APPENDIX N

Feasibility Level Geotechnical Evaluation

September 15, 2010

JENNA Development
3225 E. Pacific Coast Highway, Suite C
Signal Hill, California 90755

Attention: Mr. Jimmy Eleopoulos

Subject: Feasibility-Level Geotechnical Evaluation
Proposed Inns at Bridgecreek
SEC Jefferson Street and Highway 78
Oceanside, California
GPI Project No. 2331.I

Dear Mr. Eleopoulos:

As authorized by you, this report presents the findings of our feasibility-level geotechnical evaluation for the subject project. Our scope of services was outlined in our revised proposal dated June 6, 2010.

We understand that this feasibility-level evaluation is desired to evaluate if significant issues affect the development of the site. As outlined in our proposal, the primary purpose of our evaluation is to characterize the subsurface conditions with respect to feasible foundation types, liquefaction, long-term settlement, and geotechnical issues that would have a significant impact on the cost of developing the site.

Project Description

The proposed development will be located on a vacant 12.5± acre site adjacent to Buena Vista Creek in Oceanside, California. Current plans indicate the proposed development will include one 3-story medical office building, two 5-story hotels, a 4-story hotel, a 4-level parking structure (three supported decks), a clear span bridge crossing Buena Vista Creek, and a retention basin. The buildings will be at grade with no basements planned. Landscaping and pavement areas are also planned for the development.

Current conceptual grading plans indicate the majority of the site will have cuts of 1 to 3 feet or fills of less than 2 feet. Deeper fills of 3 to 9 feet are planned at the southwest portion of the site within the entrance drive area and the approach to the bridge crossing.

The maximum column loads for the hotels are anticipated to range from 140 to 175 kips. Maximum column loads for the medical office building and parking structure are anticipated to be 200 and 450 kips, respectively.

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The proposed site location and configuration are shown on the Site Location Map, Figure 1 and Site Plan, Figure 2, respectively.

Scope of Services

Our scope of work for this evaluation consisted of review of exiting documents, field exploration, laboratory testing, engineering analysis, and the preparation of this report.

We have been provided with previous geotechnical investigation reports (References 1 and 2) prepared for the site. We reviewed the geotechnical data contained in these reports.

Our field exploration program consisted of eight Cone Penetration Tests (CPT's) and one exploratory boring. The locations of the subsurface explorations are shown on the Site Plan, Figure 2. The CPT's were advanced to depths of 47 to 96 feet below existing grades. In addition, a seismic cone penetration test at one CPT location provided shear wave velocity measurements of the soil profile. Detailed logs of the CPT's and a summary of the equipment used are presented in Appendix A. The exploratory boring was drilled using truck-mounted, hollow-stem auger equipment to a depth of 80 feet below existing site grades. Details of the drilling and Logs of Borings are presented in Appendix B.

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the settlement characteristics and liquefaction potential of the soils. The geotechnical laboratory testing program included determinations of in-situ density, moisture contents, grain size analysis, Atterberg limits, expansion potential, consolidation, and corrosivity. Laboratory testing procedures and results are summarized in Appendix B.

Feasibility-level engineering evaluations were performed to assess the geotechnical site constraints, including static settlement, preliminary foundation evaluation, liquefaction potential, dynamic settlement, preliminary foundation design, and earthwork. The results of our evaluations are presented in the remainder of the report.

Subsurface Conditions

Our field investigation disclosed a subsurface profile consisting of man-made fill soils overlying natural soils. The fill soils were encountered within the upper 10 to 15 feet of the soil profile. The fill soils consisted of interbedded layers of clay, silty clay, sandy clay, clayey silt, sandy silt, and silty sand. The silts and clays range from soft to stiff. The silty sands ranged from loose to medium dense. Based upon our laboratory testing, the fill soils can be anticipated to moist. Documentation regarding the placement of the fill was not provided.

Information provided in Reference 1 indicates the original grades at the site were at an elevation of approximately 6 feet above mean sea level. The site was originally a relatively low lying mud area east of Buena Vista Lagoon. Reference 1 indicates that the vegetation at the site was cleared in 1972 and fill was placed to Elev. +16 to +19 feet. Reference 1 further indicates that an additional 3 to 4 feet of clayey lagoon deposits were placed at the site without compaction in 1983.

The underlying natural soils consist of interbedded layers of silt, clayey silt, sandy silt, silty clay, sandy clay, clay, silty sand, and sand. Directly below the undocumented fills, in general, we encountered a layer of normally consolidated fat clay and elastic silt ranging in thickness from 15 to 35 feet. This clay/silt layer is soft exhibiting very low strength and very high compressibility. The clay/silt layer appears to be thicker in the southern portion of the site near the creek. This clay/silt layer does have thin lenses of silty sand and sandy silt. Based on our CPT's, we estimate the thickest layers of clay/silt without intermittent thin lenses of sandy silt and silty sand may be on the order of 10 to 15 feet.

Underneath the soft fat clay/elastic silt layer, in general, we encountered layers of medium dense to dense sands and silty sands interbedded with layers of firm to stiff layers of clayey silt, silty clay and clay. In general, the dense sand layers were relatively thin with thicknesses on the order of approximately 2 to 5 feet. We estimate that these interbedded layers of clay and silt are overconsolidated with moderate compressibility characteristics. Our explorations indicated these natural soils extended to a depth ranging from approximately 55 to 80 feet below existing site grades. At this depth, we encountered very dense sands or hard clays which likely consisted of the soft bedrock material underlying the site.

The details of the subsurface conditions encountered are shown on the Logs of CPT's in Appendix A and the Log of Boring in Appendix B.

We measured the groundwater at a depth of approximately 15 feet from the existing ground surface. This corresponds to near the level of the adjacent creek and tidal fluctuations of the adjacent lagoon to the west. Based upon past reports (References 1 and 2), groundwater can be anticipated at depths on the order of 12 to 20 feet below the current grades at the site. The depth of groundwater is anticipated to fluctuate across the site and seasonally. Sandy soils are expected to cave severely below the groundwater.

Findings and Conclusions

Based on the results of our investigation, it is our opinion that, from a geotechnical viewpoint, it is feasible to develop the site as proposed provided the following geotechnical issues are incorporated into the design and construction of the project.

1) Settlement Considerations

- Past increases to the original site grade have induced long-term settlement of the underlying normally consolidated clays and silts. Based upon our consolidation testing, we estimate that the majority of the settlement from raising the grades in 1972 and 1983 has been completed. We estimate that the majority of the settlement due to areal raising of grades can be anticipated to occur over a period 10 to 30 years. We estimate that less than 1-inch of residual settlement due to the past raising of grades remains at the site.

- Future increases to the current site grade will induce long-term settlement of the underlying normally consolidated clays and silts. Based upon our consolidation testing, we estimate that for every 1-foot of fill placed above current grades, settlement on the order of ¾ to 1½ inches can be anticipated to occur over a period 10 to 30 years. The settlement will be greater along the south portion of the site adjacent to the creek as the compressible layers are thicker. Raising may cause settlements ranging from 4 to 12 inches.

- The use of surcharge fills and wick-drains could be used to accelerate future settlements at the site caused by new fill placement. The thickness of the surcharge would likely be on the order of 5 to 10 feet of nominally compacted fill above proposed finish grades. Wick drains would need to be installed to approximately 30 to 60 feet below the current existing ground surface. Wick drain installation could reduce the time to complete the majority of the settlement of embankment fill to approximately 180 to 360 days. Additional explorations and laboratory testing may result in a reduction in the estimated time needed for the settlement to finish. In building areas, the wick drains could be installed at the bottom of the removals of undocumented fills to reduce their length. Wick drains spacing would need to be fairly close, on the order 3 to 5 feet. Settlement monuments and periodic survey readings will be required to monitor the progress of the consolidation.

- Utility connections into the buildings and throughout the site will likely require flexible connections to help mitigate potential differential settlements due to building loads and areal raising of grades.

2) Earthwork/Grading Considerations

- The undocumented fill soils in the upper 10 to 15 feet are anticipated to be moist. Groundwater is anticipated to be as shallow as 15 feet and should be anticipated to fluctuate seasonally and across the site. The undocumented fill soils near the groundwater are anticipated to be very moist to wet.

- The soils exposed in the subgrade within the upper 3 feet of the undocumented fills will likely consist of clays and may be soft, wet and compressible. In general, the soils are expected be too soft or wet to support rubber-tire earthwork equipment without significant pumping and yielding of the subgrade. An excavator or wide-track-mounted equipment may be required to make required excavations. The subgrade soils will likely require stabilization for support of heavy equipment and before fills can be placed and compacted. For cost estimating purposes, the stabilization could include placement of a geogrid, such as Tensar BX1100 or equivalent, and 12 to 18 inches of aggregate base.

- Remedial grading to remove undocumented fills will be required under the buildings unless the structures are supported by pile foundations including a structural floor slab. The undocumented fills will be required to be removed to native ground at a distance of outside the building lines equal to the depth of the undocumented fills. This removal will be required if the buildings are to be supported by spread footings, mat foundations, or pile foundations including slab-on-grade floors. Removal of the undocumented fills below the buildings is the standard of practice in Southern California and is required by the Building Code. Under pavement areas, removals can be limited to 1 to 3 feet, provided the potential for future pavement settlement is acceptable.

- The soils exposed in the bottoms of building pad overexcavation, if performed, will be wet and compressible. Similar equipment restrictions and stabilization, as discussed above, will be required at the bottom of any overexcavation of undocumented fills below the building.

3) Seismic Considerations

- The site is located in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

- We assume the seismic design of the proposed development will be in accordance with 2007 CBC criteria. Based upon the shear wave velocities measured a CPT, a Site Class E may be used for the 2007 CBC. The remaining seismic code values can be determined by the Project Structural Engineer using the value above and the pertinent internet websites and tables from the building code (Reference 3 for S_{S1}, S_1, S_M, and S_D values). The actual method of seismic design should be determined by the Project Structural Engineer.

- The site is about 9 kilometers from the Newport-Inglewood (Offshore) Fault. There are no known faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

- The site is not located in a Seismic Hazard Zone for liquefaction (Seismic Hazards Mapping Act, State of California), as the site vicinity has not yet been mapped. Due to the deep alluvium and shallow groundwater at the site, we evaluated the potential for liquefaction using a groundwater depth of 15 feet below existing grades, our CPT data, and a peak ground acceleration of 0.30g. The peak ground acceleration was determined based information provided on the USGS website (Reference 3). This acceleration has been computed using 40 percent of S_{DS} for the project. .

- Our preliminary calculations indicate that total liquefaction settlements on the order of ¼ to ¾ inches may occur during a seismic event. Differential settlements may be on the order of ¼ to ½ inches. Our limited laboratory testing indicates the on-site cohesive soils do not conform to the characteristics that are necessary for liquefaction ("Chinese Criteria").

- We did not identify continuous layers of liquefiable sands in the upper 30 feet of the soil profile extending toward the open face slope at the creek. Therefore for preliminary purposes, lateral spreading is considered unlikely for this site.

4) Building Foundation Alternatives

- Depending on the structural loads and location of the buildings at the site, the buildings will be required to be supported on pile or mat foundations. With remedial grading to remove undocumented fills, the static settlements of the parking structure and the buildings supported on shallow spread footings will be about 3 to 4 inches and 2 to 3 inches, respectively. These settlements are due to surcharging the soft clays/silts underlying the undocumented fills. These estimated settlements of the structures are in excess of the limits, typical for spread footings.

- Due to the sensitive nature and the depth of the fat clays/elastic silts, the use of ground improvement methods such as deep soil-cement mixing, rammed aggregate piers (Geopiers) or stone columns to improve the on-site soils are not considered feasible to support the buildings.

- We recommend that the parking structure be supported on pile foundations including a structural floor slab. If undocumented fills are removed under the parking structure and the grades are not raised by more than 1-foot, a slab-on-grade floor may be feasible with pile supported foundations.

- A mat foundation for buildings typically needs to be limited to on the order of 3 to 4 inches of total settlement at the center of the mat. Total settlement includes static settlement due to building loads, static settlement due to raising grade, and potential seismic settlement. The ability of the mat to tolerate these magnitudes of settlements should be confirmed by the project structural engineer.

- Based upon anticipated loads of the hotels and the medical office building, we estimate that static settlements will be on the order of 2 to 5 inches for mat foundations supporting the buildings at the site, assuming remedial grading is performed to remove undocumented fills. This estimate does not included additional settlements caused by raising grades above existing grades within the building areas by more than 1-foot. Without mitigation measures, raising the site grades by more than 1-foot could induce long-term settlement having an adverse impact on the proposed structures as described above.

- Current preliminary plans indicate the hotel planned for the northwest corner of the site (Building H3) has fill of up to 4 feet planned at the southwest portion of the building footprint. Surcharging and wick drains over a portion of the building pad likely will be required to reduce the total settlement to an acceptable limit for a mat foundation.

- The hotel planned for the southern portion of the site (Building H2) is located overlying the areas of the site with the thickest layer of soft clays/silts. Static settlements due to anticipated mat pressures may be on the order of 5 inches without surcharging. Surcharging and wick drains over the building pad likely will be required to reduce the total settlement to an acceptable limit for a mat foundation.

- The hotel planned for the western portion of the site (Building H1) is located overlying the areas of the site with varying thicknesses of soft clays/silts. Static settlements due to expected mat pressures may vary from 2 inches on the northern portion of the building to 5 inches on the southern portion of the building without surcharging. Surcharging and wick drains over a portion of the building pad likely will be required to reduce the total and differential settlement to an acceptable limit for a mat foundation.

- For the medical office building planned for the western portion of the site, static settlements due to the loading of the mat will be on the order of 2 to 3 inches. A mat foundation will likely be acceptable for this building.

- If mat foundations with surcharging and wick drains are not economical, pile foundations could also be used to support the foundations of the hotel buildings, where necessary.

5) Bridge Foundation Alternatives

- Due to the highly compressible soils adjacent to the creek, the bridge abutments will likely be required to be supported on pile foundations.

- At the northern approach to the bridge over the creek, the grade is planned to be raised on the order of 9 feet above existing grades. To help limit the long-term downdrag forces on piles supporting the bridge, surcharging and wick drains would likely be required to accelerate settlement of the underlying normally consolidated clays/silts. These remedial measures will also lessen long-term maintenance issues associated with the settlement of the bridge approach and pile supported bridge abutment.

6) Pile Foundations

- Deep foundations will be required to support the foundations of the parking structure and the bridge abutments. Piles could be utilized provided the piles resist downdrag forces from liquefaction settlement and settlement from an adjustment of grades.

- Steel screw piles socketed into the soils below the highly compressible clays and liquefiable sand may be best alternative due to high capacity and low downdrag forces on the narrow shaft. We estimate the length of the screw piles would need to be on the order of 40 to 50 feet below existing site grades to penetrate into denser soils below the highly compressible clays/silts. For preliminary purposes, the allowable capacities of 8-inch diameter screw piles can be estimated to be on the order of 150 to 175 kips. This includes a reduction in the ultimate capacity of the pile due to the anticipated downdrag.

- For preliminary planning purposes, the lateral capacity of an 8-inch diameter steel screw piles would be on the order of 4 to 6 kips. This assumes a free head condition with ⅜-inch of deflection.

- Driven concrete piles or auger-cast pressure grouted piles could also be used provided they extended relatively deep below the highly compressible clays and liquefiable sand to resist downdrag forces. The length of the concrete or auger-cast piles could be significant (on the order of 60 to 80 feet) to extend below the compressible and liquefiable layers and to support downdrag forces as well as building or bridge loads. For preliminary purposes, the allowable capacities of a 12-inch square concrete or a 14-inch diameter pressure grouted pile can be estimated to be on the order of 70 to 90 kips. This includes a reduction in the ultimate capacity of the pile due to the anticipated downdrag.

- For preliminary planning purposes, the lateral capacity of a 12-inch square concrete or 14-inch diameter pressure grouted pile would be on the order of 8 to 10 kips. This assumes a free head condition with ⅜-inch of deflection.

- We estimate total static and seismic settlement of less than ½-inch could be achieved with pile foundations.

7) Other Considerations

- Resistivity testing of representative samples of on-site soils indicates that they are severely corrosive to ferrous metals. Soluble sulfate testing indicates foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for a severe level of soluble sulfate exposure for soil. Soluble chloride testing indicates corrosion of the reinforcement steel will be a consideration. We do not practice corrosion protection engineering. Further testing of on-site soils should be performed and a corrosion engineer such as Schiff Associates should be consulted for design level studies.

- The clayey soils in the upper 3 feet of the existing grade are anticipated to have a high expansion potential. Building floor slabs will require additional reinforcing and/or the building pads will require a cap of select, non-expansive soil to reduce the potential for expansion-related slab distress. Mat foundations will need to be designed to resist a moderate expansion potential. If the potential for heave of hardscape areas is not acceptable, removal of the hardscape subgrade soils and replacement with non-expansive soils will also be required.

- Although not performed during this feasibility-level investigation, R-value testing of the on-site clayey soils near the surface is anticipated to result in low values (R-value = 5 or less). A typical automobile parking and truck drive sections will be about 3 inches of asphalt concrete over 7 and 10 inches of aggregate base, respectively.

- The stability of the creek slopes need to be considered with potential loads from adjacent structures or fills (temporary surcharge or permanent fill). Stability analysis for static and seismic conditions should be performed during the design phase of the project.

Limitations

The recommendations presented herein are provided for evaluation of development constraints for the site by JENNA Development and for obtaining preliminary cost estimates for soils-related construction. This report is not intended to be a design document, as it does not contain sufficient data for a design-level investigation. When additional project details are available, a comprehensive geotechnical investigation report should be prepared to provide design level recommendations.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, expressed or implied, is included or intended in our report.

We trust this information satisfies your current needs. Please do not hesitate to call if you have any questions on the contents of this report.

Sincerely,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER
DONALD A. CORDS
No. GE 2529
Exp. 6-30-11
GEOTECHNICAL
STATE OF CALIFORNIA

Donald A. Cords, G.E.
Associate

REGISTERED PROFESSIONAL ENGINEER
JAMES E. HARRIS, IV
No. GE 2100
Exp. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

James E. Harris, G.E.
Principal

DAC/JEH:sph

SEP 15 2010

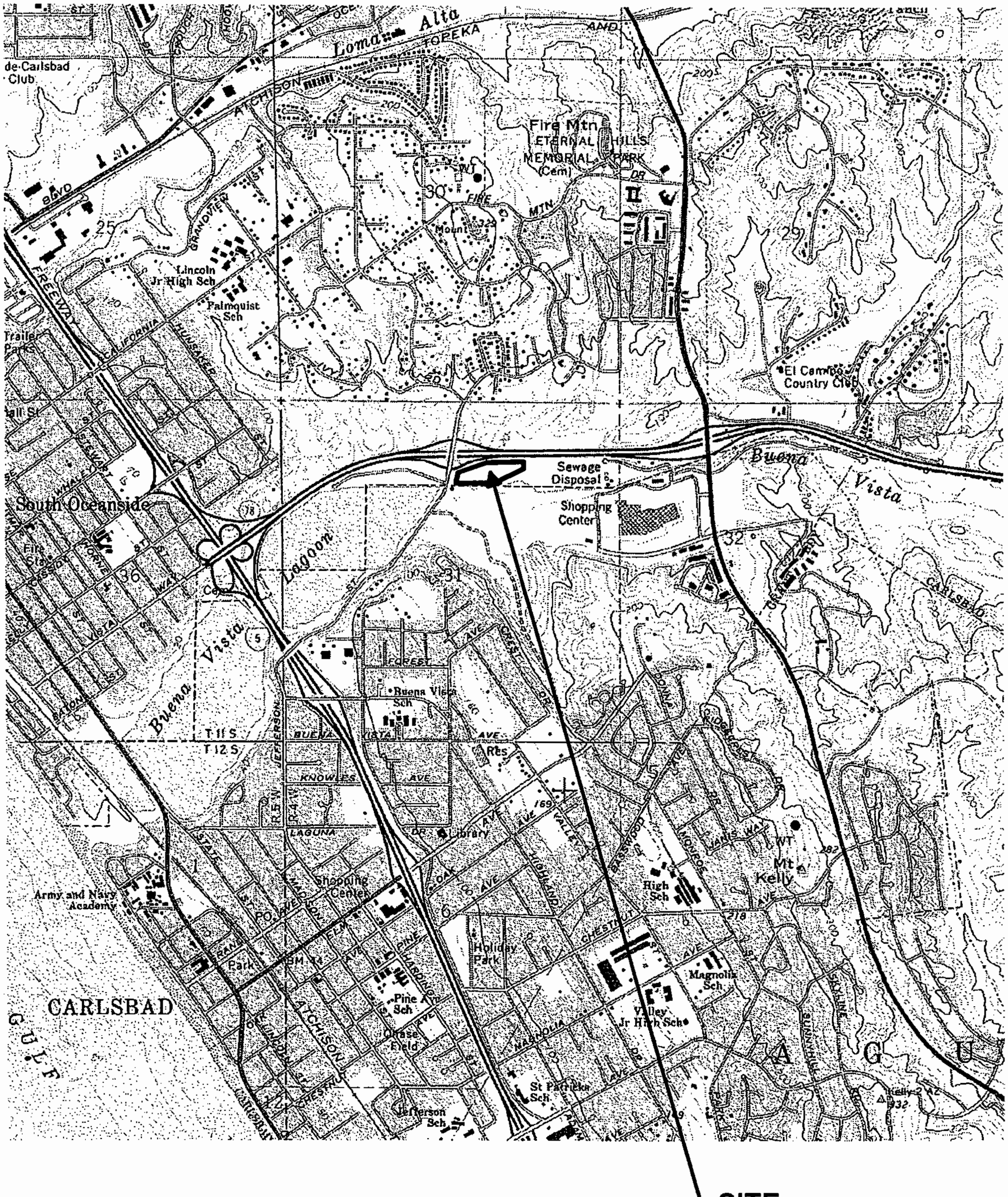
Enclosures: References
Figure 1 - Site Location Map
Figure 2 - Site Plan
Appendix A - Cone Penetration Tests
Appendix B - Exploratory Boring
Appendix C - Laboratory Tests

REFERENCES

1. Robert Prater Associates, "Geotechnical Consultations, North County Plaza II, Oceanside, California," Project No. 239-6S, 97-62, dated February 19, 1997.

2. Krazan & Associates, Inc., "Geotechnical Engineering Investigation, Proposed Hampton Inn & Suites, Homewood and Springhill Suites, State Highway 78 and Jefferson Street, Oceanside, California," Project No. 112-04054, dated October 20, 2004.

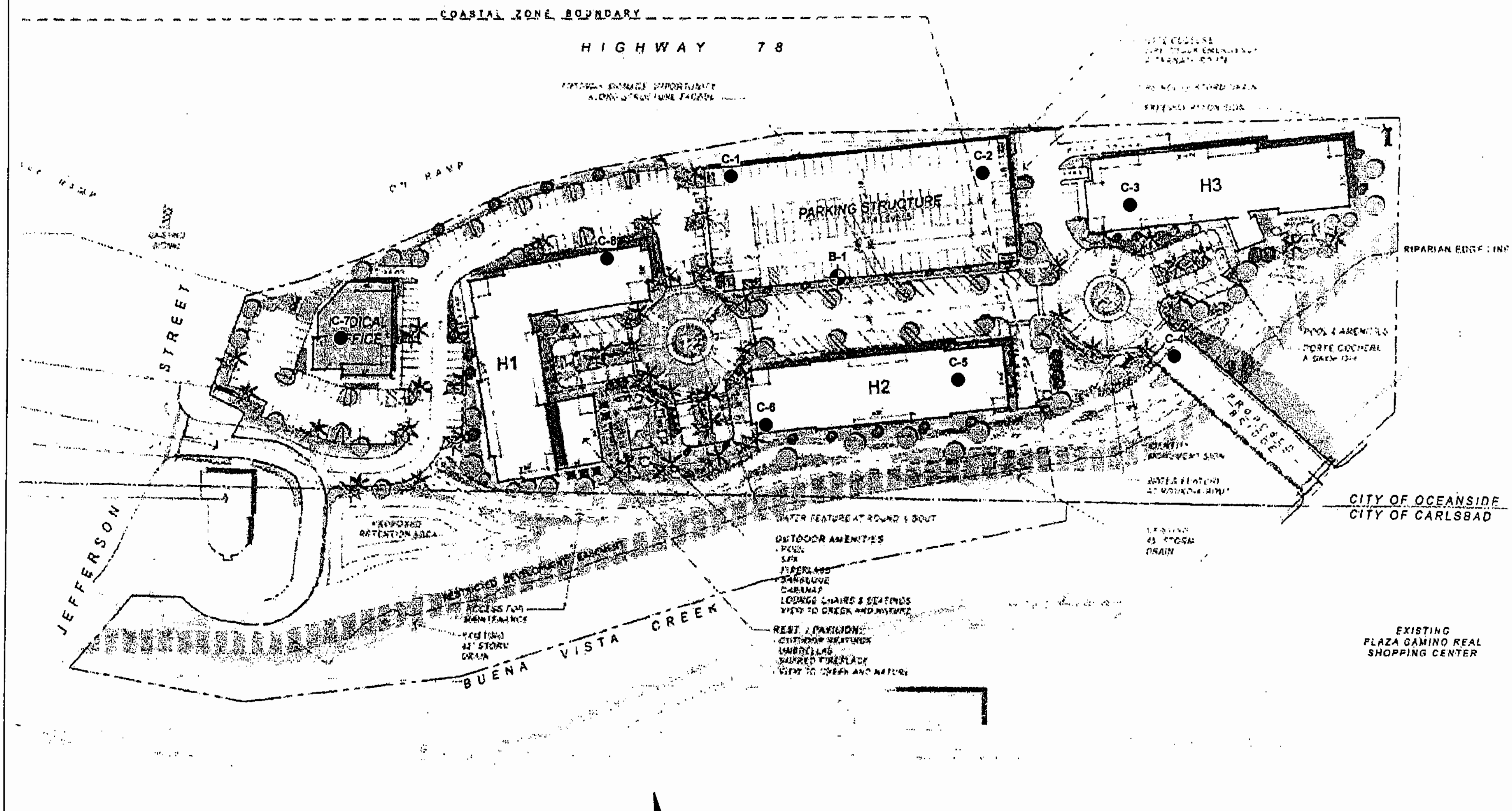
3. http://earthquake.usgs.gov/research/hazmaps/design/ for determination of Ss and S1 values.



0 2500 5000 FEET

BASE MAP REPRODUCED FROM SAN LUIS REY QUADRANGLE FROM USGS MAPS

GPI GEOTECHNICAL PROFESSIONALS, INC.	**SITE LOCATION**
INNS AT BRIDGE CREEK	
GPI PROJECT NO. 2331.I — SCALE: 1" = 2500'	FIGURE 1



EXPLANATION

B-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-2 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

BASE PLAN REPRODUCED FROM CONCEPTUAL SITEPLANS PROVIDED BY D.M. BERGERSON INC.: (UNDATED)

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INNS AT BRIDGE CREEK

GPI PROJECT NO.: 2331.I

SCALE: 1"=100'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing eight Cone Penetration Tests (CPT's) at the site. These soundings were advanced to depths of approximately 47 to 96 feet below existing grades. The locations of the CPT's are shown on the Site Plan, Figure 2.

The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a reaction to the thrust of the hydraulic rams.

Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 to A-9 of this appendix. The field testing and computer processing for the current investigation was performed by Kehoe Testing and Engineering, Inc. under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

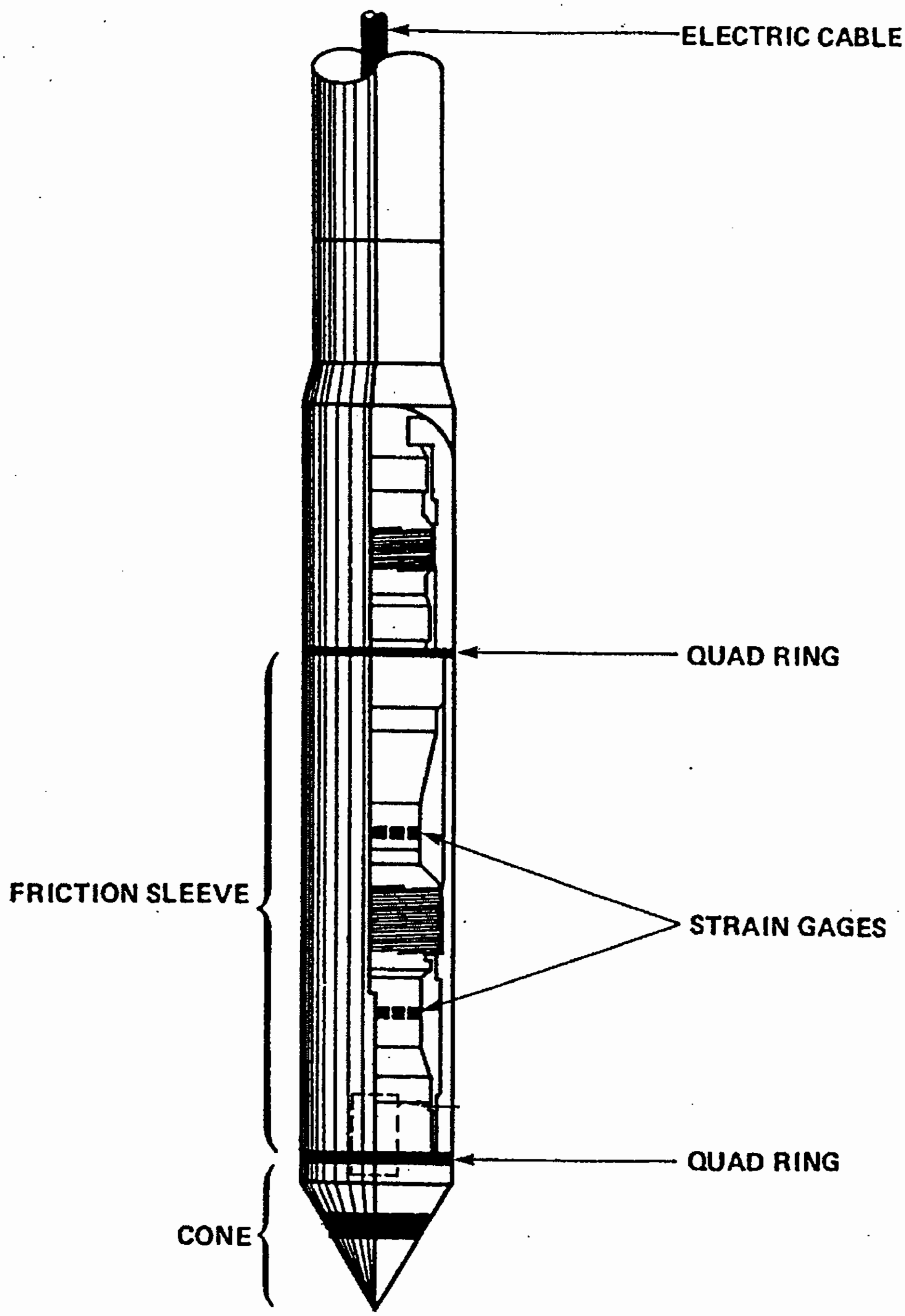
A seismic cone penetration test provided shear wave velocity measurements of the soil profile. A standard cone penetrometer is equipped with two sets of geophones located approximately 3 feet apart on the cone penetrometer. At 10 foot intervals, a shear wave source is activated at the ground surface using an air-actuated hammer. A seismograph measures the travel time of the shear wave detected at each set of geophones. The time difference provides the velocity of the shear wave in the layer between the two geophone sets.

Table A-1 provides the shear wave velocity from the surface and the interval of soil between the geophones.

The CPT locations were laid out in the field by measuring from existing site features. The ground surface elevations at the CPT locations were estimated from topographic survey data provided by JENNA Development and should be considered approximate.

TABLE A-1
CPT SHEAR WAVE MEASUREMENT

Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity From Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
10.43	11.57	19.46	594.38	
20.11	20.72	34.88	594.10	593.76
31.78	32.17	69.84	460.64	327.48
40.10	40.41	82.06	492.45	674.27
50.12	50.37	92.68	543.47	937.69
60.29	60.50	104.94	576.49	826.12
70.11	70.29	115.10	610.67	963.69
80.14	80.30	123.84	648.38	1145.05
86.12	86.27	130.18	662.66	941.51



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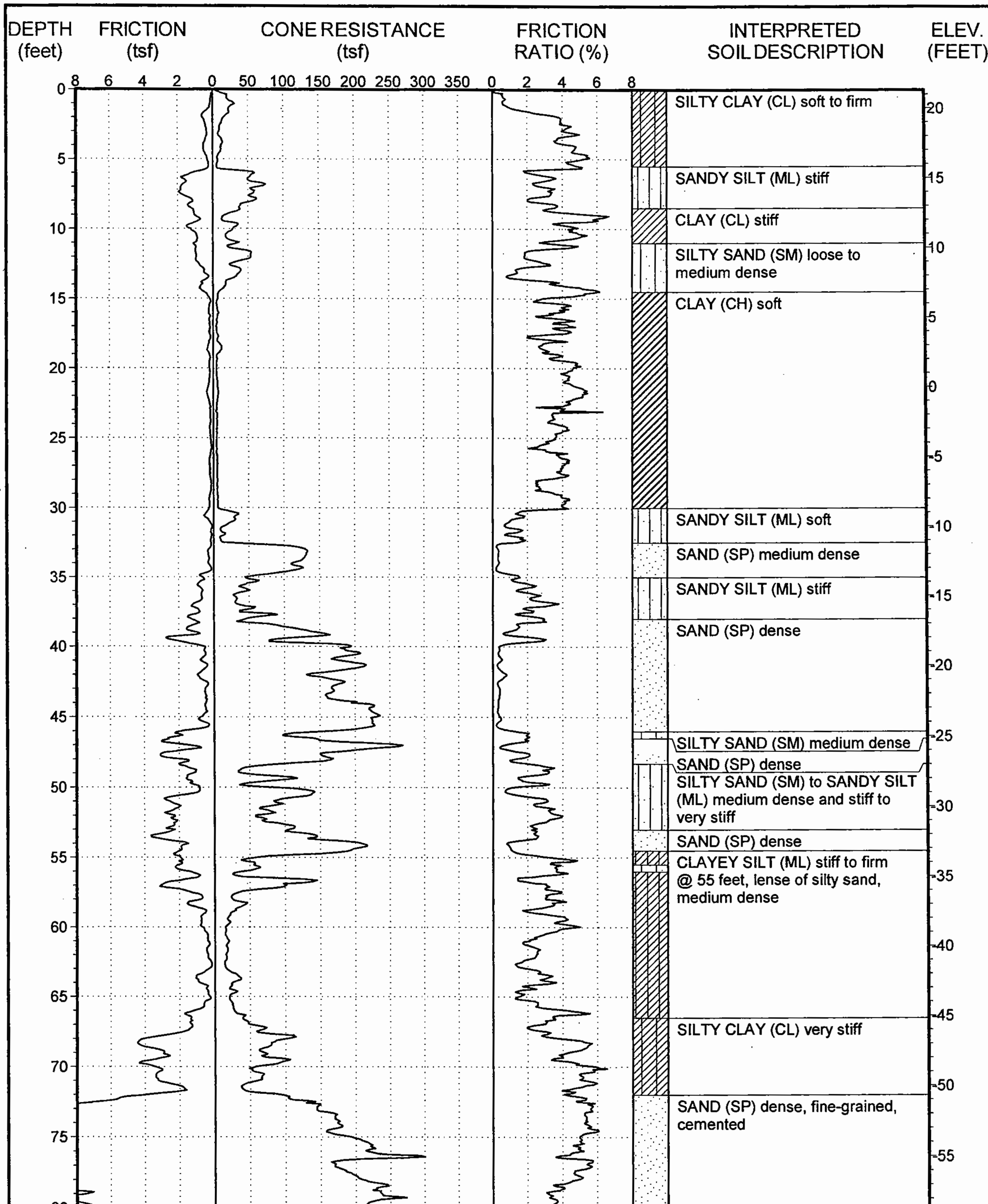
CONE PENETROMETER

FIGURE A-1



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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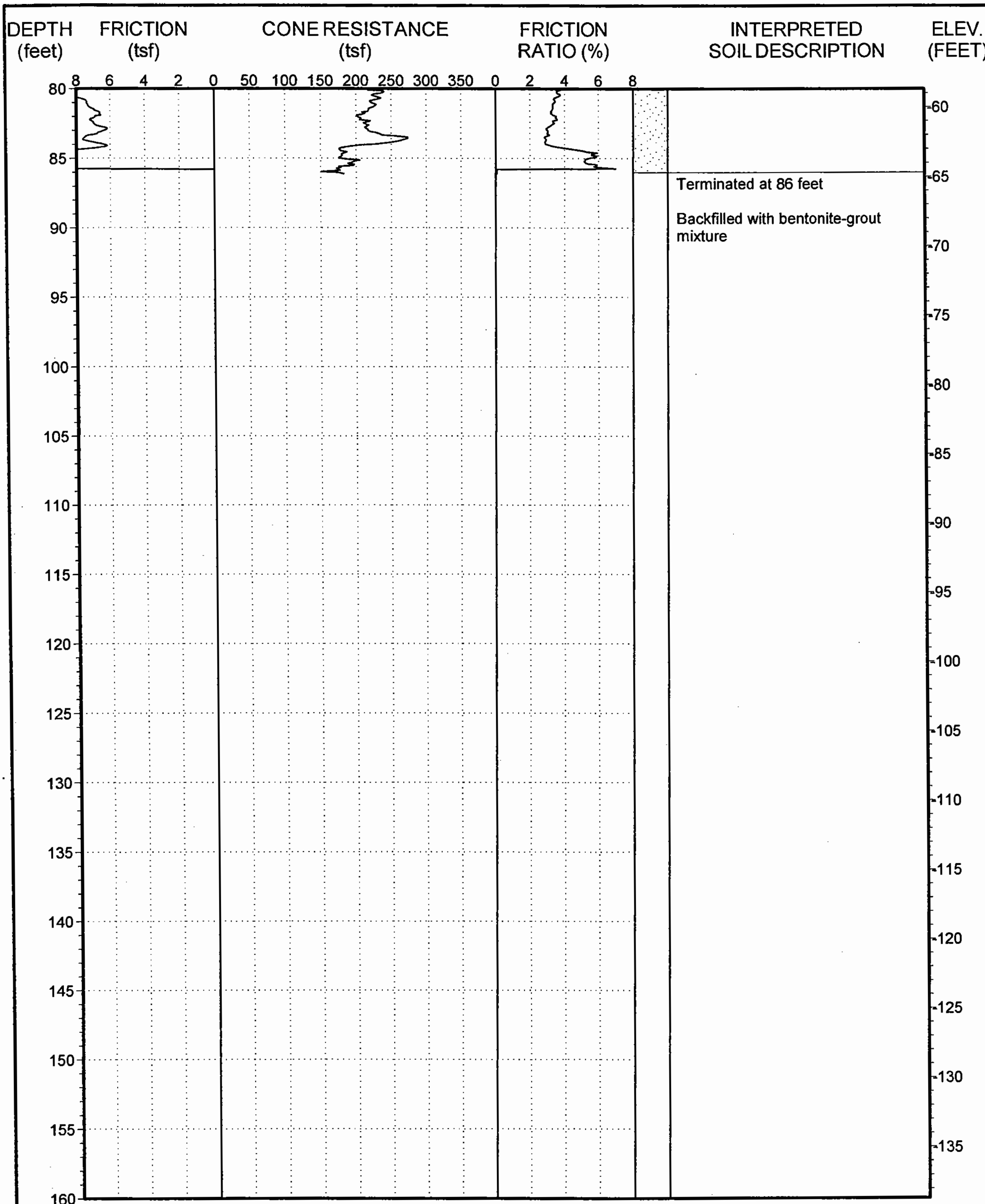
PROJECT NO.: 2331.I
INNS AT BRIDGE CREEK

LOG OF CPT NO. C-1

FIGURE A-2



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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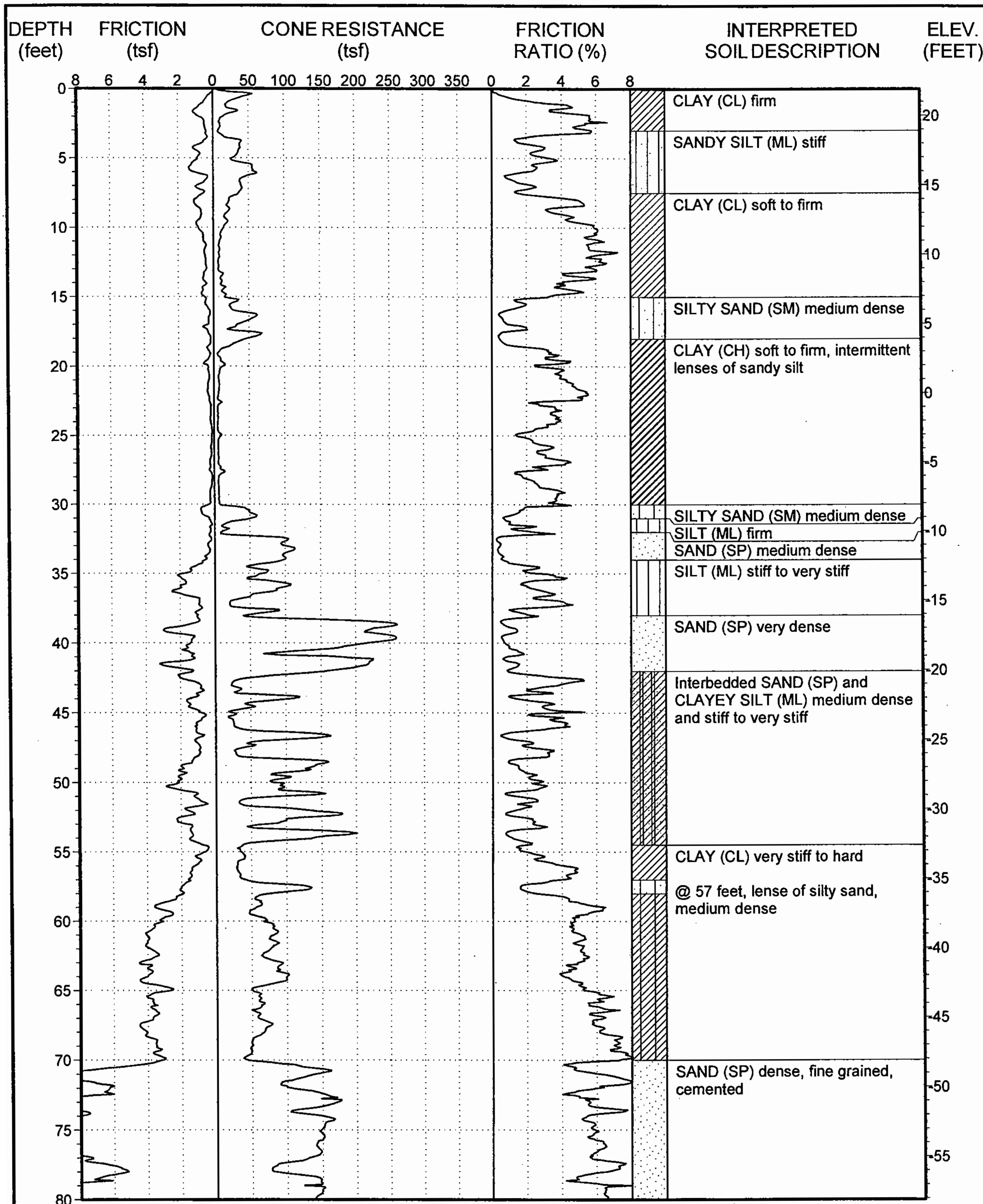
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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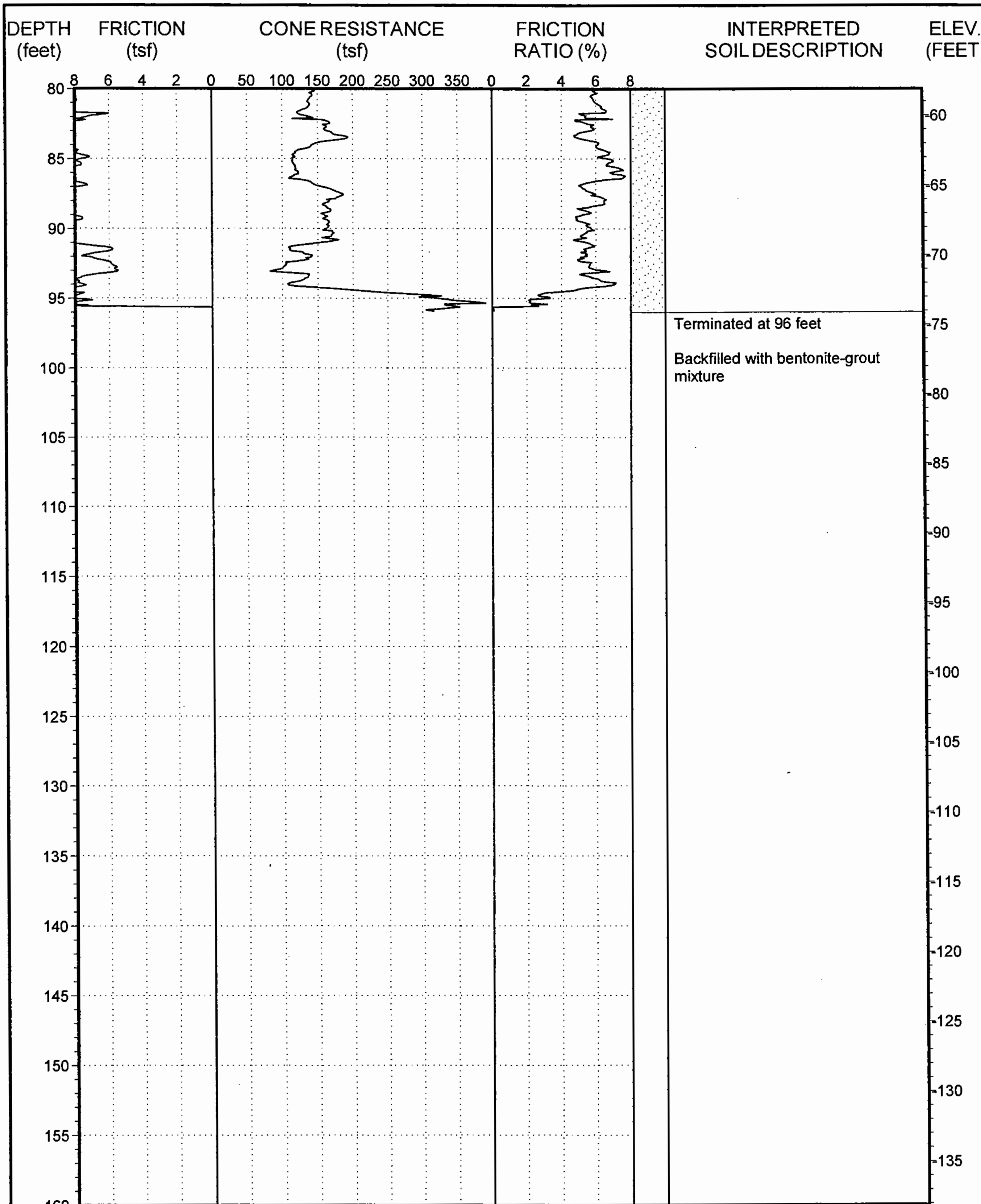
INNS AT BRIDGE CREEK

LOG OF CPT NO. C-2

FIGURE A-3



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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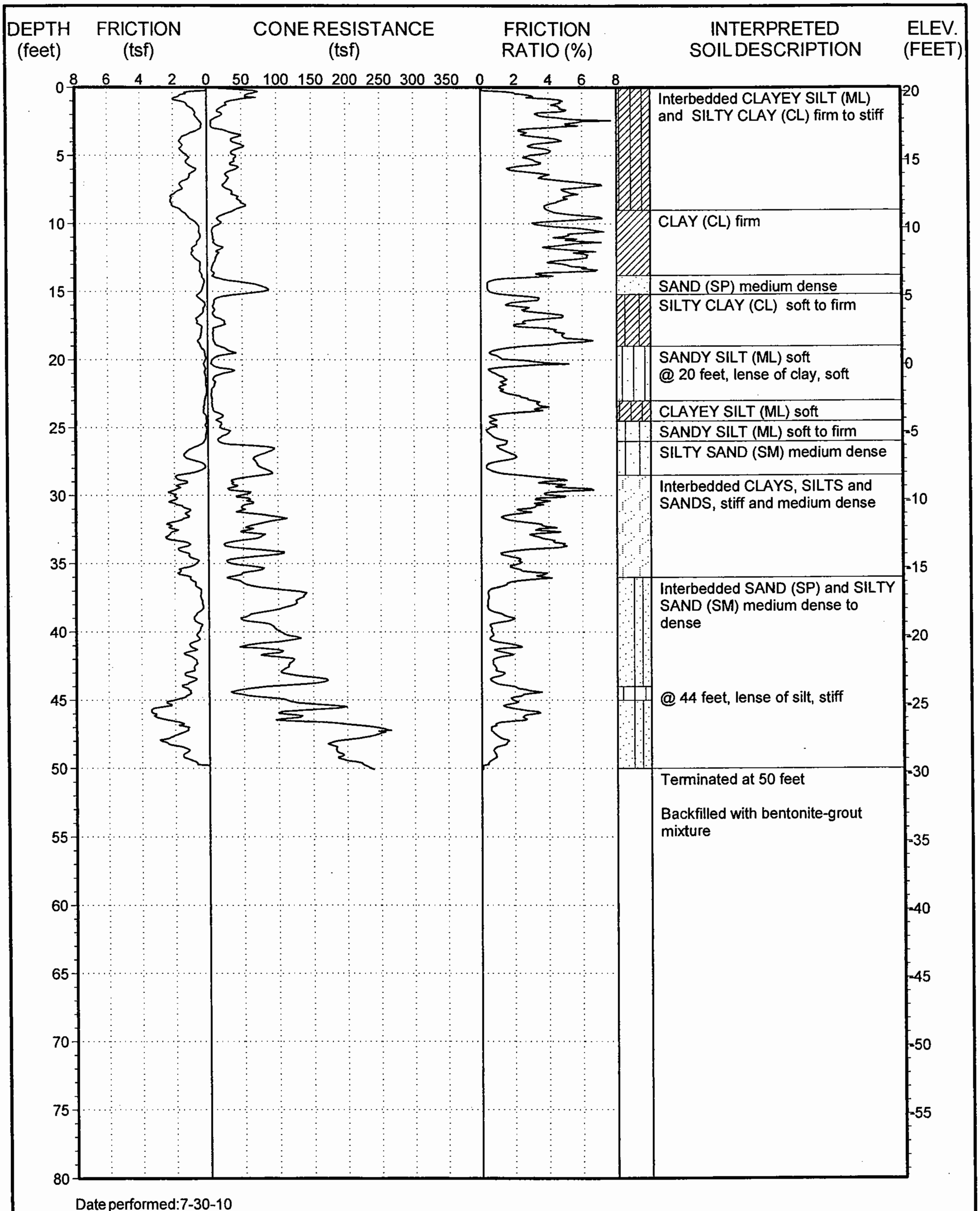
PROJECT NO.: 2331.I
INNS AT BRIDGE CREEK

LOG OF CPT NO. C-2

FIGURE A-3



This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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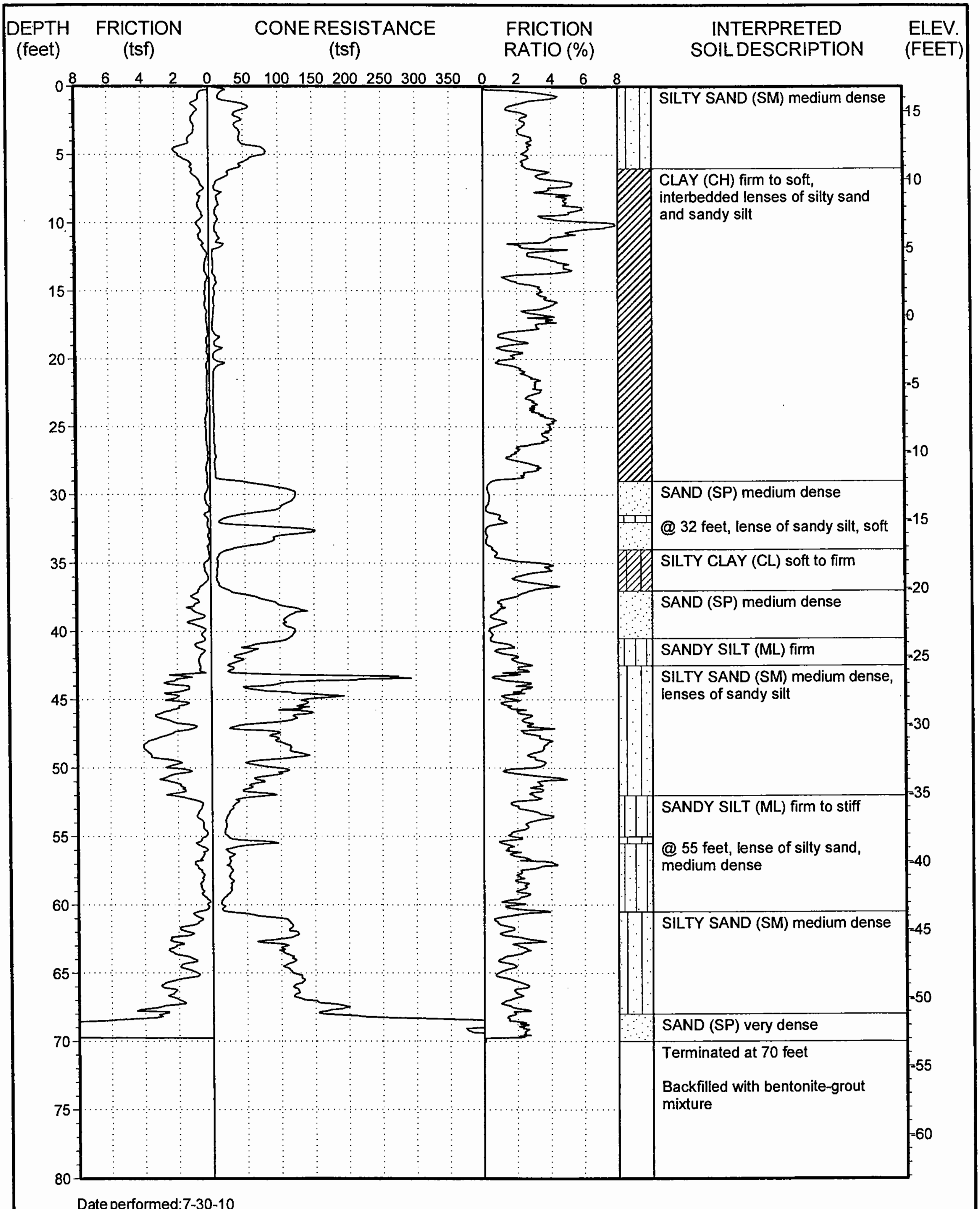
INNS AT BRIDGE CREEK

LOG OF CPT NO. C-3

FIGURE A-4



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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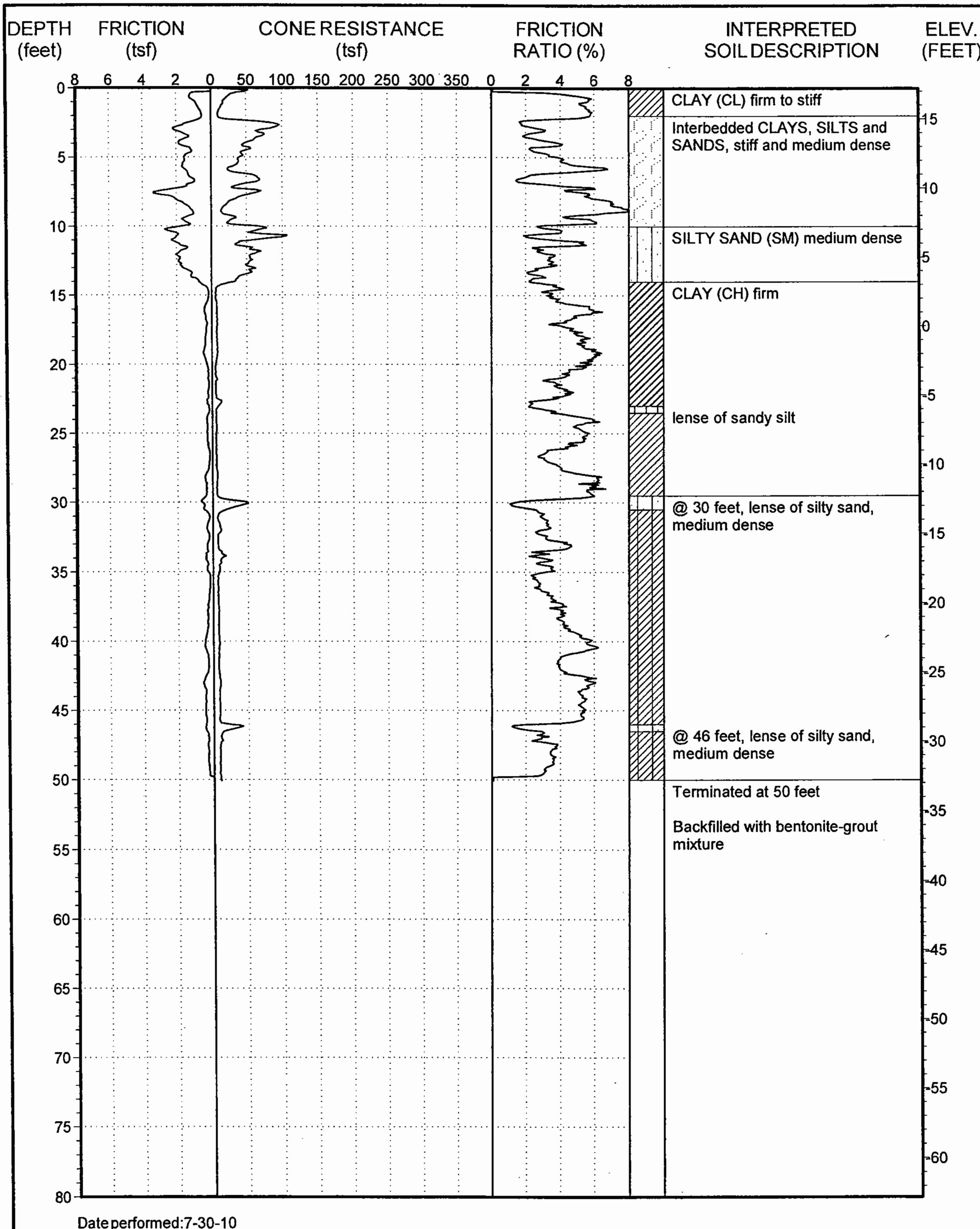
PROJECT NO.: 2331.I
INNS AT BRIDGE CREEK

LOG OF CPT NO. C-4

FIGURE A-5



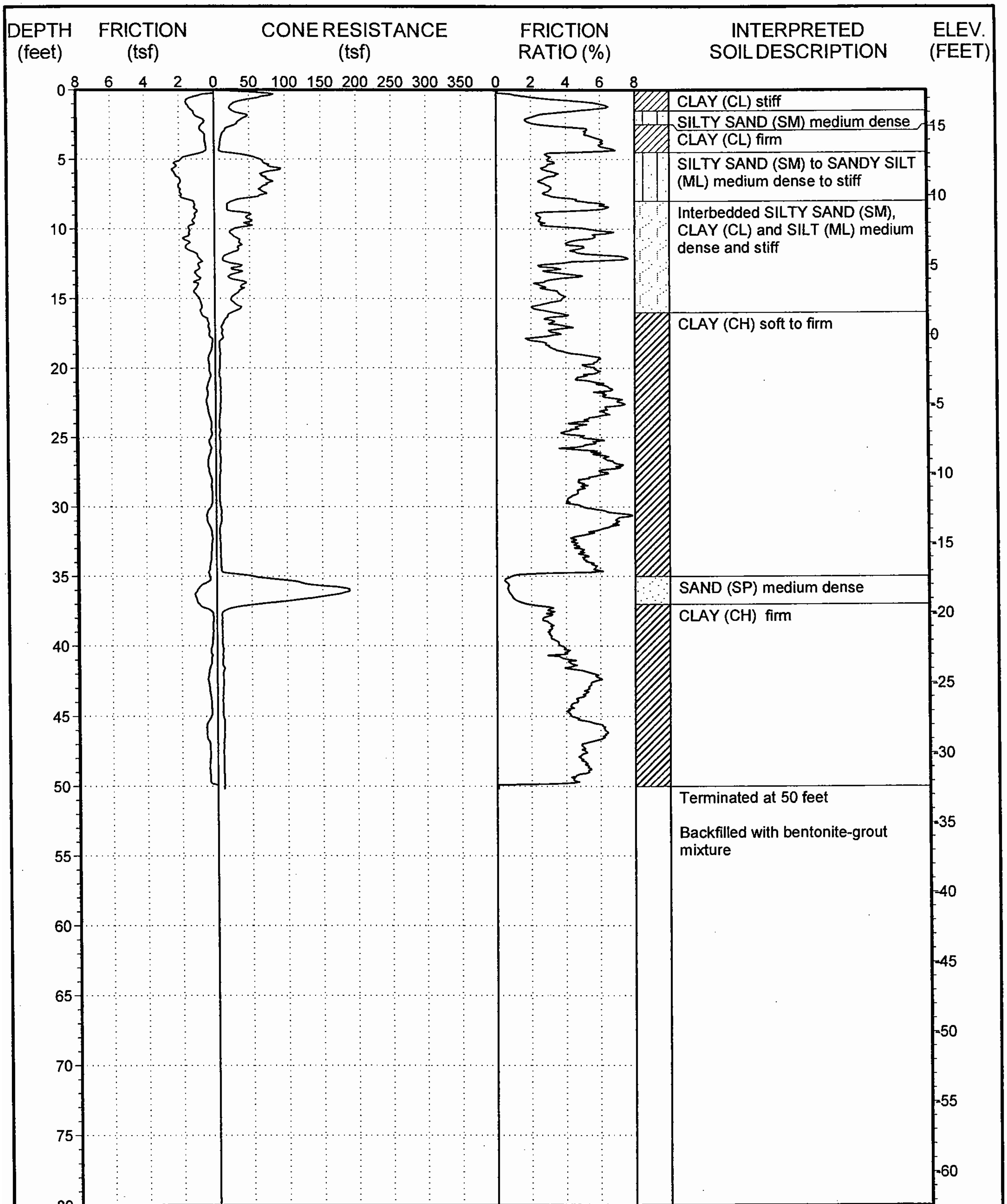
This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

PROJECT NO.: 2331.I

INNS AT BRIDGE CREEK

LOG OF CPT NO. C-5

FIGURE A-6



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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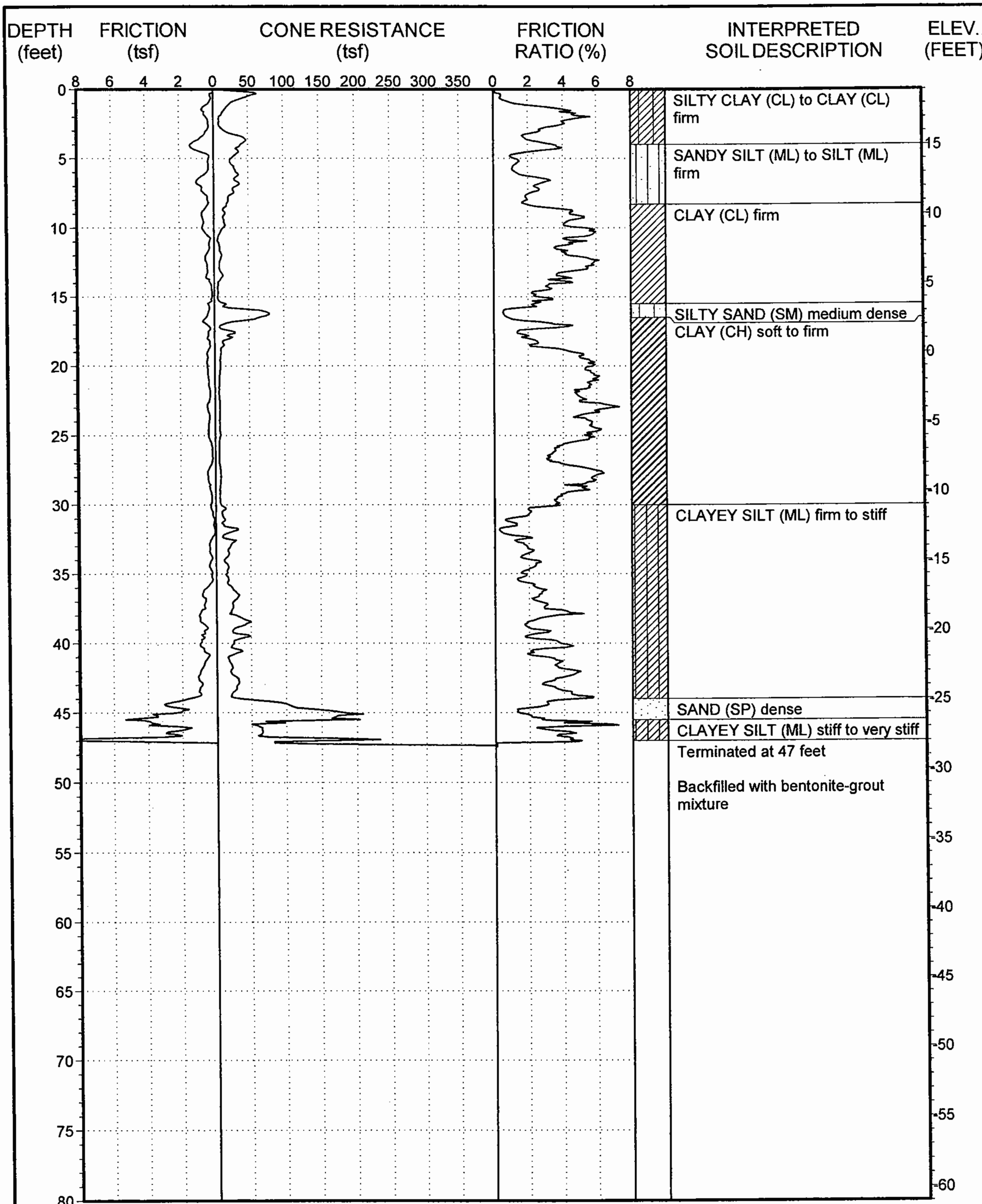
FIGURE A-7



Date performed:7-30-10

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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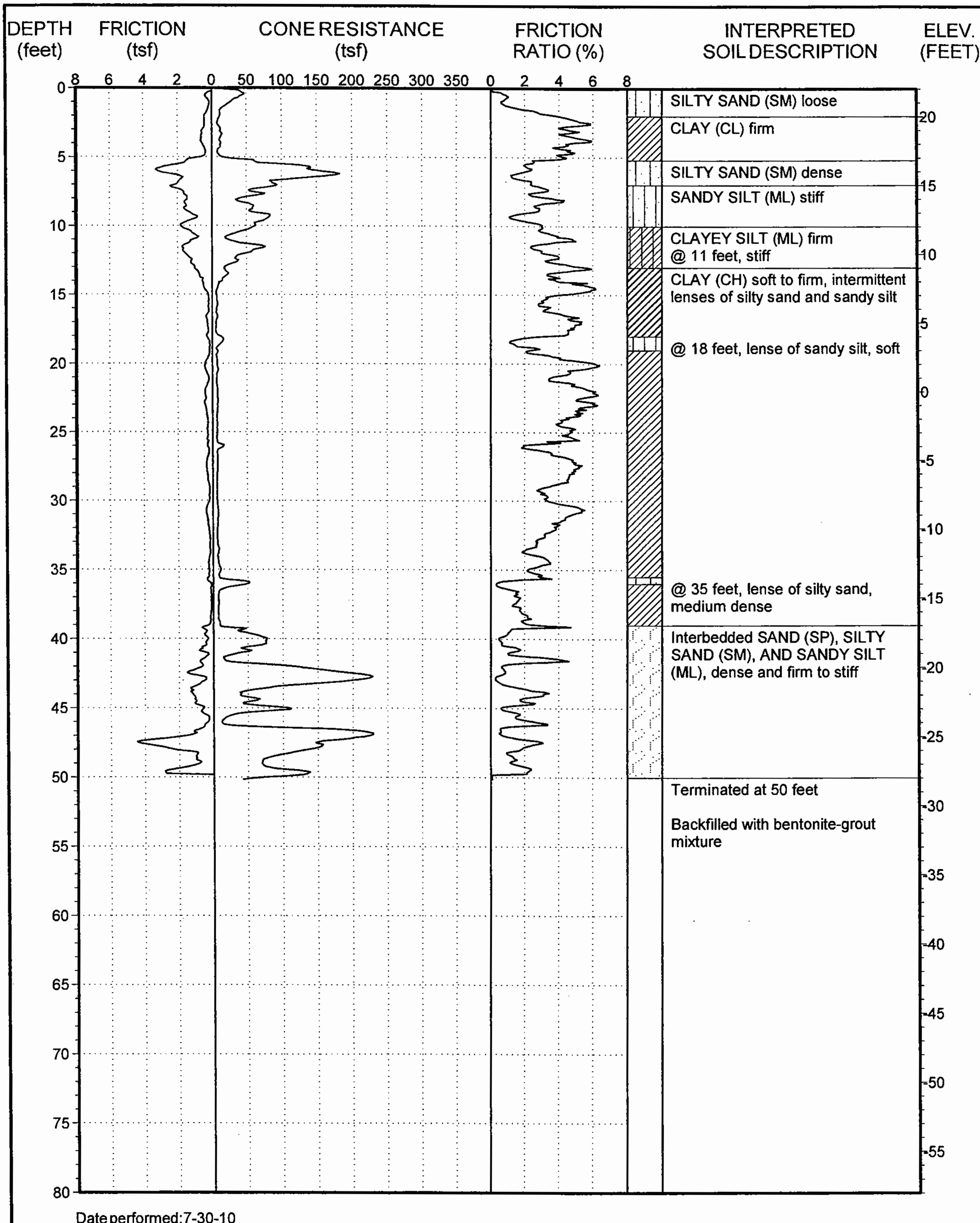
LOG OF CPT NO. C-7

FIGURE A-8



This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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LOG OF CPT NO. C-8

FIGURE A-9

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling an exploratory boring. The boring was advanced to a depth of 80 feet below the existing ground surface. The location of the exploration is shown on the Site Plan, Figure 2.

The boring was drilled using truck-mounted hollow-stem drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

The field exploration for the investigation was performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. A detailed log of the boring is presented in Figure B-1 in this appendix.

The boring was backfilled with a mixture of bentonite-grout in conformance with the requirements of the County of San Diego, Department of Environmental Health.

The boring locations were laid out in the field by measuring from existing site features. The ground surface elevations at the boring locations were estimated from topographic survey data provided by JENNA Development and should be considered approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	Fill: **CLAY (CL)** brown, moist, trace sand	20
	27.0	93	14	D		@ 2 feet, dark grey, wet. stiff	
	45.6	86	14	D	5	**SANDY CLAY (CL)** light brown, moist, stiff, trace gravel	15
	12.7	118	38	D		@ 7 feet, very stiff	
	14.3	115	19	D	10	@ 10 feet, stiff	10
	18.4	109	5	D	15	Natural: **CLAYEY SAND (SC)** grey, wet, loose	5
				B			
	40.8	76	8	D	20	**CLAY (CH)** grey, wet, firm	0
	39.8	77	9	D	25		-5
	52.5	68	11	D	30		-10
	57.5	66	9	D	35	**SILT (MH)** grey, wet, firm, trace clay	-15
	19.0	109	15	D	40	**CLAYEY SAND (SC)** grey, wet, trace shells, loose to medium dense	-20

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-2-10

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
15.0

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INNS AT BRIDGE CREEK

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
15.6	120	19	D	45		-25
16.7	104	25	D	50	**SAND (SP)** brown, wet, medium dense, coarse grained	-30
14.7	114	40	D	55	@ 55 feet, grey, coarse	-35
21.6	106	25	D	60		-40
16.9	111	69	D	65	**SILTY SAND (SM)** orangish grey, wet, dense	-45
13.7	115	55	D	70	@ 70 feet, grey/brown, very moist	-50
15.0	116	70	D	75	**SAND (SP)** grey, wet, dense, with gravel **CLAYEY SAND (SC)** grey/light brown, moist, dense, with gravel	-55
		50/1"	D	80	Total Depth 80 feet Backfilled with bentonite-grout mixture	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-2-10

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
15.0

PROJECT NO.: 2331.I

INNS AT BRIDGE CREEK

LOG OF BORING NO. B-1

FIGURE B-1

APPENDIX C

APPENDIX C

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix B.

PERCENT PASSING NO. 200 SIEVE

A total of three soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	15	Clayey Sand (SM)	27
B-1	40	Clayey Sand (SM)	32
B-1	45	Clayey Sand (SM)	24

ATTERBERG LIMITS

Liquid and plastic limits were determined for a selected sample in accordance with ASTM D4318. Results of the Atterberg Limits test are summarized on Figure C-1.

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of up to 25.6 ksf. The sample was inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Timed-deformation readings were performed on a constant load increment over a period of 24 hours on three samples. Results of the consolidation tests, in the form of percent consolidation versus log pressure are presented in Figures C-2 to C-5.

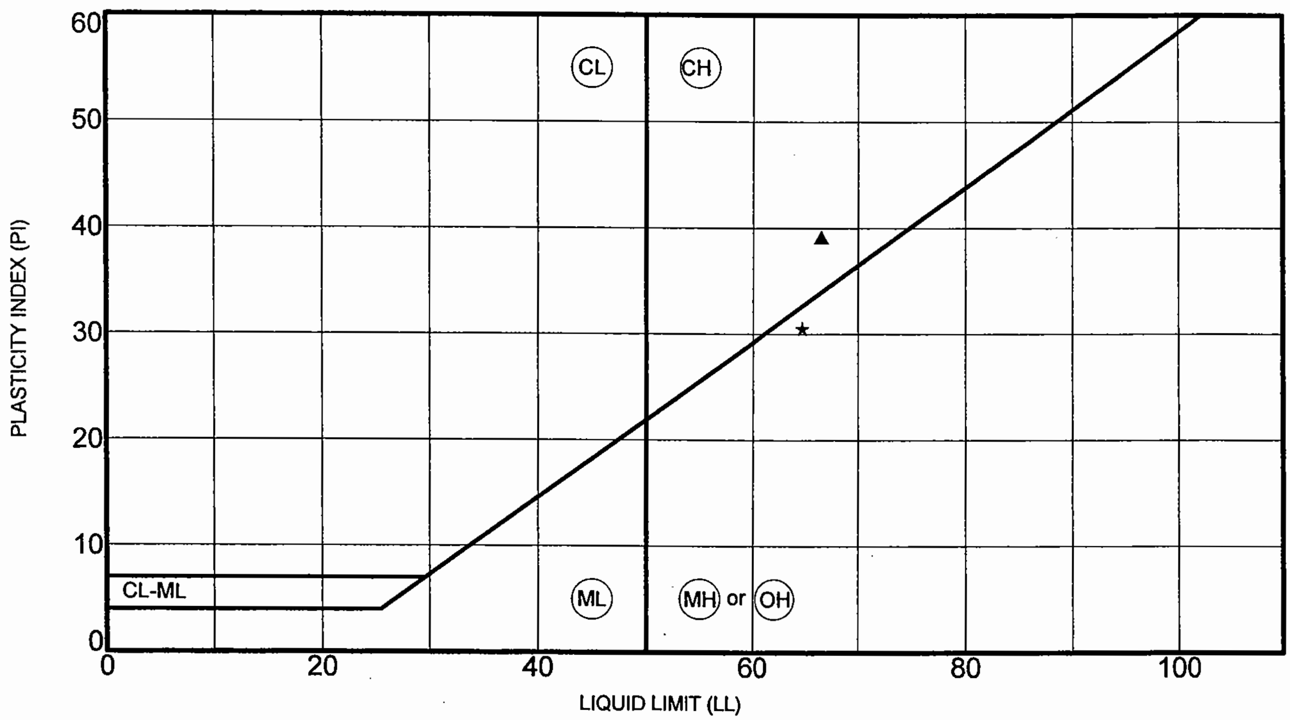
EXPANSION INDEX

An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM 4289 to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0-5	Clay (CL)	108

CORROSIVITY

Soil corrosivity testing was performed by Schiff Associates on soil samples provided by GPI. The test results are summarized in Table 1 of this appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
▲	B-1	20.0	67	27	39		FAT CLAY (CH)
★	B-1	35.0	65	34	31		SILT (MH)

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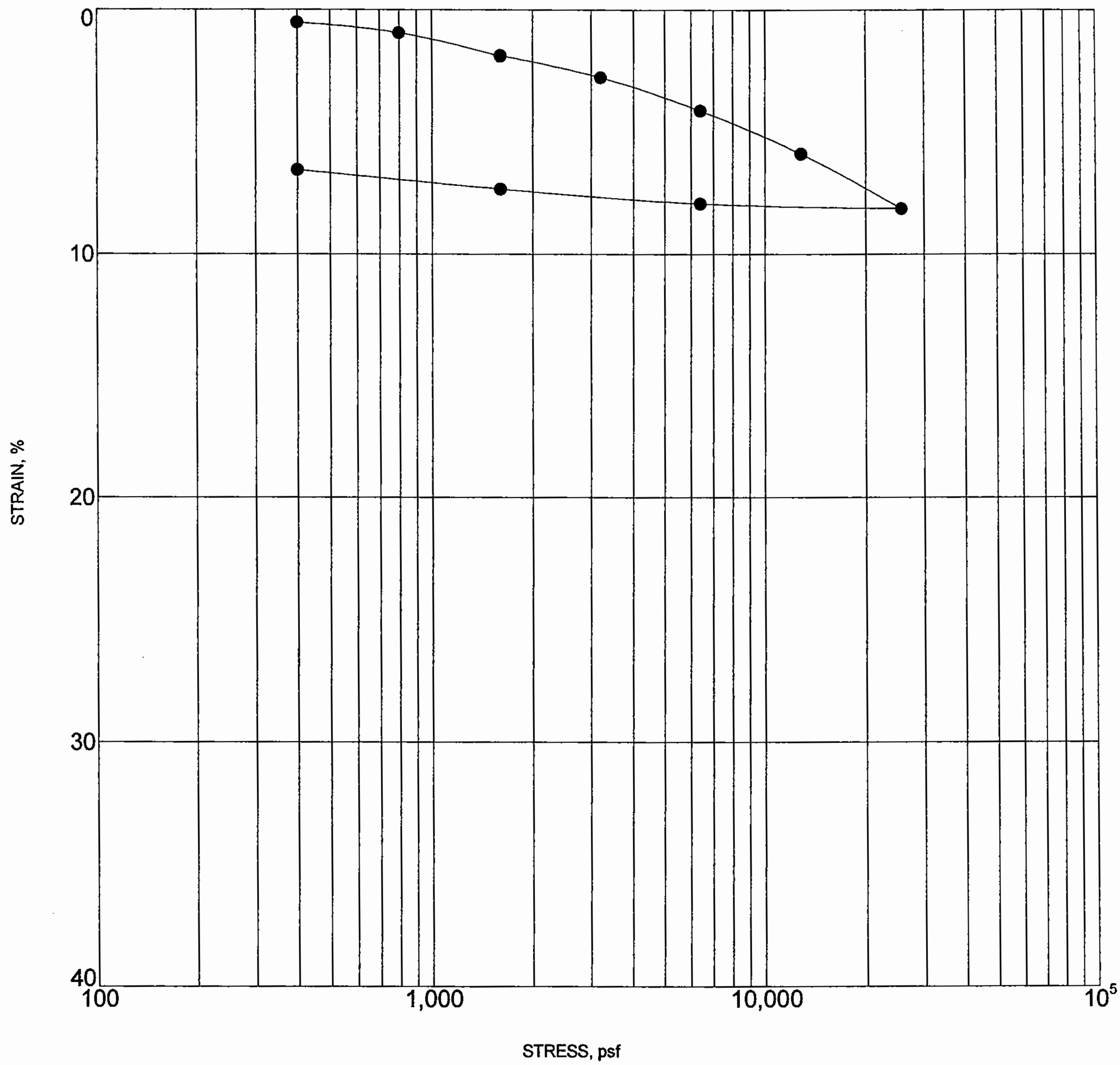
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	10.0	SANDY CLAY (CL)	115	14.3

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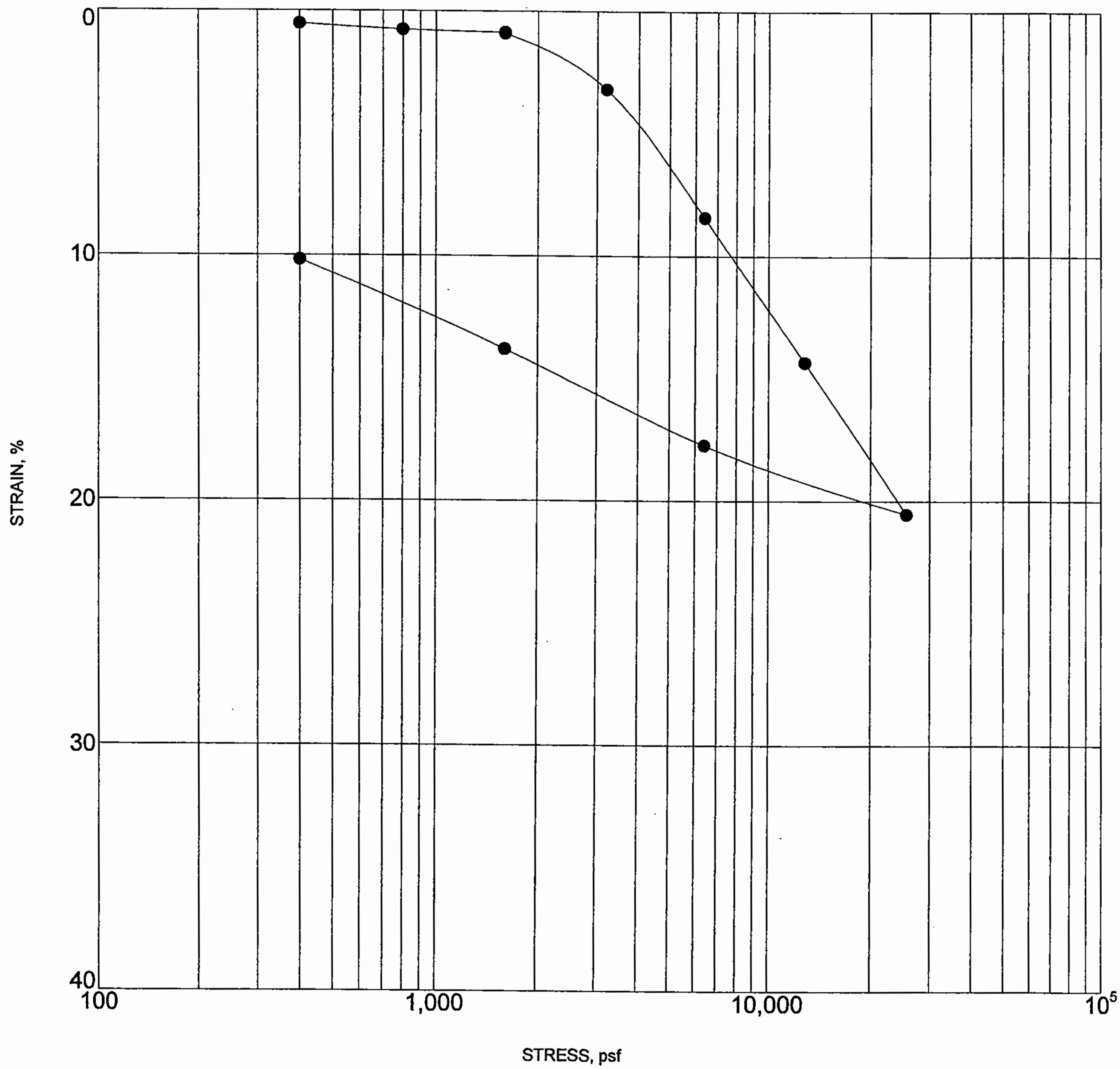
CONSOLIDATION TEST

FIGURE C-2



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	20.0	FAT CLAY (CH)	76	40.8

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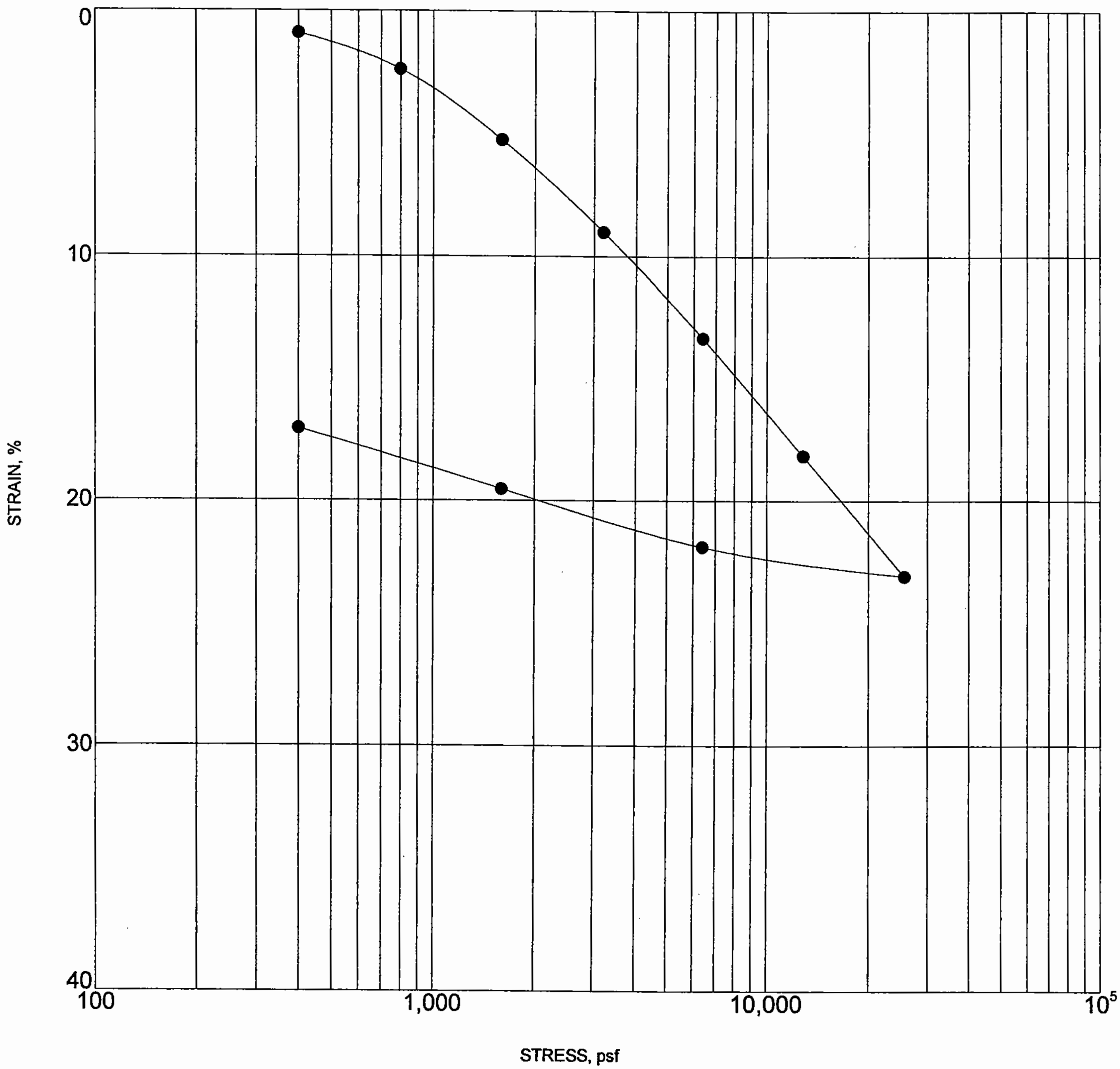
CONSOLIDATION TEST

FIGURE C-3



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	25.0	CLAY (CH)	77	39.8

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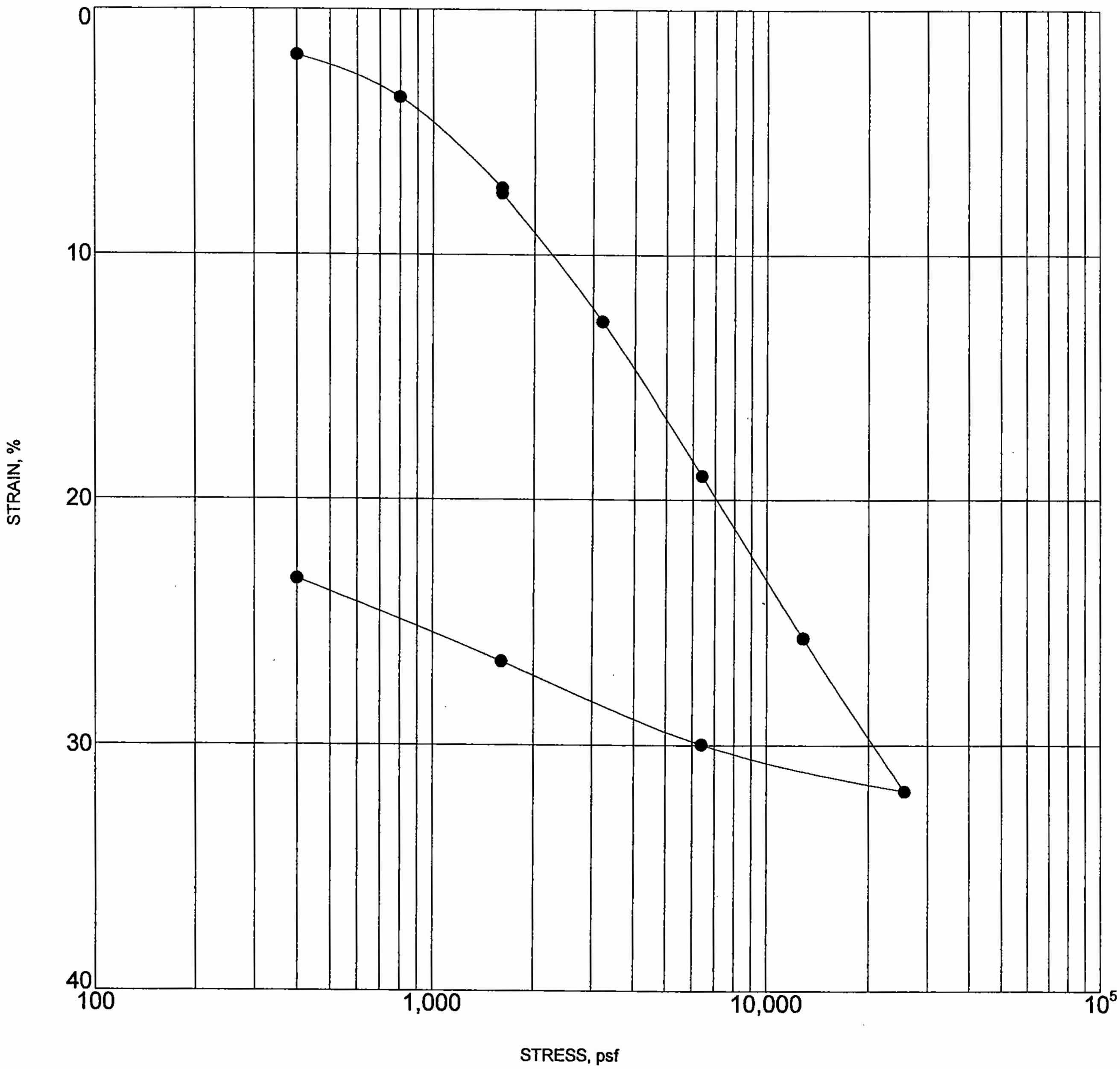
CONSOLIDATION TEST

FIGURE C-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	35.0	SILT (MH)	66	57.5

PROJECT: INNS AT BRIDGE CREEK

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CONSOLIDATION TEST

FIGURE C-5

SCHIFF ASSOCIATES

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Table 1 - Laboratory Tests on Soil Sample(s)

Geotechnical Professionals Inc.
Inns at Bridgecreek
Your #2331.1, SA #10-0889LAB
26-Aug-10

Sample ID			B-1 @ 0-4' Clay (CL)
Resistivity		**Units**	
• as-received		ohm-cm	68,000
saturated		ohm-cm	96
pH			7.4
Electrical Conductivity		mS/cm	6.59
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	3,757
magnesium	Mg^{2+}	mg/kg	972
sodium	Na^{1+}	mg/kg	5,024
potassium	K^{1+}	mg/kg	184
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	165
flouride	F^{1-}	mg/kg	5.3
chloride	Cl^{1-}	mg/kg	6,424
sulfate	SO_4^{2-}	mg/kg	12,669
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	ND
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed