Geotechnical Report
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

January 2010

Submitted To:
Stanley Consultants, Inc.
8000 South Chester Street, Suite 500
Centennial, CO 80112

By:
Shannon & Wilson, Inc.
5430 Fairbanks Street, Suite 3
Anchorage, Alaska 99518
Phone: 907-561-2120
FAX: 907-561-4483
E-mail: glj@shanwil.com

32-1-02086

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL ENGINEERING REPORT
CHUGACH ELECTRIC ASSOCIATION
INTERNATIONAL SUBSTATION SWITCHYARD
ANCHORAGE, ALASKA

1.0 INTRODUCTION

This report presents the results of subsurface explorations, laboratory testing and geotechnical engineering studies conducted by Shannon & Wilson, Inc. for design and construction of a proposed switchyard at the Chugach Electric Association (Chugach) International Substation in Anchorage, Alaska. The purpose of this geotechnical study is to evaluate subsurface conditions at the proposed location of the switchyard, and to provide geotechnical engineering recommendations for the design of these features. To accomplish this, four borings were drilled and sampled in the areas of proposed development to supplement information from six borings drilled by Hattenburg, Dilley, and Linnell (HDL) in June 2002. Follow-up laboratory testing of soil samples and engineering studies were performed to support foundation design for the proposed switchyard. Presented in this report are descriptions of the site and project, subsurface exploration and laboratory test procedures, an interpretation of subsurface conditions, and our conclusions and recommendations from our engineering studies.

Authorization to proceed with this work was received in the form of a signed contract from Mr. Robert Farrar of Stanley Consultants, Inc. on October 26, 2009. Our work was conducted in general accordance with our September 14, 2009 proposal.

2.0 SITE AND PROJECT DESCRIPTION

The project is located inside Chugach property along Electron Drive, north of West Dowling Road in Anchorage, Alaska. The proposed new switchyard is located over relatively flat-lying land that has been cleared and filled with granular material. The site was covered with snow at the time of our explorations, but vegetation appears to consist of scattered grass and weeds. In the vicinity of the proposed switchyard, a number of transformers, generators, and other equipment is stored. An existing switchyard is located to the north of the proposed addition. A vicinity map indicating the general project location is presented in Figure 1. A site plan is included as Figure 2 that provides a more detailed view of the project area including prominent site features and approximate boring locations of our work as well as previous borings by HDL in June 2002.

We understand that the project consists of design and construction of a new 138kV/115kV switchyard with associated substation structures and infrastructure. We assume the new structures will be metal-framed, lightly loaded buildings constructed with shallow foundations and concrete slab-on-grade floors (no basements) that may or may not be heated continuously throughout the year. We assume that the switchyard equipment will be founded on concrete slabs-on-grade that will remain unheated. We also assume that the project will include construction of underground and overhead utility lines.

3.0 FIELD EXPLORATIONS

Field explorations consisted of drilling four borings on the site from December 11 through 14, 2009. Borings were drilled to depths of between 31.5 and 51 feet below the ground surface (bgs). The borings were generally positioned in the footprint of the proposed switchyard to supplement the previous exploration program. The boring logs from the previous work were provided to us by Chugach. The approximate locations of our borings and the HDL borings are shown on the site plan, Figure 2.

Drilling services for this project were provided by Discovery Drilling, of Anchorage, Alaska, using a truck mounted CME-75 drill rig. Borings were advanced with $3^1/_4$-inch inner diameter (ID), continuous flight, hollow-stem augers. An engineer from our firm was present during drilling to locate the borings, observe drill action, collect samples, log subsurface conditions, and monitor groundwater if appropriate. In general, the borings were backfilled using the cuttings removed during the drilling activity and periodically hand tamped.

As the borings were advanced, grab samples were taken from the auger cuttings in the upper 2 feet of each boring and samples were recovered with a 2-inch outside diameter (OD) split spoon sampler using Standard Penetration Test (SPT) procedures. SPT samples were recovered by driving the 2-inch sampler into the bottom of the advancing hole with blows of a 140-pound hammer free falling 30 inches onto the drilling rod. The number of blows required to advance the sampler the final 12 inches of an 18-inch penetration is termed the penetration resistance. Penetration resistance values are shown graphically on the boring logs adjacent to the sample depth and give a measure of the relative density (compactness) or consistency (stiffness) of cohesionless or cohesive soils, respectively.

At the completion of Borings B-07, B-08, and B-10, 1-inch polyvinyl chloride (PVC) monitoring well casings with slotted sections were installed in the open borings prior to backfilling with auger cuttings. A summary of monitoring well construction is included on the appropriate boring logs along with the date and water level of the latest measurement. The wells were used to estimate static water level at the site after our field efforts. It should be noted that groundwater levels may take days or weeks to stabilize after drilling where fine-grained, low permeability soils are encountered and that natural groundwater levels typically fluctuate by several feet seasonally.

Sampled soils were visually classified in the field using the Unified Soil Classification System (USCS) presented as Figure 3. The field classifications were then verified through selective laboratory analysis. Frost classifications were also estimated for samples based on laboratory testing (sieve analyses). The frost classification system is presented as Figure 4. Frost classifications shown on the boring logs are followed by the method of testing which was used to estimate them, P-200 for samples in which we used the percent passing the number 200 mechanical screen sieve. Summary logs of our borings with material descriptions and frost classifications are presented as Figures 5 through 8.

Locations of Borings B-07 through B-10 (shown on the site plan, Figure 2) were recorded using a differential global positioning system (DGPS) capable of horizontal accuracies of ±3 feet. It should be noted that DGPS accuracy may be affected by weather (cloud/fog cover), geographic features, and other atmospheric anomalies. The locations of Borings B-1 through B-6 were estimated from an HDL site plan provided by Chugach. Surface elevations, shown on the boring logs, were estimated from the topographic layer of the online map at the Municipality of Anchorage's Geographical Information System (GIS) website. Therefore, boring locations and elevations presented in this report should be considered approximate.

4.0 LABORATORY TESTING

Laboratory tests were performed on select samples recovered from the borings to confirm our field classifications and to estimate the index properties of the typical materials encountered at the site.

Water content tests were performed on samples collected from the borings. Water content tests were generally conducted according to procedures described in ASTM International (ASTM) D-2216. The results of the water content measurements are presented graphically on the boring logs presented as Figures 5 through 8.

A grain size classification test was conducted to confirm the field classification of the fill soils encountered. Gradation testing generally followed mechanical sieve procedures described in ASTM C-136. Grain size testing results are presented as Figure 9 and summarized on the appropriate boring log as percent gravel, percent sand, and percent fines. Tests were also conducted on select samples to estimate the amount of material passing the Number 200 sieve (P-200). This test was performed in general accordance with ASTM C-117. The P-200 test provides an estimate of the fines (silt and clay) content. The results of this test are presented on the boring logs, indicated as percent fines. Percent fines on the boring logs are equal to the sum of the silt and clay fractions indicated by the percent passing the Number 200 sieve. Note that gradation testing indicates particle size only and visual classification under USCS designate the entire fraction of soil finer than the Number 200 sieve as silt unless Atterberg limit data shows plasticity properties consistent with clay.

In addition, an organic content test was performed on one sample of the soils underlying the fill. The organic content test was generally conducted according to procedures described in American Association of State Highway and Transportation Officials (AASHTO) T-267. The result of the organic content test is presented on the boring log for Boring B-07, presented as Figure 5.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are summarized on the boring logs presented as Figures 5 through 8 and in the HDL boring logs presented in Appendix A. In general, the soils in the project area consisted of 2.5 to 3 feet of loose to medium dense, slightly gravelly, slightly silty to silty sand fill. Underlying the fill was generally medium dense, silty sand or stiff to very stiff, sandy silt with occasional to scattered organics to approximately 10 feet bgs and medium dense to very dense, slightly silty sand to 31.5 feet bgs. Boring B-07 continued to a depth of 51 feet bgs and encountered dense, silty sand at 43 feet bgs and hard, sandy silt at 50 feet bgs to the bottom of the boring. Loose soils were encountered in samples taken in the upper 6.5 feet of Boring B-09 and at 15 feet in Boring B-10.

It should be noted that heaving sands were occasionally encountered while drilling below the water table. This condition can impact the penetration resistance values in two ways. Sands that heave prior to sampling (i.e. when the rods are removed from the augers) can create slightly looser soil conditions just beneath the augers and yield somewhat lower penetration resistance values during SPT sampling. If heave prior to sampling is less than 6 to 12 inches and the sand material is removed from the augers prior to driving the sampler, it is our opinion that the overall impact to the SPT penetration resistance values is typically low to negligible. Heaving can also occur during driving of the SPT sample. If this occurs, the sample spoon is generally overfilled and penetration resistance values can be significantly higher than actual conditions. Heaving conditions are noted on our boring logs (Borings B-07 through B-10) when encountered. In addition, we have noted where we believe heaving conditions may be biased high or low as a result of the heaving.

According to laboratory testing, the fines content of the existing fill material ranged from approximately 11 to 44 percent. Moisture content for the fill soils ranged from approximately 4 to 18 percent. The fines content of the sandy silt or silty sand material between the fill and a depth of approximately 10 feet bgs ranged from approximately 48 to 65 percent. Moisture contents for these soils ranged from approximately 6 to 32 percent.

The fines content of the native, coarse-grained material, below 10 feet bgs, generally ranged from 5 to 12 percent, but Boring B-08 encountered a silty sand layer that contained about 40.5 percent fines at a depth of approximately 15 feet bgs. These native, coarse-grained soils generally began at the depth of the groundwater table and moisture contents ranged from about 16 to 26 percent.

Groundwater was encountered during drilling in each of our borings at depths of between 9.8 and 11 feet bgs. Groundwater levels measured in the PVC monitoring wells on December 22, 2009 were found to be 8.4, 9.8, and 8.7 feet bgs in Borings B-07, B-08, and B-10, respectively. Static groundwater levels are noted on the boring logs, Figures 5 through 8. It should be noted that groundwater levels typically fluctuate by several feet seasonally. Based on conditions found in the borings, it appears that the water table is within the lower granular zone and rises to within the sandy silt/silty sand zone over time.

6.0 SEISMIC CONDITIONS

According to the 2006 International Building Code (IBC), the site classification would be Class D for a stiff soil profile, using the blow count (N) method (i.e., $15 \leq N \leq 50$). The site generally consisted of relatively flat-lying ground, sandy soils, and a water table at approximately 10 feet bgs during our explorations. Based on the site conditions and our analyses, we believe that slope failure and surface rupture should not occur at this site within the design life of the facility. Therefore, we recommend that Site Class D be selected as most representative of the overall properties of the site. Based on IBC 2006 1615.1, S_s and S_1 were estimated at 1.5 and 0.56, respectively. Consequently, the site specific modifying coefficients for the spectral response accelerations for the Maximum Considered Earthquake are $F_A = 1$, and $F_v = 1.5$ for the short and long periods, respectively.

6.1 Liquefaction

We evaluated the triggering of liquefaction at the site using the procedures described in Liquefaction Resistance of Soils: A Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on the Evaluation of Liquefaction Resistance of Soils (Youd et al. 2001).

Liquefaction triggering is estimated by comparing the liquefaction resistance properties of a soil deposit to the seismic demand placed on the soil layer by a given earthquake. The seismic load or demand placed on the soils required to cause liquefaction is a function of the intensity and duration of ground shaking. The duration of ground shaking is related to earthquake magnitude, and the intensity depends on magnitude, distance from the earthquake, and site response characteristics. The intensity of ground shaking is described by a shear stress normalized to an effective overburden pressure, referred to as the cyclic stress ratio (CSR). Generally the shear stress and CSR are estimated as a function of the peak horizontal ground acceleration, overburden stress, and a stress reduction coefficient, which accounts for a variation in the stress level or acceleration with depth.

The ability of a granular soil to resist liquefaction is expressed as a cyclic resistance ratio (CRR), which is the threshold CSR required for liquefaction initiation. Based on case history studies of liquefied and nonliquefied sites from numerous earthquakes, empirical relationships have been developed that correlate the CRR normalized for a Magnitude 9.2 earthquake with normalized/corrected values of SPT blow counts (N), identified as $(N_1)_{90cs}$. $(N_1)_{90cs}$ is the SPT blow count normalized to an overburden stress of approximately 1 ton and a hammer efficiency of 90 percent, and adjusted for the soils fines content. Blow counts are also corrected for other factors such as borehole size, sample rod length, and whether or not a sample liner was used.

The soils are potentially liquefiable when the seismic load (CSR) is greater than the resistance to liquefaction (CRR). The potential for liquefaction can be expressed in terms of a factor of safety against liquefaction defined by:

$$FS = CRR/CSR$$

Liquefaction is predicted at those depths where FS is less than 1.

Duration effects are accounted for in the characterization of liquefaction loading by earthquake magnitude. Since the PGA_{rock} value produced by a PSHA, includes contributions from all possible magnitudes and distances, there is no single magnitude value to assign to a probabilistically determined PGA_{rock} value. We therefore selected the modal magnitude (M) of the most important seismic sources contributing to the ground motion parameters from a deaggregation of the PSHA on the United States Geological Survey (USGS) website. The modal value of M from the deaggregation contributing the most to motion was 9.2. Using the USGS's Earthquake Ground Motion Parameters Version 5.0.9a and 2006 IBC, the peak ground acceleration at the site was estimated to be 0.28g. For comparison, it should be noted that the 1964 Great Alaska Earthquake had Magnitude 9.2 but generated peak ground accelerations in Anchorage on the order of about 0.17g.

Using the ground motion parameters determined above, the liquefaction potential considering the soil profiles found in Borings B-07 through B-10 were analyzed. These borings were drilled to depths of between approximately 31.5 and 51 feet bgs. Liquefaction analyses results and figures are presented in Appendix B. Boring logs show that 24 out of the 32 SPT samples taken were below the groundwater table. Of the 24 samples collected below the groundwater table, five samples yielded penetration resistance values that were significantly biased high or low (i.e., not representative of actual conditions) and were therefore not included in our analysis. Of the remaining 19 soil samples, three samples when using the Youd et al. method, two samples according to the Idriss & Boulanger method, and three samples using the Seed et al. method were found to have a factor of safety (FS) of less than 1.0 given the assumed seismic event. Each of the samples found to be potentially liquefiable were in relatively clean, sandy units between a depth of 15 and 20 feet bgs. Liquefaction of these soils could result in seismically induced settlement and/or areas of lateral spreading under the switchyard structures during a seismic event.

6.2 Seismic Induced Settlement

Densification of the granular soils above and below the water table may occur when subject to earthquake shaking, resulting in potential ground settlement at the site. We used the relationship by Tokimatsu and Seed (1987) and Pradel (1998), relating earthquake ground motion and penetration resistance with volumetric strain, to estimate the magnitude of ground settlement that may occur at the site. Our analyses suggest a potential for up to about 1.5 inches of settlement associated with seismically induced compaction. This could occur in isolated pockets of less compact soil in the 10 to 20 foot depth range during or after the design earthquake. The relationships estimate differential settlements at the existing ground surface of up to 1.5 inches over approximately 100 feet for the ground motions assumed in our liquefaction analyses.

6.3 Lateral Spreading

Typically, lateral spreading occurs in concert with liquefaction and/or slope failures adjacent to a given site. Based on the findings in our borings and the results of our liquefaction analysis, it is our opinion that lateral spreading could result from liquefaction of the relatively shallow, saturated, sandy soils in the project area. Liquefaction of the native support soils (as they lose strength during seismic acceleration) may result in lateral spreading of the structural fill supporting some of the switchyard structures. In the event of an earthquake consistent with the event assumed in our analysis, some of the structures for the proposed switchyard could experience lateral spreading up to nearly 12 inches.

We do not believe that this hazard is widespread in the area. As shown on our lateral displacement figures, presented in Appendix B, our analyses found potential lateral spreading of more than 3 inches in only one boring (Boring B-10). Therefore, we believe that because of the relatively flat-lying ground in the vicinity of the project site, the probability of lateral spreading on the order of 12 inches is low.

7.0 ENGINEERING RECOMMENDATIONS

The design of foundations for support of the equipment for the proposed switchyard and associated structures must consider the bearing support capabilities of the soils as well as the expected settlements and the effects of seasonal frost action. Our borings generally encountered fill overlying silty and/or sandy soils that are relatively compact and moderately to highly frost susceptible (generally F2 to F4). We believe the conditions encountered in our borings are adequate to support the proposed structures.

7.1 Building Foundation Design

After equipment that is currently being stored at the site is removed from the project area, surface organics (if present) should be cleared from the switchyard and building footprints. This material may be stockpiled on site for use in landscaping if desired. Excluding the loose fill and loose, silty sand soils encountered in Sample S5 at 15 feet bgs in Boring B-10, our explorations and the previous explorations conducted by HDL typically encountered undisturbed medium dense to very dense or stiff to hard native materials. It is our opinion that the native soils will provide adequate support for the proposed switchyard equipment and associated structures if foundations are designed to accommodate frost action where appropriate.

Prior to fill placement, the exposed grade should be proof-rolled to produce a firm, unyielding surface for construction. Loose soils were encountered in Borings B-09 and B-10, and could be encountered during construction in other isolated areas beneath the structures. We recommend over-excavating unsuitable soils until undisturbed, firm, unyielding native soils are encountered. If subgrade materials do not provide a firm, unyielding surface at the footing grade, we recommend the contractor sub-cut (locally as appropriate to the actual conditions exposed) a minimum of 2 feet (or as needed to remove loose or soft, unsuitable soils) below footing. The sub-cut should extend out a minimum distance equal to the sub-cut depth from the outer edges of the footings, and unsuitable material should be replaced with compacted classified structural fill. Structural fill placed beneath footings should be placed and compacted in accordance with the recommendations included in Section 7.8. We recommend that we be retained to observe excavations for the switchyard and associated structures and that a contingency be maintained to allow for isolated over-excavation and replacement beneath the proposed structure in addition to our recommendations.

Based on the information obtained from our field explorations, we recommend that buildings be supported on continuous strip or spread footings bearing directly on classified structural fill or firm, unyielding native materials. The exposed grade and classified fill placed beneath footings should be placed and compacted in accordance with the recommendations included in Section 7.8. The recommended minimum width for continuous footings is 16 inches and for spread footings 24 inches. All footings should be buried sufficiently to prevent structural damage resulting from frost action. We recommend that perimeter footings in heated buildings be placed a minimum of 4 feet below the ground surface. For interior footings in heated areas, footings may be placed directly below the floor slab such that embedment is 18 inches or more. If portions of the structures are to be unheated, the minimum burial depth for footings should be increased to 5 feet bgs for frost protection. Figure 10 presents a conventional shallow foundation floor slab and footing detail. We also recommend that cold footing excavations be sub-cut and backfilled with non-frost-susceptible (NFS) fill a minimum of 2 feet beneath the footing elevation for those structures that would be sensitive to vertical displacement due to frost heaving. Figure 11 presents a conventional shallow foundation floor slab and footing detail for movement sensitive structures.

Based on the recommended footing dimensions, depths, and site preparation recommendations, we recommend that foundations for the proposed structures be designed with an allowable soil bearing pressure of 2,500 pounds per square foot (psf). Localized loose or soft areas, whether resulting from existing conditions or disturbance during construction must be corrected prior to casting footings, or damaging differential settlements could occur. In our opinion, the above bearing value may be increased by one-third for short-term wind or seismic loading.

7.2 Concrete Slab Support

Concrete slabs are anticipated to support the proposed switchyard equipment as well as on the interior of the associated buildings. Concrete slabs in the interior of heated buildings should generally be placed as shown on Figure 10. We recommend that the exposed foundation soils be probed to locate materials that may be naturally loose or have become loosened or disturbed due to the filling and grading process. If loose areas are encountered, we recommend they be excavated and replaced. The structural fill placed beneath concrete slabs should be placed and compacted in accordance with the recommendations included in Section 7.8. We recommend developing a 4 to 6-inch lift of drainage sand and gravel fill founded on compact structural fill. The drainage soil should be a free draining granular material with a maximum grain size of 2 inches or less and not more than 6 percent fines (by weight based on the minus 3/4-inch fraction) passing the No. 200 sieve with no plastic fines. In areas to receive floor coverings, we recommend installing a vapor retarder beneath the concrete slab to maintain dry floor conditions. We recommend assuming a subgrade reaction modulus of 250 pounds per square inch per inch when designing the concrete slabs.

We envision two basic approaches to the design of concrete slabs in unheated buildings for this project. If the slab is sensitive to vertical displacement from heave and/or settlement, the entire footprint should be sub-cut to the footing sub-cut depth required for frost protection. The interior of the foundation should then be backfilled with NFS structural fill placed in accordance with the requirements of Section 7.8. Concrete slabs in the interior of unheated buildings should generally be placed as shown on Figure 11. Concrete slabs constructed in this manner can be rigidly connected to the perimeter foundation/stem wall as they should have comparable settlement/heave behavior.

If the concrete slab portion of an unheated building is not sensitive to vertical displacements, some cost saving may be realized by not sub-cutting the slab to the footing sub-cut depth, and only excavating the slab footprint to the depth necessary to remove the surface organics and unsuitable material and then backfilling to slab grade using NFS structural fill placed in accordance with the requirements of Section 7.8. Floor slabs constructed in this manner should not be rigidly connected to the perimeter foundation/stem wall since the slab could experience different seasonal frost heave movements than the footings.

We understand that concrete slabs may also be constructed to support switchyard equipment, but not be enclosed within a building. Similar to the structural sections described above for concrete slabs in unheated buildings, two basic approaches may be used to design exterior concrete slabs. If the slab is sensitive to vertical displacement from heave and/or settlement, the entire footprint should be sub-cut to a minimum depth of 7 feet below slab grade for frost protection. If the slab is not sensitive to vertical displacements, then it is only necessary to excavate the slab footprint to the depth needed to remove surface organics and unsuitable (loose or soft) material. Excavations beneath exterior concrete slabs should be backfilled with NFS structural fill placed in accordance with the requirements of Section 7.8.

7.3 Estimated Static Settlements and Frost Heave

The total static settlements that will develop are dependent upon the actual loads that are applied, the footing sizes, and the properties of the soils below the footings. Based on the penetration resistance values presented on the logs, the anticipated behavior of the native soils, allowable bearing pressures, and assuming foundations over loose sands are treated as described herein, we estimate total settlements (excluding those resulting from possible seismic events, as discussed in Section 6.1) of about 1-inch could occur and that differential settlements will be about one-half of the total over a horizontal distance of about 50 feet. Due to the generally granular nature of the native soils, we estimate that these settlements should develop almost elastically as the loads are applied such that post-construction settlements will be small and within tolerable limits.

Frost penetration beneath unheated concrete slabs could reach a depth of 8 feet or more. Frost heave related movement of the slabs will depend on the quality and uniformity of native soil and fill material in the freezing zone. We have recommended that equipment sensitive to vertical displacement from heaving or settlement be supported on at least 7 feet of NFS fill material. Vertical displacements of the slab from heave should not exceed about 1 inch. It is our opinion that differential displacement of the slabs due to frost action will be relatively small (about 0.5 inches). The slab should return to its approximate construction grade after seasonal thawing. Unheated slabs and foundations that are not underlain by NFS material to at least 7 feet bgs could experience differential vertical displacements of up to several inches from seasonal freezing.

7.4 Tower Design

We assume that the tower foundations associated with this project will consist of a rectangular cast-in-place block foundation buried below the ground surface. We recommend that the foundation bear on compacted structural fill overlying dense, undisturbed native soil. As described in Section 7.1, we recommend the bottom of the tower foundation to be embedded at least 5 feet bgs. We also recommend sub-cutting the existing soil to at least 7 feet bgs to reduce the risk of frost heaving. We assume that uplift loads will control the size and depth of the foundation (i.e., a foundation sized to resist uplift and overturning loads should likewise be sufficient to resist the applied downward loads).

The uplift resistance of a footing foundation can be estimated by summing the dead weight of the footing, the weight of the soil within a zone described by a vertical surface extending upward from the horizontal limits of the footing, and the shearing resistance of the soil across this surface. Assuming classified structural fill conforming to the specifications presented in Section 7.8 is used to backfill above the footings using the placement and compaction requirements outlined in Section 7.8, the density of the soil resisting uplift should be at approximately 135 pounds per cubic foot (pcf). The shearing resistance can be calculated using a frictional resistance of about 36 degrees for densely compacted imported fill. Using these parameters, we created a graph representing ultimate uplift resistance, presented as Figure 12. This graph represents the calculated ultimate (no Factor of Safety included) uplift resistance over a range of footing areas and foundation depths, but does not include the weight of the footing in the calculation. The relationships on the graph are such that it is safe to extrapolate outside the charted values. Additional resistance gained from friction between the block and embedment soils should be negligible in comparison and in our opinion should not be used in estimating uplift resistance for this type of foundation.

7.5 Drainage

Groundwater was observed in our borings at depths ranging from 9.8 to 11 feet bgs during drilling and as shallow as 8.4 feet bgs during static groundwater measurements on December 22, 2009. Groundwater may fluctuate seasonally by several feet and the exploration results presented herein may not necessarily coincide with high water levels. Therefore, groundwater may be encountered and water seepage could be experienced during excavation activities. In our opinion, construction efforts will not likely encounter groundwater issues unless excavations are conducted below about 10 feet bgs. There is also the potential for surface runoff to infiltrate open excavations during construction. The contractor should be responsible for maintaining site grade to prevent surface water from entering excavations during periods of high rain or snow melting. Roof down spouts should likewise carry rainwater in tight lines away from the building foundations. The contractor should be prepared to dewater the excavation with sumps and pumps during construction as needed to remove water that collects in the excavation. Assuming floor slabs are at or above the surrounding grade and site grading is designed for positive drainage control, peripheral footing drains should not be required and the details in Figures 10 and 11 would apply.

7.6 Lateral Earth Pressures and Lateral Resistance

Building walls below ground which support earth fills and floor slabs should be designed to resist lateral earth pressures. The magnitude of the pressures is dependent on the method of placement of backfill, the type of backfill material, drainage provisions, and whether the wall is allowed to deflect after or during placement of backfill. For the earth pressures provided herein, we assume that footing trenches will be backfilled with a free-draining structural fill (such as Type II or IIA material) and groundwater levels will naturally remain below the footing level.

If walls are permitted to deflect laterally or rotate an amount equal to about 0.001 times the height of the wall, an active earth pressure condition under static loading would prevail and an equivalent fluid weight of 35 pounds per cubic foot (pcf) is recommended for design of walls. For rigid walls that are restrained from deflecting at the top, an at rest earth pressure condition would prevail and an equivalent fluid weight of 56 pcf is recommended. These pressures assume that hydrostatic forces cannot develop behind the walls. To simulate seismic loading, a rectangular pressure prism with a magnitude of 12.5 pounds per square foot (per linear foot of wall) should be applied to the entire height of the wall.

Lateral forces from wind or seismic loading may be resisted by passive earth pressures against the sides of footings, exterior walls below grade and grade beams. In our opinion, these resisting pressures can be estimated using an equivalent fluid weight of 520 pcf. This value includes a factor of safety of at least 1.5 on the full passive earth pressure to limit deflections. This value assumes that backfill around the footings and stem walls are densely compacted.

Lateral resistance may also be developed in friction against sliding along the base of foundations placed on grade such as footings or floor slabs. These forces may be computed using a coefficient of 0.4 between concrete and soil.

7.7 Utility Trenches and Slopes

Buried pipes and/or cables will be needed to tie the new switchyard and associated structures into area utilities. Trenches excavated for installation of these new utilities should be constructed as presented in Figure 13. The bedding and structural fill material around the buried utility should be placed and compacted as discussed in Section 7.8 to support and hold the pipe or cable firmly in place. Bulking of backfill into the trench should be discouraged as this can cause voids and lead to large future surface settlements.

The observed static groundwater level was found to be deeper than 8.4 feet bgs. This is deeper than the anticipated burial depth of utilities for the proposed switchyard and associated structures, so groundwater is not expected to be encountered during excavation work. Provided the excavation bottom does not extend below the groundwater level in the surrounding ground, we believe that any seepage can be controlled using sumps and pumps. This assumes that the excavation and backfilling work is closely coordinated such that seepage and surface runoff are not allowed to collect and stand in open trenches thereby softening the subsoil and creating constructability problems.

The native sandy soils in this area are locally cohesionless and moist. Trench slopes above the water table in the sandy materials will tend to stand steeply for short periods of time, but as they dry they will ravel in time to their natural angle of repose, which for planning purposes is estimated at about 1.5 horizontal (H) to 1 vertical (V). Silty soils will likely tend to stand near vertical upon excavation, but, if they are exposed to moisture, silty soil slopes will tend to soften and slump. The slope and trench bottom conditions should be made the responsibility of the contractor as he or she is present on a day to day basis and can adjust their efforts to obtain the needed stability, and meet the applicable state and federal safety regulations (including OSHA).

7.8 Structural Fill and Compaction

Classified structural fill will be needed beneath footings, floor slabs, and in utility trenches. Classified structural fill that is placed should be clean, non plastic, granular soil for ease of compaction and to provide drainage and frost protection. We understand that switchyard structures are sensitive to vertical movements, so we recommend that fill soils should consist of well-graded sand and gravel containing less than about six percent (by weight, based on the minus 3/4-inch portion) passing the No. 200 sieve. Note that Municipality of Anchorage (MOA) Type II/IIA specifications call for less than six percent passing the No. 200 sieve based on the minus 3-inch portion, and thus does not meet the gradation requirements established in this report. Gradation requirements for classified fill material referenced in this report are included in Figure 14.

The existing fill encountered in our borings was found to have relatively high fines content (about 11 to 44 percent), and generally do not appear to meet the gradation requirements shown on Figure 14. This material may be selectively salvaged and utilized as unclassified fill, but should not be used beneath building footings or concrete slabs. Unclassified fills used for this project may contain up to 20 percent passing the No. 200 sieve (based on the minus 3-inch fraction); should consist of mineral soil free of organics, frozen clods, or other deleterious material; and should be compactable in accordance with the requirements of this section. Unclassified material may be used for landscaping purposes, in utility trenches, or to prepare the site for construction in areas where seasonal frost heave and subsequent loss of subgrade strength can be tolerated. Unclassified fills are often moisture sensitive and may be difficult to place in a controlled manner during wet weather or in the confines of trenches.

Structural fill material used in the construction of this project should be placed in lifts not to exceed 10 to 12 inches loose thickness and compacted to at least 95 percent of the maximum density as determined by the Modified Proctor maximum density (ASTM D-1557). During fill placement, we recommend that large cobbles or boulders with dimensions in excess of 3 inches be removed from structural fills. Non-structural fill (classified or unclassified material) should also be placed in lifts not to exceed 12 inches in loose thickness and compacted to at least 90 percent of the Modified Proctor maximum density (ASTM D-1557). Unclassified fill materials that are salvaged from site excavations may be difficult to compact with conventional vibratory methods if silt contents are near or above 20 percent (as determined by the minus 3-inch fraction). We recommend that we be retained to provide quality assurance testing for compacted fills during construction.

When backfilling within 18 inches of the building walls where the wall is not supported on both sides, material should be placed in layers not to exceed six inches loose thickness and densely compacted with hand-operated equipment. Heavy equipment should not be used as it could cause increased lateral pressures and damage walls.

8.0 CLOSURE AND LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions and recommendations contained in this report are based on information provided from the observed site conditions and other conditions described herein. The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist. It is assumed 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 or the data provided by Chugach from others.

If, during construction, subsurface conditions different from those encountered in these and prior explorations are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal 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 review those portions of the plans and specifications pertaining to earthwork and foundations to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly the compaction of structural fill, preparation of spread footing foundations and installation of shoring and site excavations, and also to make field measurements of ground displacements and such other field observations as may be necessary.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachments in Appendix C *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.

We appreciate this opportunity to be of service. Please contact the undersigned at (907) 561-2120 with questions or comments concerning the contents of this report.

Sincerely,

SHANNON & WILSON, INC.

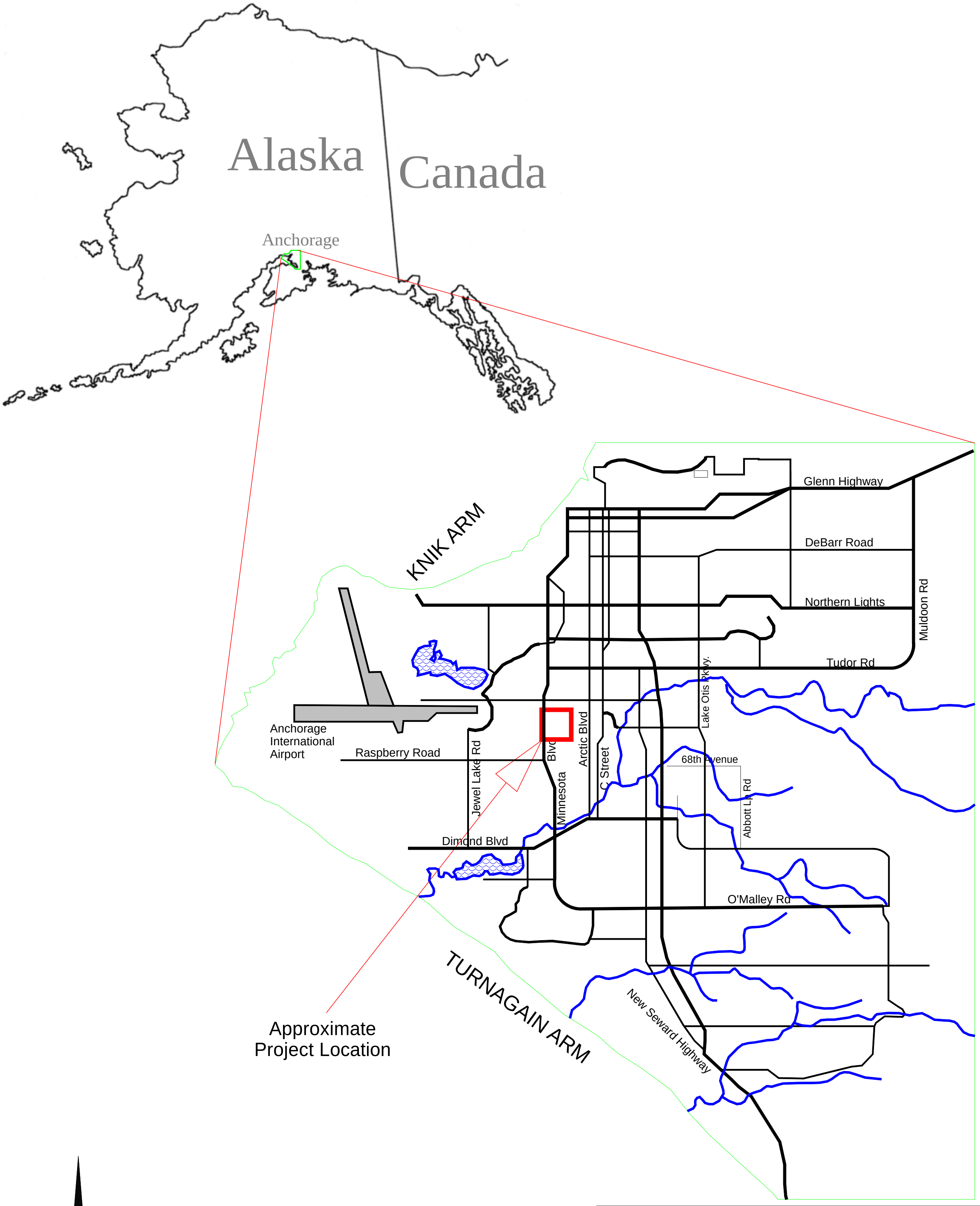
Prepared by,

Russell Hepner, E.I.T.
Geotechnical Engineer II

Reviewed by,

STATE OF ALASKA
1/28/10
49TH
GROVER L. JOHNSON
CE - 8833
REGISTERED PROFESSIONAL ENGINEER

Grover L. Johnson, P.E.
Senior Principal Engineer



NOT TO SCALE

CEA International Substation Switchyard
Anchorage, Alaska

VICINITY MAP

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 1

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth Mapping Service.

LEGEND

⊕ B-10	Approximate location of Boring B–10, advanced by Shannon & Wilson, December 2009.
⊕ B-6	Approximate location of Boring B–06, advanced by Hattenburg, Dilley, & Linnell, June 2002.

CEA International Substation Switchyard
Anchorage, Alaska

Site Plan

January 2010
32-1-02086

Shannon & Wilson, Inc.
Geotechnical & Environmental Consultants

Fig. 2

Unified Soil Classification System

GROUP NAME **Criteria for Assigning Group Names and Group Symbols**				**Soil Classification Group Symbol with Generalized Group Descriptions**	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines	▨	GW	Well-graded Gravels
			▨	GP	Poorly-graded Gravels
		GRAVELS with fines More than 12% fines	▨	GM	Gravel & Silt Mixtures
			▨	GC	Gravel & Clay Mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines	▨	SW	Well-graded Sands
			▨	SP	Poorly-graded Sands
		SANDS with fines More than 12% fines	▨	SM	Sand & Silt Mixtures
			▨	SC	Sand & Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC	▨	ML	Non-plastic & Low-plasticity Silts
			▨	CL	Low-plasticity Clays
		ORGANIC	▨	OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC	▨	CH	High-plasticity Clays
			▨	MH	High-plasticity Silts
		ORGANIC	▨	OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		▨	PT	Peat

Organic Content

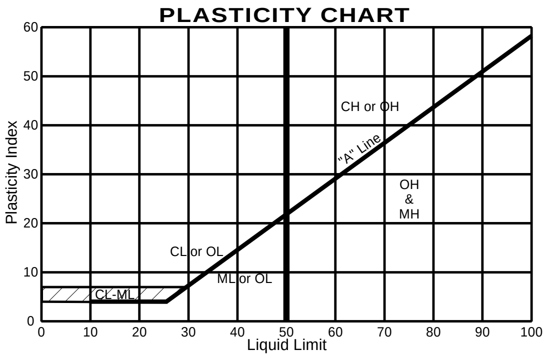
Adjective	Percent by Volume
Occasional	0-1
Scattered	1-10
Numerous	10-30
Organic	30-50, minor constituent
Peat	50-100, MAJOR constituent

Descriptive Terminology Denoting Component Proportions

Description	Range of Proportion
Add the adjective "slightly"	5 - 12%
Add soil adjective[a]	12 - 50%
Major proportion in upper case, (e.g., SAND)	>50%

[a] Use gravelly, sandy, or silty as appropriate

NOTE: The soil descriptions used in the boring logs lists constituents from smallest percentage to largest percentage.



CEA International Substation Switchyard
Anchorage, Alaska

SOIL CLASSIFICATION LEGEND

January 2010

32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 3

SHANNON & WILSON, INC.

FROST CLASSIFICATION

(after Municipality of Anchorage)

GROUP		0.02 Mil.	P-200*	USC SYSTEM (based on P-200 results)
NFS	Sandy Soils		0 to 3	SW, SP
	Gravelly Soils	0 to 3	0 to 6	GW, GP, GW-GM, GP-GM
F1	Sandy Soils	0 to 3	3 to 6	SW, SP, SW-SM, SP-SM
	Gravelly Soils	3 to 10	6 to 13	GM, GW-GM, GP-GM
F2	Sandy Soils	3 to 15	6 to 19	SP-SM, SW-SM, SM
	Gravelly Soils	10 to 20	13 to 25	GM
F3	Sands, except very fine silty sands**	Over 15	Over 19	SM, SC
	Gravelly Soils	Over 20	Over 25	GM, GC
	Clays, PI>12			CL, CH
F4	All Silts			ML, MH
	Very fine silty sands**	Over 15	Over 19	SM, SC
	Clays, PI<12			CL, CL-ML
	Varved clays and other fined grained, banded sediments			CL and ML CL, ML, and SM; SL, SH, and ML; CL, CH, ML, and SM

P-200 = Percent passing the number 200 sieve
0.02 Mil. = Percent material below 0.02 millimeter grain size

*Approximate P-200 value equivalent for frost classification.
Value range based on typical, well-graded soil curves.
P-200 criteria in absence of hydrometer data.

** Very fine sand : greater than 50% of sand
fraction passing the number 100 sieve

CEA International Substation Switchyard
Anchorage, Alaska

FROST CLASSIFICATION LEGEND

January 2010

32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 4

MATERIAL DESCRIPTION

Approx. Elevation: 88 feet

S1: 11.1% Fines (F2 [P-200])

Frozen to loose, brown, slightly gravelly, slightly silty to silty SAND; moist [FILL]

S3: 5% Organic Matter

Stiff to very stiff, gray, sandy SILT with scattered organics; moist

Medium dense to very dense, gray SAND to slightly silty SAND; wet

S6: 6 inches of sand heave in auger prior to sampling
10.9% Fines (F2 [P-200])

S7: 6 inches of sand heave in auger prior to sampling

Occasional to scattered coal fragments below 29.5 feet bgs

S9: 1.5 feet of sand heave in auger prior to sampling

S10: 1 foot of sand heave in auger prior to sampling

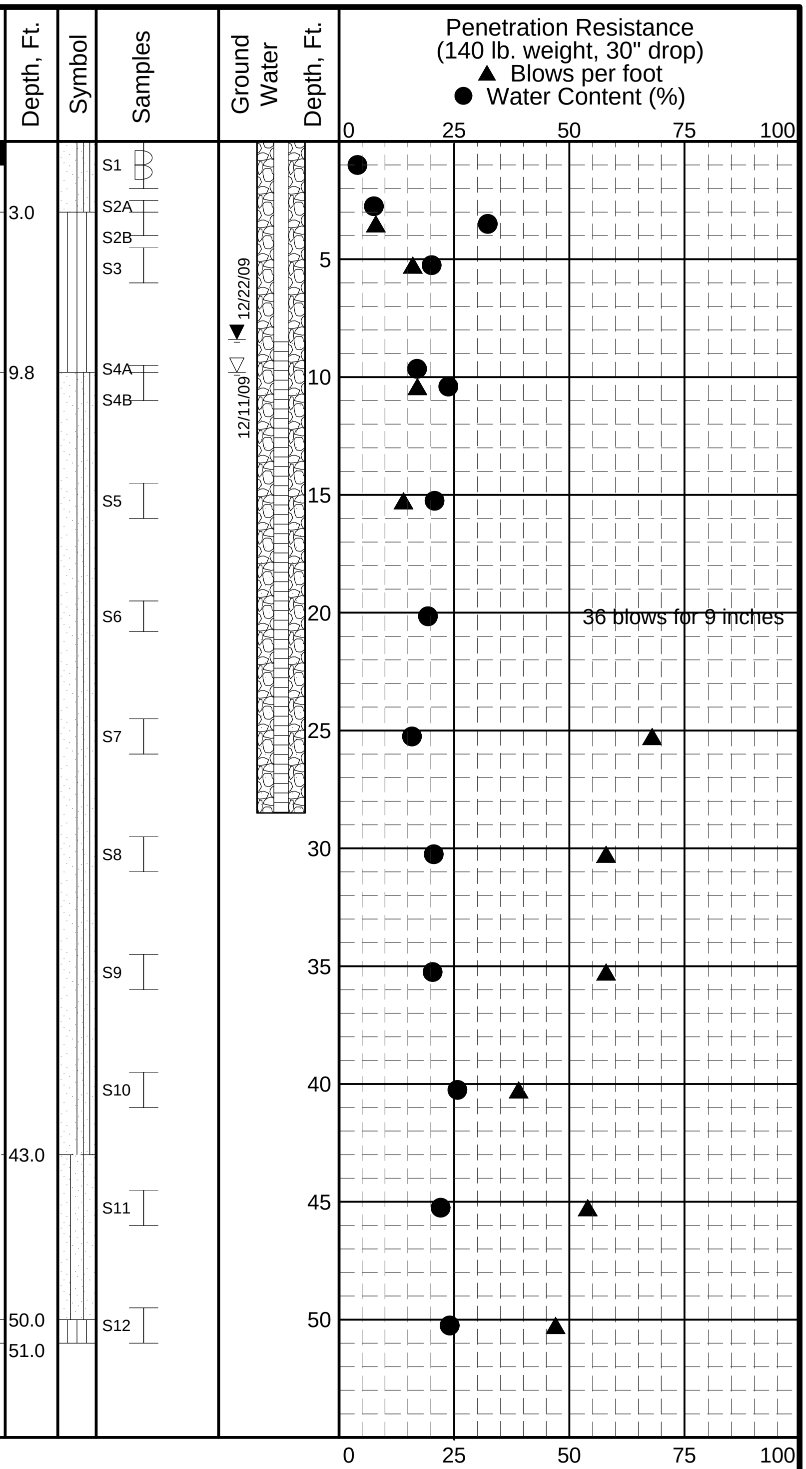
Dense, gray, silty SAND; wet

S11: Penetration resistance likely over-estimates actual conditions due to full sampler
41.1% Fines (F3 [P-200])

S12: Penetration resistance likely over-estimates actual conditions due to full sampler

Hard, gray, sandy SILT; moist to wet

Bottom of Boring
Boring Completed 12/11/2009



LEGEND

- * Sample Not Recovered
- Grab Sample
- 2" O.D. Split Spoon Sample
- ■ Frozen
- ▨ Blank Section, Cuttings Backfill
- ▨ Slotted Section, Cuttings Backfill
- ☐ Well Casing and Filter Sand
- ▥ Bentonite Seal
- ▽ Ground Water Level At Time Of Drilling
- ▼ Static Water Level

● Water Content (%)

Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

CEA International Substation Switchyard
Anchorage, Alaska

LOG OF BORING B-07

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. 5

MATERIAL DESCRIPTION

Approx. Elevation: 88 feet

Frozen to medium dense, brown, slightly silty, slightly gravelly SAND; moist [FILL]

Stiff to very stiff, gray to brown, sandy SILT; moist; occasional to scattered organics between 2.9 and 4.5 feet bgs
S3: 65.3% Fines (F4 [P-200])

Medium dense, reddish brown, silty SAND; moist

Medium dense, brown SAND; wet

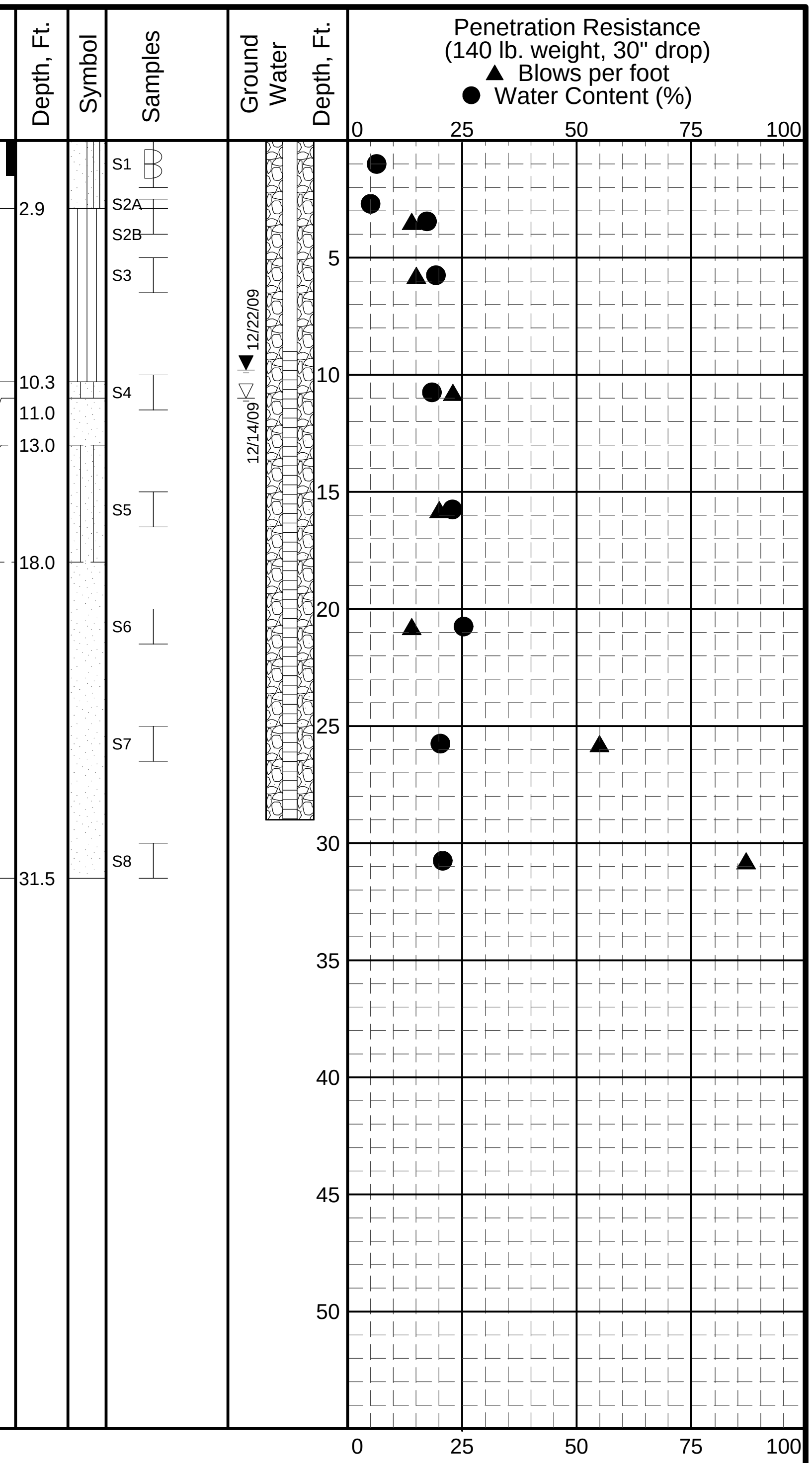
Medium dense, gray, silty SAND; wet
S5: 40.5% Fines (F3 [P-200])

Medium dense to dense, gray SAND to slightly silty SAND; wet

S7: 2 feet of sand heave in auger prior to sampling. Penetration resistance likely over-estimates actual conditions due to full sampler.

S8: 3 feet of sand heave in auger prior to sampling. Penetration resistance likely over-estimates actual conditions due to full sampler.

Bottom of Boring
Boring Completed 12/14/2009



LEGEND

- * Sample Not Recovered
- Ɓ Grab Sample
- ⌶ 2" O.D. Split Spoon Sample
- ■ Frozen
- ▨ Blank Section, Cuttings Backfill
- ▨ Slotted Section, Cuttings Backfill
- ⊞ Well Casing and Filter Sand
- ▥ Bentonite Seal
- ▽ Ground Water Level At Time Of Drilling
- ▼ Static Water Level

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

CEA International Substation Switchyard
Anchorage, Alaska

LOG OF BORING B-08

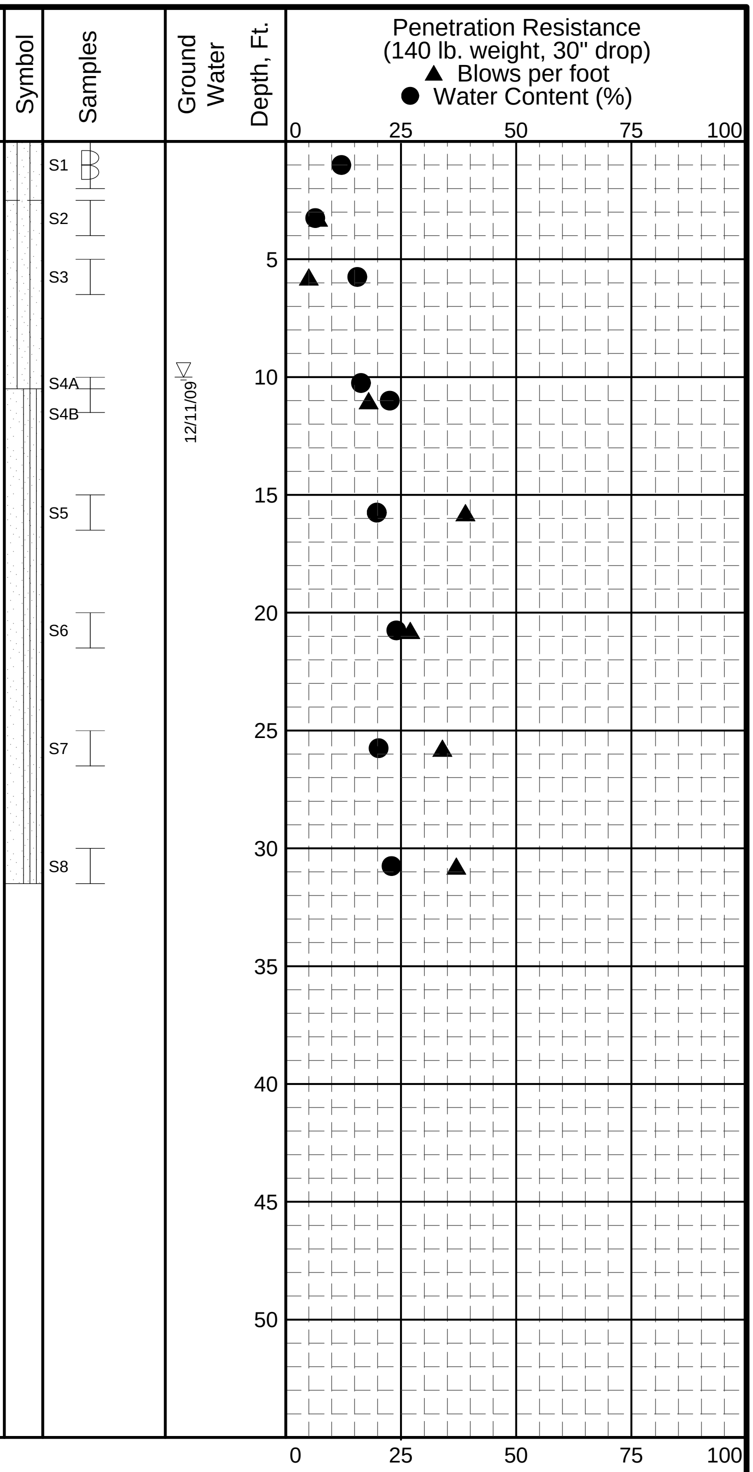
January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. 6

MATERIAL DESCRIPTION Approx. Elevation: 88 feet	Depth, Ft.
S1: 8% Gravel, 48% Sand, 44% Fines (F3 [P-200]) Frozen to loose, brown, slightly gravelly, silty, SAND; moist [FILL]	2.5
Loose to medium dense, brown, slightly gravelly, silty SAND; moist to wet	10.5
S4B: 5.1% Fines (F1 [P-200]) Medium dense to dense, brown to gray SAND to slightly silty SAND; wet S8: 1 foot of sand heave in auger prior to sampling Occasional coal fragments in Sample S8	31.5

Bottom of Boring
Boring Completed 12/11/2009



LEGEND

* Sample Not Recovered
Grab Sample
2" O.D. Split Spoon Sample
▮ Frozen
▽ Ground Water Level At Time Of Drilling

● Water Content (%)
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

CEA International Substation Switchyard
Anchorage, Alaska

LOG OF BORING B-09

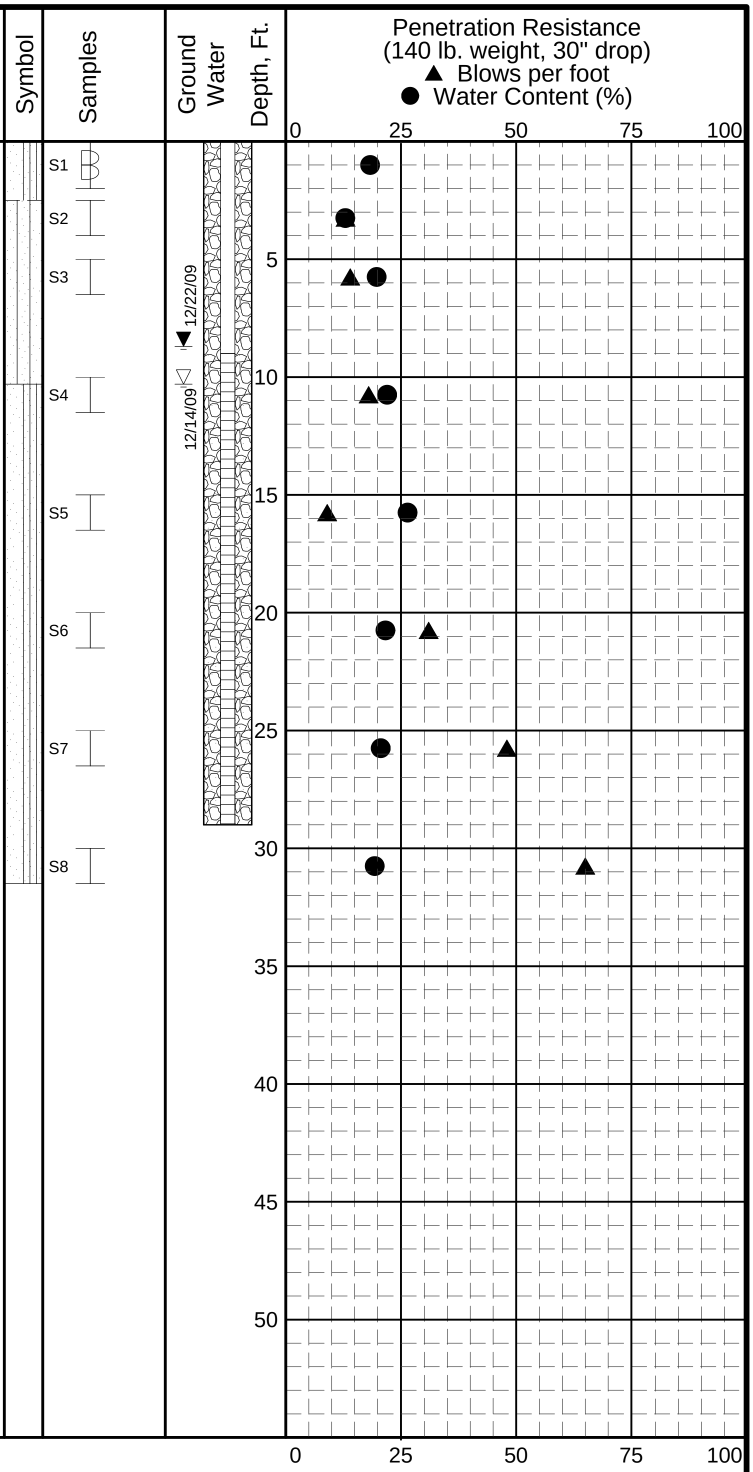
January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. 7

GEOTECHNICAL LOG 02086 LOGS.GPJ S&W GEO1.GDT 1/27/10

MATERIAL DESCRIPTION Approx. Elevation: 88 feet	Depth, Ft.
■ Frozen to medium dense, brown, slightly gravelly, slightly silty SAND; moist; scattered organics in upper 4 inches [FILL]	2.5
Medium dense, brown, silty SAND; moist S3: 47.7% Fines (F3 [P-200])	10.3
Loose to very dense, reddish brown to gray SAND to silty SAND; wet S5: 1.5 feet of sand heave in auger after sampling Occasional to scattered coal fragments and/or layers below 20 feet bgs S6: 5 feet of sand heave in auger prior to sampling. Penetration resistance likely under-estimates actual conditions due to disturbed soil beneath augers. 12.1% Fines (F2 [P-200])	31.5
Bottom of Boring Boring Completed 12/14/2009	



LEGEND

* Sample Not Recovered
Grab Sample
2" O.D. Split Spoon Sample
■ Frozen

Blank Section, Cuttings Backfill
Slotted Section, Cuttings Backfill
Well Casing and Filter Sand
Bentonite Seal
▽ Ground Water Level At Time Of Drilling
▼ Static Water Level

● Water Content (%)
Plastic Limit |—●—| Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

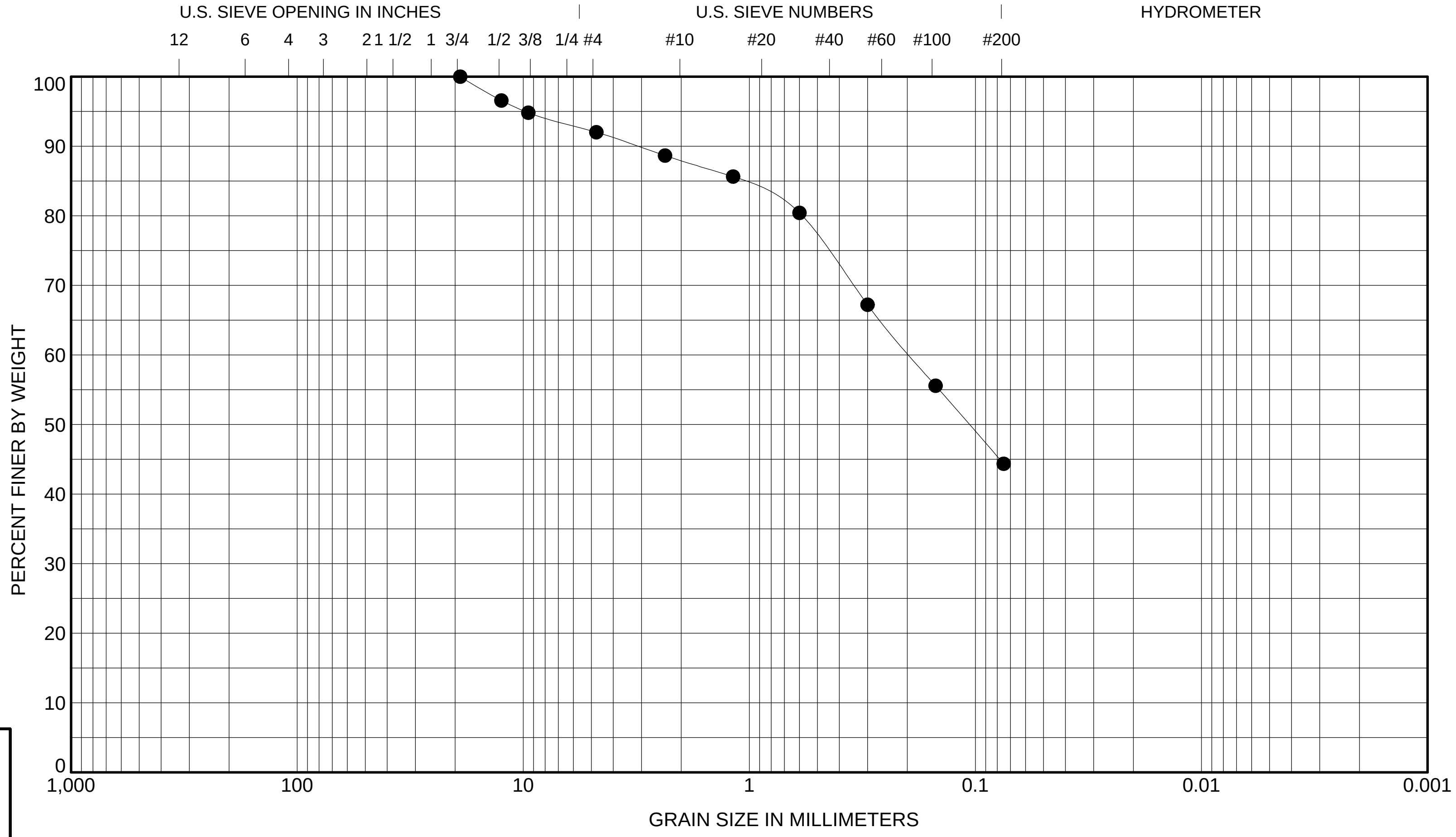
CEA International Substation Switchyard
Anchorage, Alaska

LOG OF BORING B-10

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. 8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	**B-09 S1**	**0.0 - 2.0**	**Slightly gravelly, silty SAND (SM)**					

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-09 S1**	**0.0 - 2.0**	**19**	**0.2**			**8**	**48**	**44**	

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

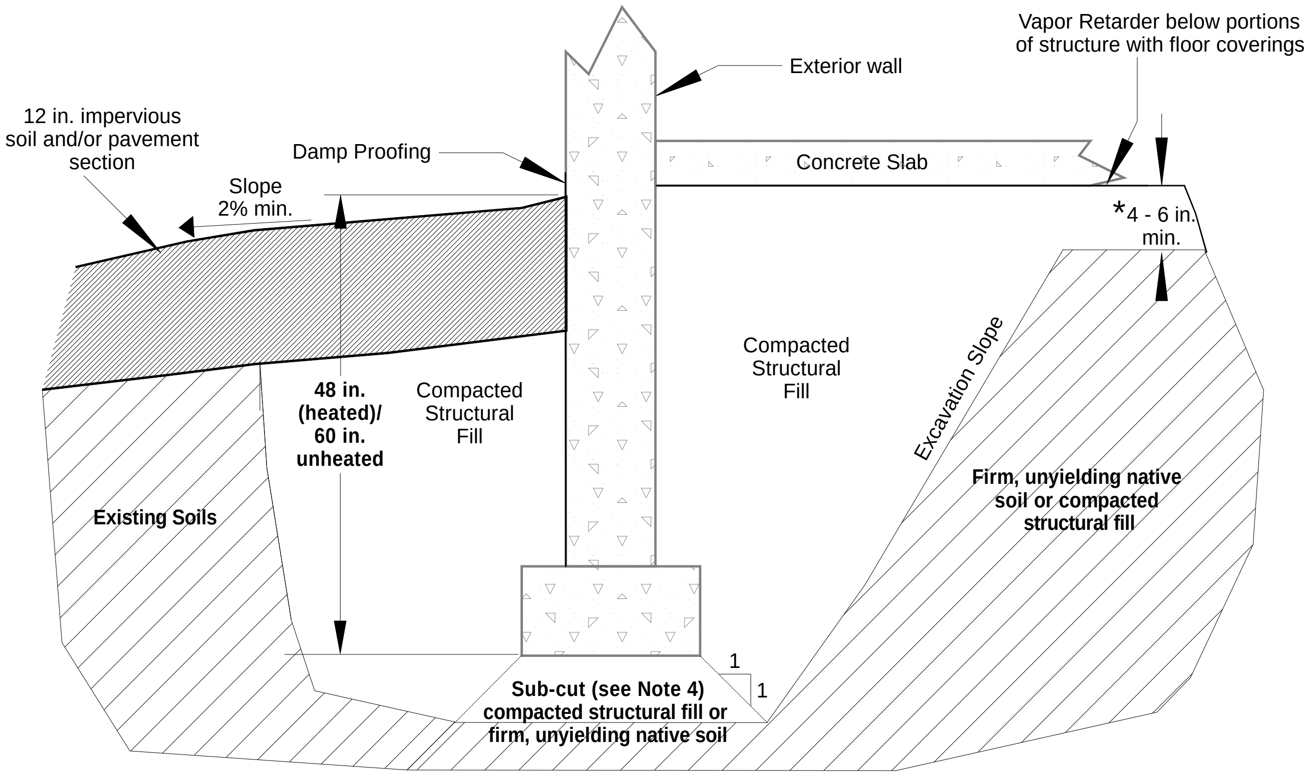
January 2010

32-1-02086

GRAIN SIZE CLASSIFICATION

CEA International Substation Switchyard
Anchorage, Alaska

Fig. 9
Sheet 1 of 1



TYPICAL FLOOR SLAB AND FOOTING DETAIL

NOTES:

1. If conditions render on-site soil unsuitable for compaction and drainage, backfill beneath footings and floor slabs with free-draining granular soil with not more than 6% (by weight based on minus 3/4" portion) passing No. 200 sieve (by wet sieving) with no plastic fines. This material would also meet the desired specifications for drainage sand and gravel.
2. All backfill should be placed in layers not exceeding 10 to 12 inches loose thickness and densely compacted. Structural fill should be compacted to 95% minimum, non-structural fill compacted to 90%, of ASTM D-1557.
3. Backfill within 18 inches of the wall should be placed in layers not exceeding 6 inches and densely compacted with hand-operated equipment. Heavy equipment should not be used for backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall.
4. If loose or organic materials are encountered beneath footings, the footing trenches should be overexcavated a minimum of 2 feet and unsuitable material removed and replaced with compacted structural fill. Excavations should extend inward as needed to remove material from beneath the slab. At a minimum, excavations should extend inward and outward at a distance equal to the depth of the excavation below footing grade.

* Drainage sand and gravel below the floor slab should be well-graded, free-draining and contain less than 6% fines (material passing the No. 200 sieve based on the minus 3/4-inch portion). It should be placed in maximum 6-inch loose lifts and compacted to 95% of its maximum density as determined by the Modified Proctor compaction procedure (ASTM D-1557).

DRAWING NOT TO SCALE

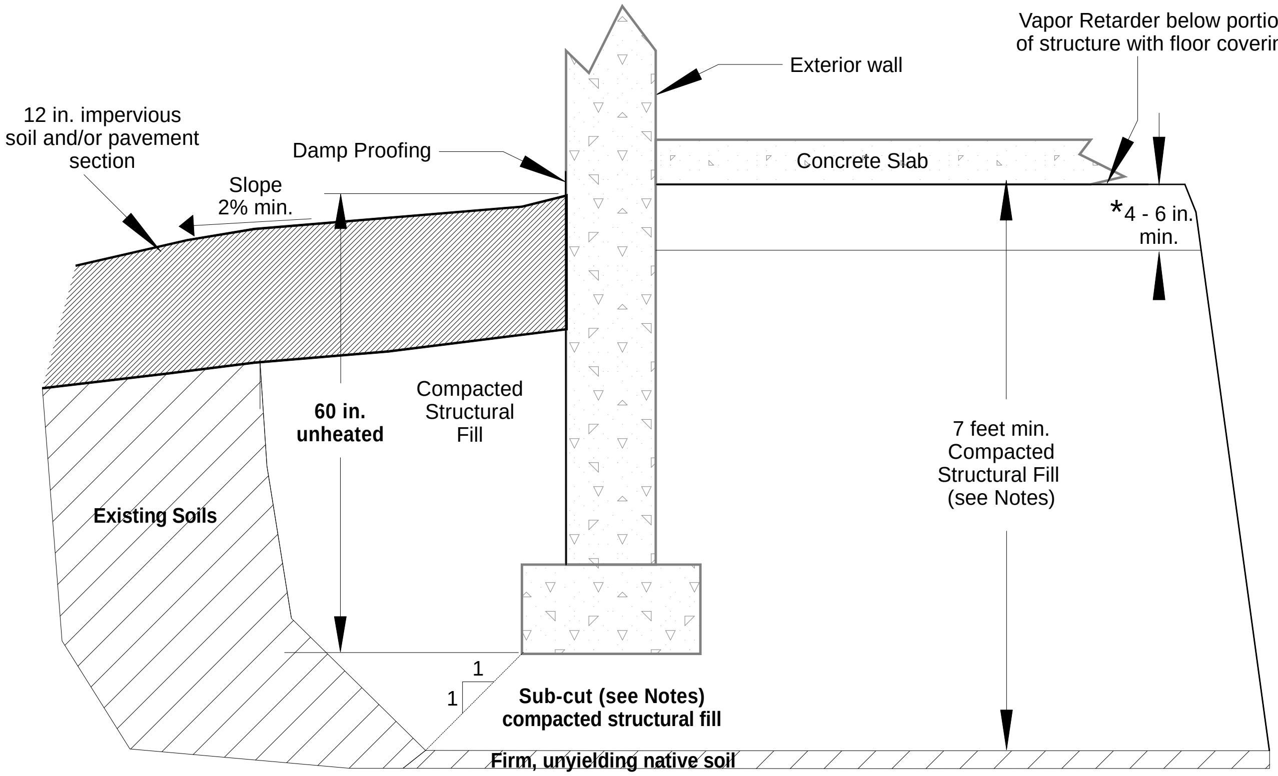
CEA International Substation Switchyard
Anchorage, Alaska

FLOOR SLAB AND FOOTING DETAIL
(Typical)

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 10



MOVEMENT SENSITIVE FLOOR SLAB AND FOOTING DETAIL

NOTES:

1. If conditions render on-site soil unsuitable for compaction and drainage, backfill beneath footings and floor slabs with free-draining granular soil with not more than 6% (by weight based on minus 3/4" portion) passing No. 200 sieve (by wet sieving) with no plastic fines. This material would also meet the desired specifications for drainage sand and gravel.
2. All backfill should be placed in layers not exceeding 10 to 12 inches loose thickness and densely compacted. Structural fill should be compacted to 95% minimum, non-structural fill compacted to 90%, of ASTM D-1557.
3. Backfill within 18 inches of the wall should be placed in layers not exceeding 6 inches and densely compacted with hand-operated equipment. Heavy equipment should not be used for backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall.
4. If loose or organic materials are encountered beneath footings, the footing trenches should be overexcavated a minimum of 2 feet and unsuitable material removed and replaced with compacted structural fill. Excavations should extend inward as needed to remove material from beneath the slab. At a minimum, excavations should extend inward and outward at a distance equal to the depth of the excavation below footing grade.
5. Sub-cut a minimum of 2 feet below unheated footing grade or 7 feet below unheated slab grade (whichever is greater) for structures sensitive to vertical displacement from heave or settlement. Sub-cut material to be replaced with compacted structural fill. Other sub-cuts as necessary to remove soft or loose soils beneath footings (see Sections 7.1 and 7.2 of report).

* Drainage sand and gravel below the floor slab should be well-graded, free-draining and contain less than 6% fines (material passing the No. 200 sieve based on the minus 3/4-inch portion). It should be placed in maximum 6-inch loose lifts and compacted to 95% of its maximum density as determined by the Modified Proctor compaction procedure (ASTM D-1557).

DRAWING NOT TO SCALE

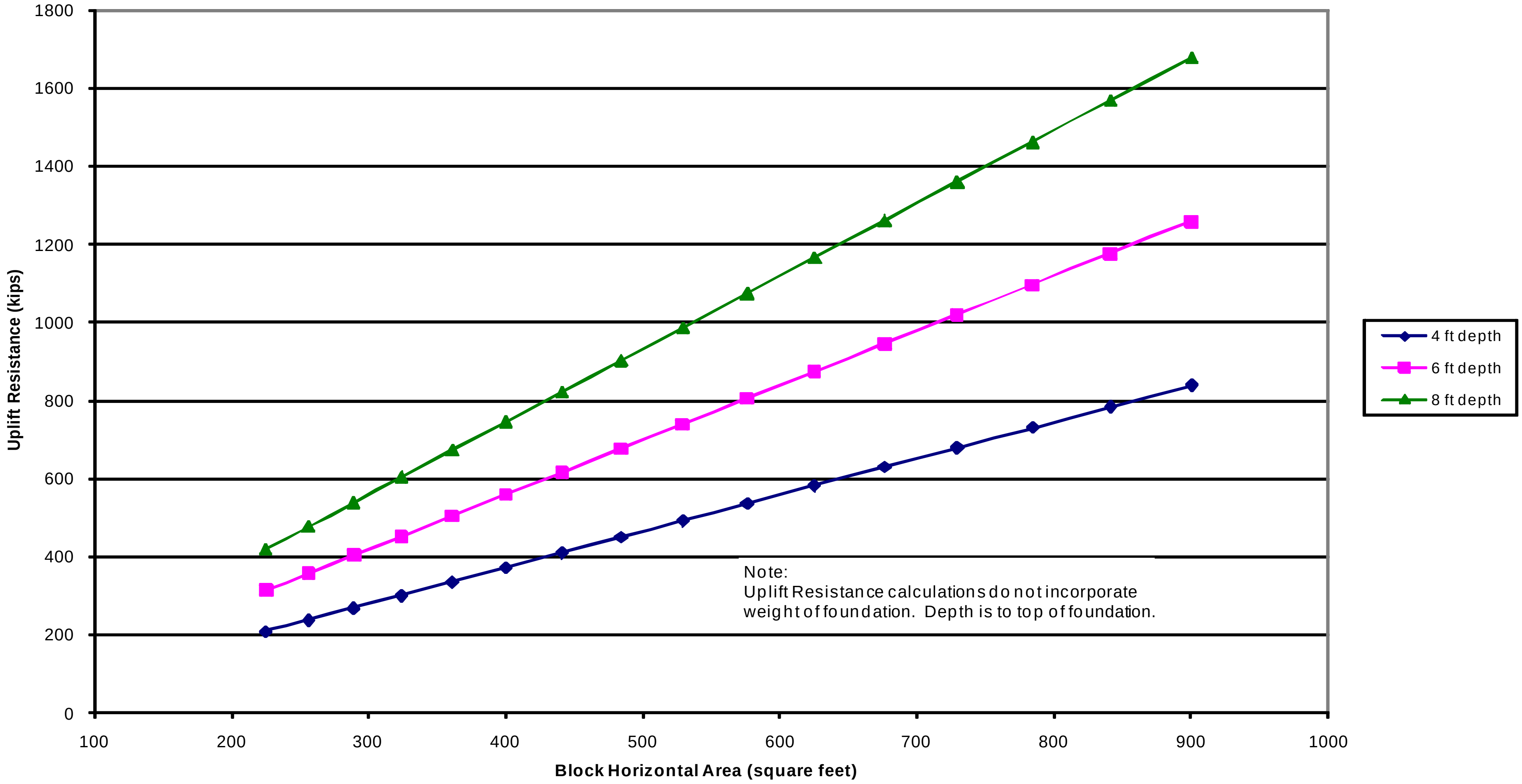
CEA International Substation Switchyard
Anchorage, Alaska

FLOOR SLAB AND FOOTING DETAIL
(Movement Sensitive)

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 11



Note:
Uplift Resistance calculations do not incorporate weight of foundation. Depth is to top of foundation.

Note: Uplift resistance is stated in terms of ultimate capacity for the case where the foundation element is lifted vertically. Assume the foundation element is embedded in compact granular fill (friction angle at least 36 degrees, unit weight at least 135 pcf or 72.6 pcf, respectively for soil above and below the seasonally high water table). Size the foundation footprint to provide an appropriate factor of safety (Shannon & Wilson recommends FS = 3 to limit deflections). Foundation element length should not exceed 1.5 times width.

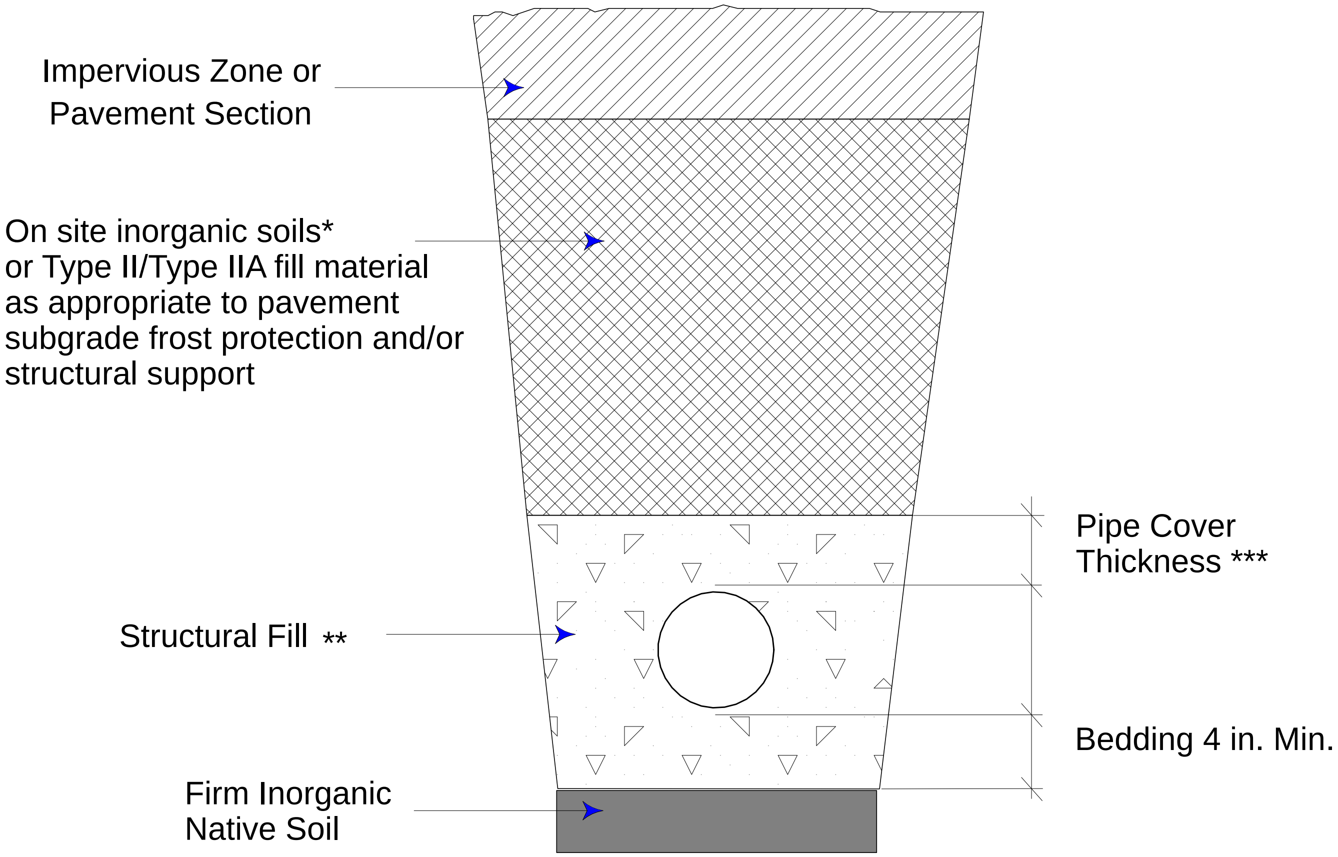
CEA International Substation Switchyard
Anchorage, Alaska

UPLIFT RESISTANCE VS. FOOTING SIZE

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 12



* Inorganic soils, 95% compaction below structural fill supporting footings, streets, etc., 90% compaction in non structural support areas.

** Inorganic clean sand or well-graded sand and gravel (max. particle size 2-inch diameter) with less than 6 percent fines. Fill to be compacted to 95% Modified Proctor maximum dry density (ASTM D 1557) or as recommended by pipe manufacturer for specific application.

*** Pipe cover thickness as specified by pipe manufacturer for specific application. Absent manufacturer specifications, pipe cover thickness depends on corrosion and structural support properties. In non-structural support and non-corrosive environment, minimum bedding fill thickness should be at or above springline of pipe. In non-structural support area with corrosive environment, pipe cover should extend at least 6-inches above top of pipe. In structural support area, minimum pipe cover should be 6-inches or one pipe diameter above top of pipe, whichever is greater.

NOTE:

OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as he/she is at the project on a day to day basis and is aware of changing conditions.

DRAWING NOT TO SCALE

CEA International Substation Switchyard
Anchorage, Alaska

UTILITY TRENCH DETAIL

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 13

GRADATION REQUIREMENTS

(Adapted from Municipality of Anchorage Standard Specifications, 2009)

LEVELING COURSE

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
English	Metric	
1 in.	25.0 mm	100
3/4 in.	19.0 mm	70 - 100
3/8 in.	9.5 mm	50 - 80
No. 4	4.75 mm	35 - 65
No. 8	2.36 mm	20 - 50
No. 50	0.30 mm	10 - 30
No. 200	0.075 mm	3 - 8*

STRUCTURAL FILL

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
3 in.	75 mm	100
3/4 in.	19.0 mm	85 - 100
No. 4	4.75 mm	50 - 90
No. 10	2.00 mm	25 - 60
No. 40	0.425 mm	4 - 30
No. 200	0.075 mm	2 - 5**

* The fraction passing the No. 200 sieve shall not exceed 75 percent of the fraction passing the No. 50 sieve.

** The fraction passing the No. 200 sieve shall not exceed 10 percent of the fraction passing the No. 4 sieve.

CEA International Substation Switchyard
Anchorage, Alaska

GRADATION REQUIREMENTS

January 2010
32-1-02086

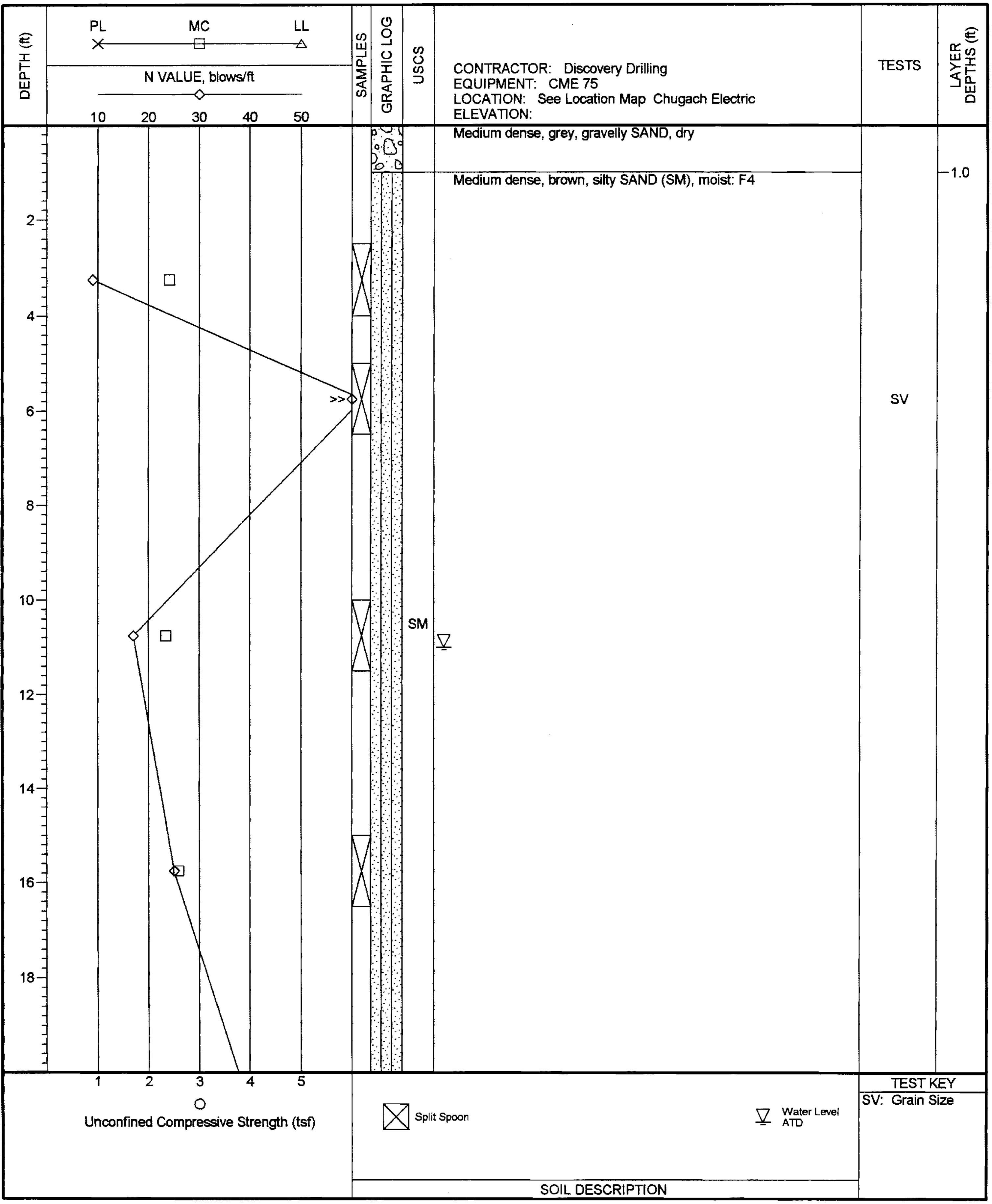
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 14

APPENDIX A

PREVIOUS EXPLORATIONS

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



Medium dense, grey, gravelly SAND, dry

Medium dense, brown, silty SAND (SM), moist: F4

TEST KEY
SV: Grain Size

LOG OF BORING CHUGACH SUBSTATION.GPJ HDEC.GDT 7/12/02 10:08

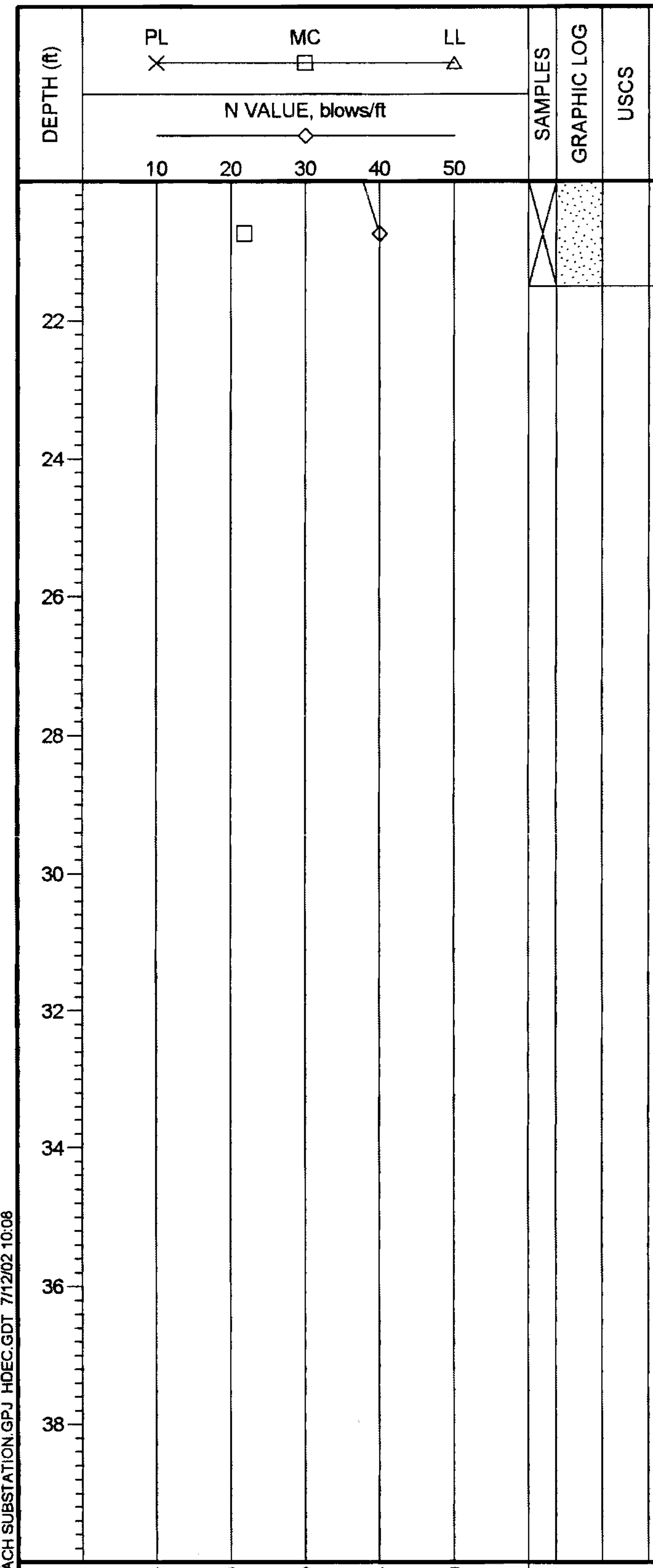
HDL HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 1
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE: FIGURE 1
SHEET: 1 of 2
JOB NUMBER: HD-02-108

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



SOIL DESCRIPTION	TESTS	LAYER DEPTHS (ft)
Dense, grey, slightly silty SAND, with charcoal pieces		20.0
BORING completed at 21.5 ft.		21.5

☒ Split Spoon

▽ Water Level ATD

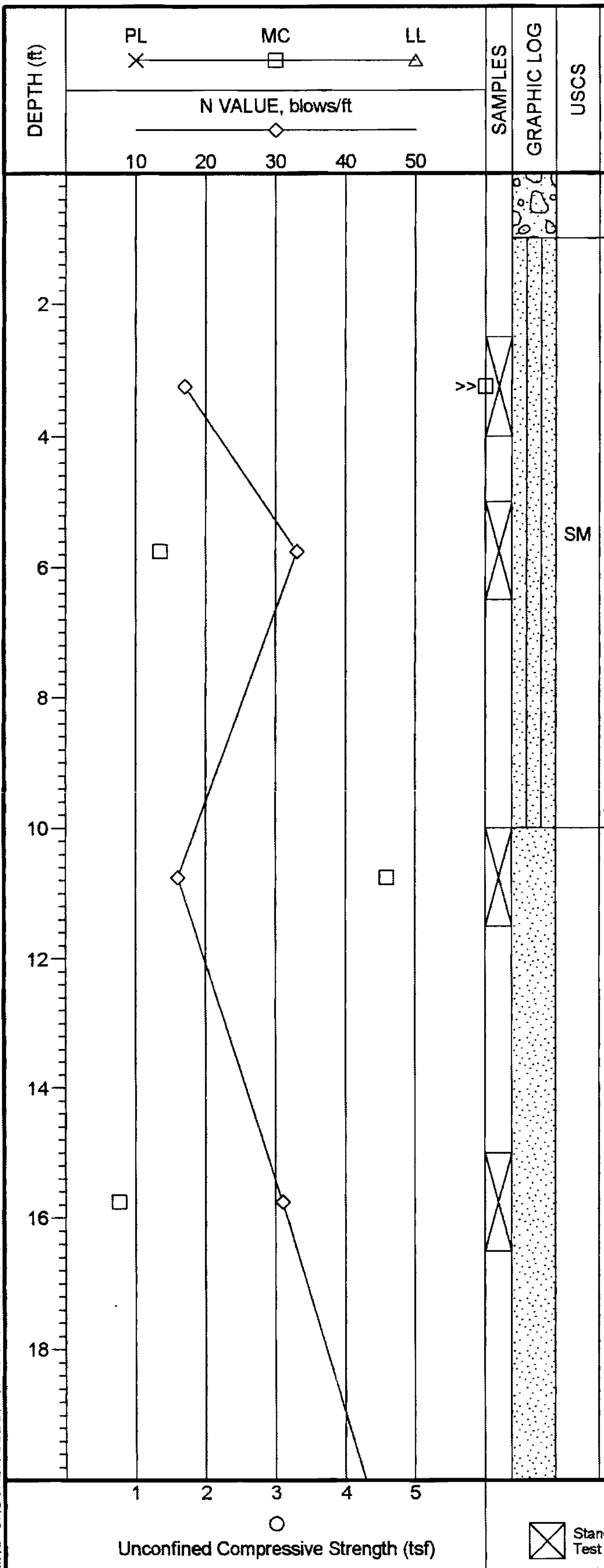
TEST KEY
SV: Grain Size

HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 1
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE: FIGURE 1
SHEET 2 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

SOIL DESCRIPTION	TESTS	LAYER DEPTHS (ft)
Medium dense, grey, gravelly SAND, dry		
Medium dense to dense, brown, silty SAND (SM), dry to moist: F4	SV	1.0
Medium dense to dense, grey, slightly silty, coarse SAND, wet, heaving sands, minor charcoal pieces.		10.0

☒ Standard Penetration Test

▽ Water Level ATD

TEST KEY
SV: Grain Size

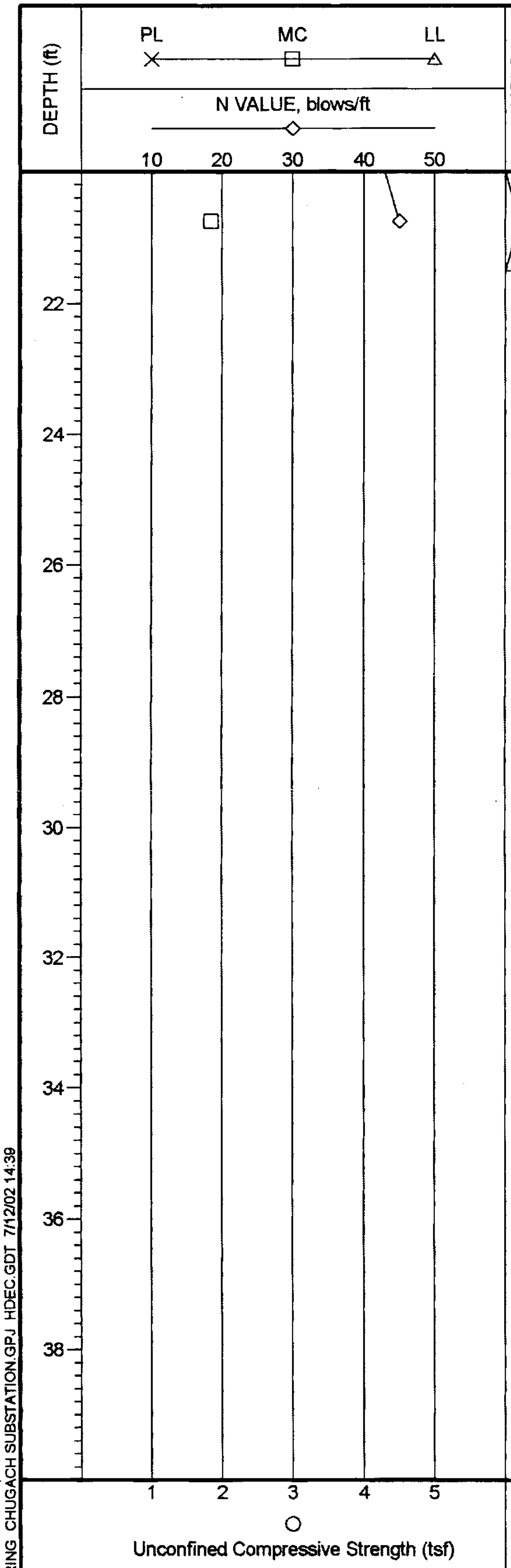
SOIL DESCRIPTION

HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 2
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE: FIGURE 2
SHEET 1 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

SAMPLES	GRAPHIC LOG	USCS		TESTS	LAYER DEPTHS (ft)
☒	▨		Medium dense to dense, grey, slightly silty, coarse SAND, wet, heaving sands, minor charcoal pieces.		21.5
			BORING completed at 21.5 ft.		

☒ Standard Penetration Test

▽ Water Level ATD

SOIL DESCRIPTION

TEST KEY
SV: Grain Size

HDL HATTENBURG DILLEY & LINNELL
Engineering Consultants

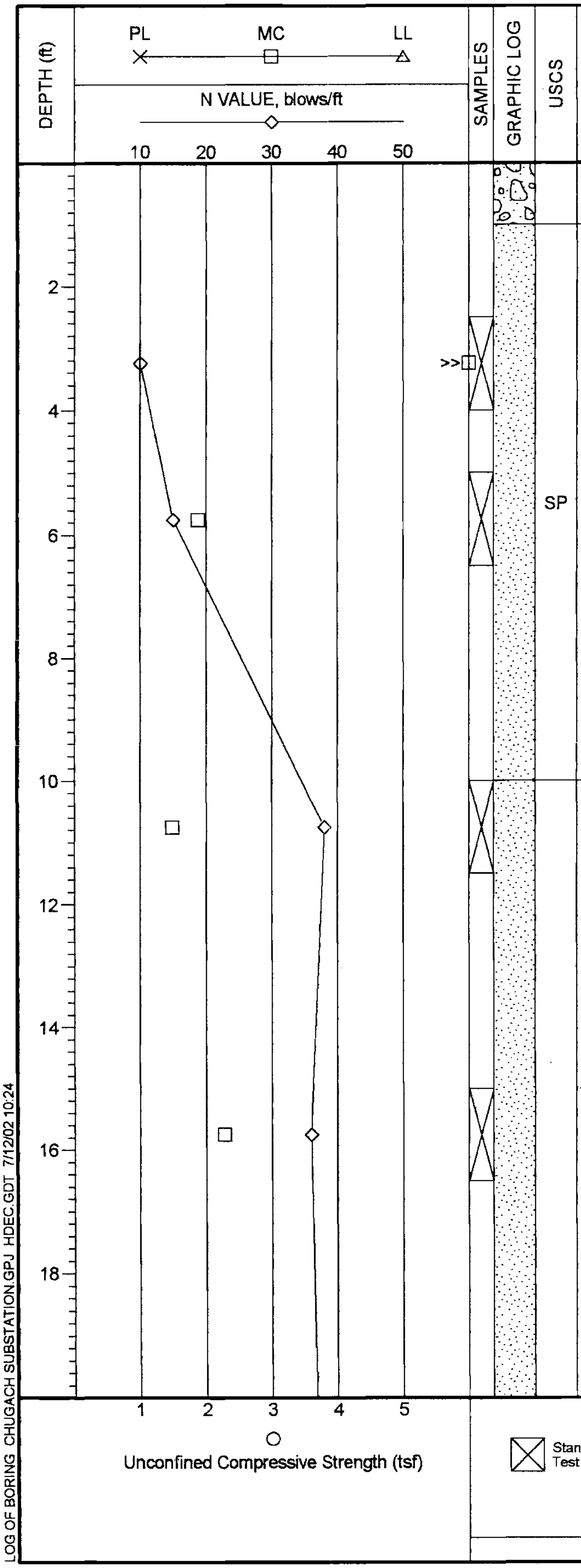
Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 2
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE
FIGURE 2
SHEET 2 of 2
JOB NUMBER: HD-02-108

LOG OF BORING CHUGACH SUBSTATION.GPJ HDEC.GDT 7/12/02 14:39

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



DESCRIPTION	TESTS	LAYER DEPTHS (ft)
Medium dense, grey, sandy GRAVEL		
A layer of geotextile Medium dense, brown to reddish brown, silty SAND, moist: F4		1.0
Medium dense to dense, brown to grey, slightly gravelly SAND (SP), pieces of charcoal, wet	SV	10.0

☒ Standard Penetration Test

▽ Water Level ATD

SOIL DESCRIPTION

TEST KEY
SV: Grain Size

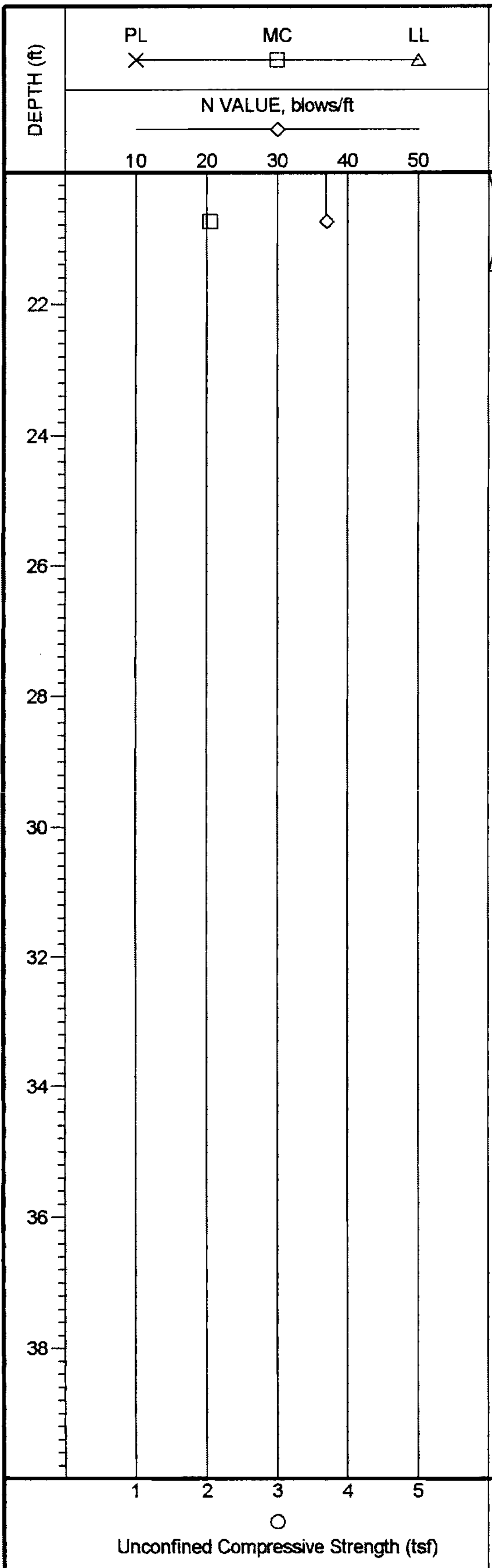
HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 3
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE
FIGURE 3
SHEET 1 of 2
JOB NUMBER HD-02-108

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



SAMPLES	GRAPHIC LOG	USCS		TESTS	LAYER DEPTHS (ft)
☒			Medium dense to dense, brown to grey, slightly gravelly SAND (SP), pieces of charcoal, wet		
			BORING completed at 21.5 ft.		21.5

☒ Standard Penetration Test

$\underline{\nabla}$ Water Level ATD

SOIL DESCRIPTION

TEST KEY
SV: Grain Size

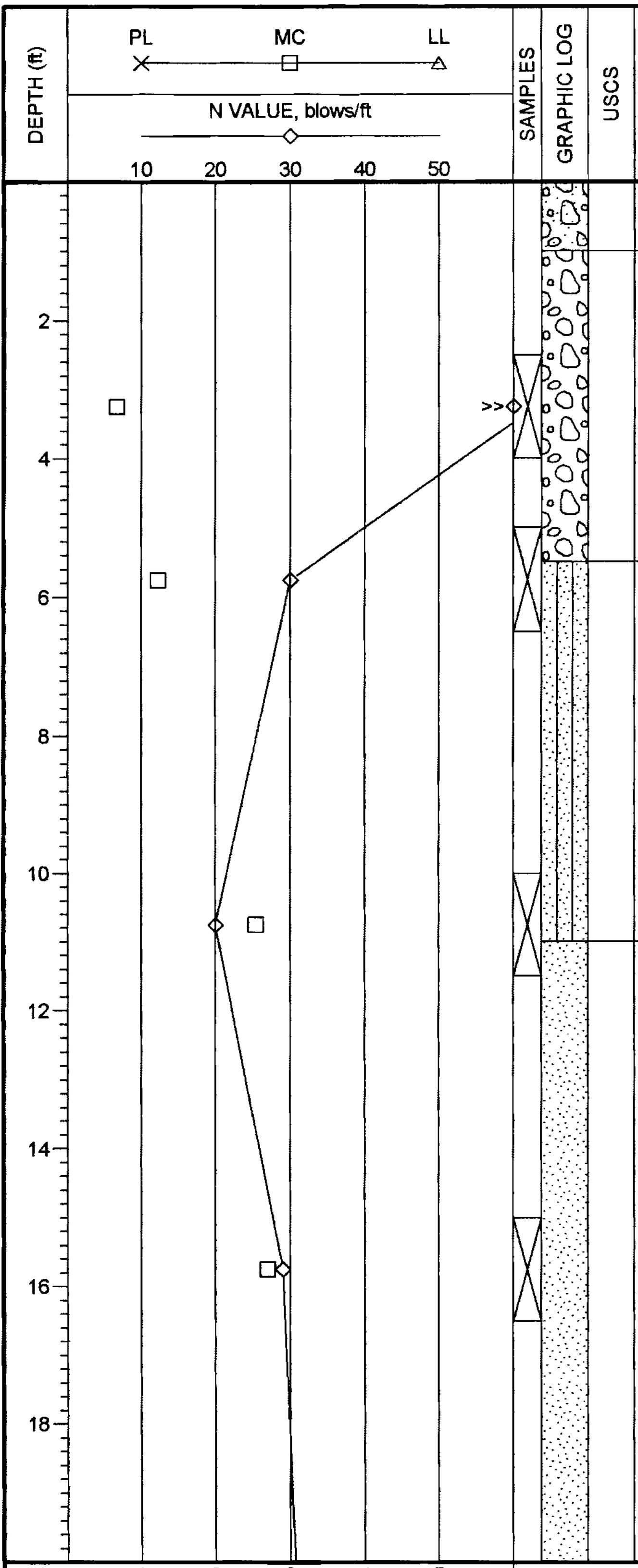
HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 3
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 3
SHEET 2 of 2
JOB NUMBER: HD-02-108

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



	TESTS	LAYER DEPTHS (ft)
Medium dense, grey, sandy GRAVEL, dry		
A layer of geotextile		1.0
Very dense, grey, sandy GRAVEL, dry, pieces of charcoal		
Dense, brown, silty, coarse SAND, moist		5.5
Dense, brown-grey, slightly silty SAND, wet large pieces of charcoal at depth		11.0

1 2 3 4 5
○
Unconfined Compressive Strength (tsf)

☒ Standard Penetration Test

▽ Water Level ATD

TEST KEY

SOIL DESCRIPTION

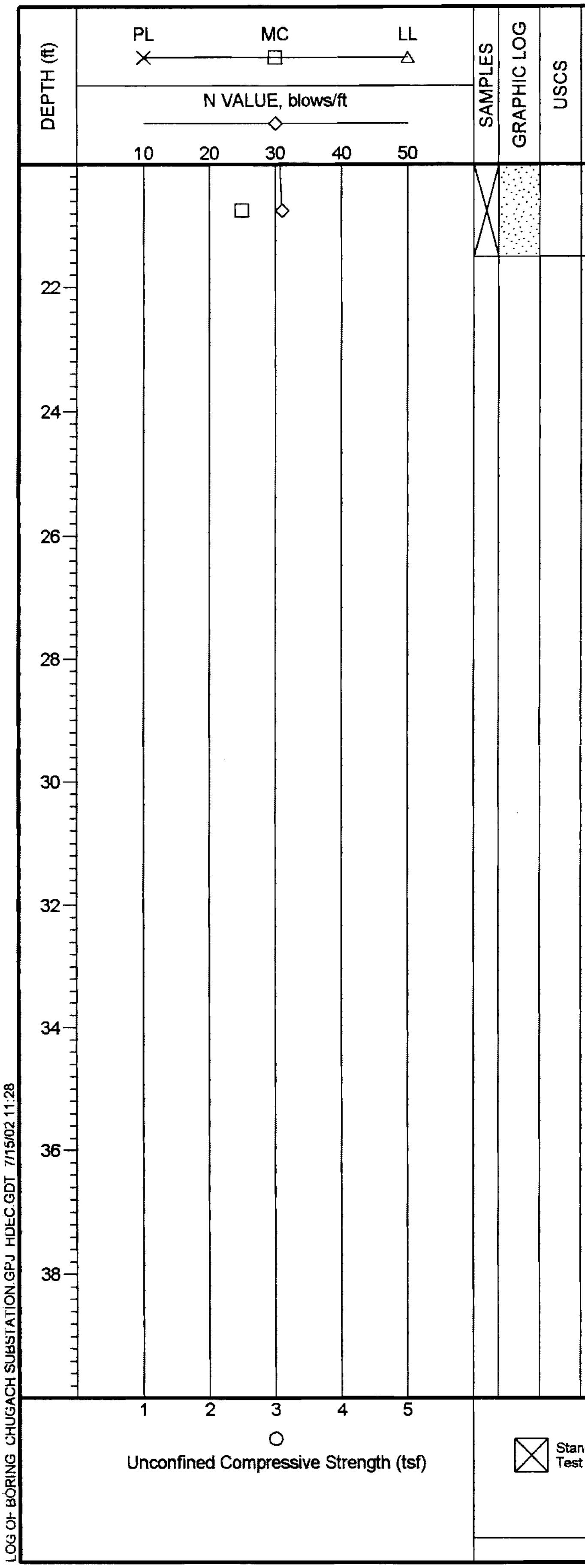
LOG OF BORING CHUGACH SUBSTATION.GPJ HDEC.GDT 7/15/02 11:28

HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 4
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 4
SHEET 1 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

SOIL DESCRIPTION	TESTS	LAYER DEPTHS (ft)
Dense, brown-grey, slightly silty SAND, wet large pieces of charcoal at depth		
BORING completed at 21.5 ft.		21.5

☒ Standard Penetration Test

$\underline{\nabla}$ Water Level ATD

SOIL DESCRIPTION

TEST KEY

LOG OF BORING CHUGACH SUBSTATION.GPJ HDEC.GDT 7/15/02 11:28

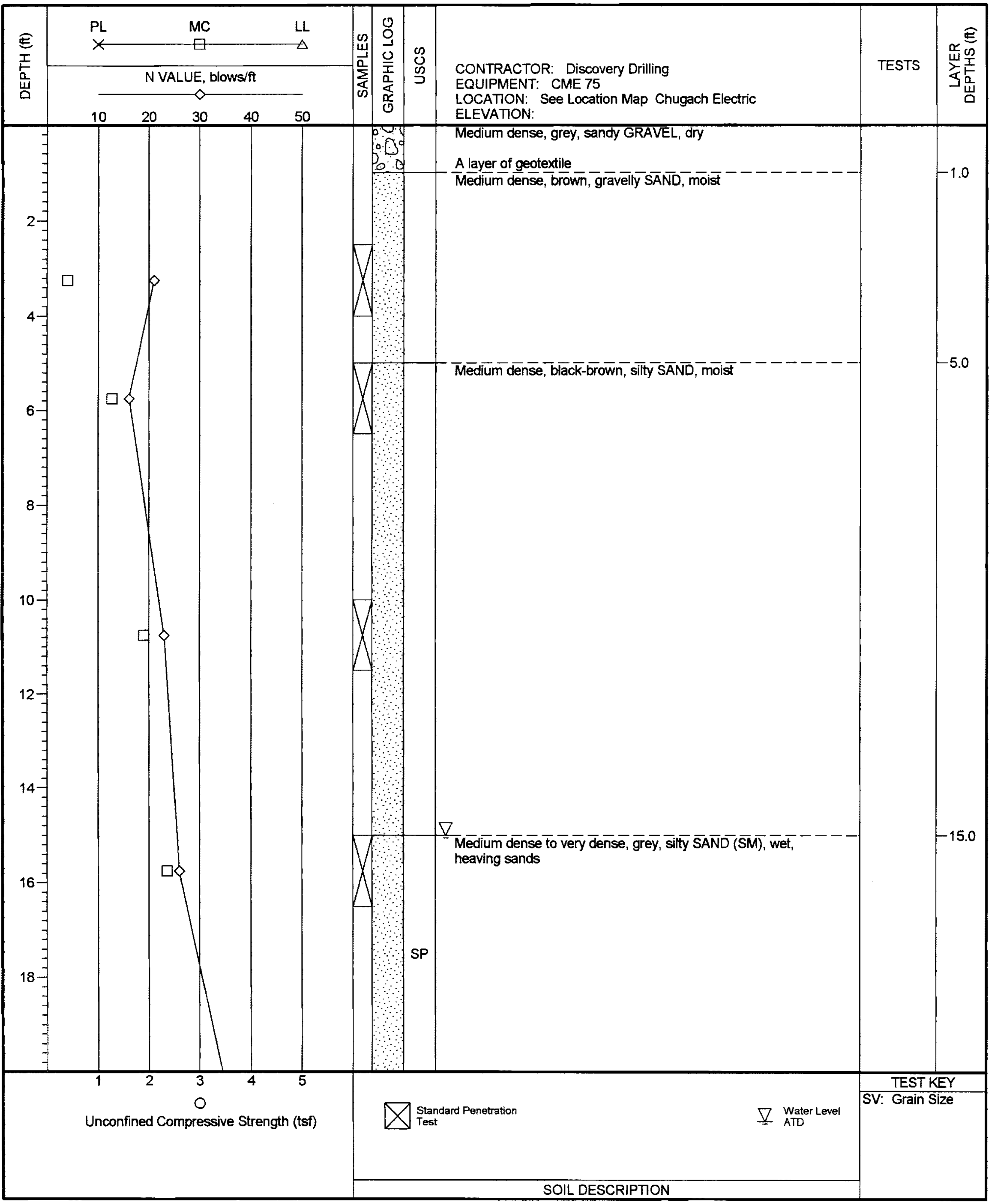
HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 4
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 4
SHEET 2 of 2
JOB NUMBER HD-02-108

CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:



Medium dense, grey, sandy GRAVEL, dry

A layer of geotextile

Medium dense, brown, gravelly SAND, moist

Medium dense, black-brown, silty SAND, moist

Medium dense to very dense, grey, silty SAND (SM), wet, heaving sands

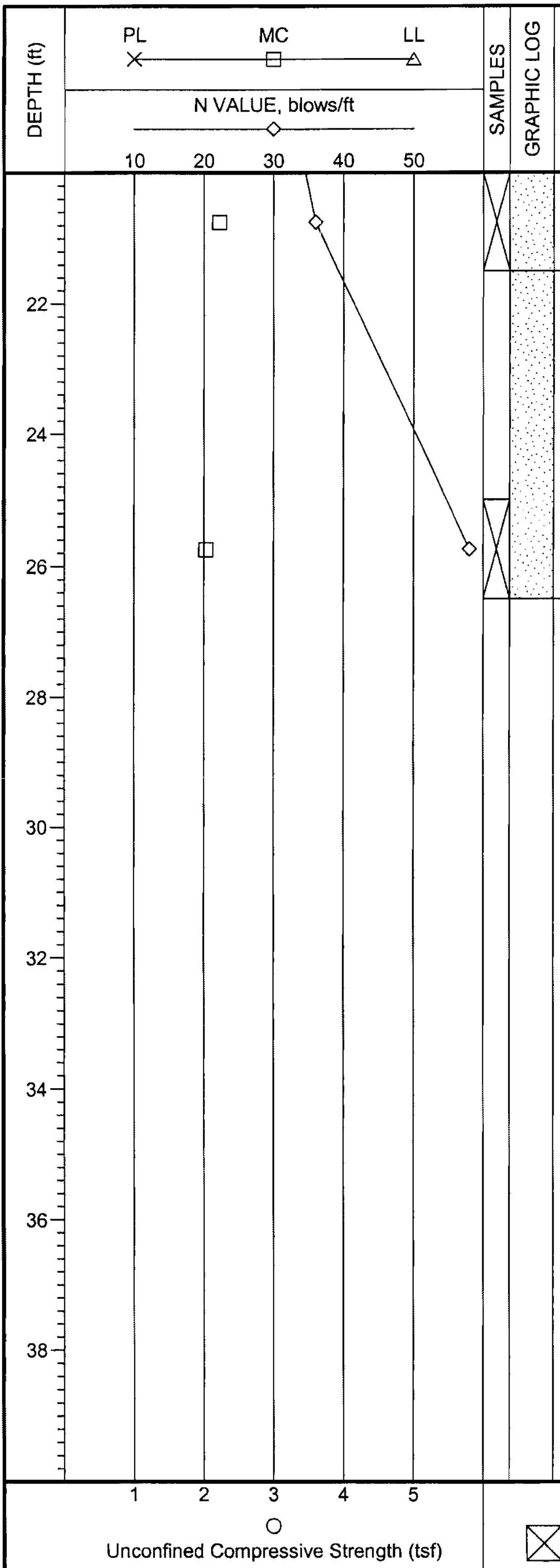
TEST KEY
SV: Grain Size

HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 5
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 5
SHEET 1 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

USCS		TESTS	LAYER DEPTHS (ft)
SP	Medium dense to very dense, grey, silty SAND (SM), wet, heaving sands	SV	
	silty SAND with slight charcoal, wet		21.5
	BORING completed at 26.5 ft.		26.5

☒ Standard Penetration Test

∇ Water Level ATD

SOIL DESCRIPTION

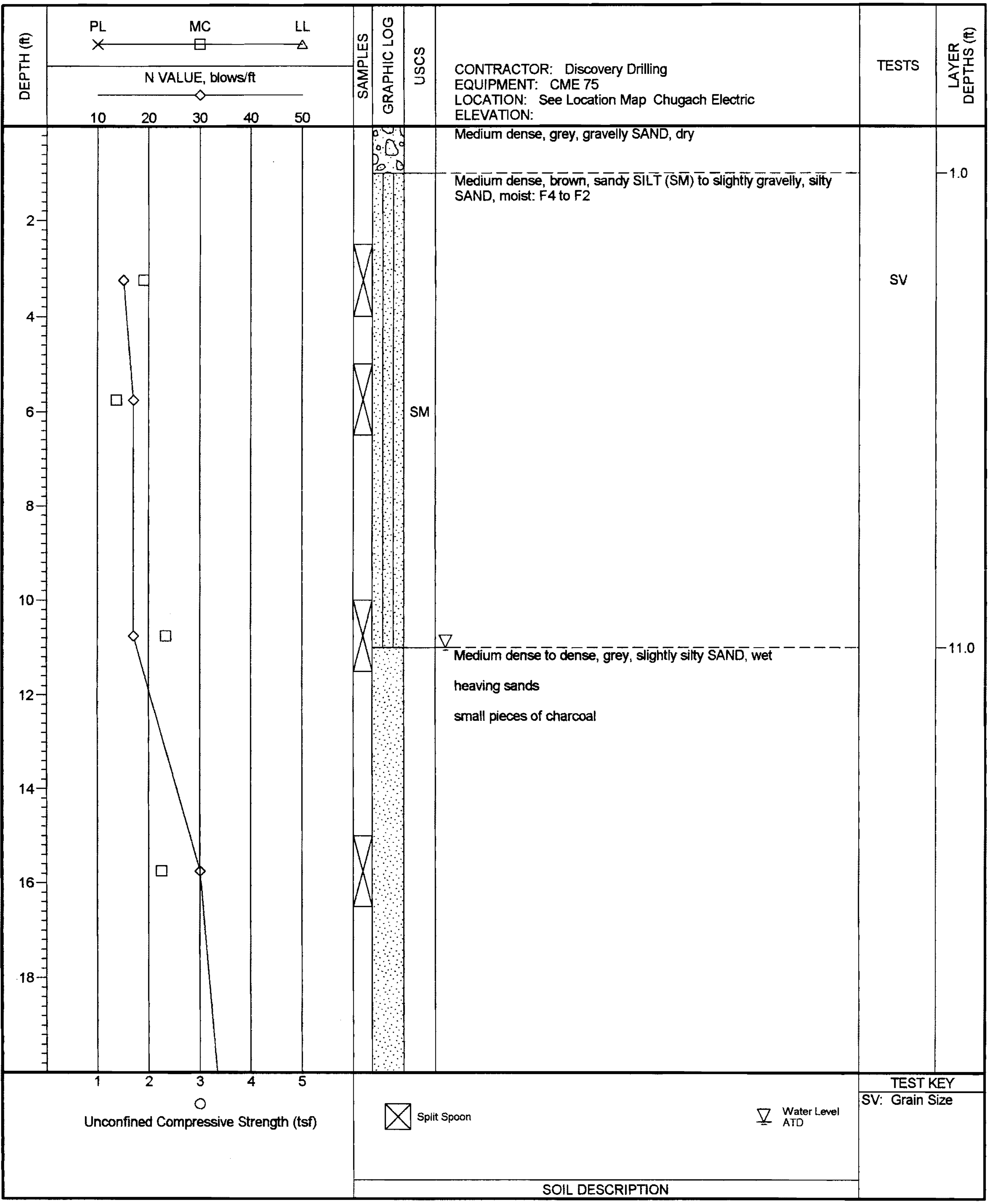
TEST KEY
SV: Grain Size

HDL HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 5
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 5
SHEET 2 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

SOIL DESCRIPTION	TESTS	LAYER DEPTHS (ft)
Medium dense, grey, gravelly SAND, dry		1.0
Medium dense, brown, sandy SILT (SM) to slightly gravelly, silty SAND, moist: F4 to F2	SV	11.0
Medium dense to dense, grey, slightly silty SAND, wet heaving sands small pieces of charcoal		

TEST KEY
SV: Grain Size

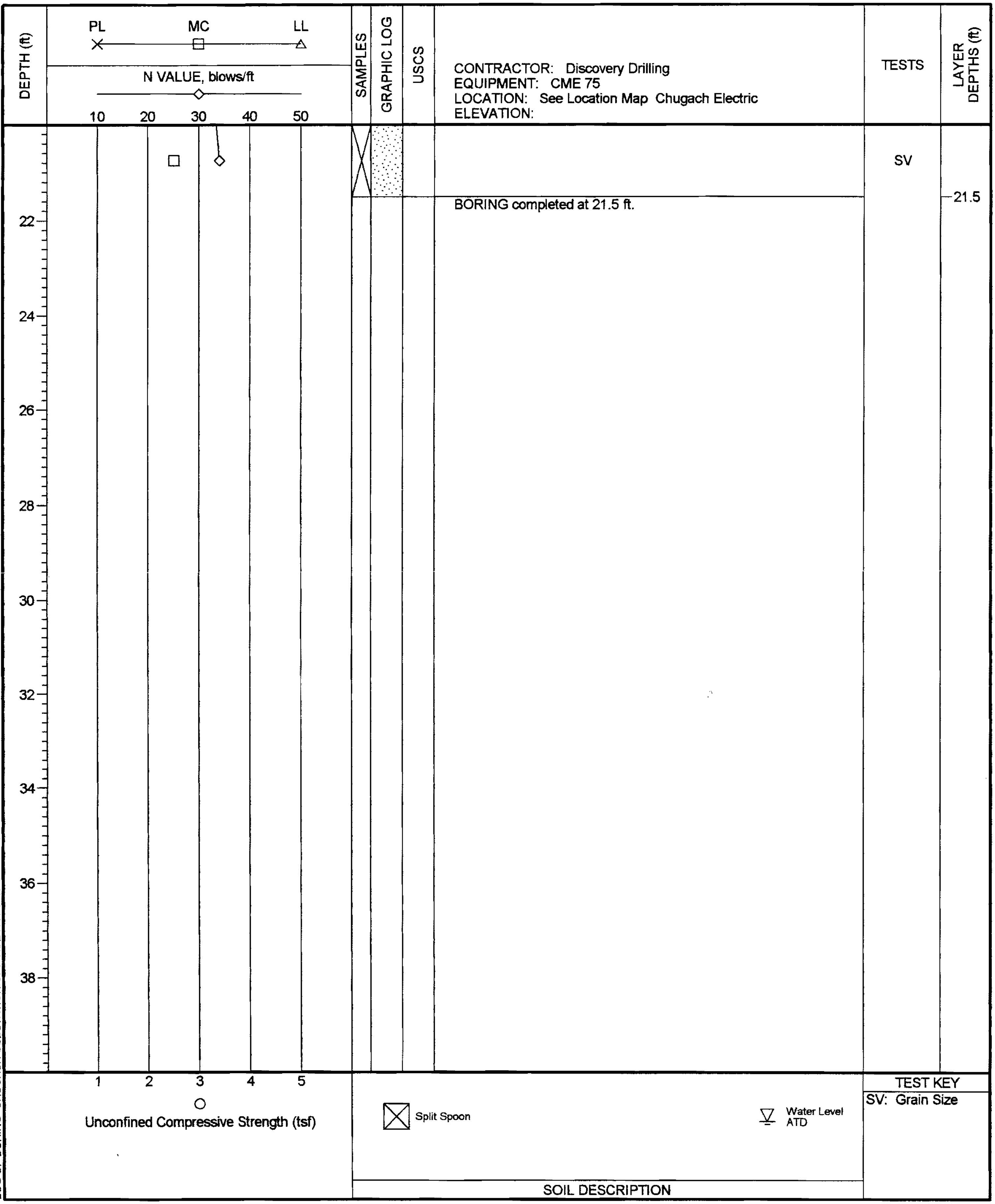
LOG OF BORING CHUGACH SUBSTATION.GPJ HDEC.GDT 7/12/02 10:36

HATTENBURG DILLEY & LINNELL
Engineering Consultants

Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 6
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE
FIGURE 6
SHEET 1 of 2
JOB NUMBER: HD-02-108



CONTRACTOR: Discovery Drilling
EQUIPMENT: CME 75
LOCATION: See Location Map Chugach Electric
ELEVATION:

BORING completed at 21.5 ft.

TEST KEY
SV: Grain Size

SOIL DESCRIPTION

HATTENBURG DILLEY & LINNELL
Engineering Consultants

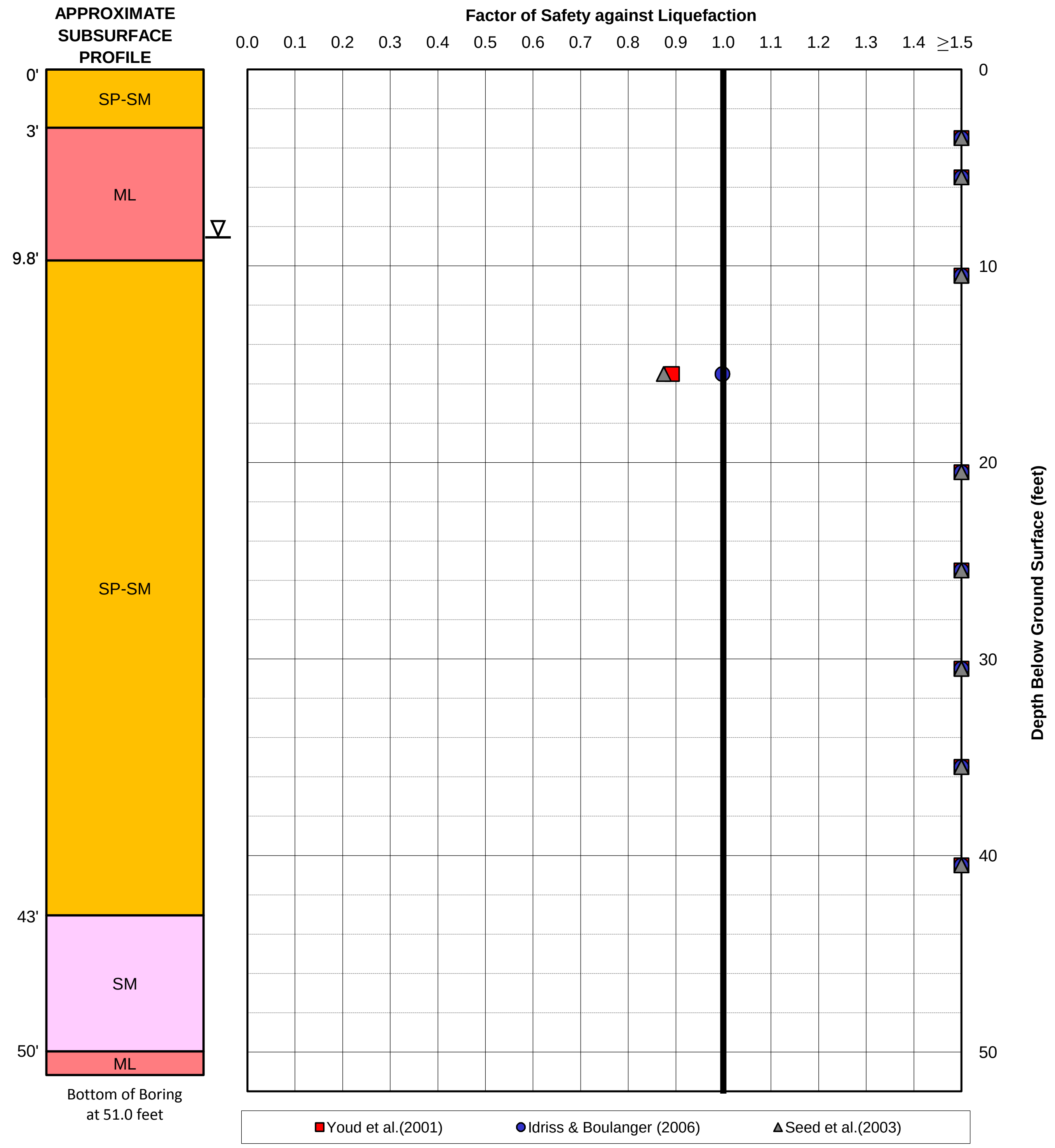
Chugach Substation
Chugach Electric
Anchorage, AK

LOG OF BORING 6
DRILLED BY: JD
DATE DRILLED: June 17, 2002

SHEET TITLE FIGURE 6
SHEET 2 of 2
JOB NUMBER HD-02-108

APPENDIX B

LIQUEFACTION ANALYSES



NOTES

1. See main text for references.

2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

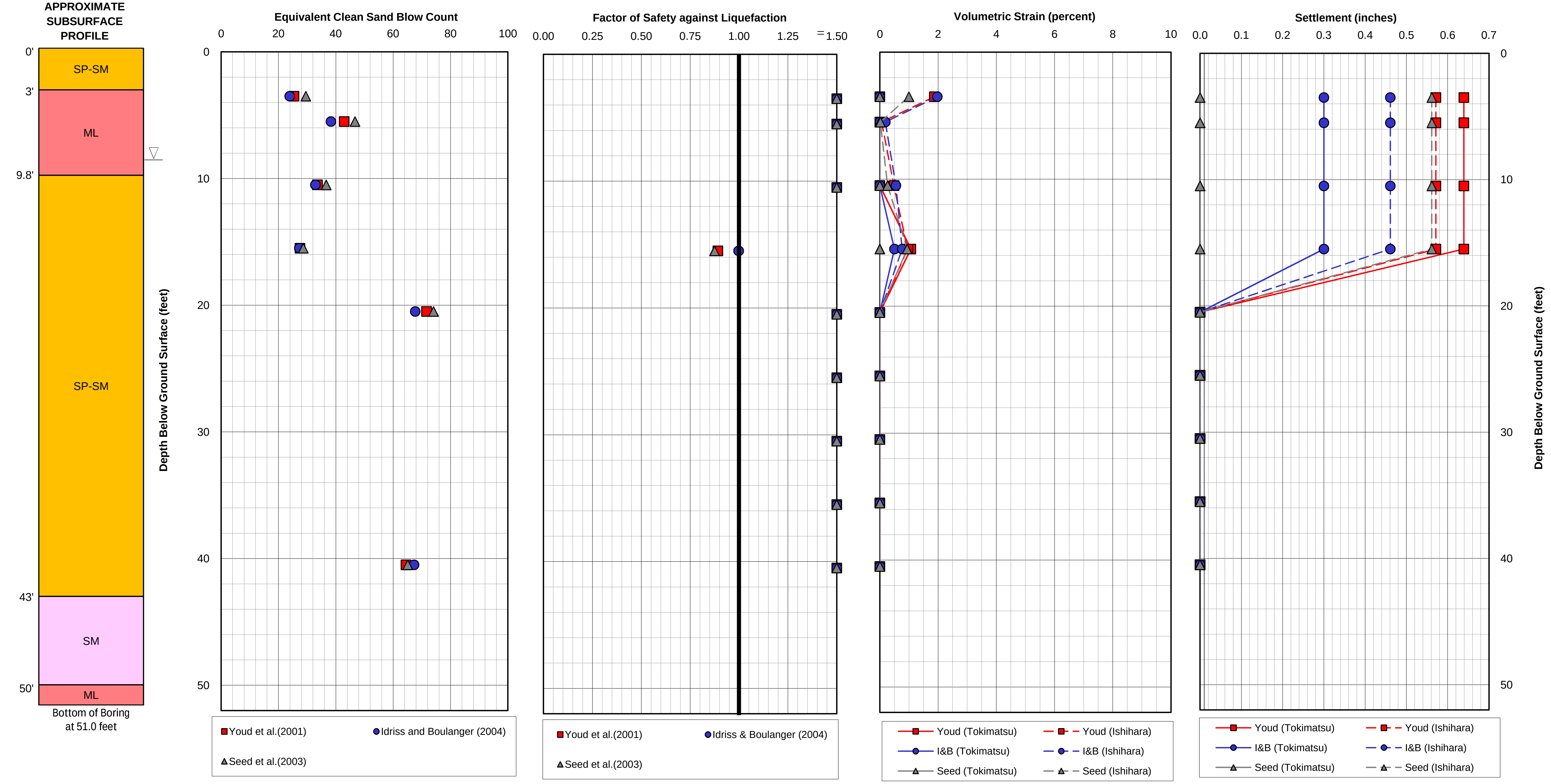
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

RESULTS OF LIQUEFACTION ANALYSES
BORING B-07
M = 9.20, PGA = 0.28

January 2010
32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-1



NOTES

1. See main text for references.

Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

SEISMIC SETTLEMENT
BORING B-07
M = 9.2, PGA = 0.28

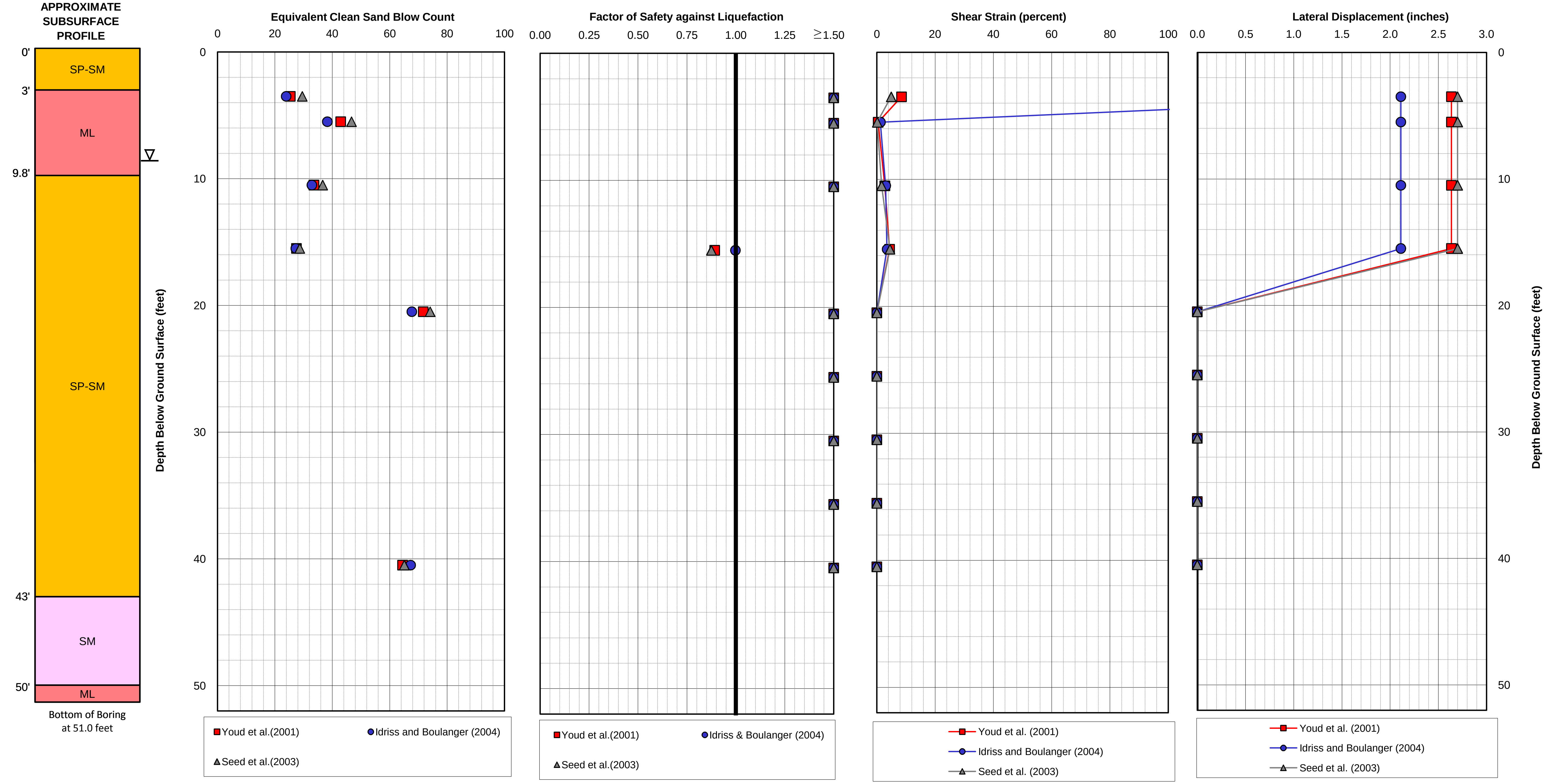
January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2

B-07 SPT_Liquefaction_v2.1.5 Printed: 1/27/2010 4:13 PM

RCH



NOTES

1. See main text for references.

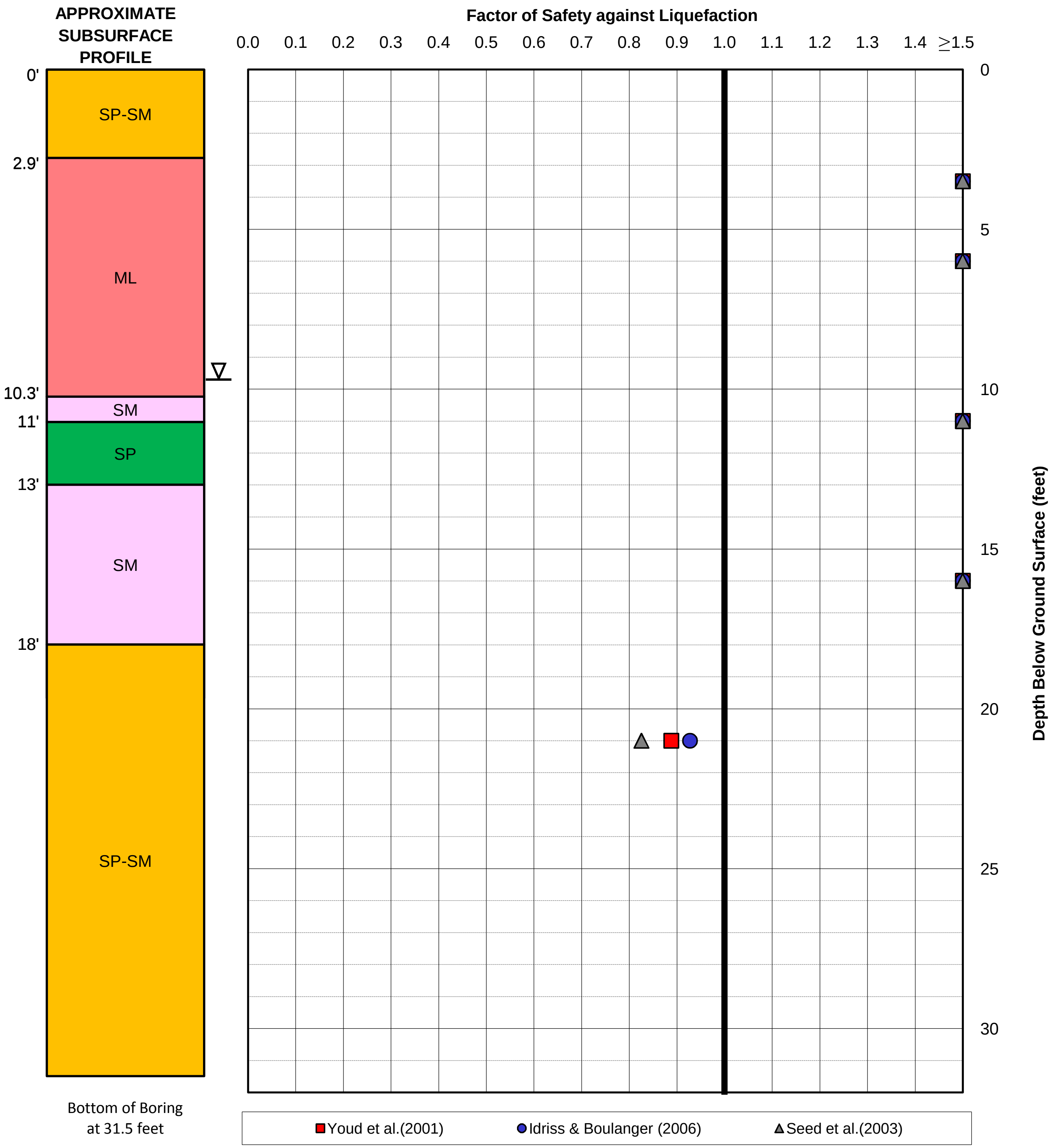
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

LATERAL DISPLACEMENT
BORING B-07
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-3



NOTES

1. See main text for references.

2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

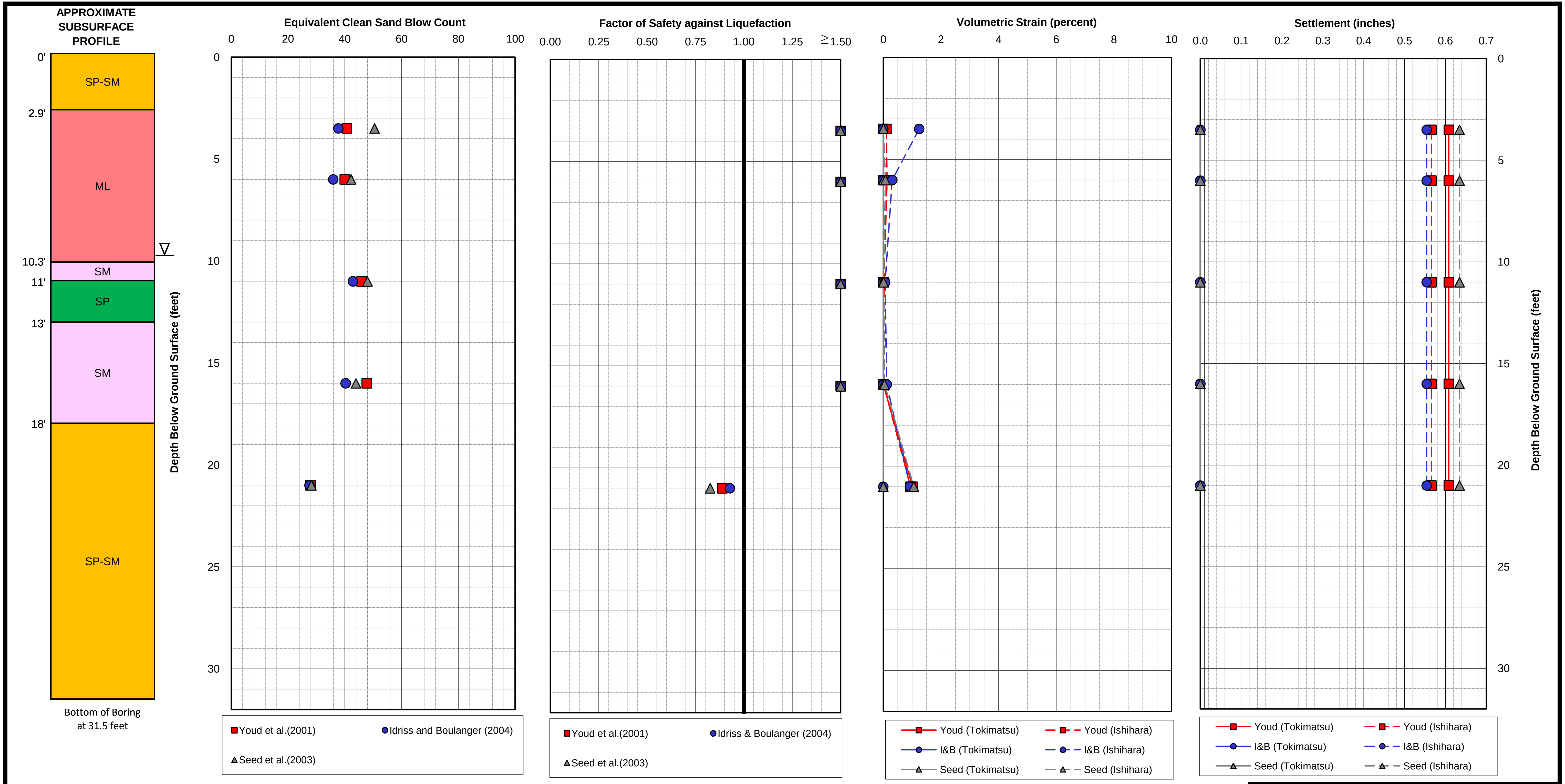
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

RESULTS OF LIQUEFACTION ANALYSES
BORING B-08
M = 9.20, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-4



NOTES

1. See main text for references.

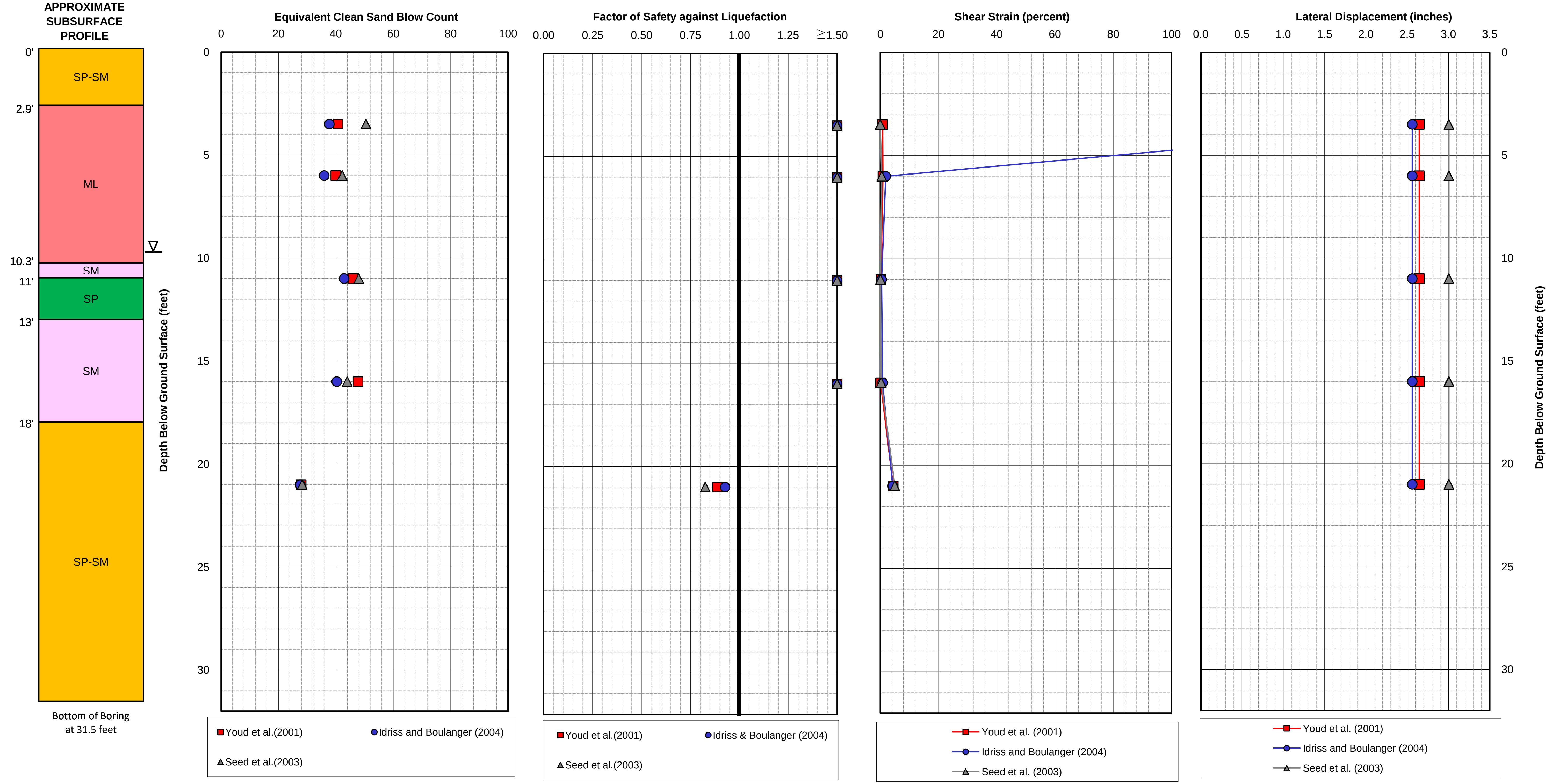
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

SEISMIC SETTLEMENT
BORING B-08
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-5



NOTES

1. See main text for references.

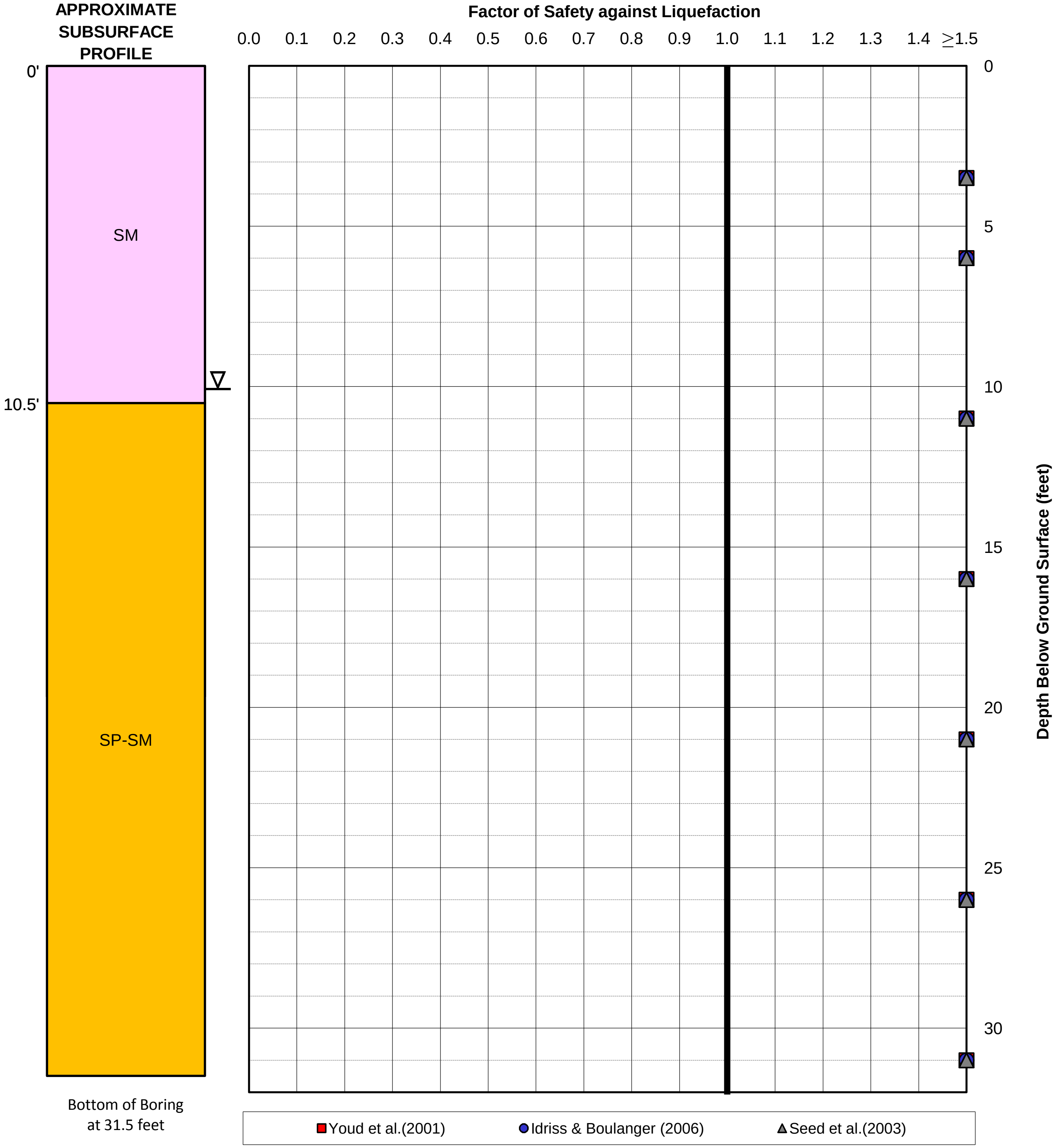
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

LATERAL DISPLACEMENT
BORING B-08
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-6



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

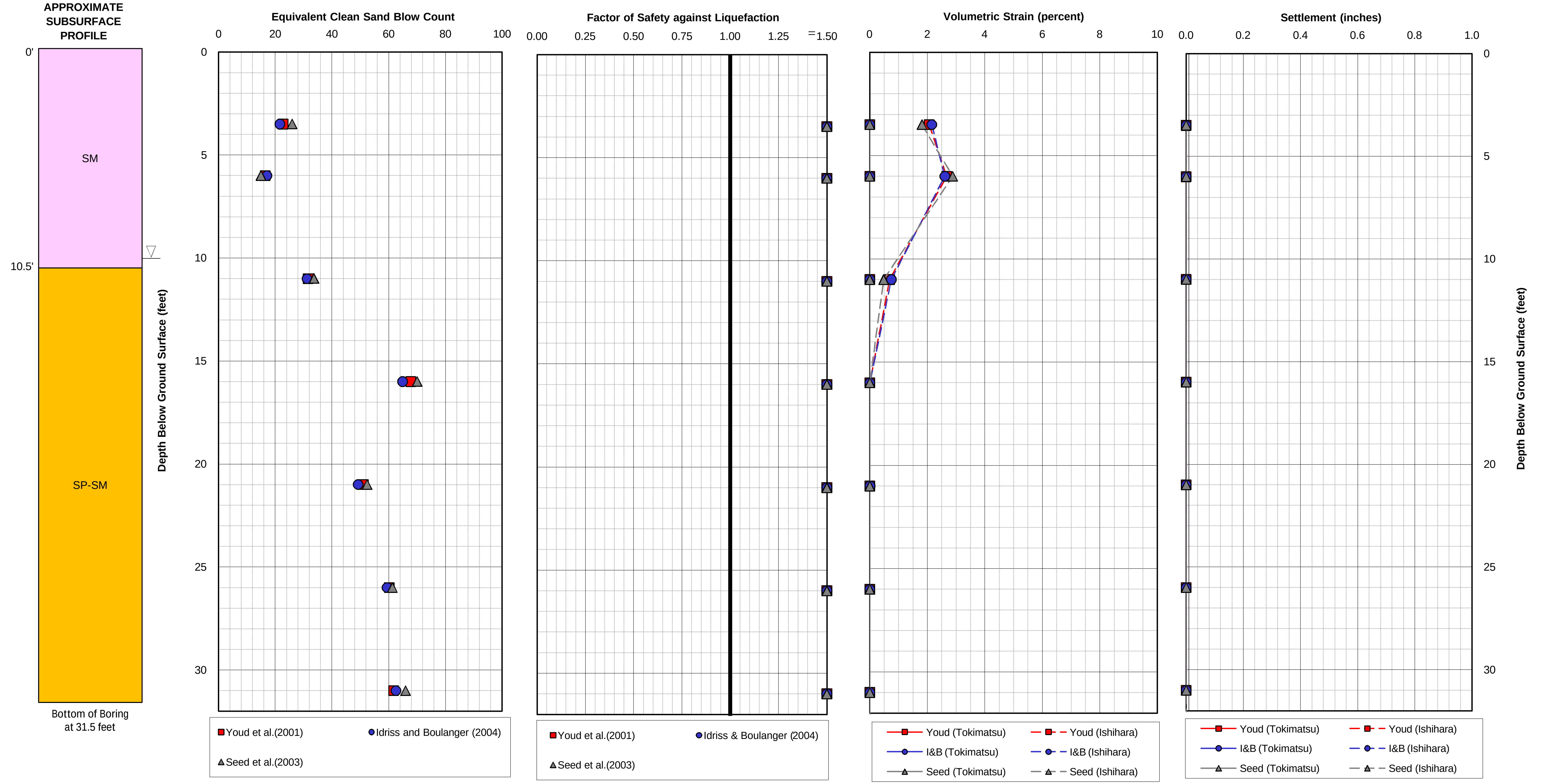
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

RESULTS OF LIQUEFACTION ANALYSES
BORING B-09
M = 9.20, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-7



NOTES

1. See main text for references.

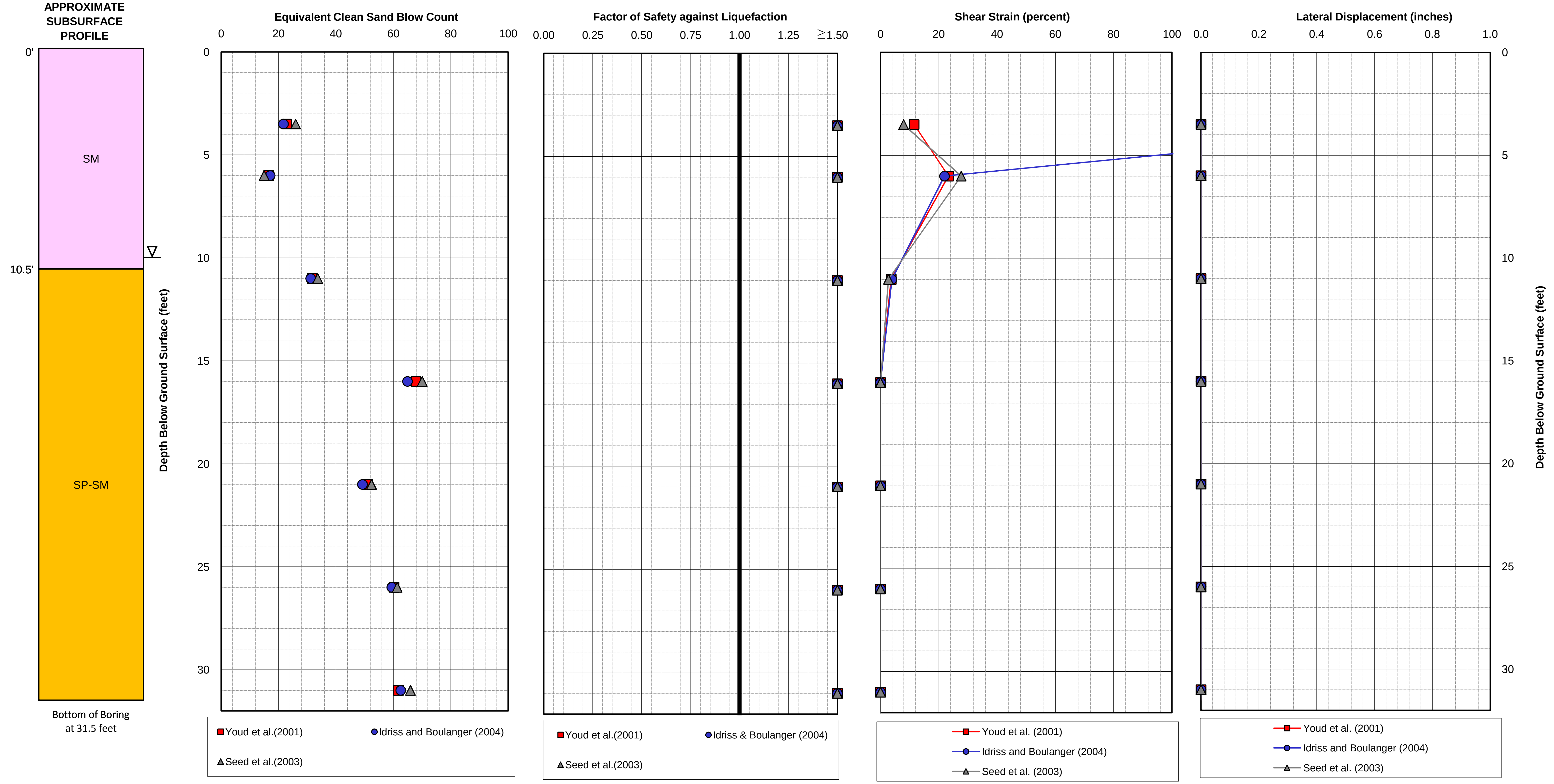
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

SEISMIC SETTLEMENT
BORING B-09
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-8



NOTES

1. See main text for references.

Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

LATERAL DISPLACEMENT
BORING B-09
M = 9.2, PGA = 0.28

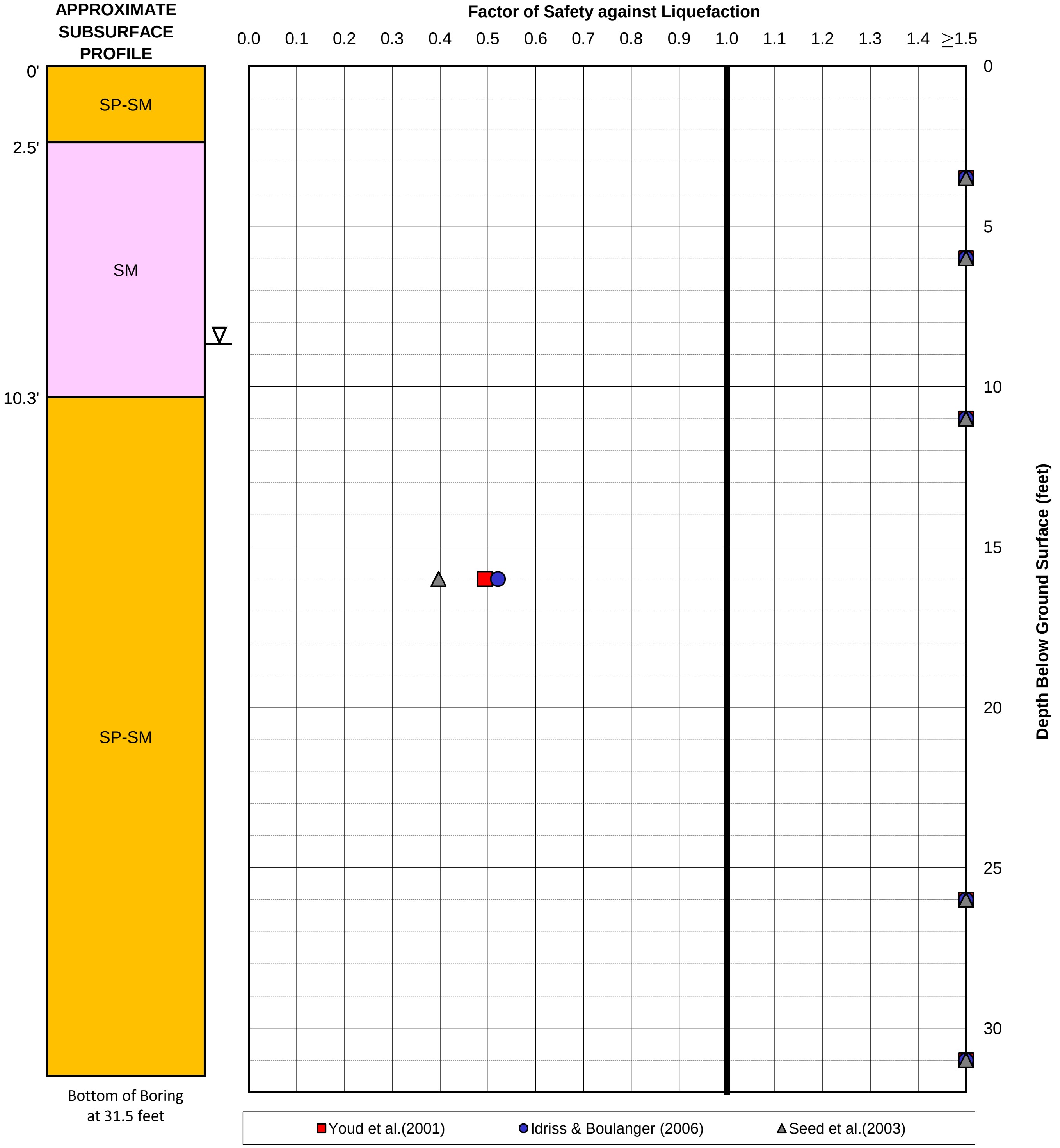
January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-9

B-10 SPT_Liquefaction_v2.1.5 1/28/2010

RCH



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

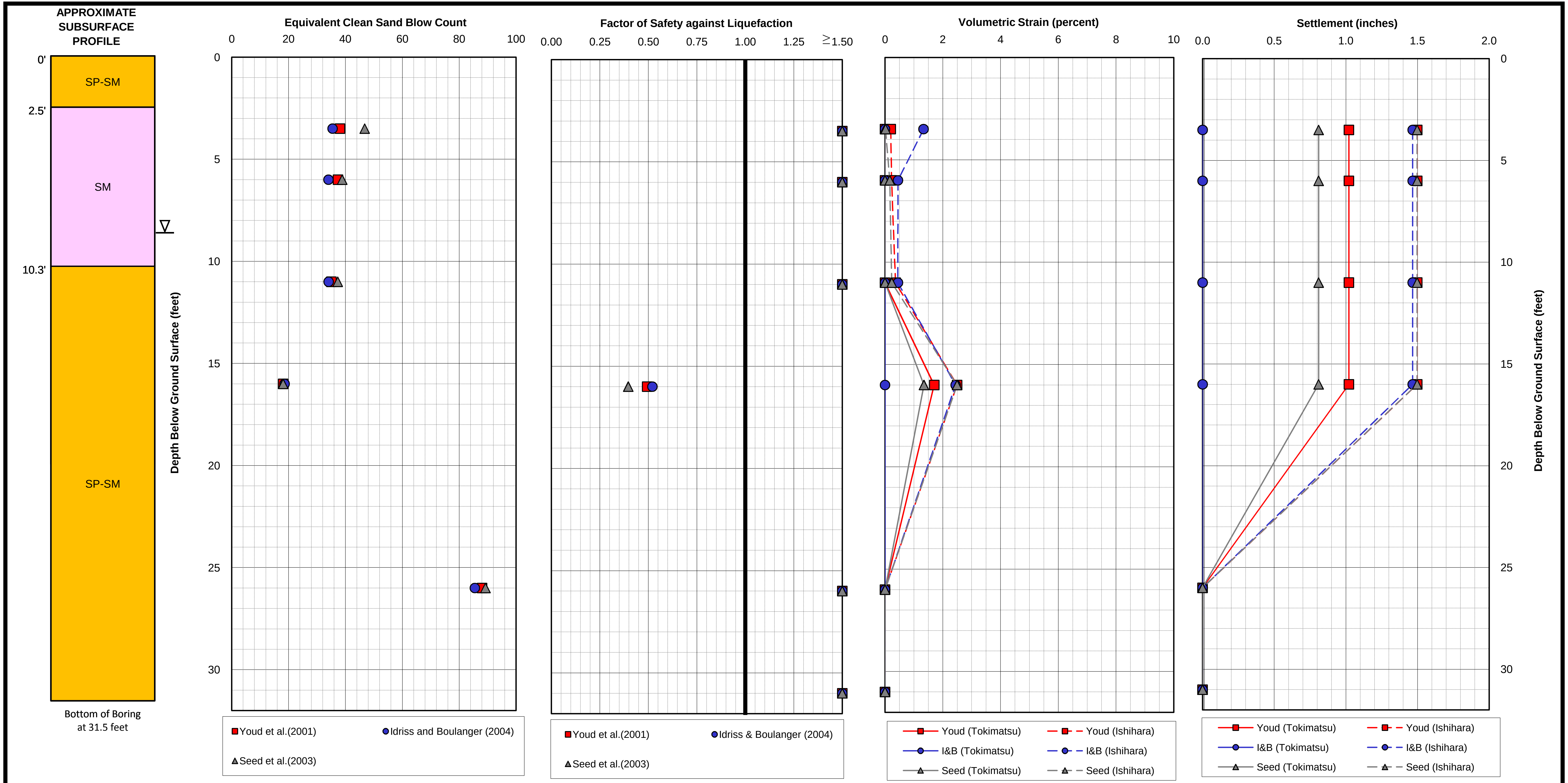
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

RESULTS OF LIQUEFACTION ANALYSES
BORING B-10
M = 9.20, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-10



NOTES

1. See main text for references.

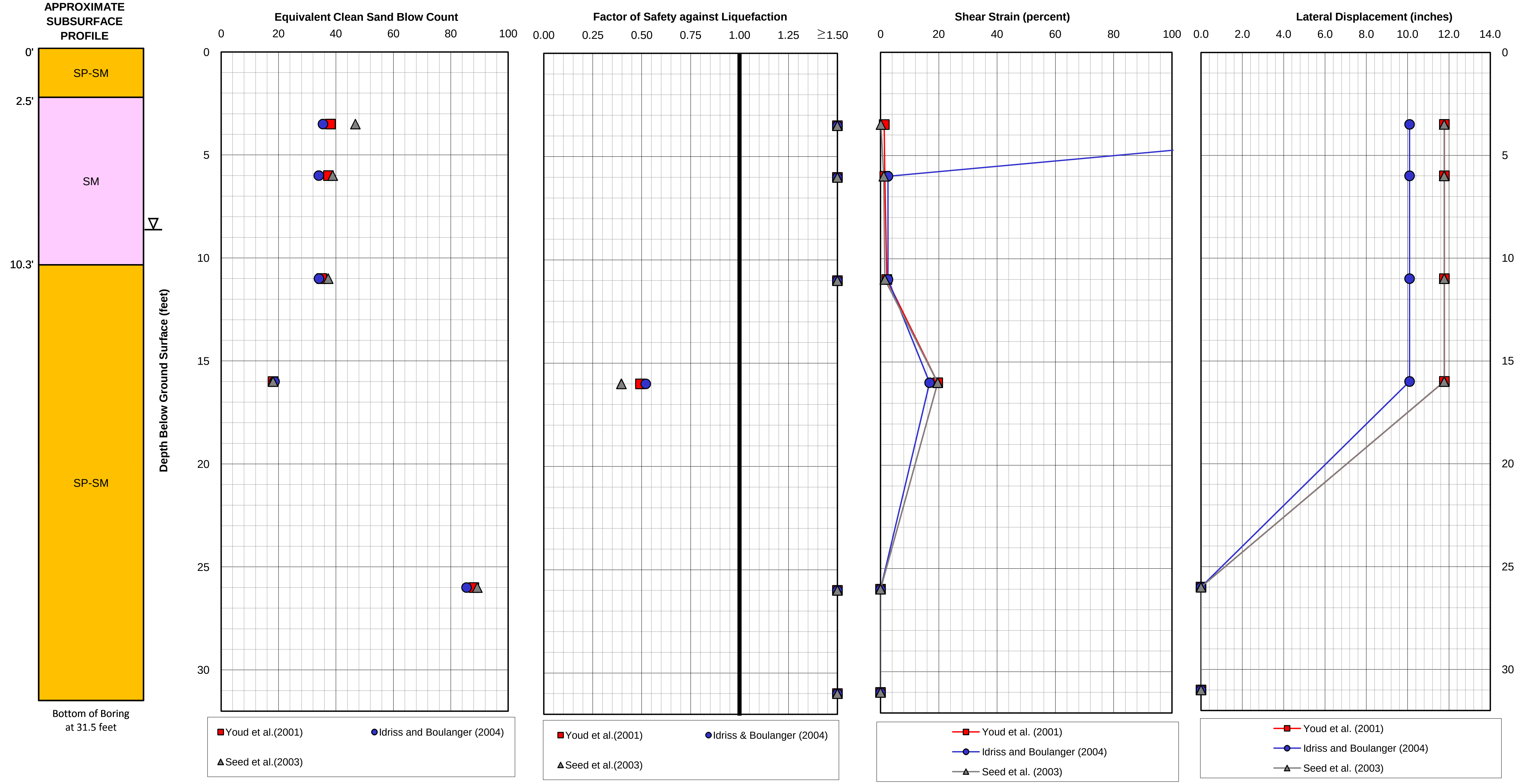
Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

SEISMIC SETTLEMENT
BORING B-10
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-11



NOTES

1. See main text for references.

Chugach Electric Association
International Substation Switchyard
Anchorage, Alaska

LATERAL DISPLACEMENT
BORING B-10
M = 9.2, PGA = 0.28

January 2010 32-1-02086

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-12

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to 32-1-02086

Date: January 2010
To: Stanley Consultants, Inc.
Re: CEA International Substation Switchyard,
Anchorage, Alaska

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland