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Geotechnical Report
16th Avenue South Bridge
Seattle, Washington

September 1991

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Kirkland, WA 98033-7354

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September 30, 1991

Sverdrup Corporation
3300 Carillon Point
Kirkland, Washington 98033-7354

Attn: Mr. Donald E. Wagner

RE: GEOTECHNICAL REPORT, 16TH AVENUE SOUTH BRIDGE, SEATTLE, WASHINGTON

We are pleased to submit one unbound and two bound copies of our geotechnical report for the above referenced project. This report only addresses the north bascule pier foundation.

Based on the results of two borings, laboratory tests, and our review of existing data and information, we performed engineering studies concerning 1) estimated capacities of existing wood piles, 2) liquefaction evaluation, 3) slope stability, and 4) settlement and soil-structure interaction analysis. On the basis of the engineering studies, recommendations are presented concerning the stabilization and rehabilitation schemes for the north bascule pier foundation.

We appreciate the opportunity to be of service on this interesting and challenging project.

Sincerely,

SHANNON & WILSON, INC.

Ming-Jiun (Jim) Wu, P.E.
Vice President

JW/lkd

Enclosures: Report (3)

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GEOTECHNICAL REPORT
16TH AVENUE SOUTH BRIDGE
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of field explorations and geotechnical engineering studies performed for the proposed stabilization and rehabilitation of the north bascule pier of the 16th Avenue South Bridge located over the Duwamish Waterway in Seattle, Washington. The purpose of the explorations and studies was to identify subsurface conditions and formulate geotechnical recommendations for design and construction of the stabilization and rehabilitation of the north bascule pier.

The work included drilling two borings, laboratory tests, review of existing design and construction data and information, and engineering studies and analyses. Based on the results of the aforementioned scope of work, conclusions and recommendations were formulated for the design and construction of the rehabilitation of the north bascule pier.

The main text of the report includes descriptions of the site and project, field explorations, laboratory testing, subsurface conditions, and our conclusions and recommendations based upon review of existing data and engineering studies and analyses. A description and the results of the field explorations are included in Appendix A. Appendix B includes a description and results of laboratory testing.

Our work was authorized by a subconsultant Agreement between Sverdrup Corporation and Shannon & Wilson, Inc. dated January 25, 1991.

2.0 SITE AND PROJECT DESCRIPTION

The 16th Avenue South Bridge crosses the Duwamish Waterway at the southerly boundary of the Seattle City limits, as shown in Figure 1. The current bridge was constructed during the 1930 and 1931 period to replace an existing structure. The structure consists of a Scherzer Rolling Leaf double-leaf bascule movable-span flanked on each side by two deck-truss approach spans and twelve concrete slab approach spans with retained fills at each end. The overall length of the structure is 1,285 feet, including the two 120-foot-long retained fills. The width of the roadway is 38 feet with 6-foot-wide sidewalks on each side, as indicated in Figure 2.

The cantilevered truss section is balanced by a counterweight consisting of concrete and steel weighing approximately 940 kips.

The bascule span, deck truss approach spans, and other accessories are supported by reinforced concrete piers. Each of the bascule pier foundations are 65.7 feet by 46 feet in plan dimensions and supported by 315 wood piles. The top of the six-foot-thick concrete pier foundations is at elevation -23.98 feet, as shown in Figure 3. Underlying the concrete pier foundation is a 15-foot-thick concrete seal down to elevation -44.98 feet.

The concrete pier foundation was installed within a steel sheet pile cofferdam. Based on the installation records, the existing wood piles were driven with a Vulcan No. 1 steam hammer. The piles supporting the north and south bascule piers were driven to average tip elevations of -97 and -73 feet, respectively, as shown in Figure 3. At the north bascule pier, the wood piles were driven after jetting to about elevation -84 feet. No reason was given why jetting was used to assist pile installation.

The concrete pier walls above the foundation are non-symmetrical and consist of two heavy walls 11 feet wide by 30 feet long in the north-south direction connected by two walls of three feet and four feet wide by 25 feet long in the east-west direction. The heights of the walls are stepped from about 27.3 feet at the south end to 36 feet at the north end.

The existing wood piles supporting the north and south bascule pier foundations were reported to have been originally designed for an allowable load of about 22 tons each. Based on calculations performed by Sverdrup Corporation, however, it is anticipated that the outer piles are carrying an estimated load of about 31 tons, while the interior ones are supporting less load. This is due to the outer piles supporting the major weight of the bridge superstructure. The center portion of the bascule bridge piers contains a hollow space for the counterweight to swing into during opening of the bridge, which reduces the loads on the interior piles.

A review of the report entitled, "16th Avenue South Bridge Investigation Engineering Report" prepared by Sverdrup Corporation in July 1987, indicates that there have been several repairs and modifications to the bridge over the years. Inspections of the bridge made between November 1986 and January 1987 indicated that the bascule piers were generally in fair condition with several cracks above waterline and areas of heavy deterioration near the mudline including cracked and spalled concrete and exposed reinforcement, with the north bascule pier in a more severe condition. The bascule piers appeared to have stabilized with the addition of

the transverse post tensioning in 1982, but the north bascule pier again began binding in 1987 on its rack support, indicating that the condition is still active for at least the north pier.

The scope of work for this project includes primarily the evaluation of the north bascule pier.

3.0 FIELD EXPLORATIONS

The subsurface conditions at the north bascule pier of the 16th Avenue South Bridge were explored by drilling two borings, designated as B-1 and B-2, at approximate locations shown on Figure 2. Both borings were drilled over water through the existing bascule bridge deck. Depths of borings below mudline were 116 and 106 feet for borings B-1 and B-2, respectively, for a total footage of 222 feet. A description of the methods and procedures used for the borings and the logs of the borings are included in Appendix A.

4.0 LABORATORY TESTING

The laboratory testing program was directed toward determining the index and engineering properties and consolidation characteristics of the foundation soils encountered at the north bascule pier of the 16th Avenue South Bridge. The program included, 1) visual classifications and water content determinations on all samples, 2) four (4) Atterberg limits determinations, 3) ten (10) grain-size analyses, 4) one (1) unconfined compression test, and 5) one (1) consolidation test. A description of the test methods and results are included in Appendix B.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the north bascule pier are generalized in a subsurface profile shown in Figure 3. More detailed subsurface conditions are shown on the individual boring logs included in Appendix A. Descriptions of the subsurface conditions encountered are presented in the following.

The mudline elevations measured during drilling were at -19 and -33 feet at borings B-1 and B-2, respectively. Underlying the mudline is a 10- to 15-foot-thick layer of very loose to medium dense, clean to silty, fine to medium SAND with scattered gravel, shells, and organics. The very loose to medium dense sand is underlain by a 30- to 50-foot-thick layer of medium dense to dense, clean to silty, fine to medium SAND with scattered silt layers and organics to about elevation -80 feet. Below this elevation, a layer of very soft, slightly clayey SILT and

sandy SILT with scattered shells and organics was encountered to elevations ranging from about -90 feet at boring B-1 to -85 feet at boring B-2. Underlying the very soft silt is a layer of medium dense, slightly clayey to clayey, silty, fine to coarse SAND with scattered gravel, shells, and organics to approximate elevations of -100 and -92 feet at borings B-1 and B-2, respectively. The aforementioned soils are typical of those alluvial deposits that exist in the Duwamish River Valley.

The alluvial deposits are underlain by glacially consolidated deposits consisting of Weathered Till and Till. The weathered till consists of very stiff, clayey SILT with scattered sand and gravel to approximate elevations of -110 and -101 feet at borings B-1 and B-2, respectively. The weathered till is underlain by a till consisting of hard clayey SILT with traces of sand and gravel to the bottom of borings B-1 and B-2.

Borings were not accomplished at the location of the south bascule pier. A review of the driving records of the existing wood piles indicates that the top of the glacial deposits could be present at an approximate elevation of -73 feet, as shown in Figure 3.

6.0 ENGINEERING STUDIES AND RECOMMENDATIONS

6.1 General

Based on the subsurface conditions encountered in the borings and our review of available data and information including driving records of existing wood piles and the 1987 16th Avenue South Bridge Investigation Engineering Report, engineering studies were performed to evaluate 1) capacities of existing wood piles, 2) liquefaction potential, 3) slope stability, and 4) pier settlements and soil-structure interaction and to provide recommendations for the stabilization and rehabilitation of the north bascule pier. The recommended stabilization and rehabilitation schemes include soil improvement and a new pile or drilled pier foundation.

During our engineering studies, several meetings and telephone discussions were held with the representatives of King County Department of Public Works and Sverdrup Corporation regarding the conditions of the existing bridge and various rehabilitation schemes. During these meetings and discussions, we were provided with the estimated structural loads and other information being considered in the rehabilitation of the north bascule pier. Our engineering studies and resulting recommendations, which are presented in the following sections, are based on the aforementioned information.

6.2 Review of Existing Data and Information

As indicated in the Sverdrup Corporation's 1987 Investigation Engineering Report, the pier supports the weight of the bascule span and the deck truss approach span. The pier also supports the counter-weights, track girders, and rack supports for the rolling lift, and houses the operating machinery, electrical equipment, and operator's control room. Due to the change of the center of gravity of the weight of the bascule movable span during closing and opening, the non-symmetrical shape and height of the wall, and the eccentrically located house for the operating machinery and control room, the loads acting on the pier are non-uniform and quite eccentric. The post-tensioning accomplished in 1982 was to limit further outward movement of pier walls due to cracking of concrete near bascule rack supports and adjacent concrete portions of the piers.

Several possible causes for the cracking had been suspected which include long-term differential settlement due to unequal loading, liquefaction of supporting soils around exterior piling caused by earthquake, intermittent overloading due to opening and closing, and wind load on the bascule span when in the open position. Another possibility is that dredging of the waterway channel resulted in reduced capacity of the exterior piles due to loss of friction and exposing the exterior piles to attack by marine organisms. This may have resulted in differential settlement of the pier foundation and piling deterioration.

Based on the possible causes for the cracking, several options are being considered to repair and improve the bascule bridge. These options include strengthening existing bascule piers to stabilize cracking and settlement, dredging deep enough on one side of pier to expose and determine the condition of timber piling, performing soil borings to confirm settlement parameters, and performing more in depth underwater inspection of piers to locate and patch cracked and spalled concrete.

Most of the aforementioned repair and improvement schemes are related to the structural integrity of the bridge members. From a geotechnical standpoint, two borings were drilled at the north and south of the north bascule pier to evaluate 1) liquefaction potential, 2) slope stability, 3) capacities of existing wood piles, and 4) settlements of the pier foundations. Based on the results of the evaluation and studies, design recommendations are presented to assist in the stabilization and rehabilitation of the north bascule pier.

Tiltmeters were installed at the north and south bascule piers by Sverdrup Corporation to measure the potential tilting and torsional movements of the pier foundations. It is understood that the data monitored to date does not indicate significant pier tilting and movements.

6.3 Estimated Capacities of Existing Wood Piles

As previously mentioned, each of the north and south bascule pier foundations are supported by 315 wood piles with an average tip diameter of 8 inches. The piles were driven with a Vulcan No. 1 steam hammer to average tip elevations of -97 and -73 feet at the north and south pier, respectively, as shown in Figure 3; and they were cut off at elevation -29 feet.

Our review of the pile driving record indicates that the final driving resistances for more than 95 percent of the piles supporting the south bascule pier were greater than 96 blows per foot (bpf). The remaining 5 percent was driven to about 50 bpf. This indicates that all of the piles supporting the south bascule pier were likely driven into glacially consolidated bearing soils. During the construction period in 1930, pull-out tests were performed on two piles which were driven to a final driving resistance of about 40 blows per inch (refusal condition). These piles were failed under a pulling force of about 45 tons. Glacially consolidated soils were observed to be stuck to the pile tip. Based on the results of the pull-out tests at the south pier, the design load of the pile was increased from 22 tons to 30 tons for the north and south bascule piers. Based on the recorded final driving resistances and the results of pull-out tests, it is our opinion that the piles driven at the south bascule pier would have an ultimate compression capacity of 50 tons and tension capacity of 45 tons.

For the north bascule pier, the 315 piles were driven through a pre-jetted hole. Jetting was terminated at about elevation -84 feet; and the piles were driven to an average tip elevation of -97 feet, as shown in Figure 3. The pile tips are generally located in a layer of very stiff clayey SILT ranging from approximately 7 to 15 feet above the top of the Glacial Till. The piles were likely driven through a layer of 2 to 10 feet thick very soft clayey SILT and sandy SILT encountered at about elevation -80 feet, as shown in Figure 3. The final driving resistances defined in inches per blow for the 315 piles supporting the north bascule pier are summarized on Figure 4. As shown, the last hammer blow caused pile penetrations averaging about 1 to 2 inches. About one-third of the piles penetrated less than 1-inch; and about 7 percent of the piles penetrated more than 2 inches and some up to 4 inches. Based on the calculations using the Engineering New Formula and final driving resistances, the estimated allowable pile capacities would range from about 6 to 12 tons which are much less than the required design loads of 22 to 30 tons. Furthermore, the presence of the very soft compressible soil layer could

have caused downdrag acting on the pile down to the bottom of the very soft soil layer. If the downdrag load is ignored and the skin friction above the very soft compressible layer is neglected, the estimated net ultimate pile capacities due to penetration below the compressible layer would be on the order of 12 to 24 tons. This would result in a factor of safety of 1 or less against allowable design loads of 22 to 30 tons per pile. This situation could have occurred sometime during and within about 1 year following construction of the north bascule pier.

For a large 315 pile group supporting a 6 feet thick reinforced concrete mat foundation, a low factor of safety against an uniform foundation loading is not the most critical factor. When the foundation loads exceed the ultimate pile capacities, the pier foundation could have settled much more than anticipated until an equilibrium is reached. However, if the pier foundation is subject to eccentric loading, differential settlements could cause permanent tilting of the foundation. Furthermore, if the outer piles are subject to greater loads than interior piles because of non-uniform superstructure loadings and/or earthquake loadings, large bending moments could be developed and cause cracking of the concrete pier foundation.

As previously mentioned, the existing wood piles could have been marginal factors of safety against allowable design loads of 22 to 30 tons. However, the factor of safety could have increased to greater than 1 sometime following the completion of the bridge construction and when the settlement of the pile group-supported pier foundation exceeded that caused by the presence of the very soft compressible layer encountered at about elevation -80 feet. When this occurred, the negative skin friction above the compressible layer would have reversed to zero or positive skin friction. This could have added an additional skin friction up to 20 to 30 tons to the ultimate compression capacity of the piles.

6.4 Liquefaction Evaluation

An evaluation was made of the liquefaction potential of the soils underlying the north bascule pier of the 16th Avenue South bridge. The evaluation was conducted to determine if there were any links between the observed cracking in the existing north and south bascule piers and any potential zones of liquefaction that may have developed during the historic 1949 or 1965 Puget Sound earthquakes. It was speculated that the potential development of liquefaction beneath the piers could have affected either the horizontal or vertical capacity of the underlying pile foundations, leading to the rocking of the piers during the historic earthquakes and the eventual distress to the piers. Thus, the studies were conducted to determine if there were any extensive zones of potentially liquefiable soils beneath the foundations that may have contributed to the observed distress to the existing bridge foundations.

The liquefaction evaluations were conducted using empirical procedures of Seed (Seed and Idriss, 1982). These empirical correlations are based upon correlations of Standard Penetration Test (SPT) blow count resistance or N-values at sites that have experienced liquefaction during prior earthquakes. Using these empirical correlations, we established plots of minimum N-values which would be needed to result in liquefaction of the soils underlying the bascule piers during the historic 1949 and 1965 earthquakes. Both the 1949 and 1965 earthquakes were recorded upon strong ground motion accelerographs in the Duwamish area. Peak ground accelerations of approximately 0.10g were recorded locally during both of these events. Therefore, the minimum N-value line to resist liquefaction was based upon a historic earthquake (magnitude 7.5) resulting in a ground surface acceleration of 0.10g. These minimum N-value lines were then compared with the actual SPT N-values obtained in the two site borings advanced at the north bascule pier.

The results of liquefaction evaluation are presented in Figures 5 and 6. Figure 5 was prepared based upon a peak ground surface acceleration of 0.10g which corresponds to the accelerations recorded locally during the 1949 and 1965 earthquakes. Figure 6 was prepared based upon a ground surface acceleration of 0.15g. This higher level acceleration was evaluated considering that there may have been some local amplification of ground motion at the site of the bascule bridge during the historic earthquakes. The data presented in Figures 5 and 6 indicate that liquefaction may have occurred within 10 to 15 feet of the mudline (average elevation -25 feet) during either the 1949 or the 1965 earthquakes. Below this level, generally high SPT N-values were observed in both borings which suggest that the underlying soils are relatively stable.

Based upon our review of the data presented in Figures 5 and 6, it is concluded that the upper 10 to 15 feet of soils below the mudline may have liquefied during the 1949 and 1965 earthquakes. This would place the maximum extent of liquefaction at approximately elevation -40. This elevation, however, is still above the base of the pile cap seals in both the north and south bascule piers (elevation -45 feet). Based upon this observation, it is concluded that there is no significant zone of liquefaction adjacent to the piles underlying the piers. The potential development of liquefaction in the soils at the mudline level, however, may have resulted in increased lateral pressures on the bridge abutments which may have resulted in horizontal movements of both of the piers toward the channel. Our review of the historic maintenance records of the bridge would not suggest any major horizontal movements occurred during either the 1949 or 1965 earthquakes.

6.5 Slope Stability

Stability analyses were conducted at the north bascule pier of the 16th Avenue South bridge to determine if the embankment slopes would be unstable during the 1949 or 1965 earthquakes. The presence of a very soft, silty clay layer between elevations -80 and -90 feet prompted the stability studies. The stability studies consider this soft clayey silt layer as having a shear strength of 200 psf. In our opinion, this relatively low value of shear strength will be applicable either during static or dynamic loading conditions. Stability analyses were then conducted using pseudo-static procedures and a seismic coefficient of 0.05g to represent the level of ground shaking which occurred during either the 1949 or 1965 earthquakes. A pseudo-static factor of safety of approximately 1.2 was computed for the slopes adjacent to the abutment for the above conditions. Based upon these analyses, it is concluded that deep-seated slope movement did not take place below the pile cap during either the 1949 or 1965 Puget Sound earthquakes. Therefore, deep-seated slope instability did not have a significant role in the formation of the cracks to the existing bascule piers.

6.6 Estimated Settlements and Soil-Structure Interaction Analysis

6.6.1 General and Loading Conditions

Long-term differential settlements due to unequal loadings are considered to be one of the possible causes for the more heavy cracking of the north bascule pier. Another possible cause under consideration is the reduced capacity of the exterior piles due to potential liquefaction of surrounding soils during a strong motion earthquake and dredging of the waterway channel which could have exposed the exterior piles to attack by marine organisms. These possible causes were considered in the settlement and soil-structure interaction analysis.

Based on the information provided by Sverdrup Corporation, the superstructure and pier wall elements loads were represented by concentrated loads at the four intersee of the bascule pier wall centerlines. Six load combinations under various conditions were provided by Sverdrup Corporation and listed as follows:

Load Combination	Bridge	Tide	Earth-quake	Point Load in Kips Southeast	Southwest	Northeast	Northwest
A	Closed	MLLW	--	4182	3798	3133	2730
B	Closed	MHHW	--	3949	3586	2934	2552
C	Open	MLLW	--	3610	3216	3704	3312
D	Open	MHHW	--	3377	3004	3505	3134
E	Closed	MLLW	N-S	4644	4260	2671	2268
F	Closed	MHHW	E-W	4523	3457	3424	2389

Based on our review of the load combinations and the elastic moduli of the soils under static and transient loading conditions, load combination A was used in our settlement and soil-structure interaction analysis.

6.6.2 Method of Analysis

Because of the large variations of the 1) final driving resistances of the 315 piles, 2) unequal loading, 3) non-symmetrical dimensions of the pier foundation, and 4) different depth intervals between the pile tip elevations and top of the glacial till, the settlement and soil-structure interaction analyses were accomplished by simulating the pier foundation as a series of rectangular grids or cross-beams connected only at their intersecting points, named as nodes. Each nodal point represents one wood pile for a total of 320 points.

The 315 piles supporting the north bascule pier generated most of their capacities from skin friction. Because of this and the presence of the very soft soil layer at about elevation -80 feet, the effective depth of the pier foundation is assumed at elevation -87 feet. The settlement of each node is determined by the contact pressures acting on each node and adjacent nodes using Boussinesq stresses and the elastic theory. The contact pressures are determined by writing and solving a series of finite-difference equations according to the basic theory of flexure of beams at each node using an existing computer program.

The loads acting on each node are determined by the effective area of the node times the buoyant weight of the concrete of the footing including the weight of the concrete seal, the weight of the concrete pier wall above the footing, and the weight of the bridge structures above the pier acting within the effective area of the node.

The load-settlement characteristics of the pile-soil column between the bottom of the concrete seal and the pile tip were determined on the basis of the final driving resistance for each of the 315 piles. The elastic moduli values of the soil layers underlying the pile tip were

determined on the basis of the standard penetration resistance values obtained in the borings and our previous experience with the similar soils.

6.6.3 Results of Analysis

The calculated settlements at each of the 320 nodal points and the settlement contours within the pile cap are presented on Figures 7 and 8. Figure 7 shows the estimated total settlements under the assumed conditions prior to the occurrence of significant cracking in the concrete pier. As shown, the estimated total settlements range from about 1.2 inches at the northwest corner to 2.2 inches at the southwest and southeast corners of the north bascule pier. The resulted differential settlement is approximately 1-inch.

Figure 8 presents the estimated total settlements under the assumed conditions following the occurrence of significant cracking in the concrete pier. The analysis was accomplished by assuming that the outermost rows of piles would retain about 10 percent of their allowable capacity. This assumption would not likely have occurred. However, it provides a reasonable simulation to the situation when the outer piles lost part of their capacities due to liquefaction and/or deterioration caused by dredging. It also provides a reasonable simulation to the condition when the outer piles had yielded because they were subjected to much greater loads than the interior piles due to non-uniform loading acting on the pier foundation. As shown on Figure 8, the estimated total settlements range from about 1.5 inches at the northwest corner to 3.4 inches at the southeast corner. The resulted differential settlement is approximately 1.9 inches.

6.7 Stabilization and Rehabilitation Schemes

6.7.1 Evaluation of Current North Bascule Pier Conditions

Based on the subsurface conditions encountered in the borings, our review of the existing pile driving records, and the estimated settlements, it is our opinion that most of the 315 piles supporting the north bascule pier could have been subjected to loadings much greater than the ultimate capacities of the piles during or shortly after construction. The presence of the very soft compressible soil layer at elevation -80 feet and pre-jetting to elevation -84 feet at every pile location could have destroyed most of the skin friction above about elevation -84 feet. Sometime after the completion of construction, the pile-supported pier foundation could have settled more than the settlement of the surrounding soils caused by the presence of the compressible layer. When this occurred, the potential downdrag load could have diminished and reversed to a neutral position or positive skin friction. It is difficult to estimate how much

positive skin friction could have been added to the ultimate capacities of the piles. The maximum possible values range from 20 to 30 tons. This may make the total ultimate capacities of the existing wood piles ranging from approximately 30 to 50 tons. Because of the non-symmetry of the pier structure and eccentric loadings, Sverdrup Corporation estimated that the outer piles could be carrying 31 tons load. Should this be the case, some of the outer piles may have marginal factor of safety against vertical loads.

Tiltmeters were installed at the north and south bascule piers to measure tilting and torsional movements. It is understood that the instrumentation data recorded to date did not show significant permanent tilting or movements. It is possible that the bascule pier foundations have stabilized under normal operating conditions. It is understood that the King County personnel have indicated that there was not any significant cracking of the bascule piers until the 1965 earthquake. Because of the marginal factor of safety against some of the outer piles supporting the north bascule pier, the north bascule pier may be vulnerable to further cracking during a future strong motion earthquake.

It is understood that the 16th Avenue South Bridge can remain in useful service for another 30 years if proper repairs and improvements are accomplished. From a geotechnical standpoint, the recommended stabilization and rehabilitation schemes include soil improvements and a new pile or drilled pier foundation.

6.7.2 Soil Improvements - Compaction Grouting

As discussed in Section 6.4, the upper loose sand located above elevation -40 feet around the north bascule pier may have liquefied during the 1949 and 1965 earthquakes. These two earthquakes generated a peak ground acceleration of about 0.10g. If future earthquakes are greater than the 1949 and 1965 magnitude, the potentially liquefiable zone would extend deeper than elevation -40 feet. From the foundation support standpoint, it is imperative that liquefaction be mitigated because it would reduce the vertical capacity of the piles and the lateral stability of the pier foundation.

The methods and techniques for soil improvements consist of compaction grouting, vibro-compaction, stone columns, jet grouting, chemical grouting, and others. Based on our review of the subsurface conditions encountered, existing foundation conditions, construction impact, and other factors, it is our opinion that compaction grouting is one of the most suitable soil improvement methods for stabilizing the north bascule pier foundation. This technique uses low-slump cement grout injected under high pressures. Normally, the primary pipes are placed

on a 10-foot grid pattern with the secondary pipes placed in the center of the grid. In addition to the columns thus formed, the high pumping pressure of the dense grout can densify the soils between the columns to a relative density in excess of 75 percent. In addition to providing densification and strengthening of soils, compaction grouting can also be used to increase the bearing capacity of the less competent soils located between the pile tip elevation and the top of the glacially consolidated soils. The key advantage of compaction grouting is that it would not create vibration and cause problems to existing foundation and piling.

Based on our discussions with Hayward Baker Company, the first row of grout columns may be placed about 7 feet from the edge of the pile cap. This spacing would not overstress the existing piles but would adequately densify the soils around the outer piles. It is recommended that the densification zone be extended to at least 40 feet from and around the edges of the entire pile cap. In addition, all grout columns should be extended to the top of the glacially consolidated soils. Some grout columns should also be extended to below the existing pile tip.

If the compaction grouting scheme is adopted, a test program should be performed to 1) determine the grout column spacings, 2) evaluate the pressure/grout volume relationship, 3) measure ground and surface displacements, and 4) define the degree of densification achieved. The test program can be accomplished during the initial phase of the compaction grouting work.

During the compaction grouting process, care must be taken to prevent the grout from seeping into the river channel. Also, a ground movement monitoring program should be developed to prevent disturbance to existing piles and mudline. In order to evaluate the effectiveness of the densification program, standard penetration and cone penetration tests should be performed before and after grouting. The recorded standard penetration resistances after grouting should satisfy the required penetration resistance values presented in Figure 6. Accurate records of pumping time, pressures, volumes, locations, slumps, and other data should be kept by the field representatives and submitted to the owner's engineer for review.

6.7.3 New Pile or Drilled Pier Foundation

As an alternate to compaction grouting, new piles or drilled piers may be installed some distances to the east and west of the existing pile cap to replace the existing wood piles. The new piles or drilled piers should be structurally connected to the existing pile cap. This would require installation of cofferdam during construction.

Installation of drilled piers may require large and heavy equipment depending on the sizes of the drilled piers selected. This may be difficult in a small confined area. However, installation of drilled piers will not create vibration and cause problems to existing foundation and piling. Based on the subsurface conditions encountered, the drilled piers will develop most of their capacities from penetrations into the glacially consolidated hard clayey silt layer. For preliminary design purposes, the recommended allowable unit skin friction in the hard clayey silt layer is 0.75 tons per square foot (tsf) and the allowable end bearing capacity is 20 tsf.

The new pile foundation may consist of prestressed concrete piles, steel pipe piles, or steel H-piles. Based on the subsurface conditions encountered, existing pier foundation conditions, construction impacts, and other factors, it is recommended that steel pipe piles be considered in the preliminary design. Steel pipe piles ranging from 18 to 24 inches in diameter may be designed for allowable loads ranging from 150 to 250 tons. For a close-end 24-inch-diameter steel pipe pile with a wall thickness of 1/2-inch, the estimated penetration into the glacially consolidated soils would be on the order of 30 feet to carry a design load of 200 tons. In order to minimize vibration during pile driving, it is recommended that the close-end steel pipe piles be fitted with a conical tip. The aforementioned information may be used for preliminary design purposes. It is recommended that we be consulted when the pile type and size and design load become available.

7.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If, during construction, subsurface conditions different from those encountered in the exploratory holes are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to evaluate the development of the stabilization and rehabilitation schemes for the north bascule pier to determine if they are consistent with our

recommendations. In addition, we recommend that our firm be retained to observe construction, particularly compaction grouting or installation of steel pipe piles or drilled piers.

This report was prepared for the exclusive use of King County and Sverdrup Corporation in the development of the rehabilitation schemes of the subject facilities. It should be made available to prospective contractors and/or the Contractor for information on factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the boring logs and discussions of subsurface conditions included in this report.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or making test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly designed and constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

SHANNON & WILSON, INC.

Kuo-Hsia Chang
Staff Consultant

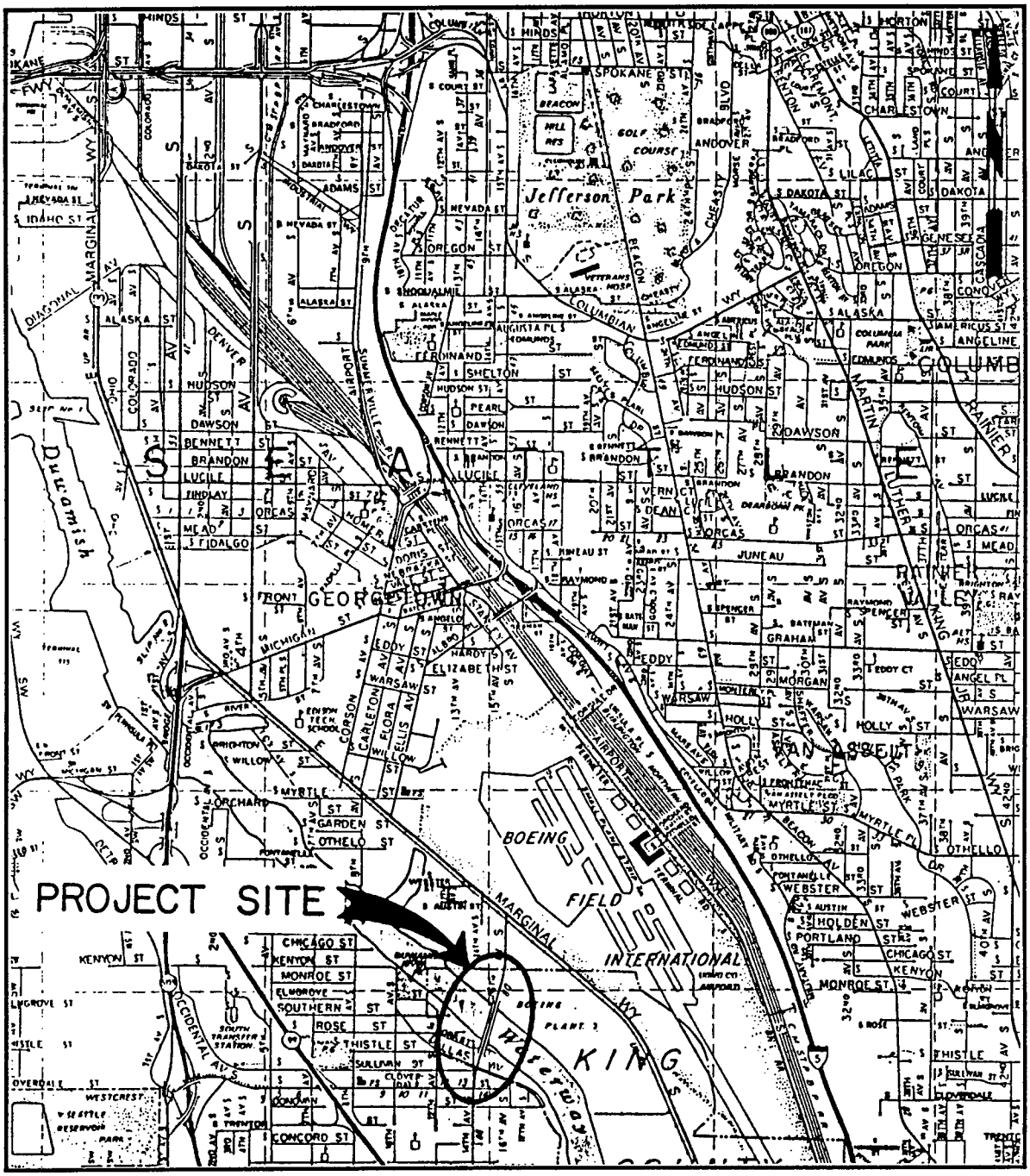
MING JIUN WU
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

9/30/91

Ming-Jiun (Jim) Wu, P.E.
Vice President

KHC:JW/lkd

9-30-91/W5749-01.RPT/JW-lkd/lkd



NOTE

Map reproduced from Sverdrup Corporation's report titled "16th Avenue South Bridge Investigation Engineering Report", dated July 1987.

16th Avenue Soth Bridge
Seattle, Washington

VICINITY MAP

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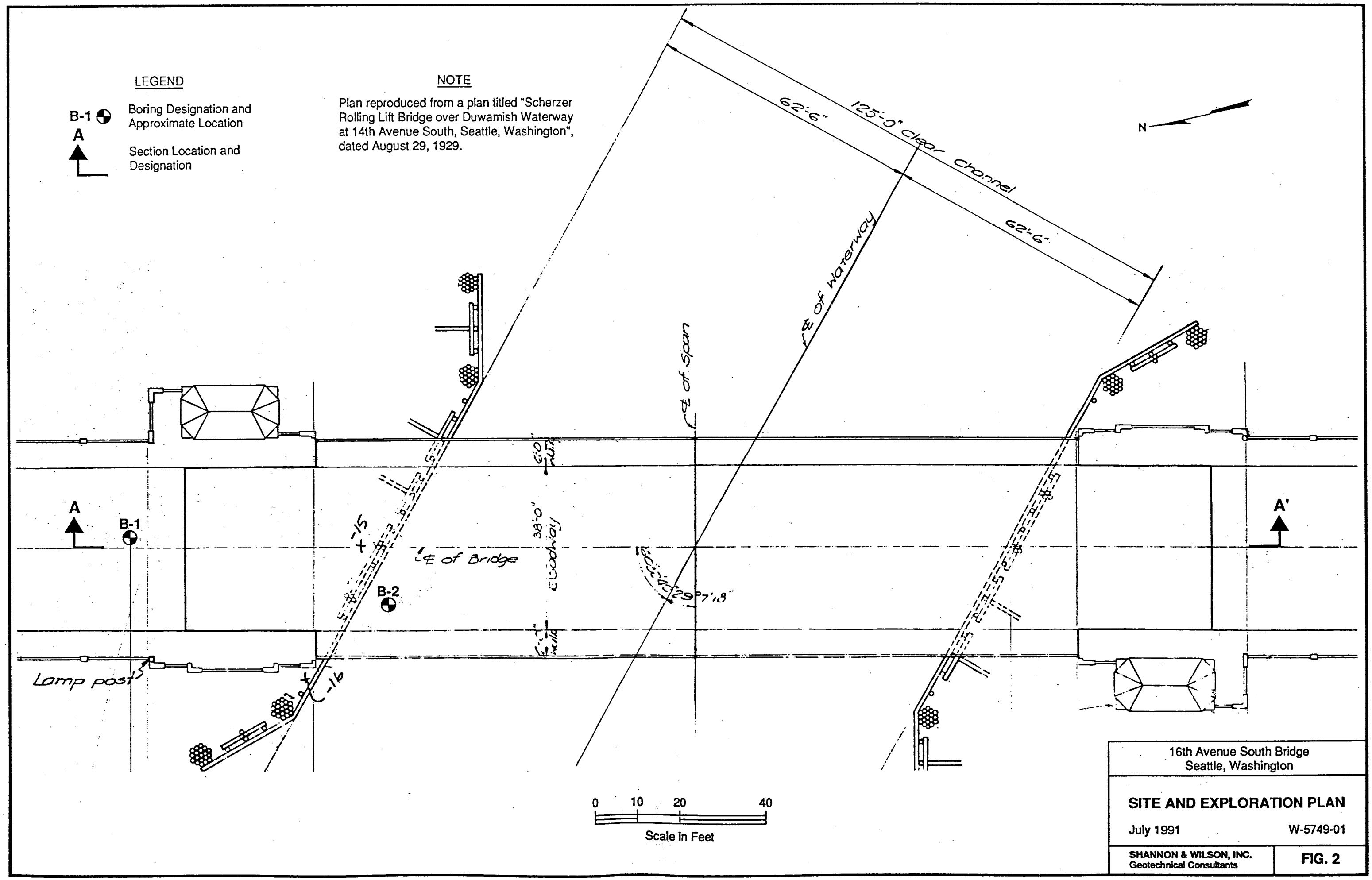
FIG. 1



16th Avenue South Bridge
Seattle, Washington

SITE AND EXPLORATION PLAN

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FIG. 2

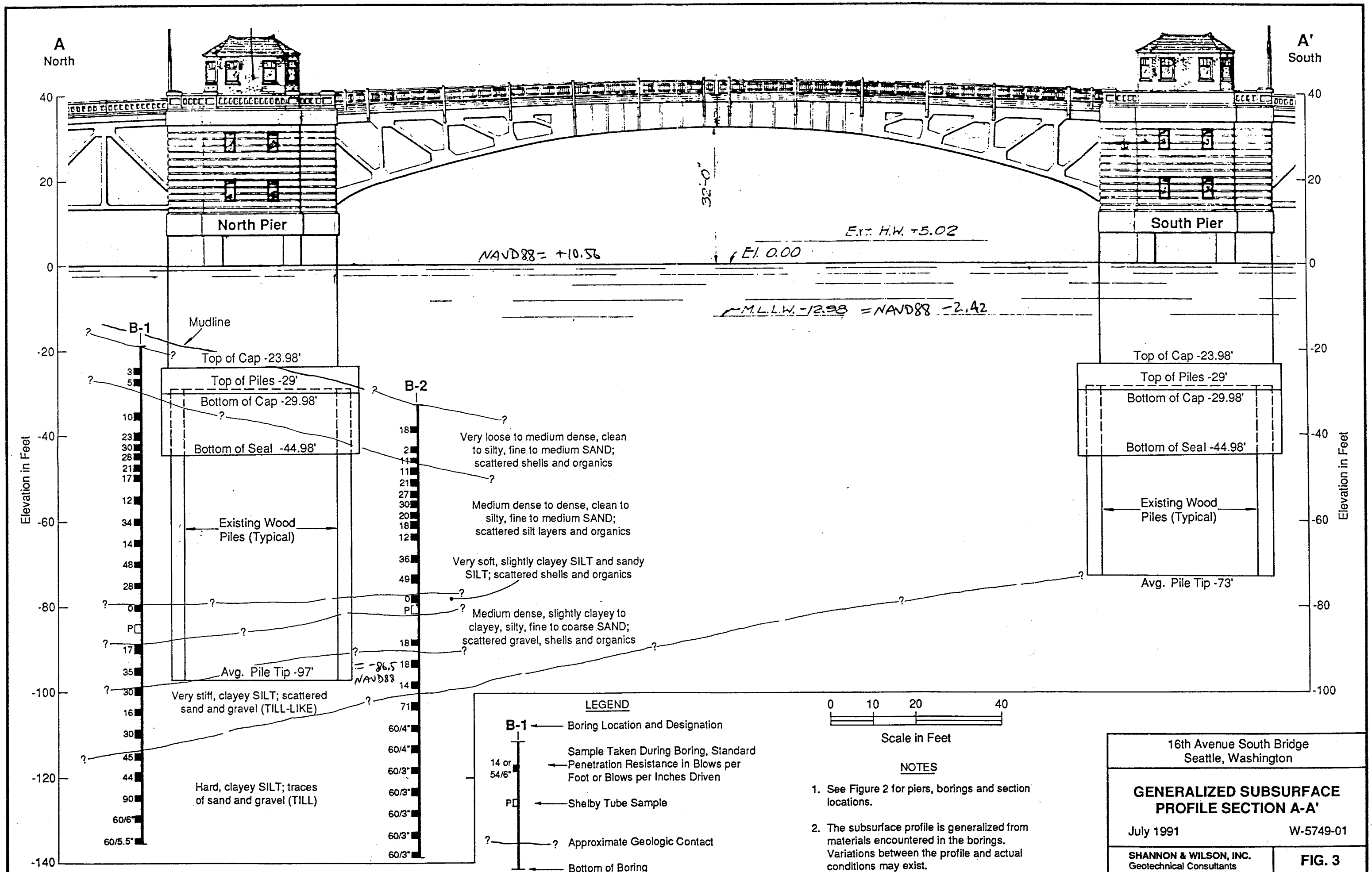
A
North
A'
South
North Pier
South Pier
32'-0"
Ext. H.W. +5.02
NAVD88 = +10.56
El. 0.00
M.L.L.W. -12.98 = NAVD88 -2.42
Elevation in Feet
40
20
0
-20
-40
-60
-80
-100
-120
-140
B-1
Mudline
Top of Cap -23.98'
Top of Piles -29'
Bottom of Cap -29.98'
Bottom of Seal -44.98'
Existing Wood Piles (Typical)
Avg. Pile Tip -97'
B-2
Top of Cap -23.98'
Top of Piles -29'
Bottom of Cap -29.98'
Bottom of Seal -44.98'
Existing Wood Piles (Typical)
Avg. Pile Tip -73'
Very loose to medium dense, clean to silty, fine to medium SAND; scattered shells and organics
Medium dense to dense, clean to silty, fine to medium SAND; scattered silt layers and organics
Very soft, slightly clayey SILT and sandy SILT; scattered shells and organics
Medium dense, slightly clayey to clayey, silty, fine to coarse SAND; scattered gravel, shells and organics
Very stiff, clayey SILT; scattered sand and gravel (TILL-LIKE)
Hard, clayey SILT; traces of sand and gravel (TILL)
= -86.5 NAVD88
LEGEND
B-1 Boring Location and Designation
14 or 54/6" Sample Taken During Boring, Standard Penetration Resistance in Blows per Foot or Blows per Inches Driven
P Shelby Tube Sample
Approximate Geologic Contact
Bottom of Boring
0 10 20 40
Scale in Feet
NOTES
1. See Figure 2 for piers, borings and section locations.
2. The subsurface profile is generalized from materials encountered in the borings. Variations between the profile and actual conditions may exist.
16th Avenue South Bridge
Seattle, Washington
GENERALIZED SUBSURFACE PROFILE SECTION A-A'
July 1991
W-5749-01
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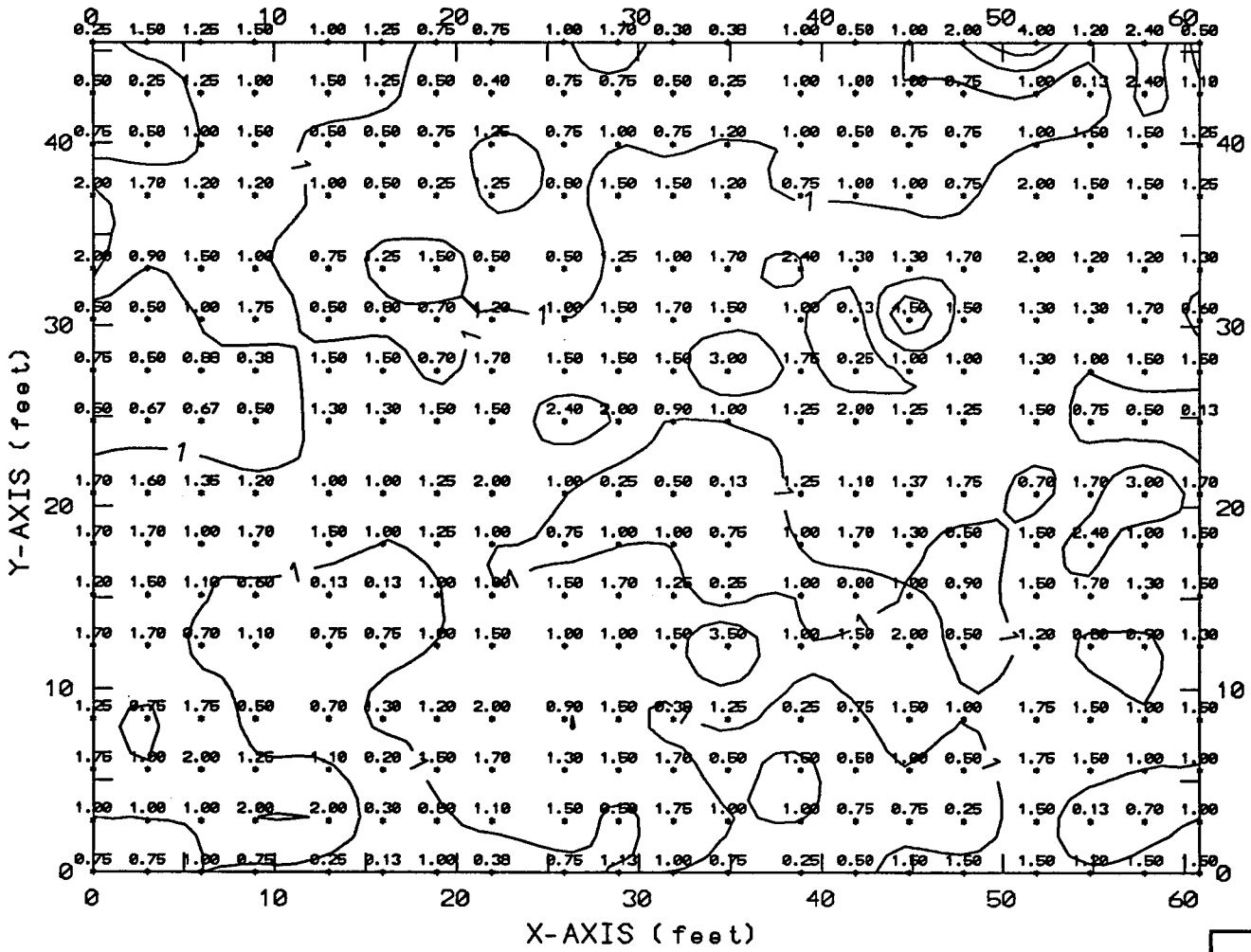
FIG. 3



NOTE

Driving resistance is shown at each pile in inches per blow. The contour interval is 1 inch per blow.

16th Avenue South Bridge
Seattle, Washington

FINAL DRIVING RESISTANCE AT NORTH BASCULE PIER

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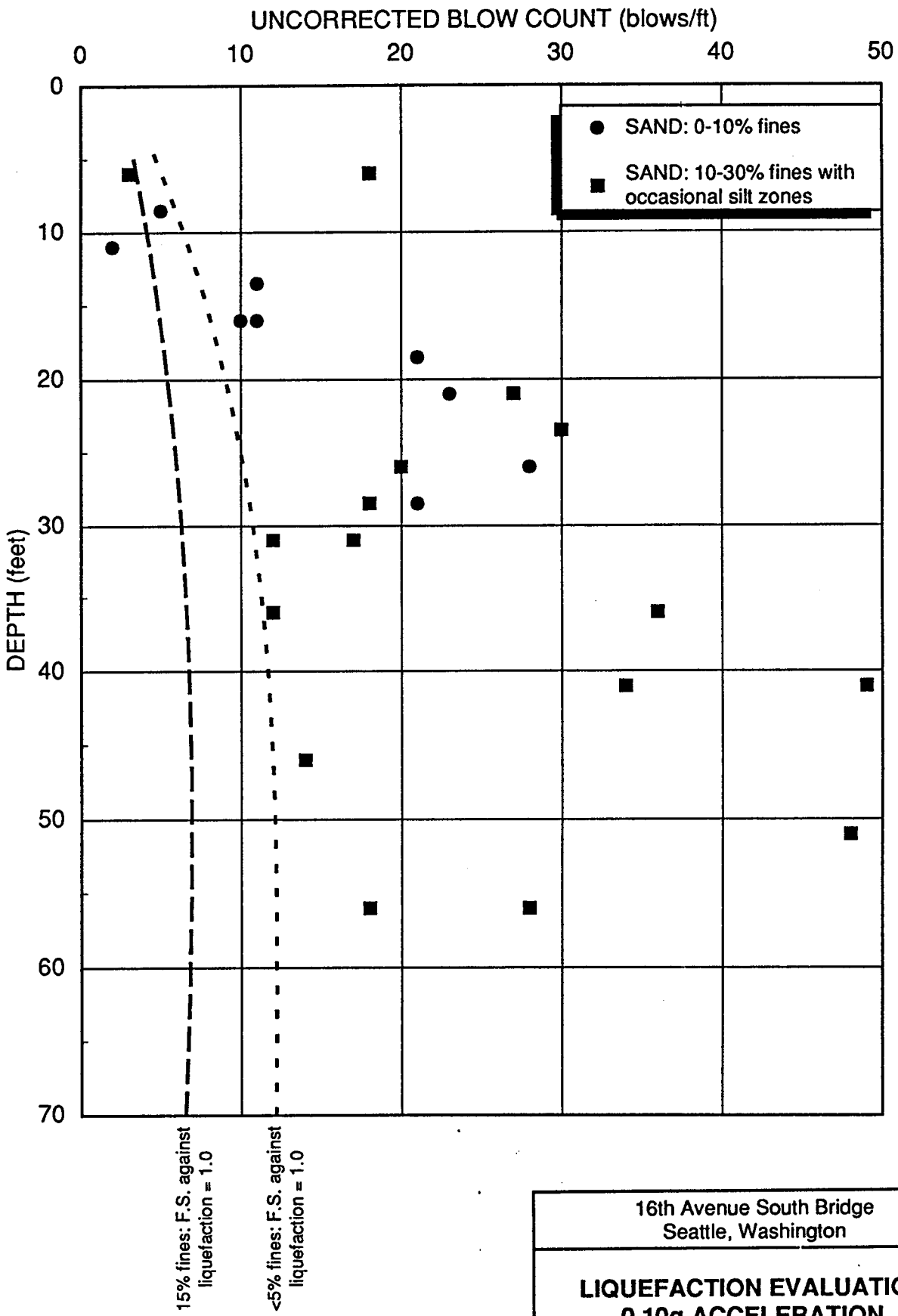
FIG. 4

FIG. 4



16th Avenue South Bridge
Seattle, Washington

LIQUEFACTION EVALUATION
0.10g ACCELERATION

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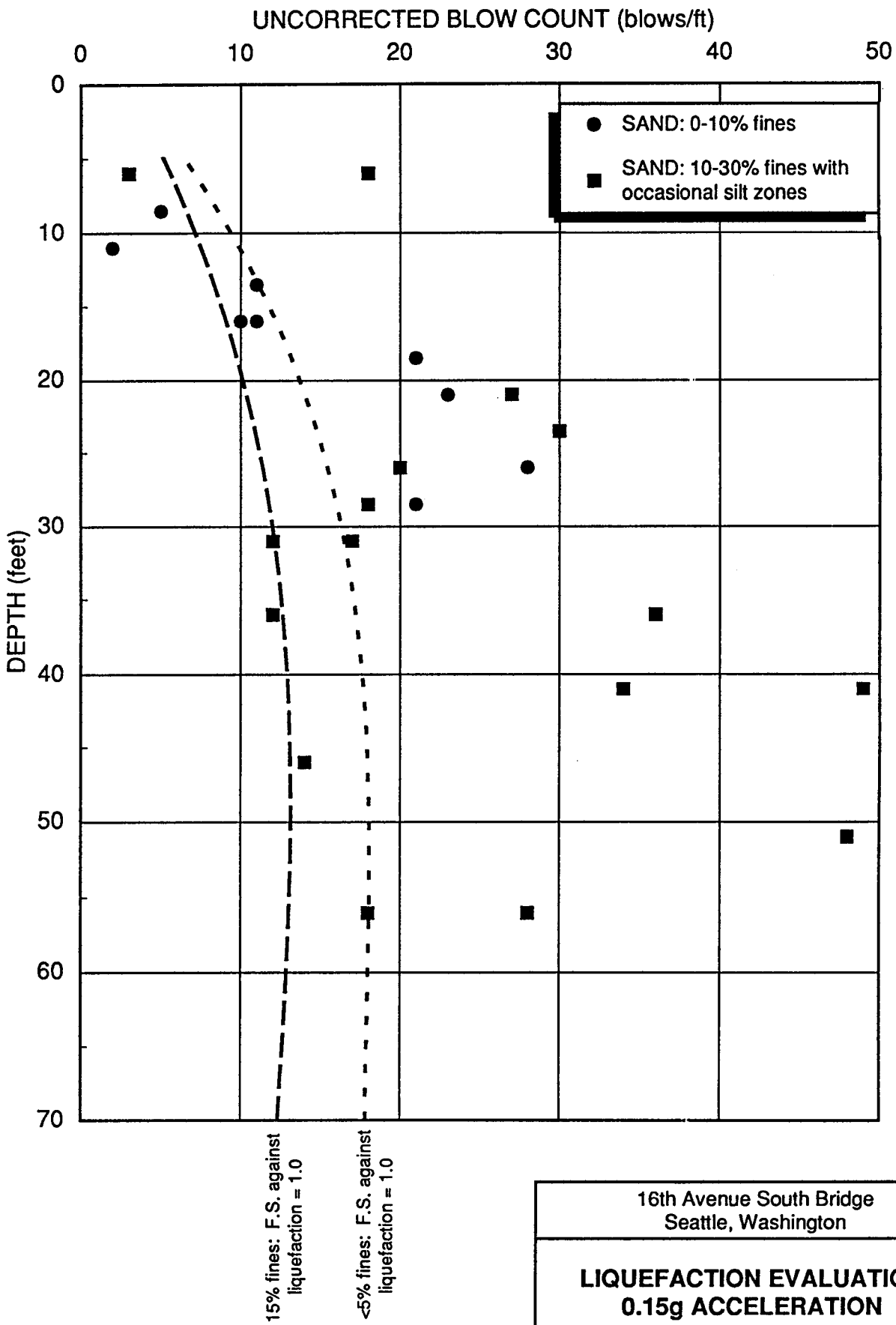
FIG. 5



16th Avenue South Bridge
Seattle, Washington

LIQUEFACTION EVALUATION
0.15g ACCELERATION

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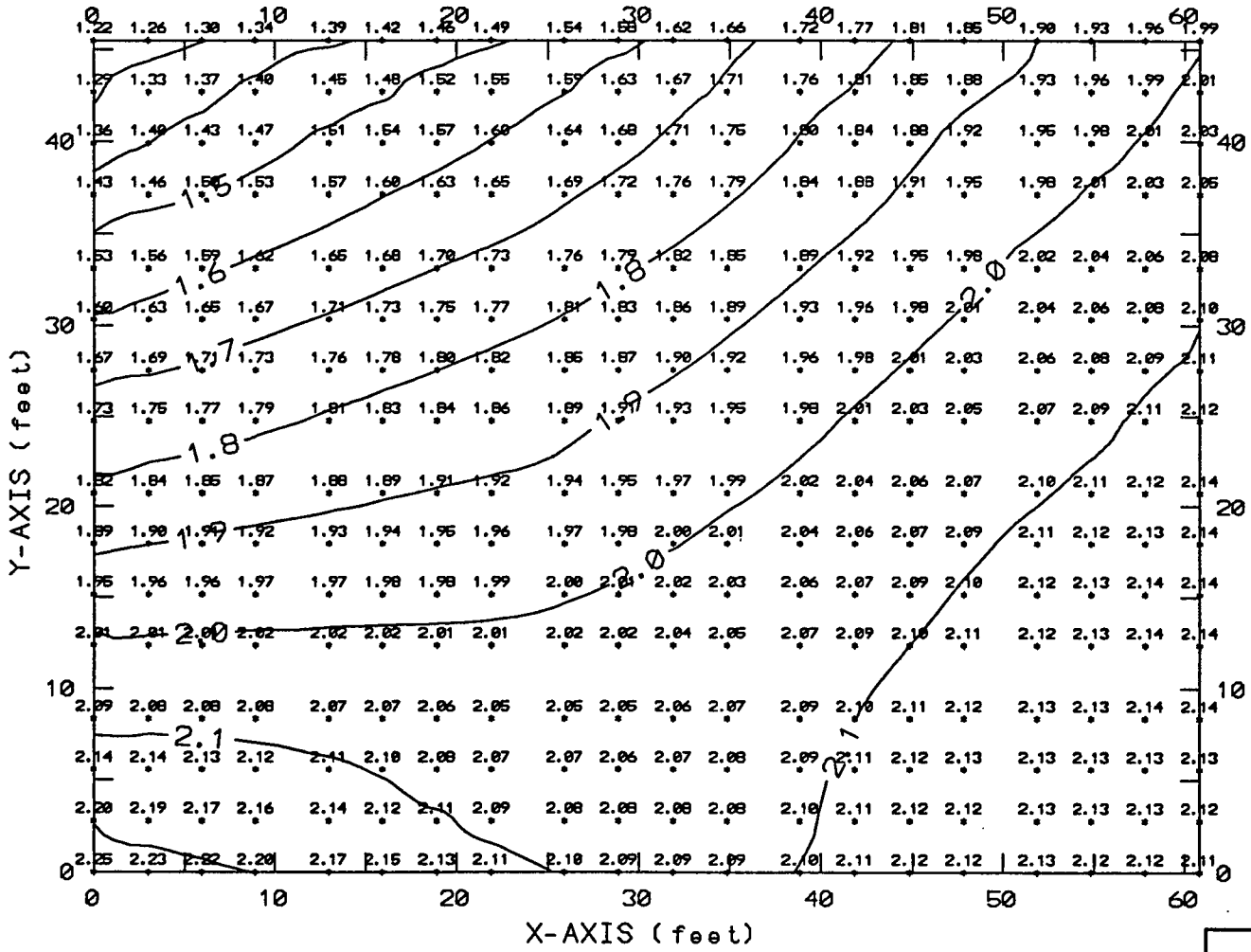
FIG. 6



NOTE

Settlement is shown at each pile in inches. The contour interval is 0.1 inch.

16th Avenue South Bridge
Seattle, Washington

ESTIMATED TOTAL SETTLEMENT
PIER NOT CRACKED

July 1991 W-5749-01

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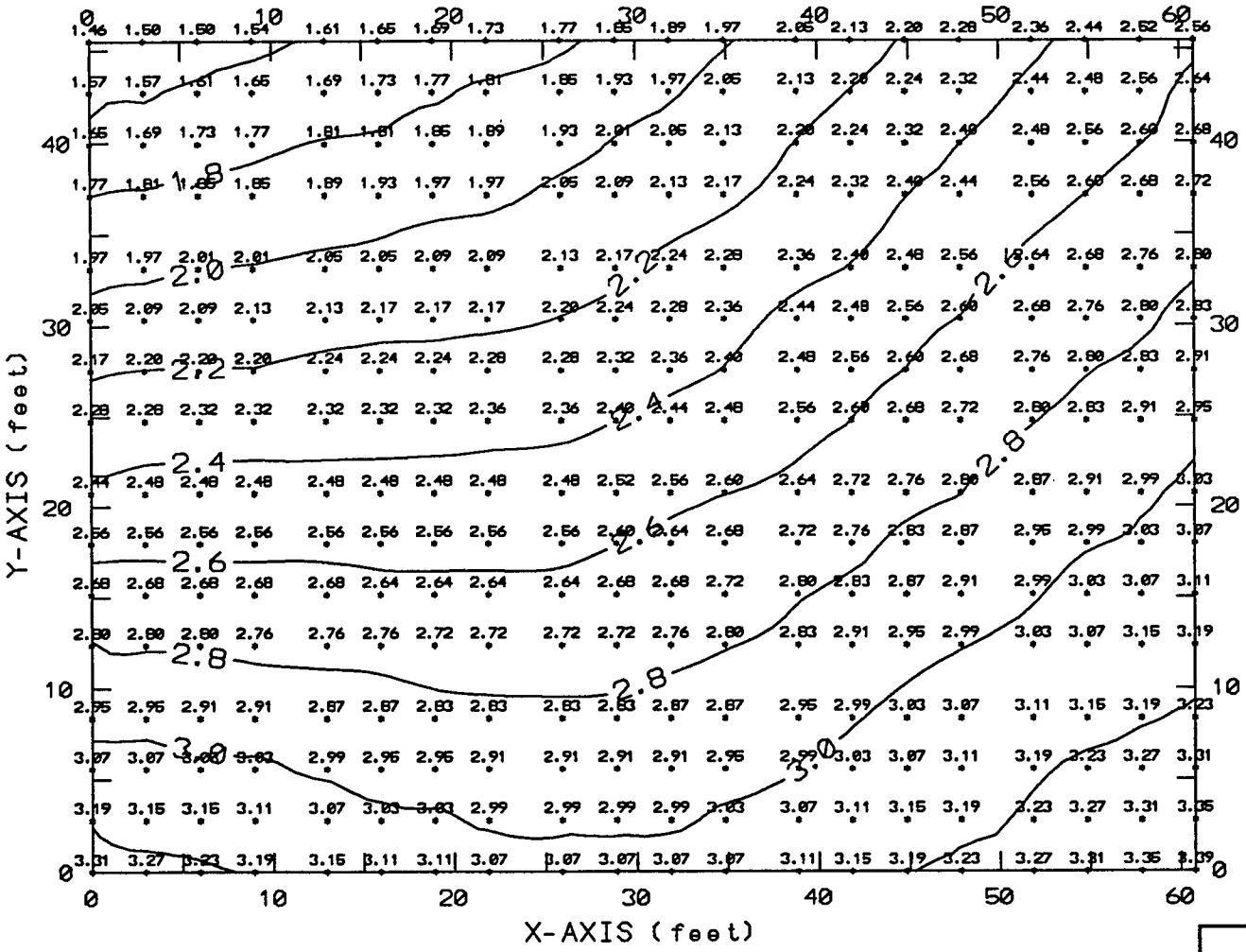
FIG. 7

FIG. 7



NOTE

Settlement is shown at each pile in inches. The contour interval is 0.2 inch.

16th Avenue South Bridge
Seattle, Washington

ESTIMATED TOTAL SETTLEMENT PIER CRACKED

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FIG. 8

FIG. 8

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

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APPENDIX A

FIELD EXPLORATIONS

A.1 INTRODUCTION

The subsurface conditions at the north bascule pier of the 16th Avenue South Bridge were explored by drilling two borings, designated as B-1 and B-2, at approximate locations shown on Figure 2. Both borings were drilled over water through the existing bascule bridge deck. This appendix contains descriptions of the general procedures performed for, and the logs of, the explorations.

The approximate locations of the borings were obtained by a Shannon & Wilson field representative by taping from existing features. The elevations of the borings at the bridge deck were surveyed by the King County crew. The mudline elevations at the boring locations were obtained by measuring the distances between the bridge deck and the mudline and the elevations at the bridge deck. The elevations indicated are based on the datum shown in Figure 3.

A.2 DRILLING AND SAMPLING

A.2.1 Drilling Procedures

The borings were drilled using a truck-mounted, modified Failing 1500 rotary drill rig provided by Donald A. Kenner Drilling of Oregon, Inc., of Sherwood, Oregon. All borings were advanced using a 4-7/8-inch tricone bit and fish tail. A 5-1/4-inch O.D. casing was used as necessary to prevent excessive loss of drilling mud and to stabilize slough soils encountered in the borings. The lengths of casing required were about 25 and 35 feet below mudline in borings B-1 and B-2, respectively. The rotary drilling procedure consisted of drilling the formation materials and removing the cuttings by circulation of drilling mud. The cuttings were deposited in a settling tank at the ground surface. The drilling mud used was a mixture of baroid-zeogel (bentonite) and baroid-barite weighting additive combined with water.

Boring B-1 was drilled and sampled on January 26 and 27, 1991 to a depth of 116 feet below mudline. Boring B-2 was drilled and sampled on February 9 and 10, 1991 to a depth of 106 feet below mudline. All drilling and sampling operations were observed by an experienced engineer from our firm. The engineer also collected and classified samples and prepared field boring logs.

Disturbed samples were retrieved in conjunction with the Standard Penetration Resistance Test. Relatively undisturbed samples were obtained with thin-wall tube sampling techniques.

The retrieved samples were classified by our field engineer, sealed in jars or tubes, and returned to our laboratory for further testing.

A.2.2 Sampling Procedures

A.2.2.1 Standard Penetration Test Samples

Disturbed samples were retrieved from the borings in conjunction with the Standard Penetration Test (ASTM Designation: D 1586). The Standard Penetration Test involves driving a 2-inch O.D., split-spoon sampler 18 inches using a 140-pound hammer, free-falling 30 inches. The number of blows required to drive the sampler 18 inches in 6-inch increments is recorded. The number of blows required for the last 12 inches of penetration is termed the Standard Penetration Resistance. When 50 blows were required for 6 inches or less of penetration, the test was stopped and the number of blows and the corresponding penetration recorded. The Standard Penetration Resistance values are plotted on the boring logs, Figures A-1a through A-2b. These values provide a means for evaluating the relative consistency (stiffness) of cohesive soils and the relative compactness or density of granular soils.

For describing the consistency of fine-grained, cohesive soils, the following terminology was used:

Relative Density	Standard Penetration Resistance or N-Value (blows/foot)
Very soft	Less than 2
Soft	2-4
Medium	4-8
Stiff	8-15
Very stiff	15-30
Hard	Over 30

For describing the consistency of fine-grained, cohesive soils, the folloiwng terminology was used:

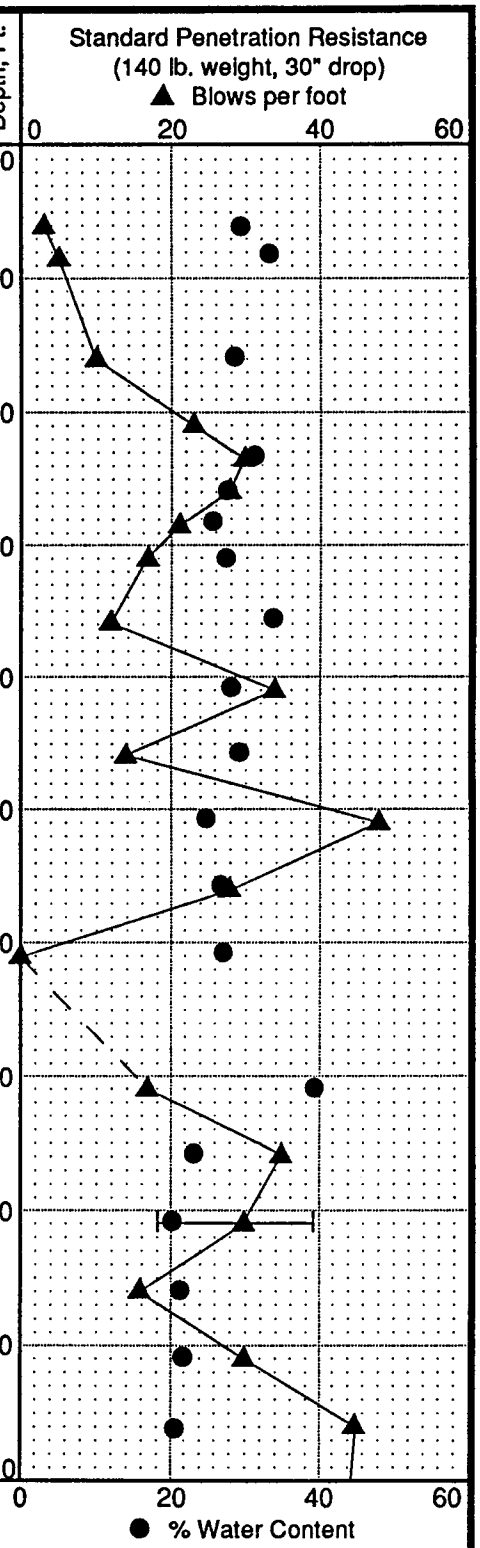
Relative Density	Standard Penetration Resistance or N-Value (blows/foot)
Very loose	0-4
Loose	4-10
Medium dense	10-30
Dense	30-50
Very dense	Over 50

A.2.2.2 Thin-Wall Tube Samples

Relatively undisturbed thin-wall steel tube samples were obtained in accordance with ASTM Designation: D 1587-67. This sampling method employs a thin-wall steel tube connected to a sampling head that is attached to drill rods. The tube is pushed by the hydraulic ram of the drill rig into the soil below the bottom of a drilled hole and then retracted to obtain a sample.

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples
Very loose to loose, dark gray, slightly silty to silty, fine to medium SAND; wet with some gravel (Organic smell at 7.5')		1, 2
Medium dense, dark gray, clean to silty, fine to medium SAND; wet with zones of silt	10	3, 4*, 5, 6, 7, 8
Medium dense to dense, dark gray, slightly silty to silty, fine SAND; moist to wet with layers and zones of silt	35	9, 10, 11, 12, 13
Very soft, gray, slightly clayey, fine to medium sandy SILT; moist with shells and scattered gravel	60	14, 15
Medium dense, gray, slightly clayey to clayey, silty fine to coarse SAND; moist, with shells and scattered gravel	70	16, 17
Very Stiff, gray-brown, clayey SILT; moist with scattered coarse sand and a trace of gravel (TILL-LIKE)	80	18, 19
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	90	20, 21

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-1

February 1991 W-5749-01

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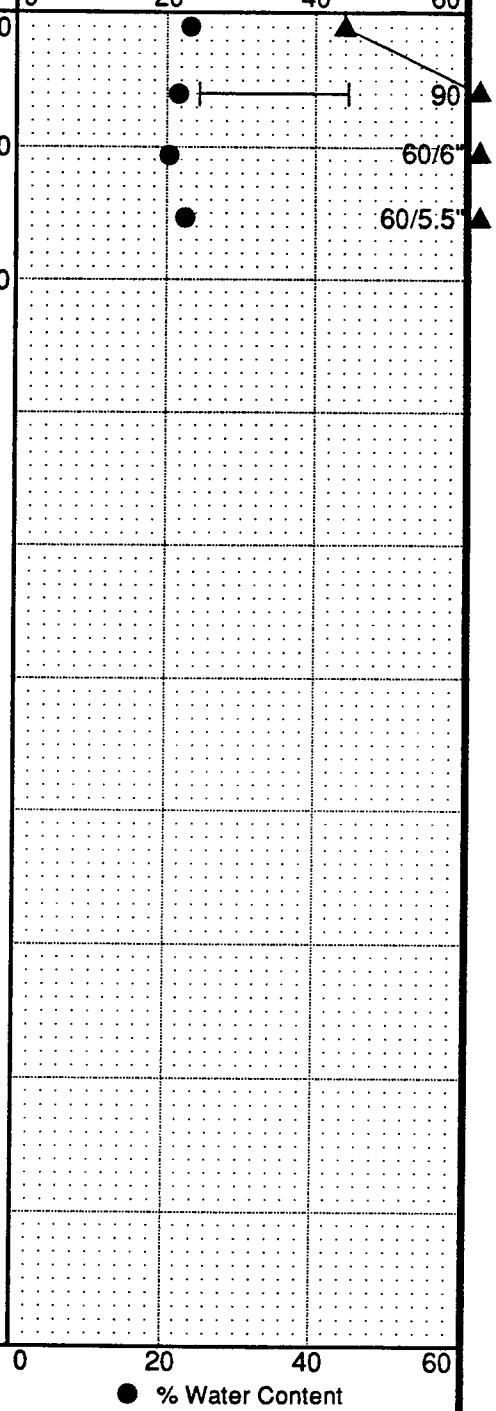
FIG. A-1a

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	100	22		100
		23		
		24		110
	116	25		
BOTTOM OF BORING COMPLETED 1-27-91				120



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-1

February 1991 W-5749-01

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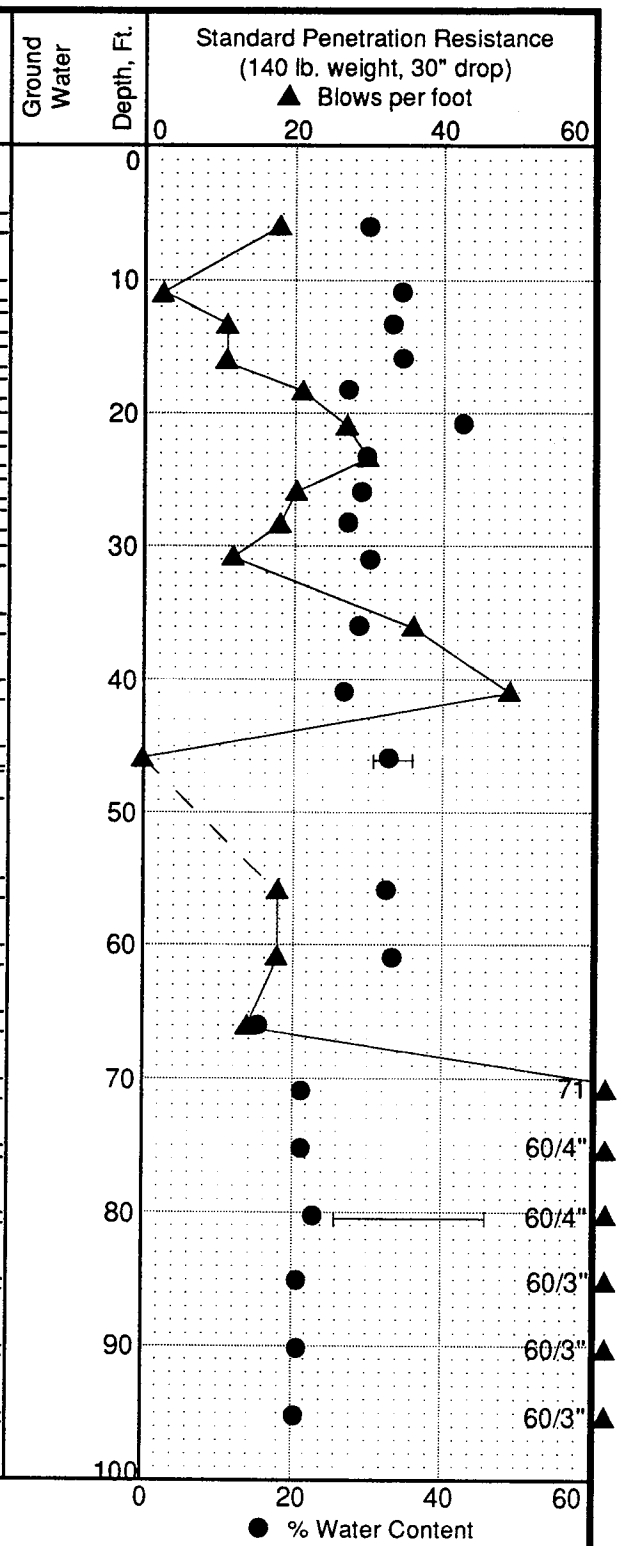
FIG. A-1b

SOIL DESCRIPTION Mudline Elevation: Approx. -33 Feet (See Fig.3)	Depth, Ft.	Samples
Very loose to medium dense, dark gray, clean to silty, fine to medium SAND; wet with some shells and organics (Organic smell)		S-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8, S-9, S-10, S-11, S-12
Medium dense to dense, dark gray, slightly silty to silty, fine to medium SAND; wet with scattered organics and layers and zones of silt	15	
Very soft to soft, gray-brown, silty CLAY; moist to wet with a trace of sand and organics	45	S-13, S-14
Medium dense, gray, slightly clayey, silty, fine to coarse SAND; moist to wet with gravel, organics and shells	50	S-15
Very stiff, light brown and gray, clayey SILT; moist with laminated layers of: silt, sand and gravel	58	S-16
Stiff, gray-brown, gravelly, fine to coarse sandy, clayey SILT; moist (Till-Like)	63	S-17
Hard, gray, clayey SILT; dry with a trace of sand (Till)	68	S-18, S-19, S-20, S-21, S-22, S-23

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue Bridge
Seattle, Washington

LOG OF BORING B-2

February 1991 W-5749-01

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FIG. 2a

SOIL DESCRIPTION Mud Line Elevation: Approx. -33 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water	Depth, Ft.	Standard Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot (0 20 40 60)
Hard, gray, clayey SILT; dry with a trace of sand (TILL)	100	24		100	60/3" ▲
	105.7	25			60/3" ▲
BOTTOM OF BORING COMPLETED 2-10-91				110	

0 20 40 60
● % Water Content

LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-2

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FIG. A-2b

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

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B.4	ATTERBERG LIMITS DETERMINATIONS	B-1
B.5	GRAIN-SIZE ANALYSIS	B-2
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B-5	Grain-Size Distribution, Boring B-2
B-6	Grain-Size Distribution, Boring B-2
B-7a & 7b	Unconfined Compression Test No. QU-1, Boring B-2, Sample S-14
B-8a thru 8f	One-Dimensional Consolidation Test No. C-1, Boring B-2, Sample S-14

APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

B.1 INTRODUCTION

This appendix contains descriptions of the procedures and the results of laboratory tests performed on soil samples obtained from the test borings performed for the 16th Avenue South Bridge Project. The samples were tested to determine the basic index and engineering properties and consolidation characteristics of the foundation soils.

All laboratory testing was performed at the Shannon & Wilson, Inc. laboratory in Seattle, Washington.

B.2 VISUAL CLASSIFICATION

All soils samples obtained from the test borings were visually classified in our laboratory using a system based on ASTM[1] Designation: D 2487-69, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D 2488-69, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The individual sample classifications are incorporated in the exploration logs, Appendix A.

B.3 WATER CONTENT DETERMINATION

The water content of all soils samples obtained from the test borings was determined in general accordance with ASTM Designation: D 2216-17, Standard Method of Laboratory Determination of Moisture Content of Soil. Water contents are shown on the boring logs, Appendix A.

B.4 ATTERBERG LIMITS DETERMINATION

The Atterberg limits were determined on selected samples in general accordance with ASTM Designation: D 423-66, Standard Test Method for Liquid Limit of Soils, and ASTM

[1] American Society for Testing and Materials, "Natural Building Stones; Soil and Rock," 1980 Annual Book of Standards, Part 19.

Designation: D 424-59, Standard Test Method for Plastic Limit and Plasticity Index of Soils. The results are shown on the boring logs and are presented graphically on Figure B-1.

B-5 GRAIN-SIZE ANALYSIS

The grain-size distribution of selected samples was determined in general accordance with the Department of the Army Manual of Laboratory Soils Testing, Appendix V: Grain-Size Analysis.[2] Only sieve analyses were performed.

Results of these analyses are plotted on the grain-size distribution curves, Figures B-2 through B-6. Along with each grain-size distribution is a summary containing group symbol, sample description, and natural water content.

B.6 UNCONFINED COMPRESSION TEST

One unconfined compression test was performed on a selected undisturbed sample in general accordance with ASTM Designation: D 2166-66, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The test results are presented on Figures B-7a and B-7b.

B.7 ONE-DIMENSIONAL CONSOLIDATION TEST

A one-dimensional consolidation test was performed on one undisturbed sample obtained in Boring B-2 in general accordance with ASTM Designation: D-2435-70, Standard Test Method for One-Dimensional Consolidation Properties of Soils.

The specimen was incrementally loaded using a floating ring consolidometer, with each load increment approximately doubling the previous load. Drainage was allowed from both the top and bottom of the specimens. The specimen was inundated with water after the first load increment. Additional load increments were applied at the end of primary consolidation or each day. Upon reaching the maximum normal stress, the specimen was unloaded in increments of about one-fourth of the previous load.

The results of the consolidation test are presented on Figures B-8a through B-8f. The results presented include summary of test data and plots of percent compression versus logarithmic

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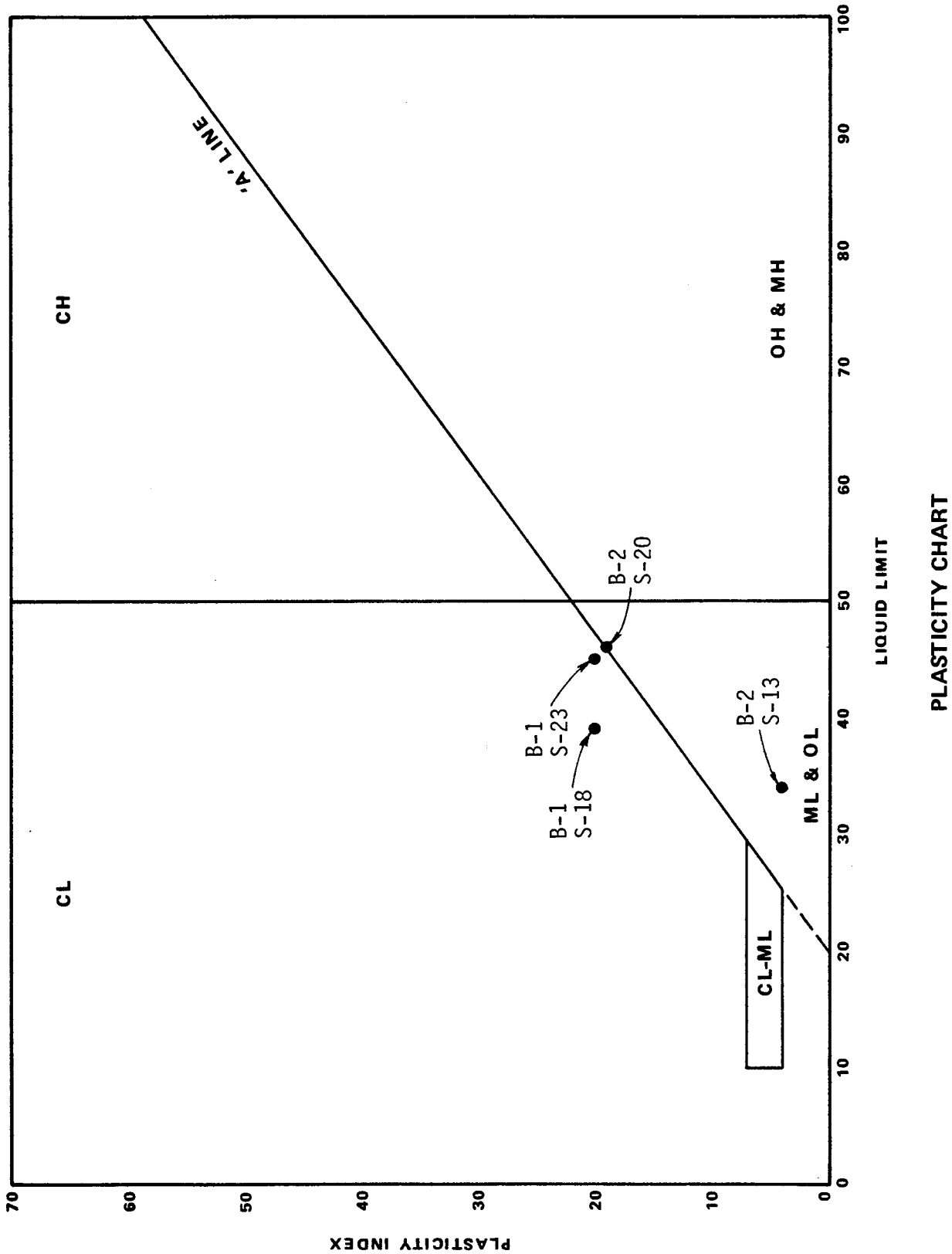
[2] Department of the Army, Engineering and Design, "Laboratory Soils Testing," Engineering Manual EM 1110-20-1960, 30 November 1970.

stress, void ratio versus logarithmic stress, and settlement versus logarithmic time for each load increment and decrement.



PLASTICITY CHART

16th Avenue South Bridge
Seattle, Washington

PLASTICITY CHART

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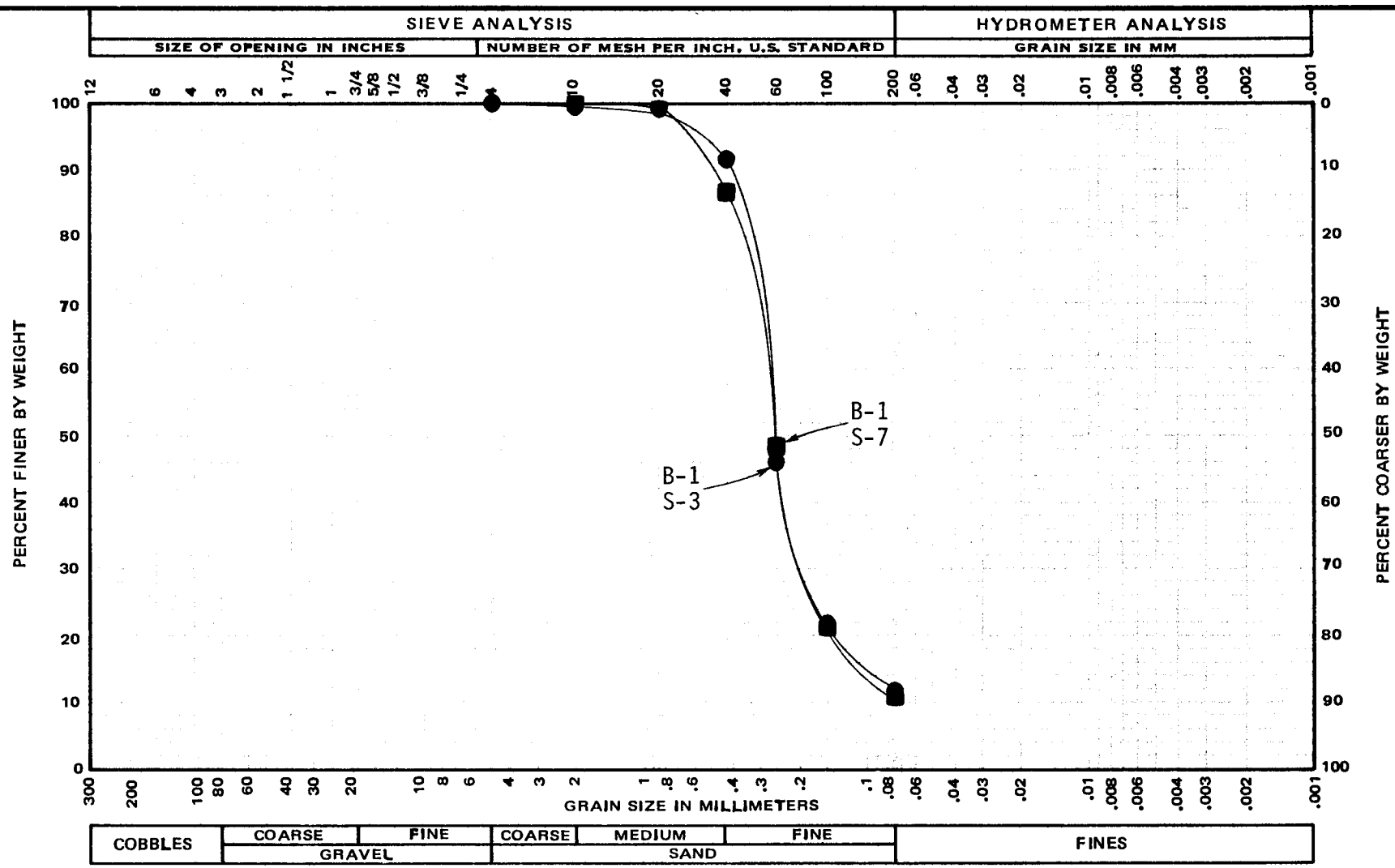
FIG. B-1



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-1 S-3	15.0-16.5	SP-SM	Dark gray-brown, slightly silty SAND.	28.3			
B-1 S-7	27.5-29.0	SP-SM	Dark gray-brown, slightly silty SAND.	25.7			

16th Avenue South Bridge
Seattle, Washington

GRAIN SIZE DISTRIBUTION

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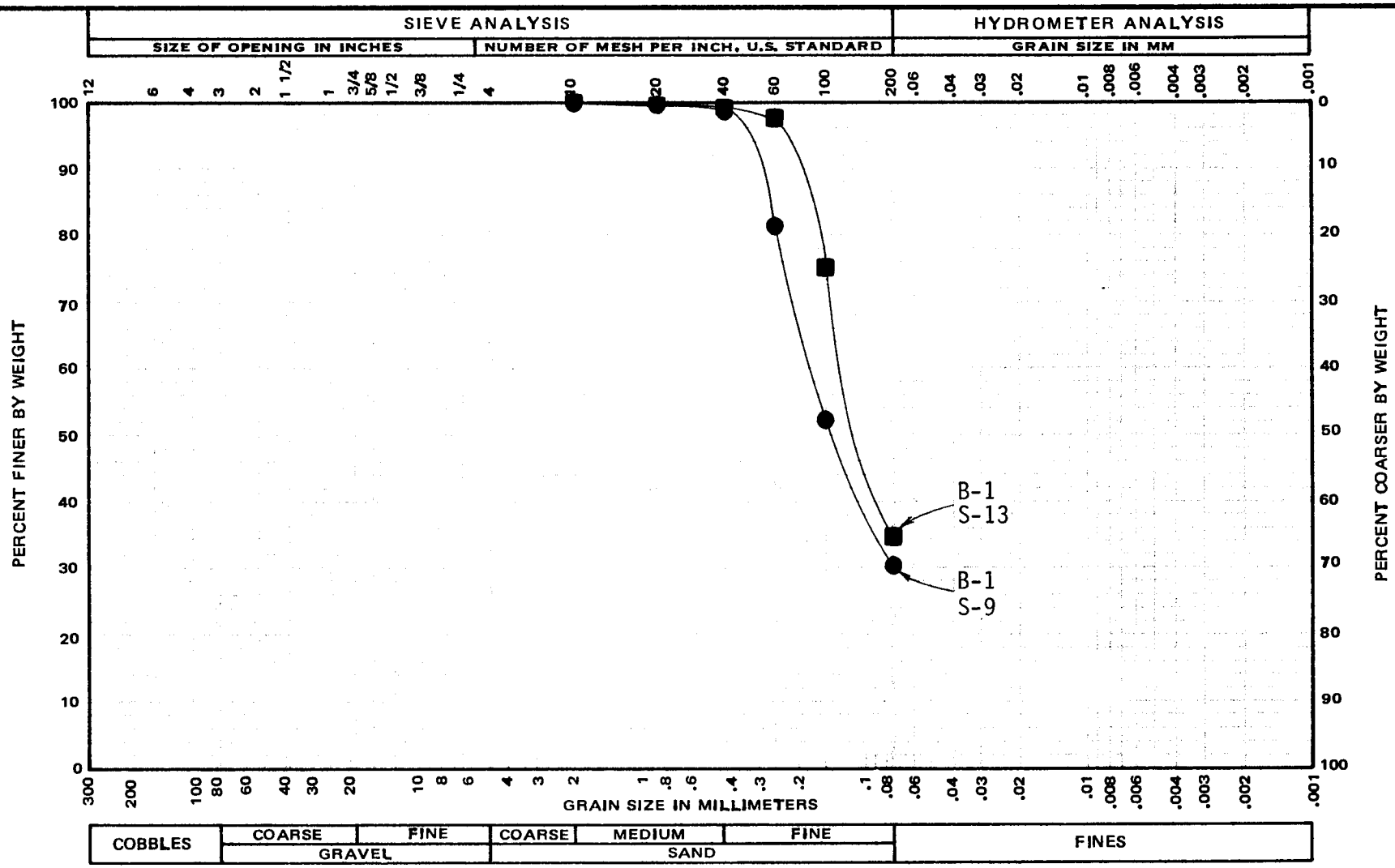
FIG. B-2

FIG. B-2



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-1 S-9	35.0-36.5	SM	Dark gray-brown, silty SAND.	33.9			
B-1 S-13	55.0-56.5	SM	Dark gray-brown, silty SAND.	26.8			

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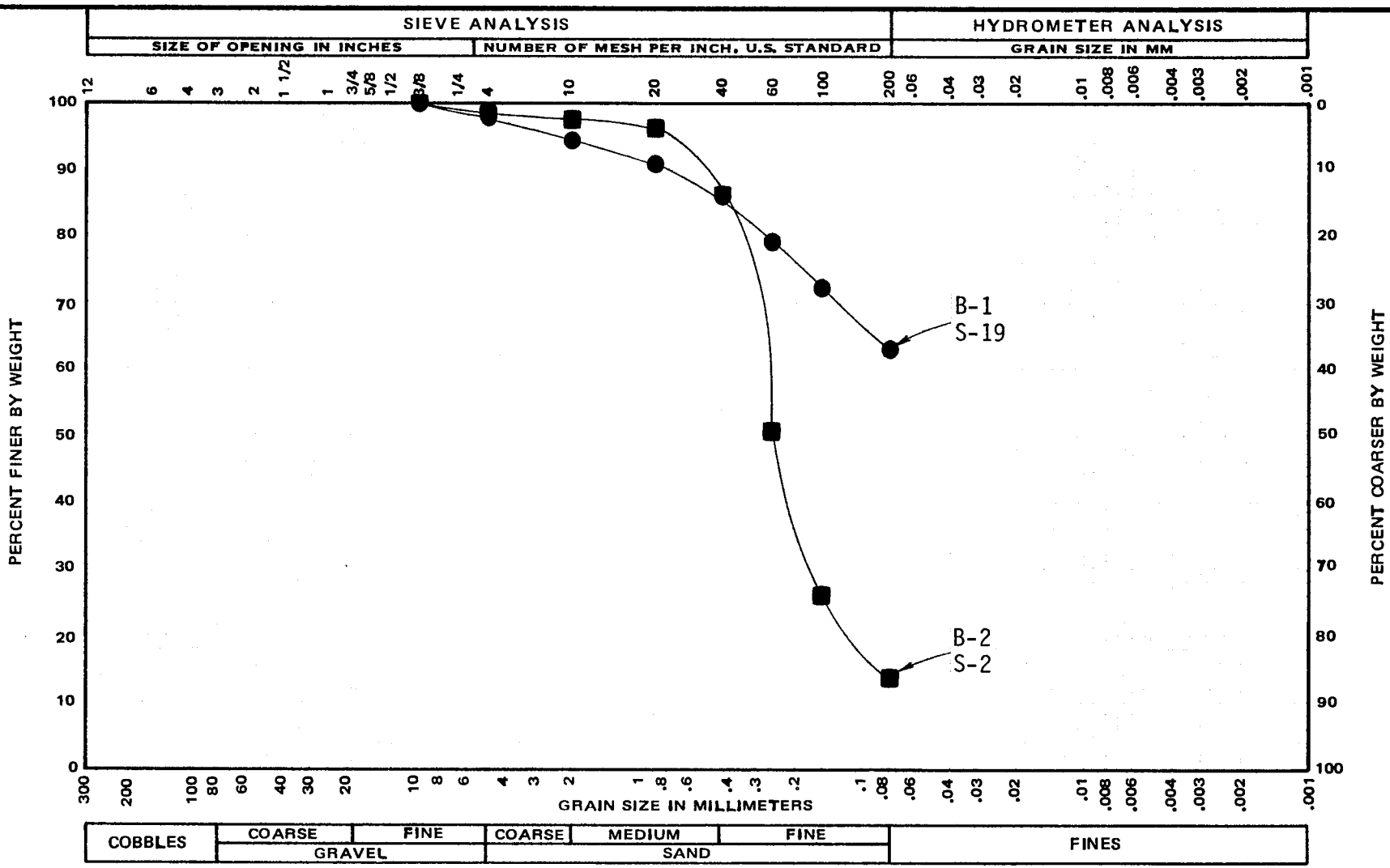
FIG. B-3

FIG. B-3



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-1 S-19	85.0-86.5	ML	Brown, sandy, clayey SILT.	21.3			
B-2 S-2	10.0-11.5	SM	Dark gray, silty SAND.	34.7			

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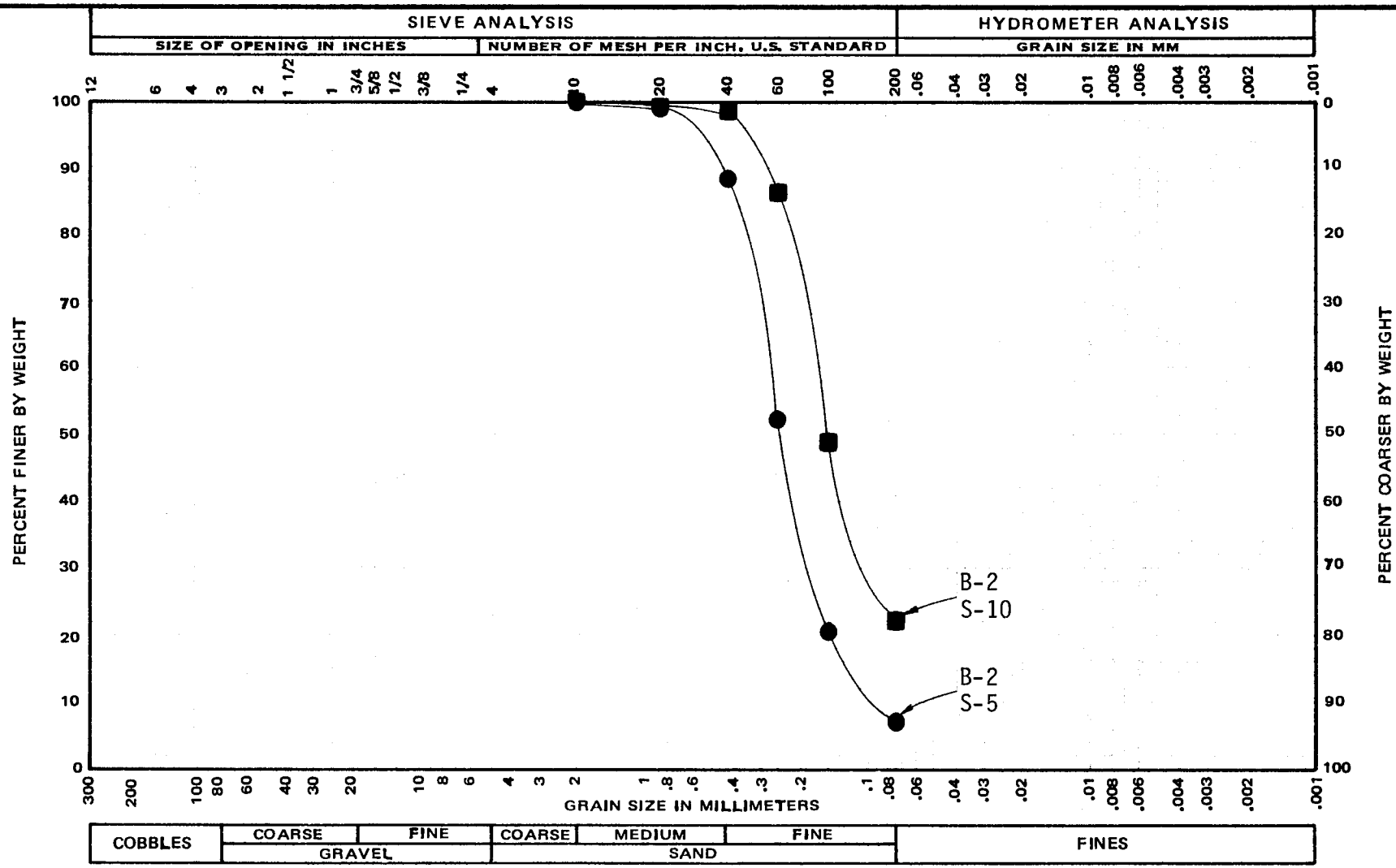
FIG. B-4

FIG. B-4



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-2 S-5	17.5-19.0	SP-SM	Dark gray, slightly silty SAND.	27.6			
B-2 S-10	30.0-31.5	SM	Dark gray-brown, silty SAND.	30.6			

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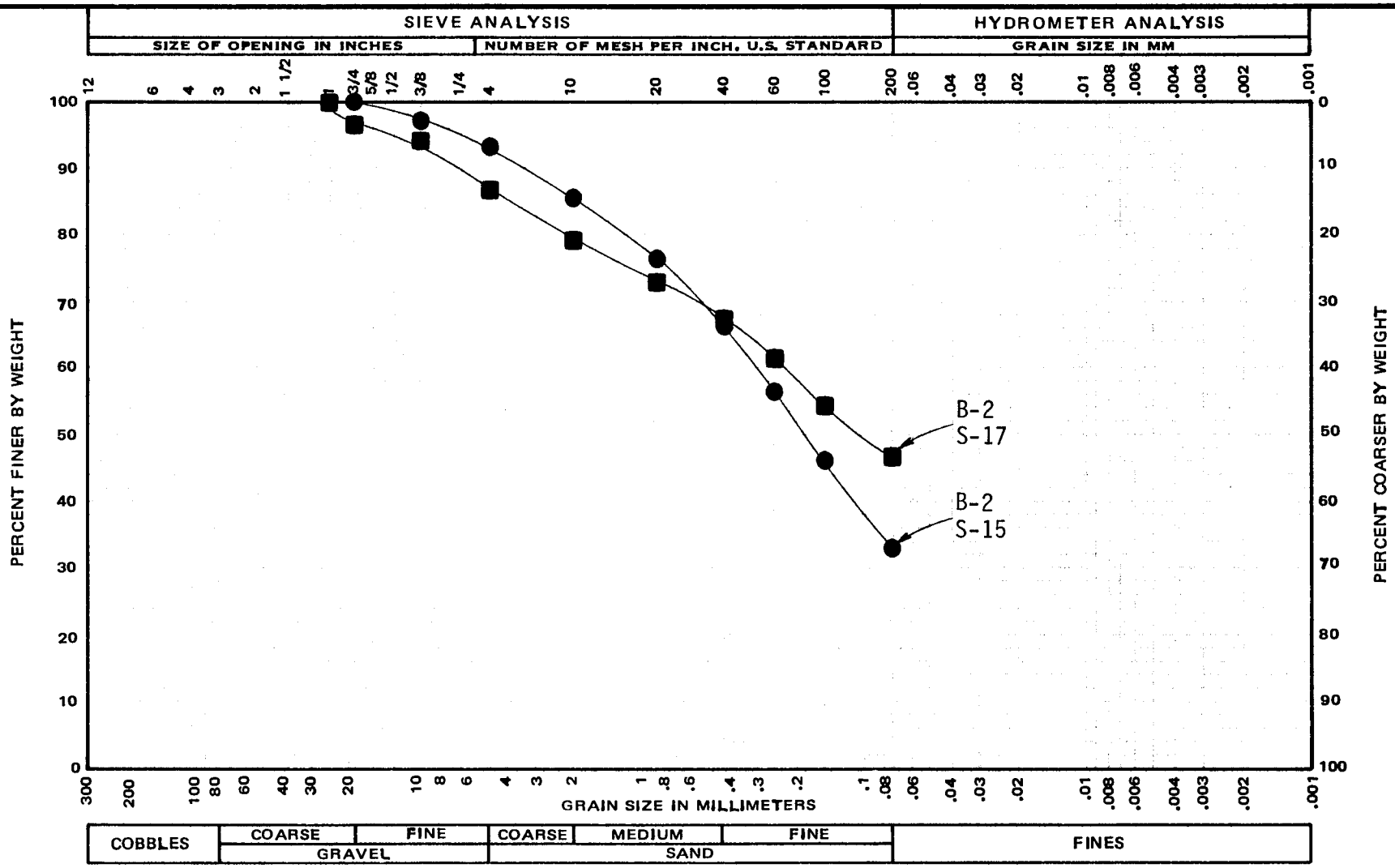
FIG. B-5

FIG. B-5



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
B-2 S-15	55.0-56.5	SM	Gray, slightly gravelly, silty SAND.	32.8			
B-2 S-17	65.0-66.5	SM	Brown, gravelly, clayey, silty SAND.	15.6			

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FIG. B-6

FIG. B-6

UNCONFINED COMPRESSION TEST NO. QU-1

SUMMARY OF TEST DATA

Boring B-2
Sample S-14
Depth, ft. 47.6-48.1

Tested By/Date ND 03-06-91
Calc. By/Date ND 03-07-91
Check. By/Date _____

CLASSIFICATION:
Gray, slightly sandy, silty CLAY
trace of shell fragments.

Specific Gravity 2.70
(Estimated)
Specimen Undisturbed

SPECIMEN DATA:

	Before Test
Height, in.	5.807
Diameter, in.	2.882
Height/Dia. Ratio	2.01
Volume, cc	37.88
Weight, gm.	1162.13
Wet Density, pcf	116.8
Dry Density, pcf	88.8
Water Content, %	31.6
Saturation	95

FAILURE SKETCH

INSTRUMENT CONSTANTS:

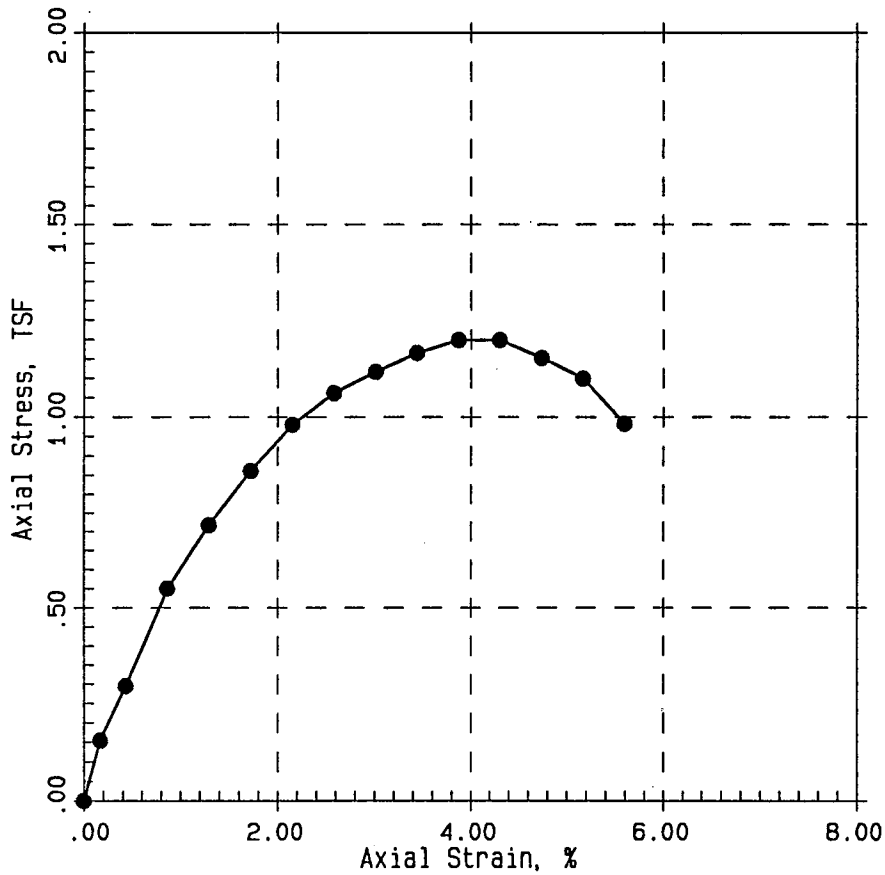
Deformation, in./div.	0.0010
Load, kg./div.	1.0980
Avg. Rate of Strain, %/min	0.6

MEASURED DATA: / **CALCULATED RESULTS:**

Axial Def. Read. div.	Load Read. div.	Axial Strain %	Axial Stress tsf
10.0	5.8	0.17	0.15
25.0	11.1	0.43	0.30
50.0	20.8	0.86	0.55
75.0	27.2	1.29	0.72
100.0	32.8	1.72	0.86
125.0	37.5	2.15	0.98
150.0	40.8	2.58	1.06
175.0	43.1	3.01	1.12
200.0	45.2	3.44	1.17
225.0	46.7	3.87	1.20
250.0	46.9	4.31	1.20
275.0	45.3	4.74	1.15
300.0	43.4	5.17	1.10
325.0	39.0	5.60	0.98

UNCONFINED COMPRESSION TEST NO. QU-1

Boring B-2
Sample S-14
Depth, ft 47.6-48.1



W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-7b**

ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1

SUMMARY OF TEST DATA

Boring B-2
Sample S-14
Depth, ft. 47.2-47.5

Tested By/Date ND 03-01-91
Calc. By/Date ND 03-22-91
Check. By/Date _____

CLASSIFICATION:

Gray-brown, slightly sandy, clayey SILT; scattered shells.

SPECIMEN DATA:	Before Test	After Test
Height, inches	0.787	0.657
Diameter, inches	2.505	2.505
Wet Density, pcf	118.8	131.3
Dry Density, pcf	90.8	108.6
Water Content, %	30.9	20.9
Void Ratio	0.8559	0.5495
Saturation, %	97	102

Specific Gravity 2.70 (Estimated)
Lever Arm Ratio:10.0 to 1

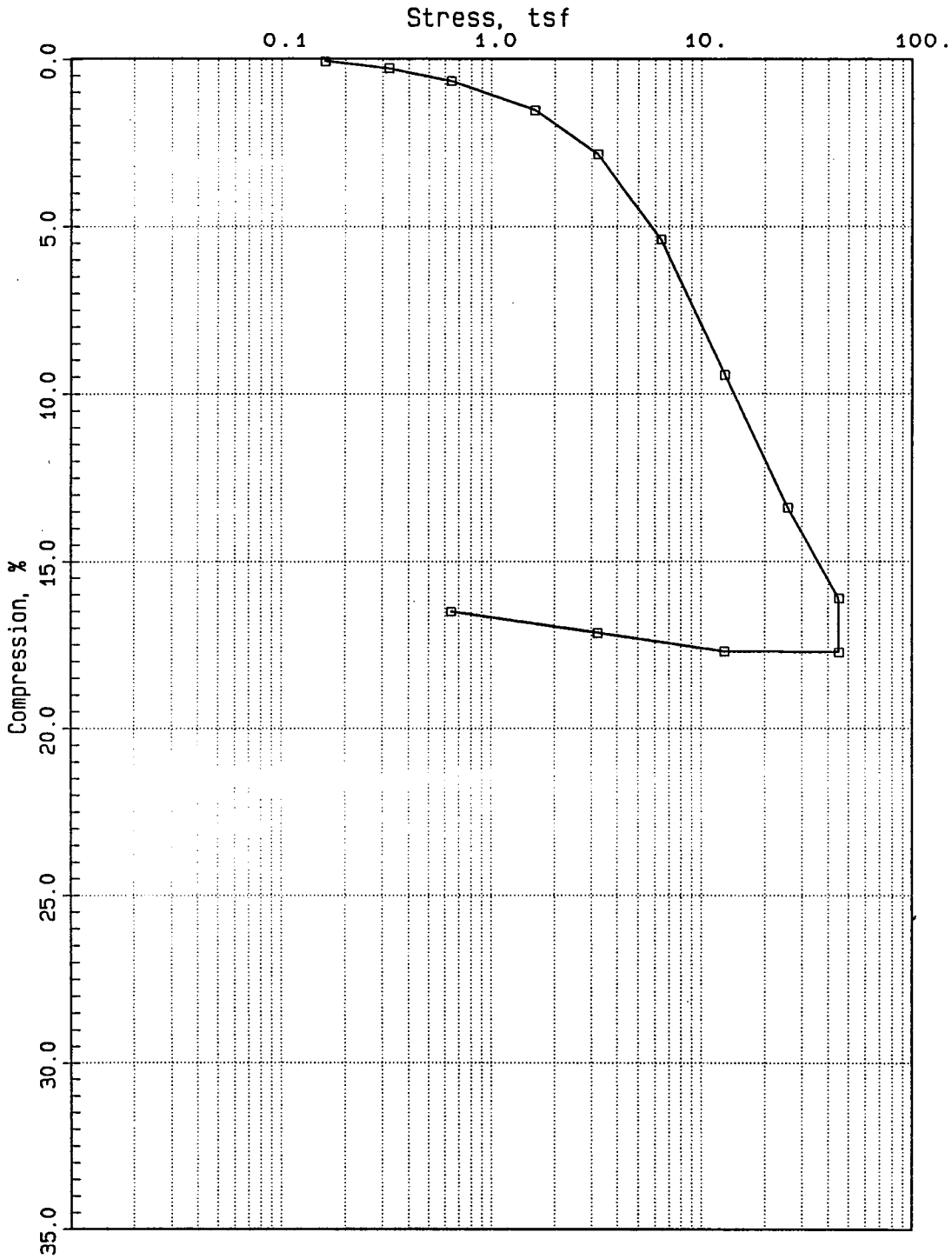
Frame No. E
Ring No. 7

Seating Load 0.03 tsf
Specimen Inundated at 0.06 tsf
Specimen Undisturbed

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Stress tsf	Sett %	Void Ratio	t 50 min	d 50 0.01mm	Coeff of Consolidation cm2/sec	Coeff of Permeability cm/sec
0.5	3.0	1.8	0.16	0.06	0.8547	0.12	2.6	0.027311	
1.0	8.2	2.8	0.32	0.27	0.8509	0.13	7.6	0.025110	0.0000003276
2.0	17.5	4.4	0.64	0.66	0.8437	0.14	16.4	0.023148	0.0000002768
5.0	38.2	7.5	1.61	1.54	0.8274	0.17	36.1	0.018746	0.0000001708
10.0	67.4	10.7	3.22	2.84	0.8032	0.19	63.2	0.016368	0.0000001322
20.0	122.3	14.5	6.44	5.39	0.7558	0.19	111.6	0.015627	0.0000001240
40.0	208.2	19.7	12.88	9.43	0.6809	0.19	191.3	0.014426	0.0000000904
80.0	294.2	26.5	25.77	13.39	0.6073	0.24	277.6	0.010449	0.0000000321
140.0	357.0	34.7	45.09	16.12	0.5566	0.25	349.2	0.009316	0.0000000132
140.0	389.2	34.7	45.09	17.73	0.5267				
40.0	379.5	25.6	12.88	17.70	0.5273				
10.0	363.3	20.5	3.22	17.15	0.5376				
2.0	346.1	16.1	0.64	16.51	0.5495				

ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1

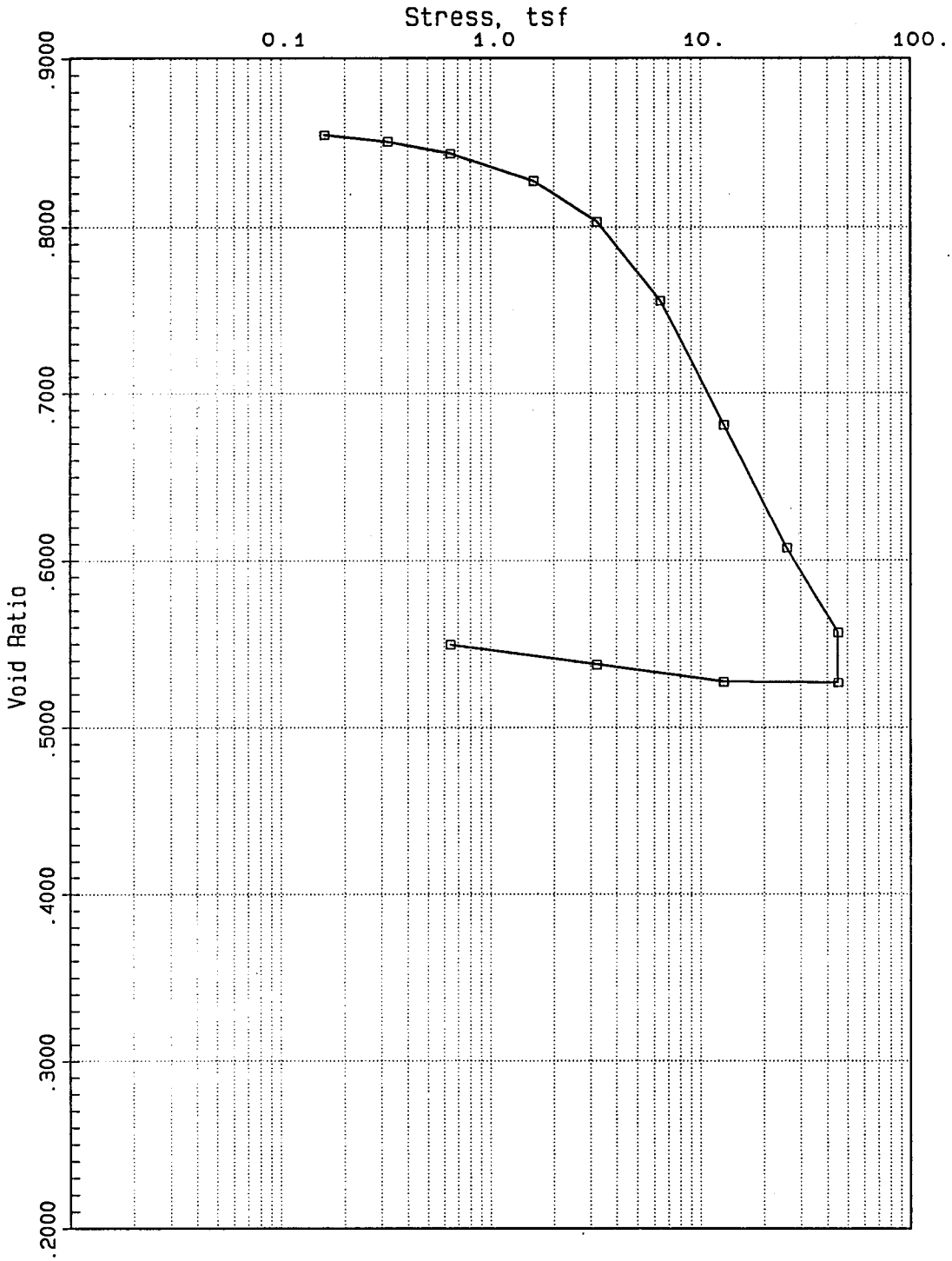
Boring B-2
Sample S-14
Depth, ft 47.2-47.5



W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-8b**

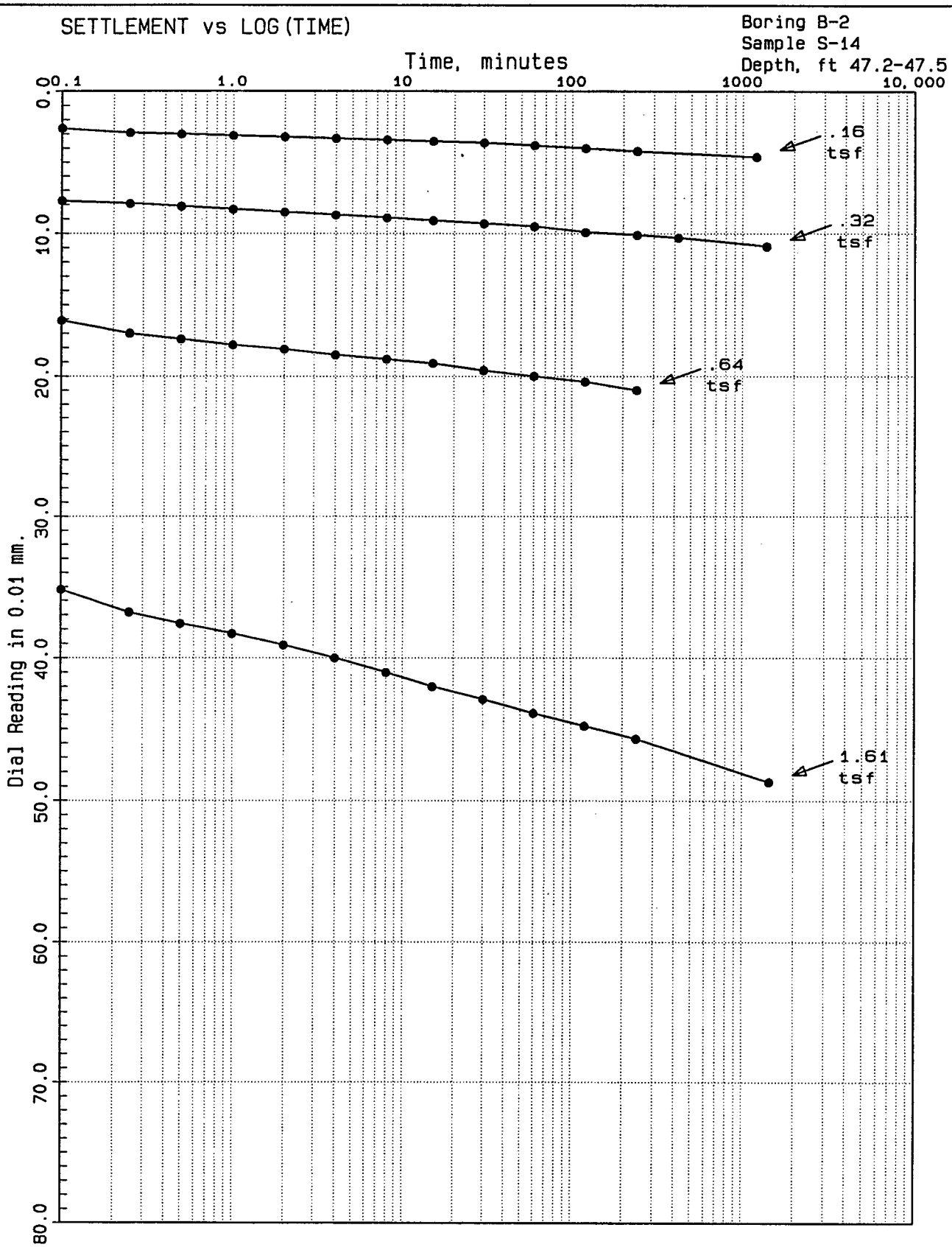
ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1

Boring B-2
Sample S-14
Depth, ft 47.2-47.5



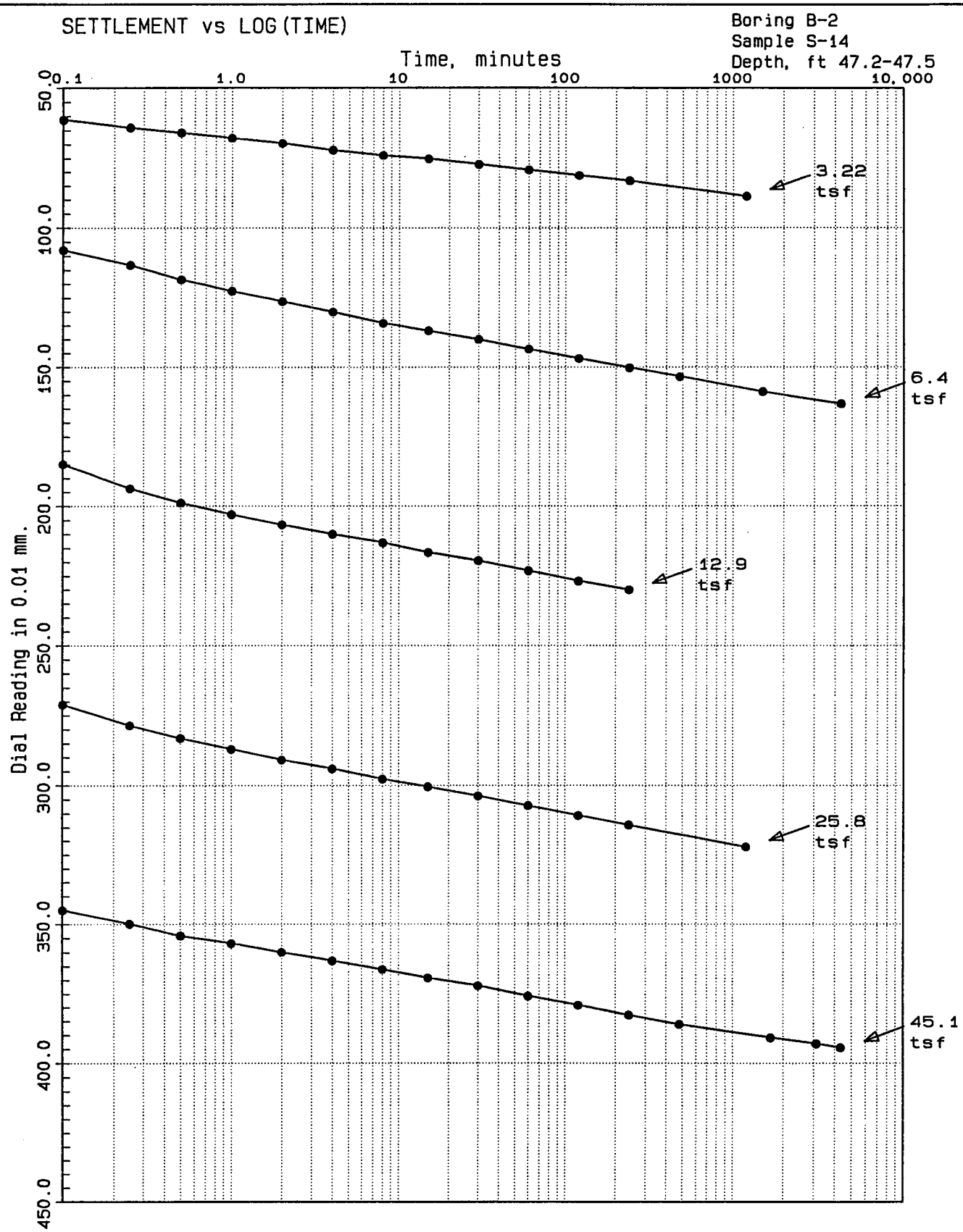
W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-8c**

ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1



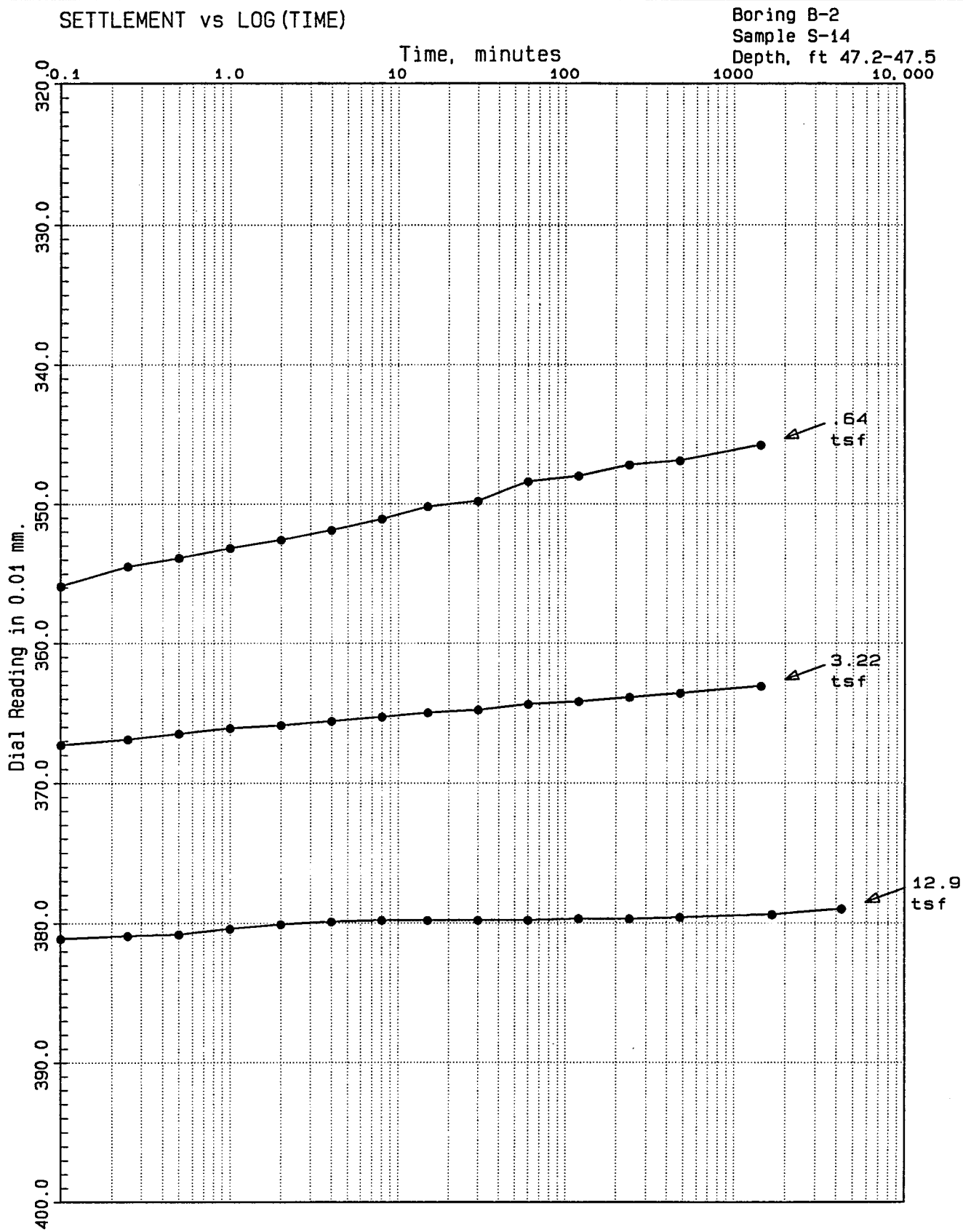
W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-8d**

ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1



W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-8e**

ONE DIMENSIONAL CONSOLIDATION (ASTM D2435-80) TEST NO. C-1



W-5749-01 Shannon & Wilson, Inc. Geotechnical Consultants	16th Avenue South Bridge Seattle, Washington	Mar 1991 **FIG. B-8f**