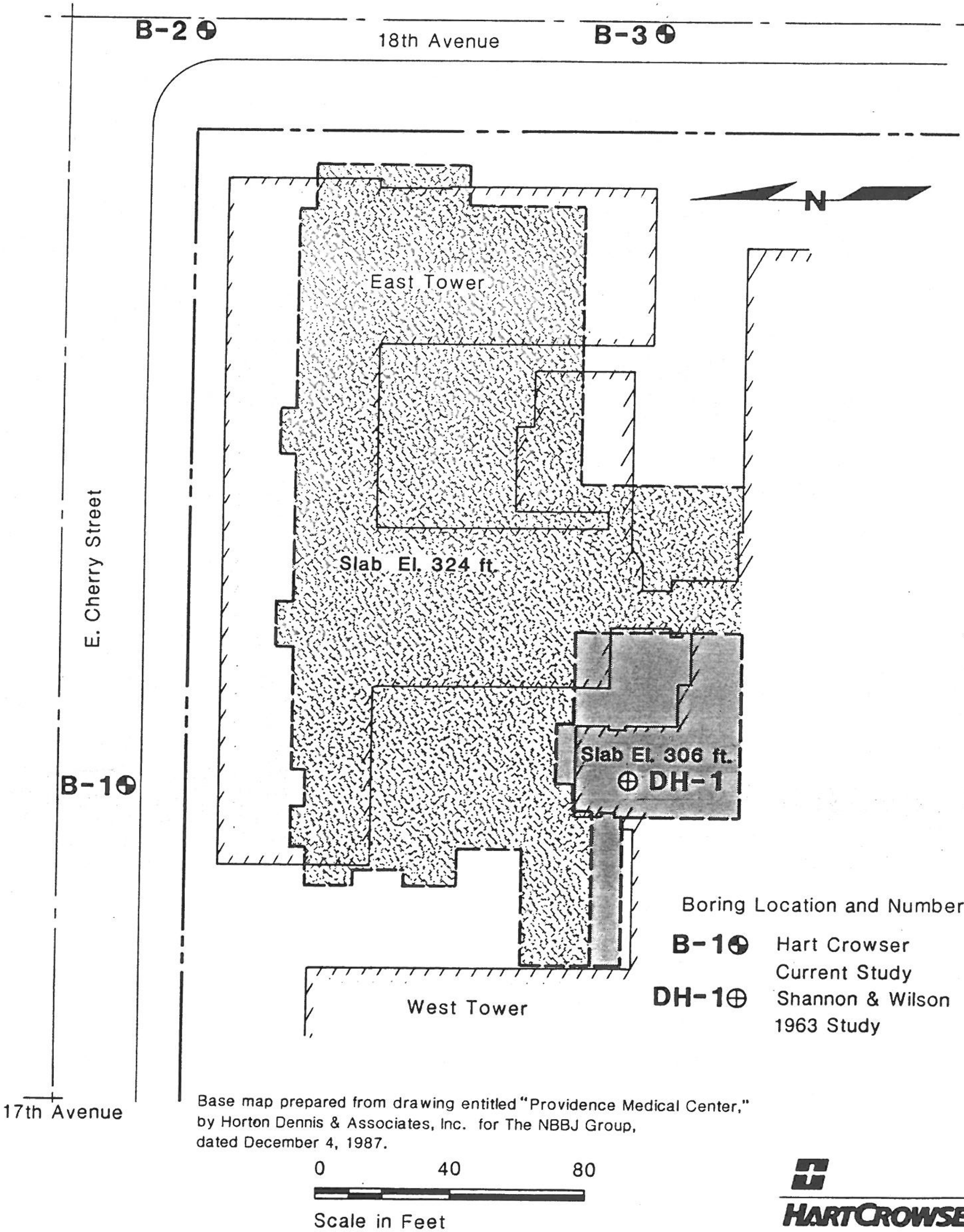
HART CROWSER, INC.

David G Winter

DAVID G. WINTER, P.E.
Senior Project Engineer

DGW:stk

Site and Exploration Plan



Base map prepared from drawing entitled "Providence Medical Center," by Horton Dennis & Associates, Inc. for The NBBJ Group, dated December 4, 1987.

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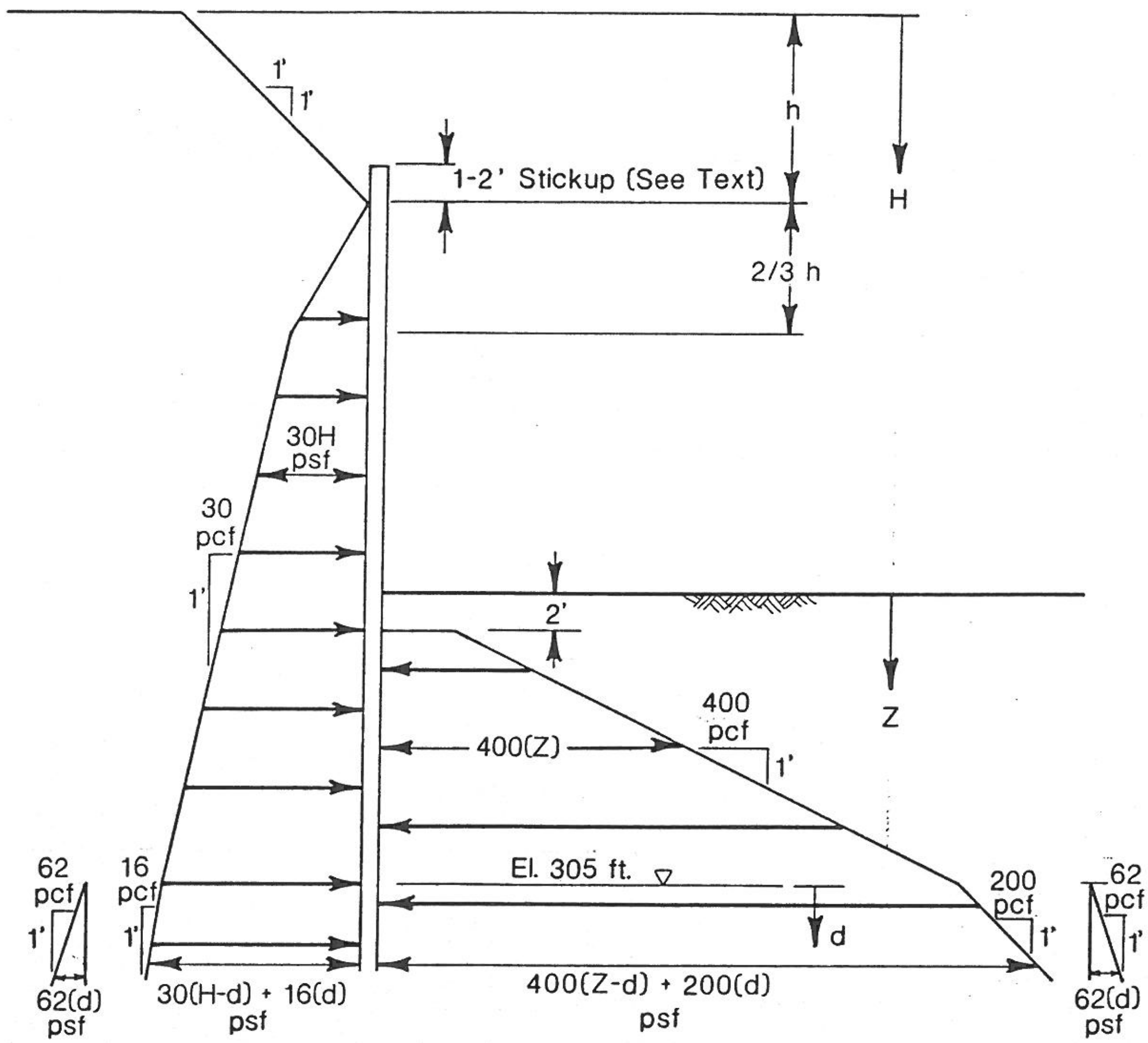
Figure 1

Design of Shoring

Single Support or Cantilever

Inclined Backslope

Active Pressure Conditions



Not to Scale

Notes: 1. Active Pressure assumed to act over pile spacing above excavation level, and twice the grouted soldier pile diameter or the pile spacing, whichever is smaller, below excavation level.

2. Passive Pressures assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive Pressures include Factor of Safety of about 1.5.

3. All dimensions in feet.

4. See text for discussion.

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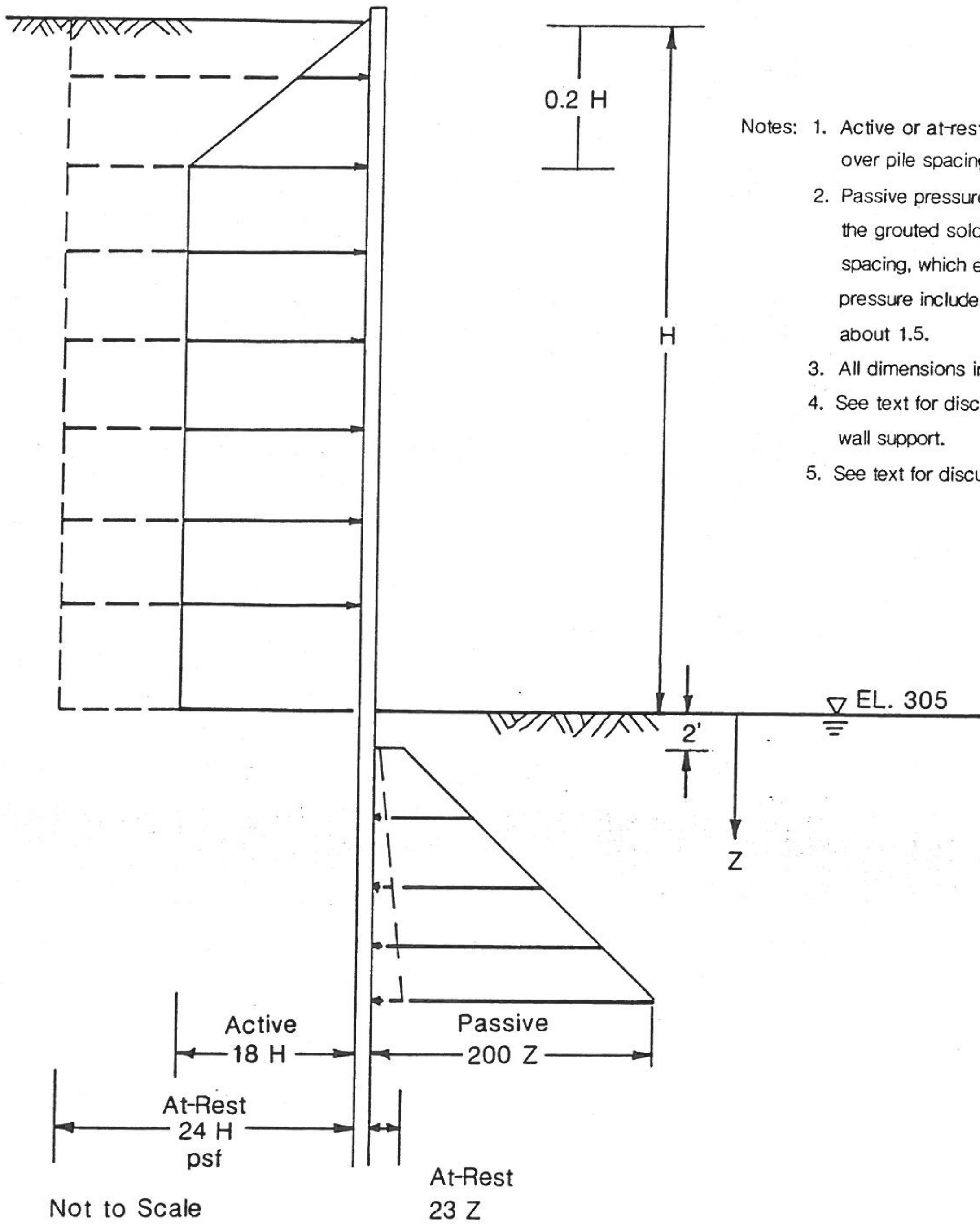
Figure 2

Design of Shoring
Multiple Supports
Horizontal Backslope
Active or Rest Pressure Conditions



Notes:
1. Active or at-rest pressure assumed to act over pile spacing.
2. Passive pressure assumed to act over twice the grouted soldier pile diameter or the pile spacing, which ever is smaller. Passive pressure includes Factor of Safety of about 1.5.
3. All dimensions in Feet.
4. See text for discussion of design of wall support.
5. See text for discussion of application.

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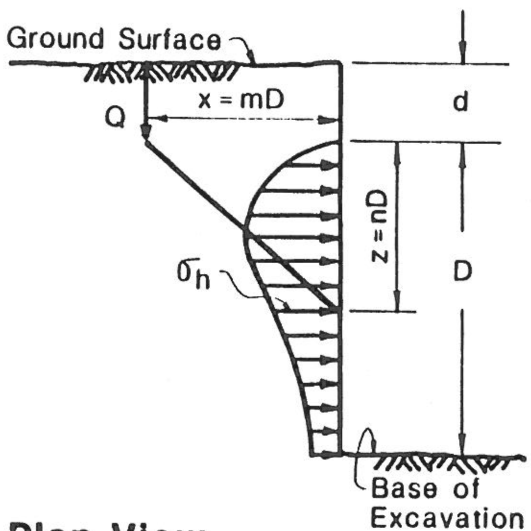
Figure 3

Surcharge Pressures

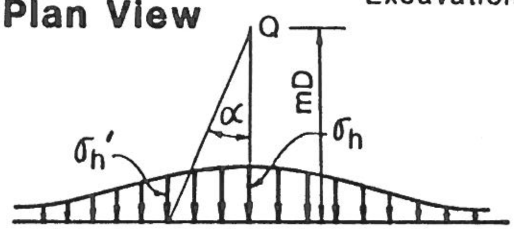
Determination of Lateral Pressure Acting on Adjacent Walls Due to:

A. Small Isolated Footing

Cross Section View



Plan View



$\sigma_h' = \sigma_h \cos^2 (1.1\alpha)$

(For $m > 0.4$)

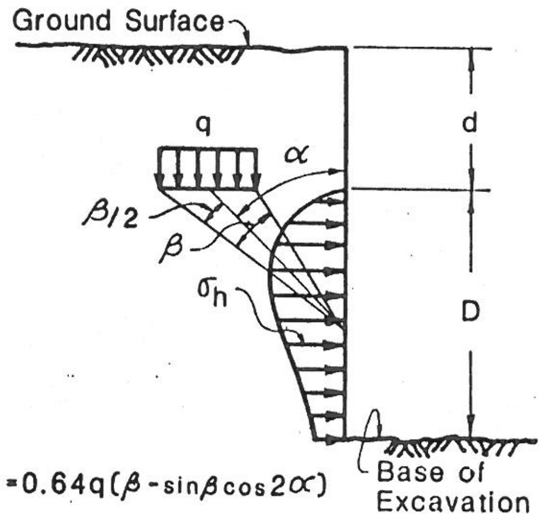
$$\sigma_h = \frac{1.77Q}{D^2} \frac{m^2 n^2}{(m^2 + n^2)^3}$$

(For $m \leq 0.4$)

$$\sigma_h = \frac{0.28Q}{D^2} \frac{n^2}{(0.16 + n^2)^3}$$

B. Wide Continuous Footing

Cross Section View



$\sigma_h = 0.64q(\beta - \sin\beta \cos 2\alpha)$

Definitions and Units

Q Footing Load In Pounds

D Excavation Depth below Footing in Feet

d Depth to Base of Footing in Feet

σ_h Lateral Soil Pressure in PSF

q Unit Loading Pressure in PSF

q' Unit Line Loading Pressure in Pounds per Foot

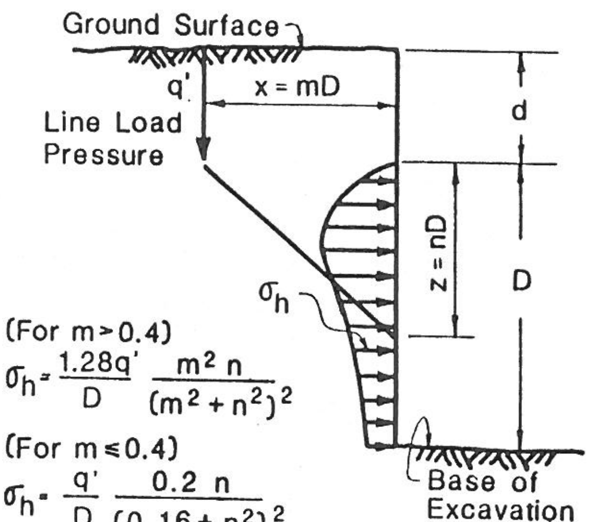
α, β Radians

General Notes

1. Lateral Soil Pressures due to adjacent structure should be added to At Rest Lateral Pressures.
2. See text for area of application.
3. Wall footings acting other than parallel to the excavation or large isolated footings can be treated as series of discrete point loads, using approach A.

C. Continuous Wall Footing Parallel to Excavation

Cross Section View



(For $m > 0.4$)

$$\sigma_h = \frac{1.28q'}{D} \frac{m^2 n}{(m^2 + n^2)^2}$$

(For $m \leq 0.4$)

$$\sigma_h = \frac{q'}{D} \frac{0.2\ n}{(0.16 + n^2)^2}$$

D. Uniform Area Load

$\sigma_h = 0.5q$

q Uniform Pressure in PSF

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Figure 4

APPENDIX A
FIELD EXPLORATIONS

The program of subsurface explorations for this project included drilling of three hollow-stem auger borings. The results of our exploration program are presented on the exploration logs within this Appendix. The exploration logs are a representation of our interpretation of the drilling, sampling, and testing information. The depth where the soils or characteristics of the soils changed is noted. The change may be gradual. Soil samples recovered in the explorations were visually classified in the field in general accordance with the method presented on Figure A-1. A legend for the field exploration logs defining symbols and abbreviations utilized is also presented on Figure A-1.

The exploration locations are presented on Figure 1. The explorations were located in the field by hand taping or pacing from existing physical features. The approximate ground surface elevation at the exploration locations, as presented on the exploration logs, are interpreted from elevations presented on a site survey entitled "Providence Medical Center" completed by Horton Dennis & Associates and The NBBJ Group. The location and elevation of the explorations should be considered accurate to the degree implied by the method used.

A total of 3 hollow-stem auger borings, designated B-1 through B-3, were drilled from November 23 to 24, 1987. The borings were completed to depths ranging from 60 to 70 feet below the ground surface. The borings were advanced with a truck-mounted drill rig under subcontract to Hart Crowser, Inc. using a 3-3/8-inch inside diameter hollow-stem auger. The drilling was accomplished under the continuous observation of an engineering geologist from our firm. Detailed field logs were prepared of each boring. Samples were obtained on approximately 5-foot-depth intervals using the Standard Penetration Test (SPT) procedures.

The Standard Penetration Test procedure as described in ASTM D 1587, was used to obtain disturbed samples. A standard 2-inch outside diameter, split-spoon sampler is driven into the soil a distance of 18 inches using a 140-pound hammer, free-falling 30 inches. The number of blows required to drive the sampler the last 12 inches is the Standard Penetration Resistance. This resistance, or blow count, provides a measure of the relative density of granular soils and consistency of cohesive soils. The blow counts are plotted on the boring logs at the respective sample depths. Samples were recovered from the split-barrel sampler, field classified and placed in water-tight jars and taken to our laboratory for further testing. The Standard Penetration Test is a useful quantitative tool from which density/consistency is determined. The results must be used in conjunction with other tests and engineering judgment.

If the high penetration resistance encountered in very dense materials precluded driving the total 18-inch sample interval, the penetration resistance for the partial penetration is entered on logs as follows: if the total penetration is greater than 6 inches and less than 18 inches, then the noted blow count is the sum of the number of blows completed after the first 6 inches of penetration, over the number of inches driven in excess of the first 6 inches. For example, a blow count series of 12 for 6 inches, 30 for 6 inches, and 50 for 3 inches, would be recorded as 80/9 inches. A blow count series of 32 for 6 inches and 50 for 4 inches would be reported as 50/4 inches. In the case where total penetration is less than 6 inches, the total number of blows and penetration are indicated.

The boring logs are presented on Figures A-2 through A-4. The log of a boring, DH-1, drilled in 1963 for the Shannon & Wilson study is reprinted in this Appendix on Figure A-5.

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

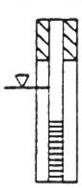
BORING SAMPLES

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

TEST PIT SAMPLES

- ☒ Grab (Jar)
- ◩ Bag
- ⧅ Shelby Tube

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)

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

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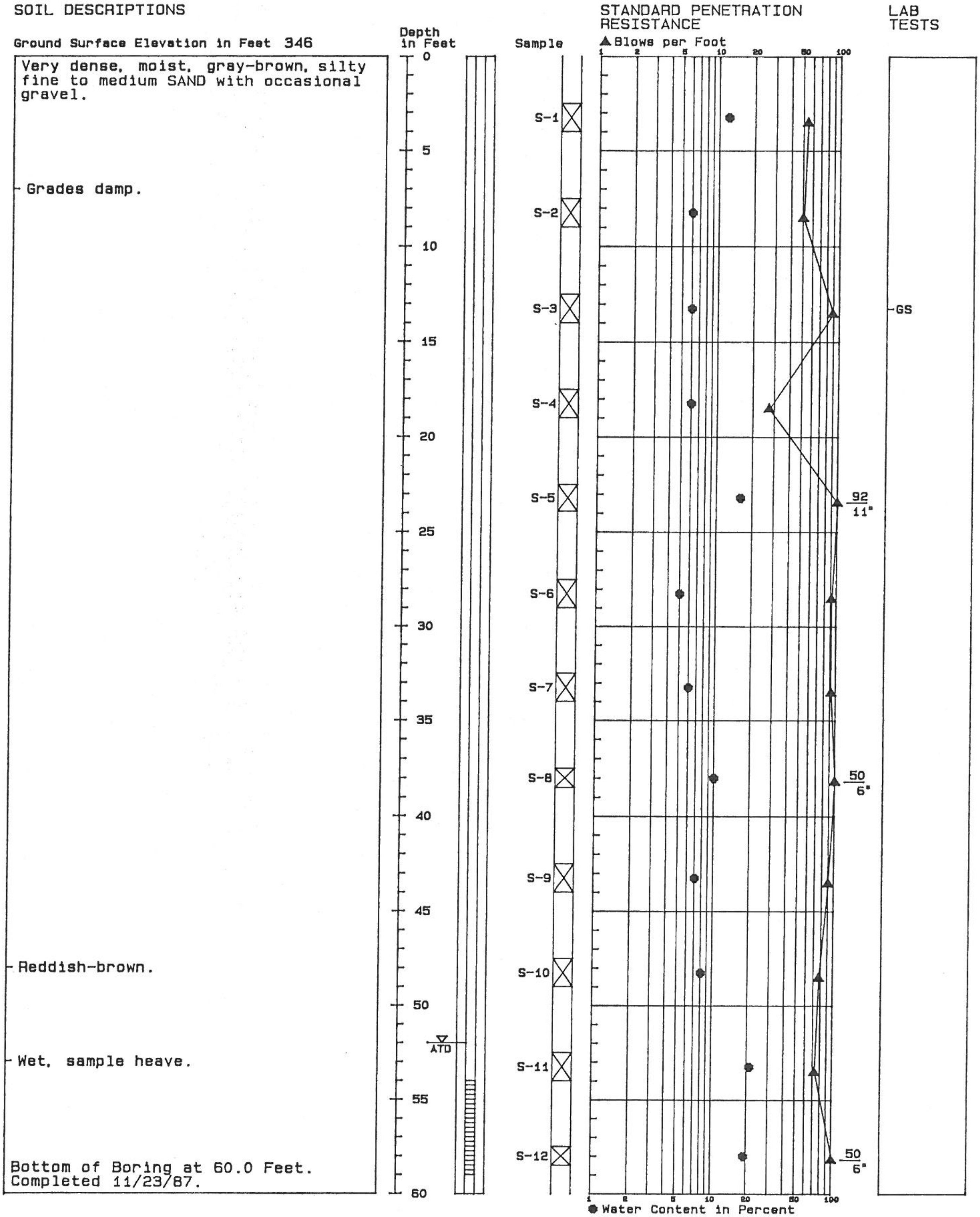
Figure A-1

Boring Log B-1



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. Ground water 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

Boring Log B-2

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 353

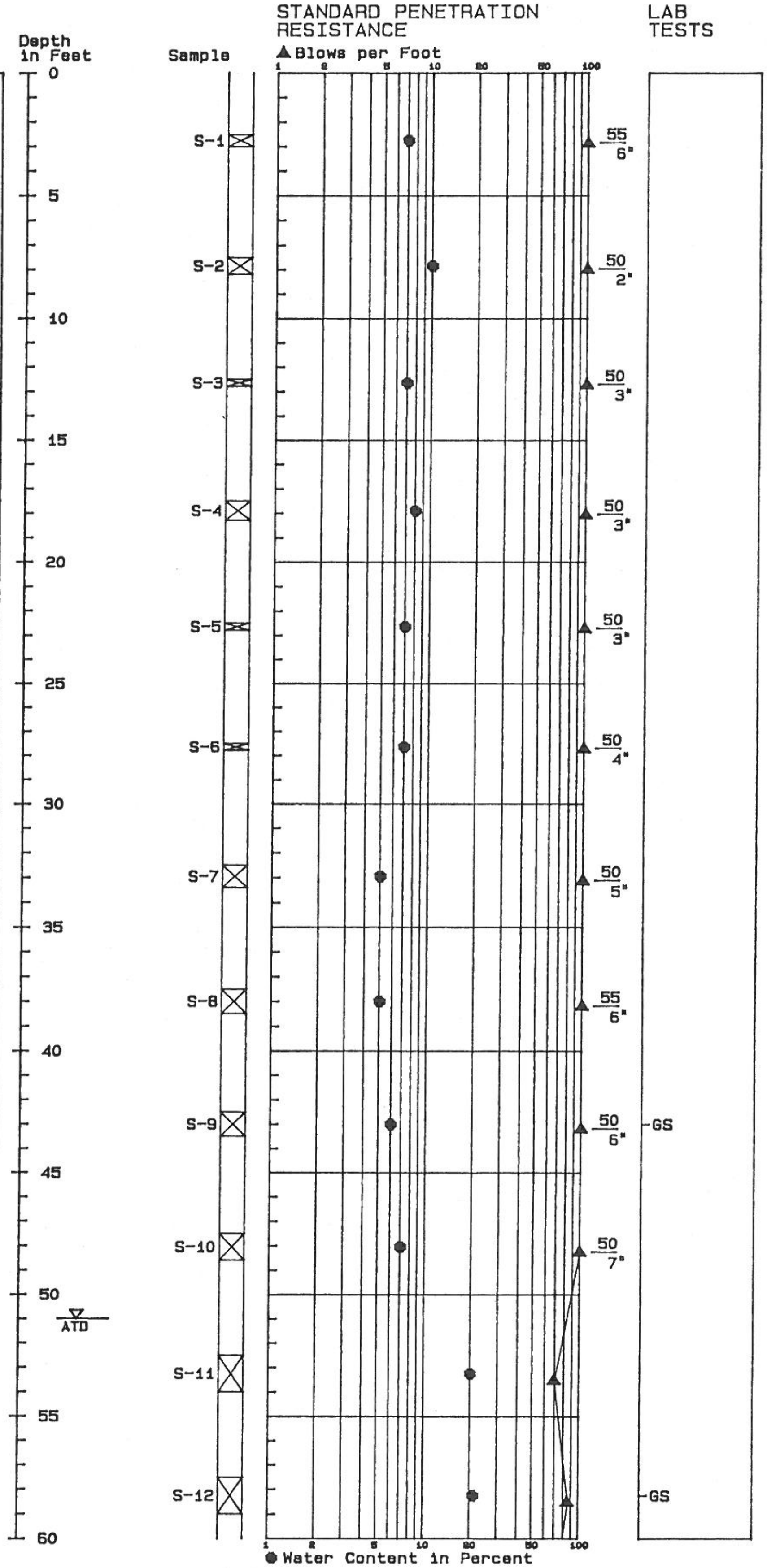
Very dense, damp, gray, silty, gravelly SAND. (TILL)

- Grades less gravelly.

Very dense, damp, gray-brown, slightly silty, fine to medium SAND.

- Grades to wet, brown, silty, fine SAND.

- Sample heave.



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Sheet 1 of 2 Figure A-3

Boring Log B-2

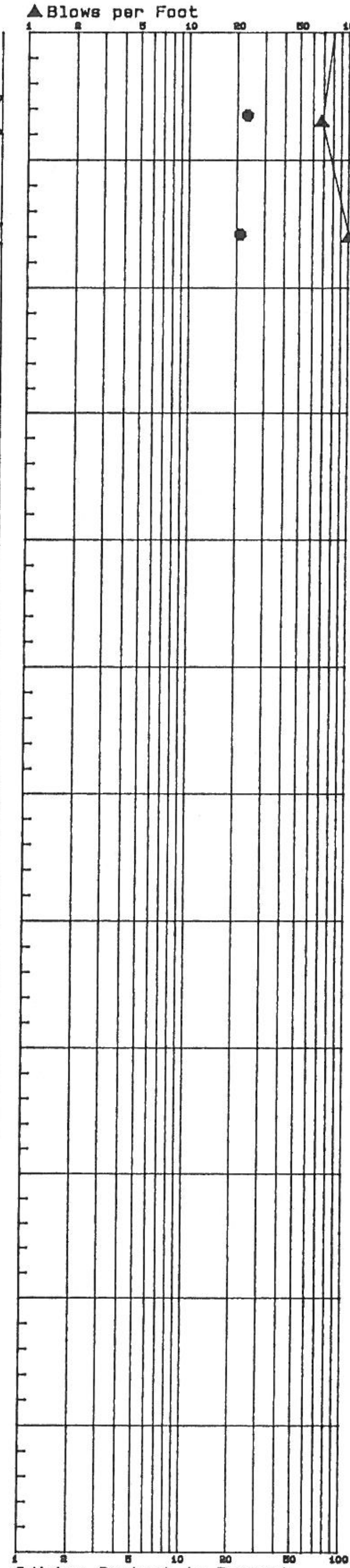
SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 353

Soil Descriptions	Depth in Feet	Sample
Very dense, wet, brown, silty, fine SAND.	60–70	S-13, S-14
Bottom of Boring at 70.0 Feet. Completed 11/24/87.	70	

STANDARD PENETRATION RESISTANCE

▲ Blows per Foot



● Water Content in Percent

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. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Sheet 2 of 2 Figure A-3

Boring Log B-3

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet 343

Depth in Feet

Loose, moist, brown, slightly silty, slightly gravelly to gravelly SAND. (FILL)

Very dense, damp, gray, gravelly, very silty SAND. (TILL)

- Samples indicate variable silt and gravel contents.

- Drill action indicates 1-foot-zone of water.

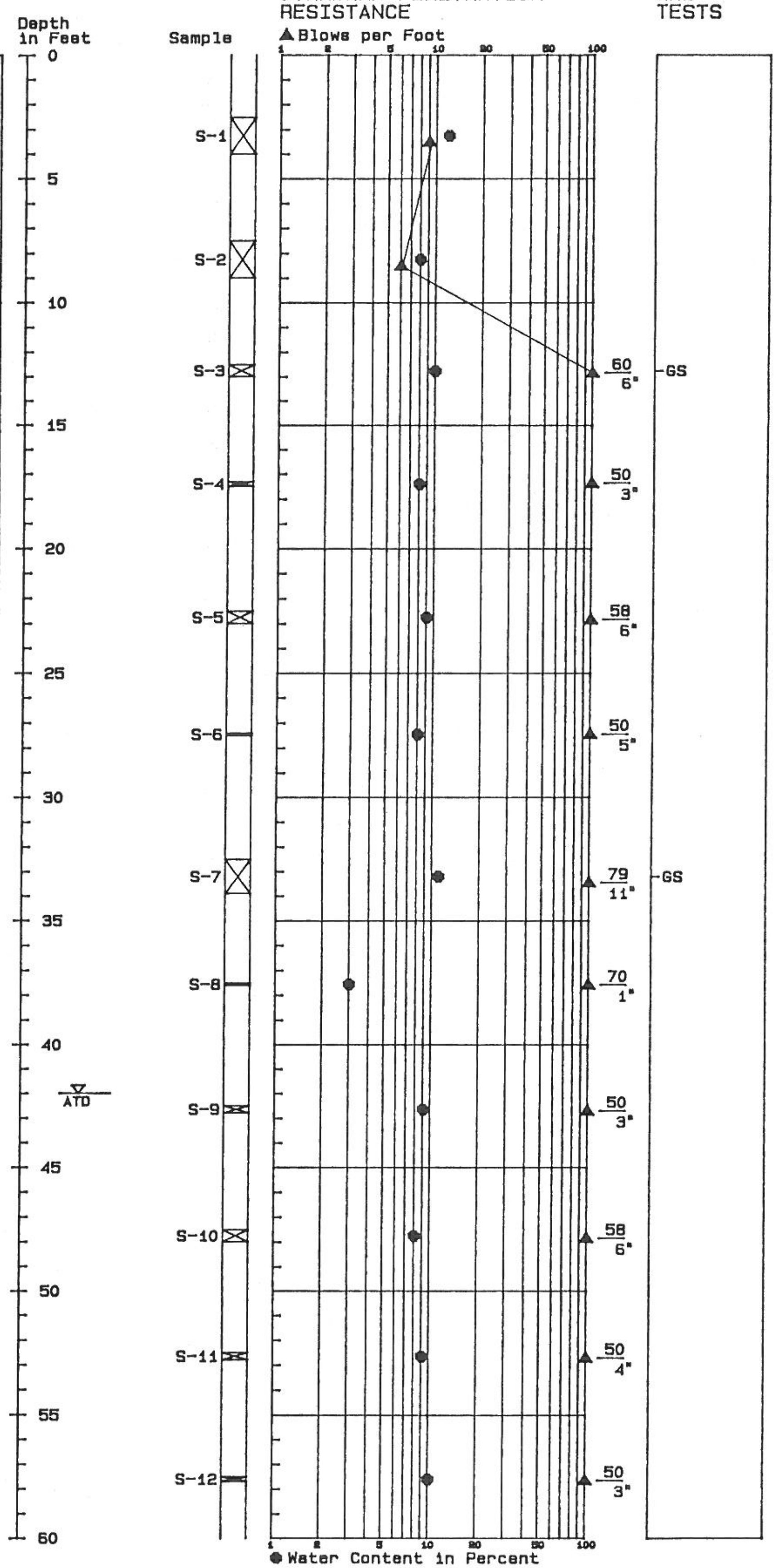
- Grades dark gray.

- Drill action indicates cobbly zones.

Bottom of Boring at 60.0 Feet.
Completed 11/24/87.

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. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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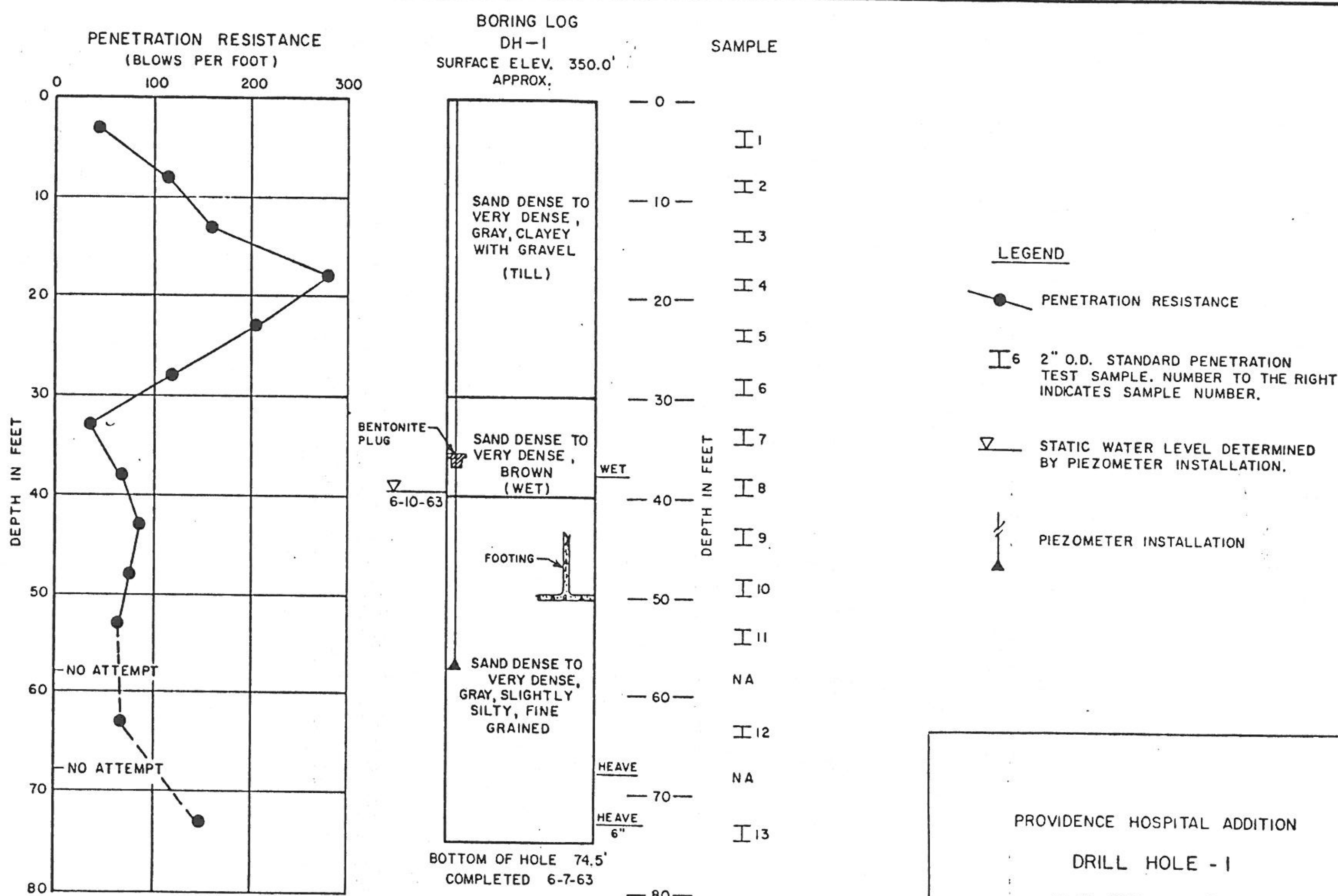
Figure A-4



PROVIDENCE HOSPITAL ADDITION

DRILL HOLE - 1

W-63-209 JUNE, 1963

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle

FIGURE 2

Figure A-5

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. Laboratory tests were performed on disturbed samples. The laboratory tests performed and the procedures followed are outlined below.

Soil Classification

Soil samples recovered in the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled environment. Visual-manual field and laboratory observations include density, moisture condition, and grain size estimates.

The classifications of selected samples were checked by performing laboratory tests such as 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

Water contents were determined for most samples recovered in the explorations in general accordance with ASTM D 2216 as soon as possible following their arrival in our laboratory. Water contents were not determined for very small samples nor samples where large gravel contents would result in values considered unrepresentative. The results of these tests are plotted at the respective sample depth on the exploration logs. In addition, the water contents of samples subjected to other testing have been determined and are presented on the exploration logs as well as with the various test results which follow in this appendix.

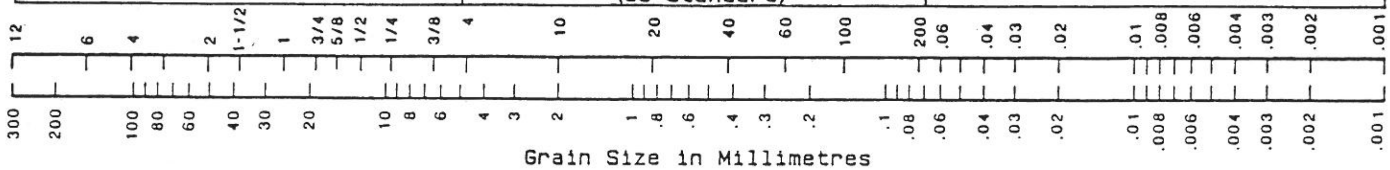
Grain Size Analysis (GS)

Grain size analyses were performed on representative samples in general accordance with ASTM D 422. The wet sieve analysis method was used for most samples and determines the size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figure B-2 plotting percent finer by weight versus grain size.

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 & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

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						

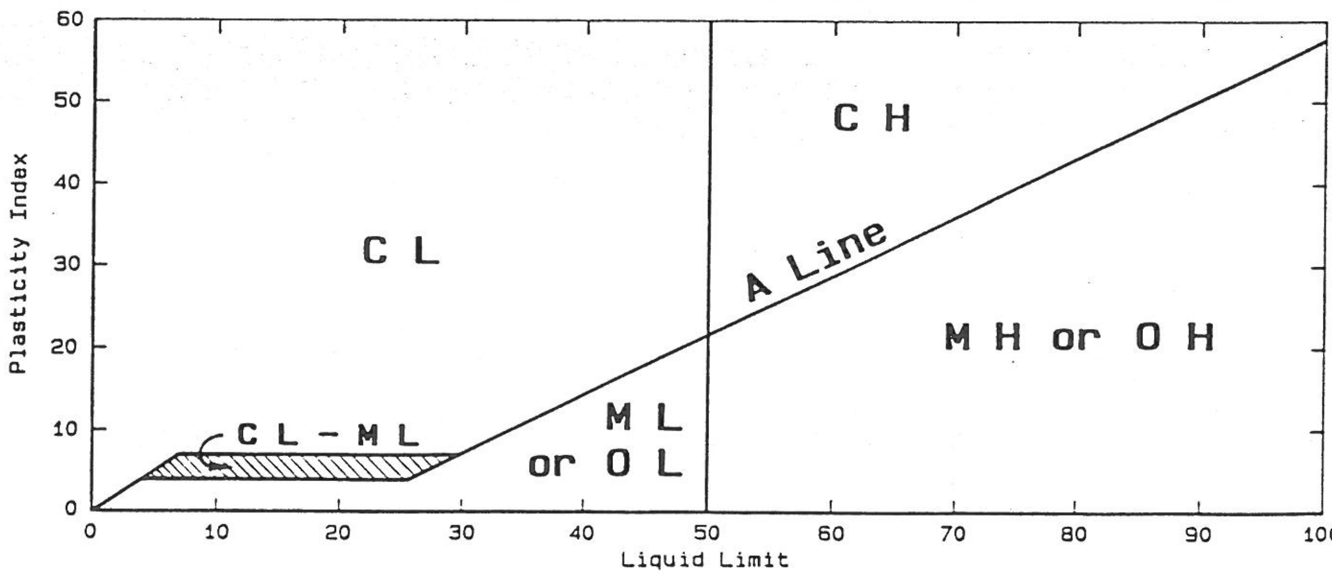


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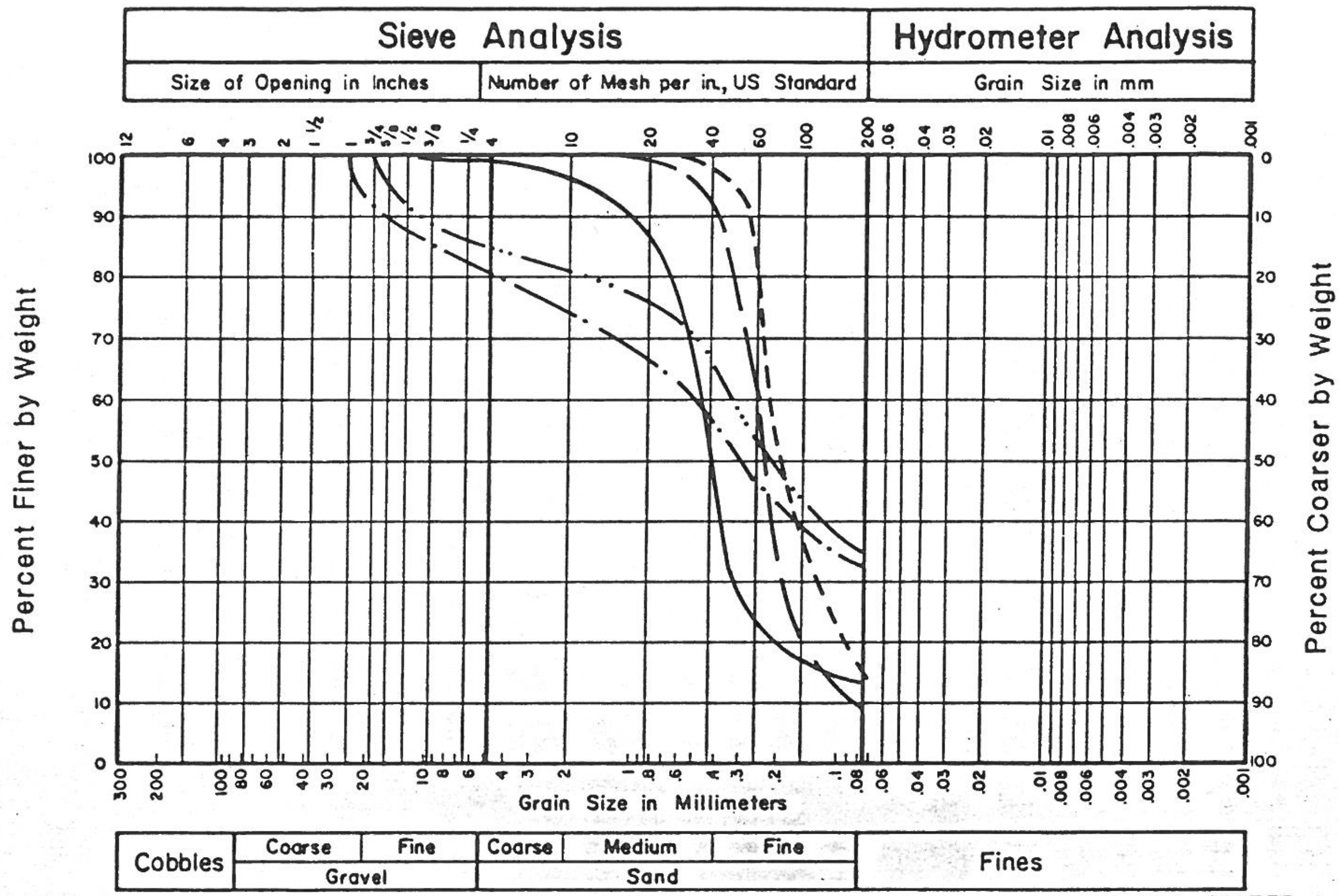
Figure B-1

Grain Size Classification



LINE SYMBOL	BORING NUMBER	SAMPLE NUMBER	DEPTH IN FEET	CLASSIFICATION	UNIFIED SOIL CLASS.	WATER CONTENT PERCENT
————	B-1	S-1	12.5 - 14.0	Silty, medium to fine SAND.	SM	8
—— ——	B-2	S-9	42.5 - 44.0	Slightly silty, medium to fine SAND.	SP-SM	6
- - - -	B-2	S-12	57.5 - 58.0	Silty fine SAND.	SM	22
——·——	B-3	S-3	12.5 - 14.0	Gravelly, very silty SAND.	SM	8
——···——	B-3	S-7	32.5 - 34.0	Gravelly, very silty, medium to fine SAND.	SM	11

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Figure B-2