CHARACTERIZATION OF "T-Z" MODEL DESIGN PARAMETERS FOR AUGERED CAST-IN-PLACE PILES USING FIELD LOAD TEST DATA

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ABSTRACT

With the increasing use of augered cast-in-place piles in new construction, it is important that proper design parameters be incorporated when evaluating pile capacity and performance using reliability-based design methods. This paper focuses on developing “t-z” model parameters from analysis of static axial compression and tension load test data from a project site along the Missouri River floodplain in northwest Missouri. Data was collected from a total of twelve axial load tests (six compression and six tension) and includes dial gauge readings from the pile heads as well as vibrating wire strain gauge data from multiple locations throughout several of the test piles. The “t-z” method has been used extensively as a soil-structure interaction model to evaluate the settlement of deep foundations. The soil-structure interaction modeled in this analysis was based on hyperbolic load displacement behavior using effective (drained) stresses. The development of the “t-z” model parameters has been accomplished using finite difference methods to analyze the non-linear soil-structure interaction along the sides of the piles. During the analysis, the mean shear modulus of soil-structure interface subgrade reaction, K_{init}, and the mean ultimate shear strength of the soil-structure interface, τ_u, were back-calculated from each set of load test data and were based on the assumption of a single-layer, homogenous soil profile. These “t-z” model parameters were then compared to standard field investigation data, including standard penetration tests (SPT) and cone penetrometer test (CPT) soundings, and effective overburden stress to develop correlations suitable for service limit state design of augered cast-in-place piles. While there was some indication of a linear relationship between K_{init} and the field investigation data, there was not a sufficient quantity of data in the analysis to properly identify any statistical trends. The relationship between τ_u and the field investigation data was much more variable and did not provide any distinct correlation. The plot of the data relating the model parameters to the effective overburden stress exhibited some grouping but the sample size and distribution was not sufficient to identify any statistical trends.

The faculty listed below, appointed by the Dean of the School of Computing and Engineering, have examined a thesis titled “Characterization of “t-z” Model Design Parameters for Augered Cast-In-Place Piles Using Field Load Test Data,” presented by Bradley Scott Gardner, candidate for the Master of Science degree, and hereby certify that in their opinion it is worthy of acceptance.

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CHAPTER 1

INTRODUCTION

Augered cast-in-place (ACIP) piles, also referred to as auger cast piles, augered pressure-grouted (APG) piles, and continuous flight auger (CFA) piles, among others, have been in use in the United States for over 60 years. Throughout this time, ACIP piles have been utilized on a wide variety of project types including industrial and commercial buildings, bridges, equipment foundations, transmission structures, retaining walls and various other structures (DFI 1990). Due to the relative speed of installation, economy and high capacity, ACIP piles have come to be a common deep foundation element selected by geotechnical engineers for a wide variety of soil types. Unlike drilled shafts, the installation methods utilized for ACIP piles are somewhat independent of the soil and groundwater conditions encountered at a project site.

ACIP piles are a type of drilled foundation in which the pile is drilled to a targeted depth in one continuous process using a continuous flight auger with a hollow core. As the auger is drilled into the ground, the flights of the auger are filled with soil which provides lateral support and stability of the drilled hole. As the auger is withdrawn from the hole, a grout mixture typically consisting of sand, cement, and water is pumped continuously under pressure through the hollow center of the auger to the base of the auger. As the grout is pumped, the auger is lifted smoothly in one continuous operation. Because the auger provides support during the drilling process, and the fluid grout provides support during the auger removal, there is no need for the use of casing or drilling slurry when this pile type is installed in soils prone to caving.

Immediately following placement of grout in the hole, and complete removal of the auger, any soil cuttings remaining in the grout at the top of the pile are removed. Following the completion of the grouting process, a steel reinforcing cage is placed into the grout. The cage is then tied off at the ground surface to prevent the cage from settling into the fluid grout and to maintain proper elevation for the top of the reinforcement.

ACIP piles are typically installed with diameters ranging from 12 to 36 inches and lengths of up to 100 feet, with longer piles occasionally used. The reinforcing cages are often confined to the upper 30 to 50 feet of the pile for ease of installation and due to the fact that relatively low bending stresses are transferred beyond these depths (Brown et al. 2007). When tension loads are included in the design, full length center bars can be used to transfer the design tension loads to the full pile depth.

ACIP Pile Axial Capacity Design

In general, there are three design conditions that must be met when evaluating the axial capacity of a deep foundation: the geotechnical ultimate limit state or ultimate bearing capacity; the service limit state or settlement performance under service loads; and the structural ultimate limit state or structural capacity. For many projects, the ultimate limit state capacity will control the design of individual ACIP piles rather than the service limit state capacity. For that reason, the typical design approach is to first evaluate the ultimate limit state capacity and then check the service limit state performance. Once a design has been selected to satisfy the requirements of the ultimate limit state capacity and the service limit state performance, the structural capacity will be evaluated and the steel reinforcement designed to accommodate the internal stresses (Brown et al. 2007).

While it is important to evaluate the lateral capacity of ACIP piles, the focus of this research was on the evaluation of axial capacity. Therefore, the methods and steps required for the evaluation of lateral capacity are not specifically addressed herein.

Ultimate Limit State Capacity

Currently, a large portion of the deep foundations designed in the United States, including ACIP piles, are designed utilizing Allowable Stress Design (ASD) procedures, also referred to as “working stress”, and only consider the ultimate limit state capacity of the foundation.

With ASD design, the foundation capacity is evaluated by assuming full resistance through skin friction and end bearing. Extensive research has been conducted and many methods have been developed to evaluate the skin friction and end bearing components that contribute to the ultimate limit state capacity. Brown et al. (2007) includes a summary and comparison of 16 methods that can be used for estimating the axial capacity of ACIP piles. These include methods that were initially developed for the design of drilled shafts and driven piles but are also considered applicable to the design of ACIP piles. Also included are summaries of four comparisons of design methods, the results of which were used as the basis for the selection of recommended methods to be used for estimating the axial capacity of ACIP piles.

The FHWA 1999 Method for the design of drilled shafts presented by O'Neil and Reese (1999), and originally developed by Reese and O'Neill (1988), is the recommended method for the estimation of both the skin friction and end bearing capacity for ACIP piles in cohesive and cohesionless soils. This method is one of the more widely recognized methods for the prediction of drilled shaft capacities and comparison studies have shown it to be reasonably accurate at estimating capacities for ACIP piles.

The method generally relies on soil strength data collected from conventional soil borings including the undrained shear strength, S_u, of cohesive soils and the N_{60} values from cohesionless soils. The preferred method for evaluating S_u in cohesive soils is through laboratory triaxial compression tests or unconfined compressive strength tests performed on relatively undisturbed samples collected from the soil borings. N_{60} values are the standard penetration test (SPT) *N*-value adjusted to represent a hammer efficiency of 60 percent but they are not corrected for depth.

For an incremental length of pile, the ultimate unit skin friction in cohesive soils, τ_u, is calculated as:

$$\tau_u = \alpha S_u \tag{1-1}$$

where, α is a strength reduction factor that accounts for soil disturbance during construction, water migration from the concrete, and other similar factors (O'Neil and Reese 1999). The strength reduction factor varies as follows:

$$\alpha = 0.55 \qquad \text{for } S_u/P_a \leq 1.5, \text{ and} \tag{1-2}$$

$$\alpha = 0.55 - 0.1(S_u / P_a - 1.5) \qquad \text{for } 1.5 \leq S_u/P_a \leq 2.5 \tag{1-3}$$

where, P_a is the standard atmospheric pressure typically approximated as 1.06 tons per square foot (tsf).

The ultimate unit end bearing resistance for cohesive soils, q_t, is calculated as:

$$q_t = N_c^* S_u \tag{1-4}$$

where, N_c^* is the bearing capacity factor and S_u is the average undrained shear strength of the soil with two pile diameters below the tip of the pile. For cohesive soils where $S_u \geq 2$ tsf, the value of $N_c^* = 9$ can be used with reasonable accuracy. For cohesive soils where $S_u \leq 2$ tsf, the value of N_c^* is reduced to as low as 6.5 as a function of S_u and the undrained Young's modulus of the soil.

The ultimate unit skin friction in cohesionless soils, τ_u, is calculated based on correlations with the lateral earth pressures and the drained angle of internal friction as follows:

$$\tau_u = K_{ep}\, \sigma_v' \tan\delta \;\leq\; 2.0 \text{ tsf} \tag{1-5}$$

where, K_{ep} is the lateral earth pressure coefficient, σ_v' is the vertical effective stress, and δ is the drained angle of friction for the pile-soil interface. The values of K_{ep} and δ are difficult to evaluate and are significantly dependent on the type of deep foundation being installed and the quality of construction practices. Due to the difficulty of accurately evaluating the values of K_{ep} and δ, the simplified "β method" has been developed such that:

$$\beta_{ep} = K_{ep} \tan\delta \tag{1-6}$$

And Equation 1-5 can be re-written in the form:

$$\tau_u = \beta_{ep}\, \sigma_v' \tag{1-7}$$

Where definitive information is not available regarding the values of K_{ep} and δ, it is considered reasonable to use an empirical relationship for β_{ep} that is near the lower bound of values obtained from a database of load tests (O'Neil and Reese 1999). The values for β_{ep} are typically correlated directly with the SPT N_{60} values which are expressed in blows per foot (bpf). The values recommended for use with the design of ACIP piles following the FHWA 1999 Method are estimated as:

$$\beta_{ep} = 1.5 - 0.135\, Z^{0.5} \qquad \text{for } N_{60} \geq 15 \text{ bpf} \qquad (1\text{-}8)$$

$$\beta_{ep} = \frac{N_{60}}{15}(1.5 - 0.135\, Z^{0.5}) \qquad \text{for } N_{60} < 15 \text{ bpf} \qquad (1\text{-}9)$$

where, Z is the depth (in feet) from the ground surface to the midpoint of each respective soil layer or pile segment.

Within the FHWA 1999 method, the ultimate unit end bearing resistance value for cohesionless soils, q_t, is also correlated directly with SPT N_{60} values as follows:

$$q_t \text{ (tsf)} = 0.6N_{60} \qquad \text{for } 0 \leq N_{60} \leq 75 \qquad (1\text{-}10)$$

$$q_t = 45 \text{ tsf} \qquad \text{for } N_{60} > 75 \qquad (1\text{-}11)$$

where, N_{60} is considered within an interval from approximately one pile diameter above to two or three diameters below the tip of the pile.

Where cone penetration test (CPT) soundings are available, the Laboratorie Des Ponts et Chausses (LPC) method, originally developed by Bustamante and Gianeselli in 1982 for drilled shafts and driven piles, has been shown to provide reliable estimates of axial capacity for ACIP piles in both cohesive and cohesionless soils (Brown et al. 2007).

Using the LPC method, the ultimate unit skin friction and end bearing resistance values are correlated with the cone tip resistance, q_c, as follows:

$$\tau_u = \frac{q_c}{\alpha} \qquad \text{(skin friction)} \qquad (1\text{-}12)$$

$$q_t = k_c\, q_c \qquad \text{(end bearing)} \qquad (1\text{-}13)$$

where, τ_u is the ultimate unit skin friction, α is a friction coefficient, qt is the ultimate unit end bearing, and k_c is a bearing capacity factor. Recommended values for α and k_c for the design of ACIP piles, which are referred to in the LPC method documentation as "hollow auger bored piles" are dependent upon soil type as summarized below in Table 1. In addition, maximum values for τ_u are specified as part of the LPC method and are summarized in Table 1. For select soil types, a higher maximum value for τ_u, noted in parentheses in Table 1, is included for situations where careful execution and minimum disturbance of soil can be accounted for during construction.

Table 1. – LPC method α coefficients and k_c factors (Robertson and Robertson 2010)

Soil Type	q_c (MPa)	α Coefficient	Maximum τ_u (MPa)	k_c Factor
Soft clay and mud	< 1	30	0.015	0.40
Medium stiff clay	1 to 5	40	0.035 (0.08)	0.35
Very stiff clay	> 5	60	0.035 (0.08)	0.45
Loose silt/sand	< 5	60	0.035	0.40
M. dense sand/gravel	5 to 12	100	0.080 (0.12)	0.40
V. dense sand/gravel	> 12	150	0.120 (0.15)	0.30

Once the ultimate limit state capacity has been calculated using one of the many methods for axial capacity design, an assumed safety factor is then applied to that value to obtain the allowable axial capacity to be used for design. The factor of safety is used to account for variations in soil material strengths, inaccuracies in design equations, construction methods and the potential for errors to occur during construction, and the consequences of foundation failure (Phoon, Kulhawy and Grigoriu 2000). Typical factor of safety values range from 2 to 3 but can vary widely from engineer to engineer based on a variety of factors such as personal experience, quantity and quality of subsurface information available, and use of quality control measures such as static or dynamic load testing prior to or during construction. For the design of ACIP piles, Brown et al. (2007) recommend a minimum safety factor of 2.5 unless four specific conditions are met which permit the use of a safety factor of 2.0. Those conditions include: (1) the performance of at least one conventional static load test to a load exceeding the computed ultimate by 50 percent or to a load producing displacement equal to 5 percent of the pile diameter, (2) use of automated monitoring equipment on production pile, (3) the site geology stratigraphy, and soil properties are not highly variable, and (4) the site conditions do not pose difficult construction conditions for the piles.

Service Limit State Performance

Once the ultimate and allowable capacities have been calculated, the service limit state or settlement performance of the deep foundation can then be evaluated. The settlement analysis methods for single piles can generally be grouped into three broad categories: (1) load transfer methods which incorporate the relationship between pile resistance and pile movement at select points along the pile, (2) methods based on the theory of elasticity using equations by for subsurface loading within a semi-infinite mass published by Mindlin in 1936, and (3) numerical methods such as finite-element analysis (Poulos and Davis 1980).

The method based on elastic theory is developed from equations for stress and deformation at any point in the interior of semi-infinite, elastic, and isotropic solids resulting from a force applied at another point in the solids. (Reese, Isenhower and Wang 2006). In addition, the method is generally based on the assumption that no slip occurs at the pile-soil interface. The method does not effectively consider the soil-structure interaction between the foundation and the surrounding soil. Modifications to the basic approach have been developed that allow for slip but the displacements that occur after slip occurs are still based on elastic theory. Several numerical methods have been developed based on the elastic theory method but which permit the consideration of variations to the method such as soil layering and bilinear or elasto-plastic soil performance. (Poulos and Davis 1980).

Empirical curves were developed by Reese and O’Neill (1988) to improve on the methods for prediction of settlement of drilled shafts by evaluating the load transfer and settlement performance for side resistance and end bearing separately. The curves were developed from the analysis of a database of compression load tests performed on single, full-sized drilled shafts. The use of these curves requires iterative process of estimating the settlement of the foundation element until the corresponding tip and side resistance values added together equal the applied design load for the foundation. The load-settlement curves for side resistance were developed from tests performed on drilled shafts ranging diameter from 18 inches to 60 inches and the curves for end bearing were developed from tests performed on drilled shafts ranging in diameter from 30 inches to 132 inches. Use of these curves on drilled shafts with diameters outside those ranges should be verified with load testing (O'Neil and Reese 1999). While Brown et al. (2007) suggests that these curves can also be used to evaluate the load-settlement performance of ACIP piles, it is also recommended that the results be verified with load testing.

While there are separate curves for cohesive and cohesionless soils, as shown in Figures 1 and 2, this empirical method for evaluating settlement does not specifically consider the soil material properties along the side or at the tip of the foundation element. Rather, the method provides an estimate of settlement based on the ratio of design side load relative to the ultimate side load capacity as well as the ratio of the design end bearing pressure relative to the ultimate end bearing capacity. Furthermore, the evaluation of the overall load-settlement performance requires an iterative process to account for various aspects of the design, such as soil layering and stiffness of the soil relative to the stiffness of the foundation element, and to identify the proportioning of side load and end bearing load that result in comparable settlement performance.

The development of theoretical load transfer methods that consider the soil-structure interface performance, which can be applied to a variety of deep foundation types, and which can be adjusted for site-specific conditions, would be beneficial. Such an approach is discussed in Chapter 2

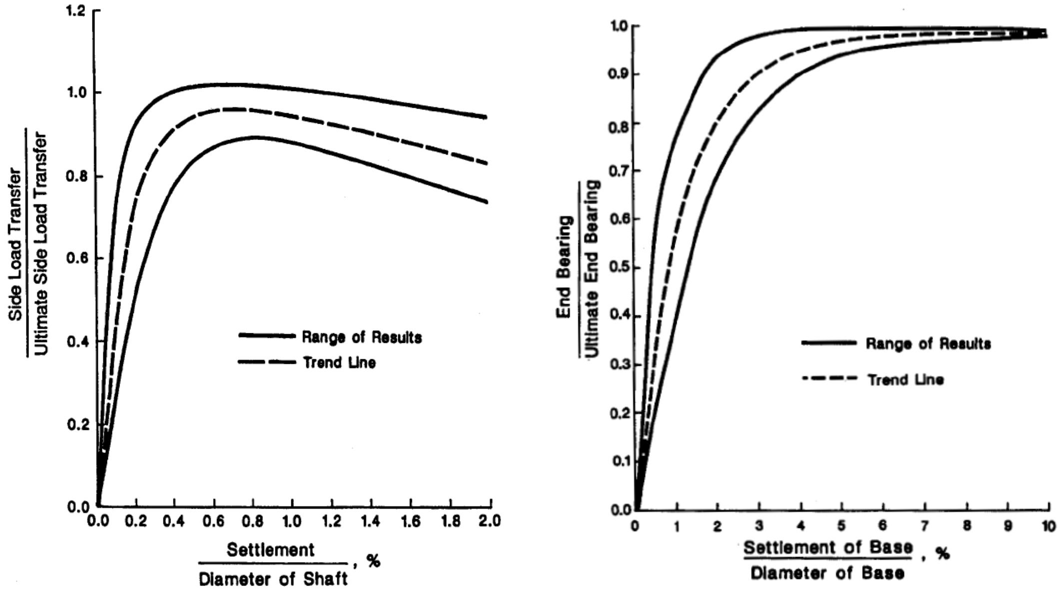


Figure 1. Normalized load-settlement curves for cohesive soils (Brown et al. 2007).

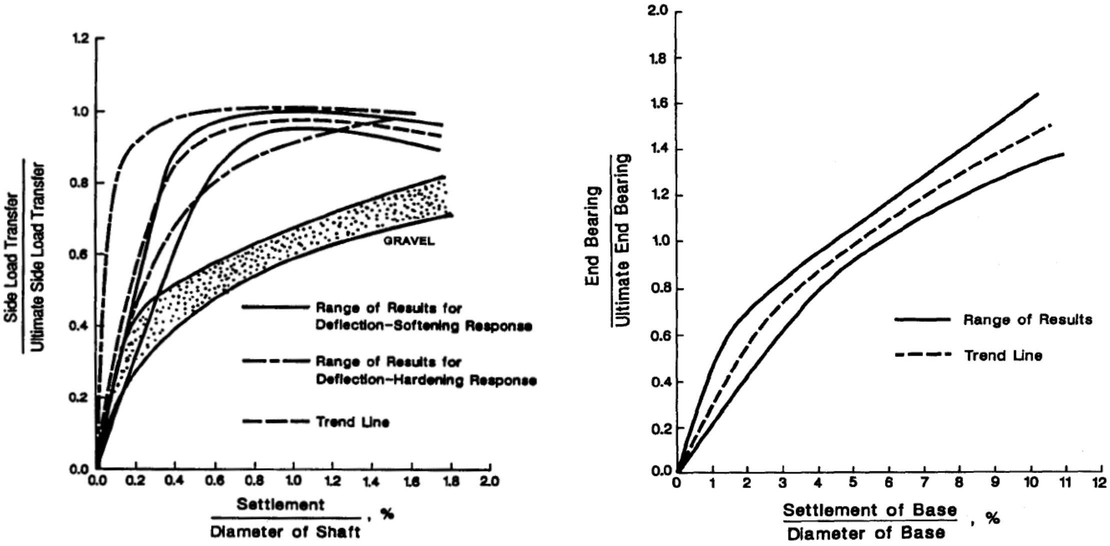


Figure 2. Normalized load-settlement curves for cohesionless soils (Brown et al. 2007).

CHAPTER 2

LOAD-DISPLACEMENT ANALYSIS USING THE “T-Z” METHOD

Due to the uncertainty of load-settlement performance inherent in the ASD methods, a theoretical method of evaluating load-settlement performance for a variety of deep foundation types, and which takes into consideration the soil-structure interaction, would be beneficial. Recently, the “t-z” method, first developed by Seed and Reese (1957), has become more widely used to evaluate and predict the load-transfer performance of deep foundations including ACIP piles. While the ASD methods consider the side and tip capacities and settlements separately and then add them together, the “t-z” method utilizes data relating the load transfer from the pile to the soil, *t*, to the foundation displacement, *z*, to evaluate the evaluate the capacity and settlement along the length of the pile and at the tip of the pile. With numerical modeling techniques, load-settlement performance of the sides and tip of the pile, as well as the elastic shortening of the structural foundation element, can be modeled simultaneously. Furthermore, the “t-z” method has the ability to incorporate site specific strength and deformation properties of soils (Zhu and Chang 2002).

Numerous empirical and theoretical models have been developed to evaluate the load transfer performance of deep foundations which are primarily based on drilled shafts and driven piles. Few models have been developed specifically for ACIP piles and those that have been developed usually take the form of modifications to models originally developed for drilled shafts or driven piles (Brown et al. 2007). The study conducted herein follows previous research and analysis by Misra and Roberts using a theoretical "t-z" model to explicitly describe the load-displacement behavior of deep foundations (Misra and Roberts 2006, Roberts 2006).

The load-displacement behavior of the soil-structure interface can generally be described by two different theoretical "t-z" models: (1) a linear ideal elasto-plastic model, and (2) a non-linear hyperbolic model. The ideal elasto-plastic model can be evaluated using closed-form analytical relationships to describe the load-displacement. However, the hyperbolic model requires the solution of differential equations using numerical analysis techniques to describe the soil-structure interaction and evaluate the load-displacement behavior. The equations and methods for evaluating the non-linear model, on which this analysis is based, are summarized herein. For further derivation and reference of the equations for evaluating both the elasto-plastic model and the non-linear model, the reader is referred to Roberts (2006).

Soil-Structure Interaction Model

The soil-structure interaction that acts along the length of the pile in the "t-z" model method of evaluating load-displacement performance can be represented by the spring-slider system shown in Figure 3. This assumption is common with analytical and numerical models of the load-displacement behavior of drilled shafts and piles (Kraft, Ray and Kagawa 1981, Reese and O'Neill 1988).

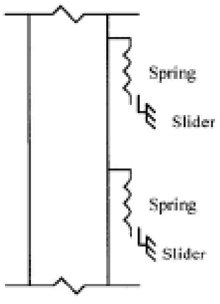


Figure 3. Spring-slider model of pile-soil interface (Roberts 2006).

The analysis of the pile-soil interface resistance, represented by a series of springs along the length of the pile and at the tip, can be assumed to behave either as a linear ideal elasto-plastic material or as a non-linear material. If an ideal elasto-plastic material is assumed, the displacement that occurs during loading is assumed to be recoverable such that the displacement returns to zero when unloading occurs. For the non-linear model, the displacement that occurs during loading is non-recoverable and thus permanent when unloading occurs. For this analysis, the interface was modeled with the non-linear, behavior which better represents the typical observed load-displacement behavior of deep foundations.

Figure 4 shows the hyperbolic curve representative of a non-linear force-displacement behavior where the displacement, u, is plotted relative to the shear force per unit length of pile, q. The value K_{init} represents the initial tangent shear modulus of the subgrade reaction at the soil-structure interface and q_o represents the asymptotic value of the ultimate strength at the soil-structure interface. The value of q_o is defined as the product of the pile perimeter and the ultimate shear strength of soil-structure interface, τ_u.

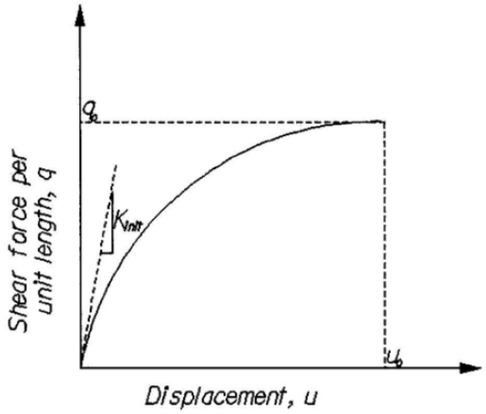


Figure 4. Non-linear force-displacement relationship for soil-structure interface (Roberts 2006).

For deep foundations, the load transfer occurs through the soil-structure interface along the length of the foundation element referred to as the interaction zone, L_b. Portions of the foundation with zero or negligible shearing resistance are considered to be the non-interaction zone. The length of the non-interaction zone can be affected by the diameter of foundation, frost depth, seasonal variations in moisture content of the soils, presence of fill, construction techniques, and the presence of excessive lateral loads. The non-interaction zone for deep foundations will typically range from as little as one foot, where cohesionless soils are present at the ground surface, to five feet or the full depth of seasonal moisture change, whichever is greater, where cohesive soils are present. Until recently, it has been common practice to also include a non-interaction zone of one diameter at the bottom of drilled shaft and auger cast piles which bear in cohesive soils. This approach was based on numerical modeling that predicts the development of a zone of tension at the shaft-soil interface in the zone immediately above the base of the shaft. Results of field load test data do not support this approach and Brown, Turner and Castelli (2010) recommend that side resistance should not be neglected along the bottom one diameter.

As the pile is loaded in compression, the soil-structure interface within the interaction zone goes through deformation based on the assumed hyperbolic force-displacement relationship described previously. As the load increases, the soil-structure interface will begin to yield near the top of the interaction zone and then progress downward toward the bottom of the pile. Throughout the loading sequence, the pile is assumed to behave as an elastic element. The load required to reach the yield strength of the pile grout is much higher than the load required to cause yielding of the soil-structure interface.

Load Transfer Model

As described by Bowles (1997), the force balance of the pile-soil interface can be represented by the following equilibrium equation:

$$q(z) - Ku(z) = 0 \qquad (2\text{-}1)$$

Where *q(z)* is the shear force per unit length along the pile and *u(z)* is the pile deformation at that location. The axial force in the pile can be represented by K_m *du/dz*, where K_m is the axial stiffness of the pile and is the product of the pile area and the modulus of elasticity of the pile material. The shear force per unit length, *q(z)*, can then be obtained with the following equation:

$$q(z) = K_m \frac{d^2u}{dz^2} \tag{2-2}$$

The governing equilibrium Equation 2-1 can then be written as:

$$K_m \frac{d^2u}{dz^2} - K\, u(z) = 0 \tag{2-3}$$

The non-linear force-displacement relationship behavior has been effectively described for both clay and sand soils using the following hyperbolic model developed by Kondner et al. (Duncan and Chang 1970):

$$q = \frac{u}{\left[\dfrac{1}{K_{init}} + \dfrac{uR_f}{q_o}\right]} \tag{2-4}$$

where, K_{init} represents the initial tangent shear modulus of the subgrade reaction at the soil-structure interface. R_f, is a factor described as the failure ratio relating the theoretical ultimate strength, q_o, of the load-displacement curve to the observed failure strength, q_f, as follows (Duncan and Chang 1970):

$$R_f = \frac{q_f}{q_o} \tag{2-5}$$

Duncan and Chang note that the value of R_f will always be less than unity and that it has been found to be between 0.75 and 1.00 for a variety of different soils. They also note that the value of R_f is essentially independent of the confining pressure.

The load transfer model described above is applicable to piles subject to tension loads where the soil-structure interaction is limited to side forces. For piles subject to compression loads, the tip performance needs to be considered in addition to the side forces. The tip force, P_t, developed for each increment of compression load acting on the pile is proportional to the tip displacement, u_t, and is described by:

$$P_t = K_t \, u_t \tag{2-6}$$

where, K_t is the tip soil stiffness. The tip soil stiffness can be related to the pile diameter and the elastic properties of the tip soil using the theory for rigid punch bearing on an elastic half-space using (Johnson 1985):

$$K_t = \frac{0.3 \, \pi \, D \, E_s}{\left(1 - \mu_s^2\right)} \tag{2-7}$$

where, E_s is the elastic modulus of the tip soil, μ_s is the Poisson's ratio of the tip soil, and D is the diameter of the pile. To evaluate non-linear force-displacement relationship behavior at the tip of the pile, Equation 2-4 can be rewritten to represent the tip force:

$$P_t = \frac{u_t}{\left[\dfrac{1}{K_{ti}} + \dfrac{u_t R_f}{P_{utip}}\right]} \tag{2-8}$$

where, K_{ti} is the initial tip soil stiffness. Equation 2-7 can be used to calculate the initial value of tip soil stiffness. The failure ratio, R_f, is now used to relate the theoretical ultimate capacity of the tip soil, P_{utip}, to the observed failure strength of the tip soil, P_{ftip}, similar to Equation 2-5:

$$R_f = \frac{P_{ftip}}{P_{utip}} \tag{2-9}$$

$$P_{utip} = q_t \ A_m \tag{2-10}$$

Finite Difference Methodology

While the assumption of hyperbolic load-displacement for the soil performance allows for the modeling of non-linear behavior at the soil-structure interface, it is not possible to solve the equations that describe that performance in closed form. For a pile modeled using the non-linear behavior, the shear modulus of soil-structure interface sub-grade reaction, *K*, along the length of the pile and at the pile tip are dependent on pile displacement. Therefore, a numerical method, such as the finite difference method, must be used to evaluate the load-displacement behavior.

Using the finite difference methodology, the pile is divided into a series of equidistant nodes along the length of soil-structure interface beginning at the tip (or deepest node) and proceeding to the head of the pile. As derived by Roberts (2006), and summarized herein, the central-difference methodology can be used to develop a set of algebraic equations that can be used to solve for the displacement at each node using the governing Equation 2-3:

$$u_{i+1} = \frac{K}{K_m} u_i (\Delta z^2) + 2u_i - u_{i-1} \tag{2-11}$$

where, u_i is the nodal displacement and Δz is the distance between each node. The subscript *i* refers to the i^{th} node along the soil-structure interface, and the nodes are numbered sequentially from the top of the pile (node *0*) to the head (node *x*). As a result, a total of (*x+1)* nodes exist which enables the development of (*x+1)* equations to define nodal displacements. Equation 2-11 includes two additional unknown displacements which requires two additional known boundary conditions to solve for nodal displacements. At node *0*, the term u_{i-1} becomes u_{-1}, and at node *x,* the term u_{i+1} becomes u_{x+1}. In the case of the pile foundation loaded in compression, the applied load at the head of the pile, *P*, and the tip force, P_t, given by Equation 2-6, are known. Using the central difference methods, the boundary conditions for u_{-1} and u_{x+1} can be written in terms of nodal displacement for the tip force and the applied pile load, respectively:

$$u_{-1} = u_0 \frac{K_t}{K_m}(2\Delta z) + u_1 \qquad (2\text{-}12)$$

$$u_{x+1} = u_{x-1} + \frac{P}{K_m}(2\Delta z) \qquad (2\text{-}13)$$

These algebraic equations can be utilized for piles subjected to either compression or tension loading. For a pile under tension loading, the boundary condition for the tip force, given by Equation 2-12, is simplified by the fact that the value for K_t will be equal to zero. In addition, for a pile subjected to tension loading, the value of the unknown displacement u_{-1} becomes equal to u_1.

With the addition of these two boundary conditions, a total of (*x+3*) equations can be written and arranged in matrix form. When the head of the pile is subjected to an initial load, the algebraic equations can be solved for displacement at each discrete node using standard matrix algebra. The use of the finite difference methodology requires that the nodal displacements be solved at small load increment steps. This allows for the values of shear modulus of soil-structure interface subgrade reaction, K, and the tip soils stiffness, K_t, to be updated at each load increment, and it accounts for the fact that the soil-structure interface will begin to yield from the head of the pile to the tip as the load is incrementally increased.

To incorporate the hyperbolic force displacement relationship into the analyses, Equations 2-4 and 2-5 can be rearranged by substituting $q_o = \pi D \tau_u$ and then by dividing by the displacement at node i, u_i, which yields the following expression:

$$K_i = K_{init} \frac{q_o}{q_o + u_i R_f K_{init}} \tag{2-14}$$

where, K_i is the secant shear modulus of soil-structure interface subgrade reaction at the node of interest corresponding to the node displacement, u_i. At each load increment, the value of K_i can be calculated for the i^{th} node based on the nodal displacement calculated from the previous step load.

A similar equation can be written for the tip soil performance by dividing Equation 2-8 by the tip displacement, u_0, which yields the following expression:

$$K_t = K_{ti} \frac{P_{utip}}{P_{utip} + u_0 R_f K_{ti}} \tag{2-15}$$

where, K_t is the secant stiffness of the tip soil corresponding to the tip displacement, u_0. At each load increment, the value of K_t can be calculated based on the tip displacement calculated from the previous step load. Note that Equation 2-15 will note apply to piles subject to tension loading.

Once the values for K_i and K_t are known, the displacements at each node are calculated. In addition, for piles subject to compression loading, the tip force, P_t, can be calculated using finite difference methods based on tip displacement represented as:

$$P_t = K_m \frac{(u_1 - u_{-1})}{2\Delta z} \tag{2-16}$$

The process continues by incrementally increasing the applied shaft load, updating the load and stiffness vectors, and solving for the new nodal displacements. The process is completed for each node along the length of the pile and at the pile tip. As the load increases, and the displacement at each node increases, the value of K_i will approach zero and the soil-structure interface will fail progressing from the head of the pile to the tip. As this process continues for a pile under compression loading, the tip soil will carry a larger portion of the total load until the soil-structure interface yields at all nodes and the full load is applied to the tip soil. Once the tip force, P_t represented by Equation 2-16, reaches the tip baring capacity, P_{utip} represented by Equation 2-10, the pile will fail by plunging. For a pile subject to tension loading, ultimate failure of the pile occurs once all of the soil-structure interface nodes have failed since no load is carried by the pile tip.

CHAPTER 3

PROJECT SUMMARY

The project from which the research data was collected included the construction of a new coal-fired power plant near Weston, Missouri. Figures 5 shows the site vicinity of the project area and Figure 6 is a topographic map of the area. Construction of the new 850-megawatt generator was completed in 2010 adjacent to an existing coal-fired generator. Throughout the course of construction, more than 7,000 auger cast piles were installed for the support of various structures and equipment. During the design of the power plant, an extensive geotechnical subsurface investigation was conducted and multiple static pile load tests were performed to evaluate the performance of auger cast piles at this site.

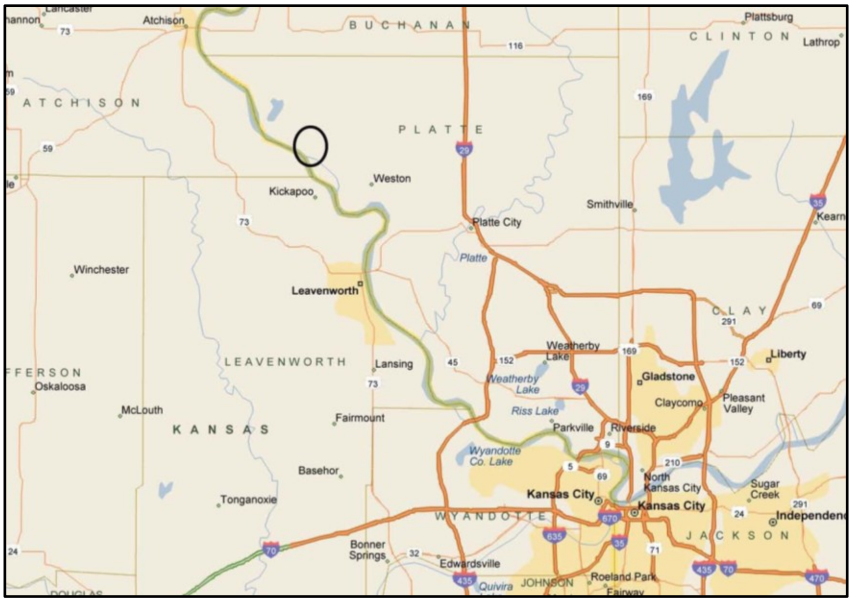


Figure 5: Project vicinity map (Microsoft 2009).

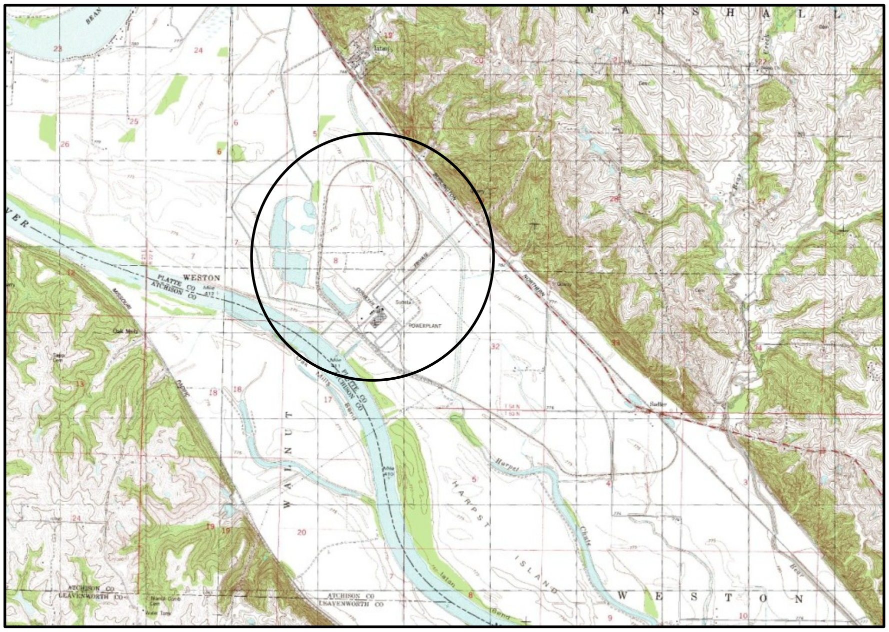


Figure 6: Topographic map of project area (USGS 1984).

Project Geology

The project site is located along the east bank of the Missouri River approximately 4.5 miles northwest of Weston, Missouri. The site is within the unconsolidated alluvial deposits of the Missouri River floodplain and located between the east bank of the river and an upland bluff marking the flood plain boundary.

The uppermost soils within the flood plain are considered to be recent stage Holocene alluvial deposits consisting of fine grained clays, silty clays and clayey silts. The upper Holocene soils are underlain by thick layers of sand and gravel alluvium believed to be of Wisconsinan-age within the Pleistocene Series and which are believed to be of glacial origin. The Missouri River in its current location is considered to be the approximate southern-most limit of continental glaciation. The Wisconsinan alluvium can be more than 50 feet thick in terraces (Hasan, Moberly and Caoile 1988) and borings at the project site indicate that the alluvium extends to depths ranging from 76 to 91 feet below existing grade where bedrock is encountered at elevations between 703 feet and 695 feet above mean sea level. The Geologic Map of Missouri (Middendorf et al. 2003) indicates that the bedrock underlying the alluvium likely consists of Pennsylvanian-age shale, limestone or sandstone.

Field Investigation

The initial geotechnical subsurface field investigation was performed in March and April 2006 and included the completion of 45 soil borings and 27 cone penetration test (CPT) soundings. A second phase of investigations was performed in October and November 2007 and included an additional 22 soil borings. The soil borings and CPT soundings were completed to pre-determined depths or to practical refusal, whichever occurred first. Final boring depths ranged from 10 feet to 91.5 feet below the ground surface. Final CPT sounding depths ranged from 30 to 90 feet below the ground surface.

Borings and CPT soundings were number sequentially from 1 through 94 with borings denoted as B-## and CPT soundings denoted as C-##. The borings and CPT soundings used for the current research were selected based on their general proximity to the test pile locations and are summarized in Table 2. Locations for most of the borings and CPT soundings are shown in Figure 7. However, some locations which were further away from the main project area are not shown. Logs of soil borings and CPT soundings applicable to the current research are included in Appendix A and Appendix B, respectively.

Table 2. – Borings and CPT soundings evaluated.

Test Pile Area	Boring / CPT Sounding	Distance from Test Pile (ft.)	Grade Elev. (ft.)	Total Depth (ft.)
Powerhouse	B-01	150	784.5	87.0
	C-02	75	783.6	85.0
	B-03	150	784.9	102.5
	B-67	150	784.6	50.0
	B-68	90	782.9	50.0
Boiler	C-04	220	785.7	85.1
	B-05	160	785.6	86.0
	B-06	200	786.4	103.5
	C-07	80	785.9	73.3
	B-08	195	787.0	88.0
	C-29	280	786.3	50.0
Chimney	C-10	80	786.5	76.0
	B-11	130	788.2	90.5
	B-12	50	787.2	105.0
	C-13	115	786.9	79.2
	B-14	170	786.1	50.0
Coal Yard	B-58	700	785.7	88.0
	B-60	380	775.7	50.0
	C-61	135	785.7	50.0
	B-71	140	786.7	76.5
Cooling Tower	B-35	560	773.7	50.0
	B-36	320	774.9	76.7
	B-37	95	773.0	50.0
	B-38	255	772.7	50.0
Water Tanks	C-48	60	785.6	30.0
	B-49	195	785.6	20.0
	B-76	140	785.1	50.0
	B-77	90	784.5	85.0
	B-78	135	785.0	50.0

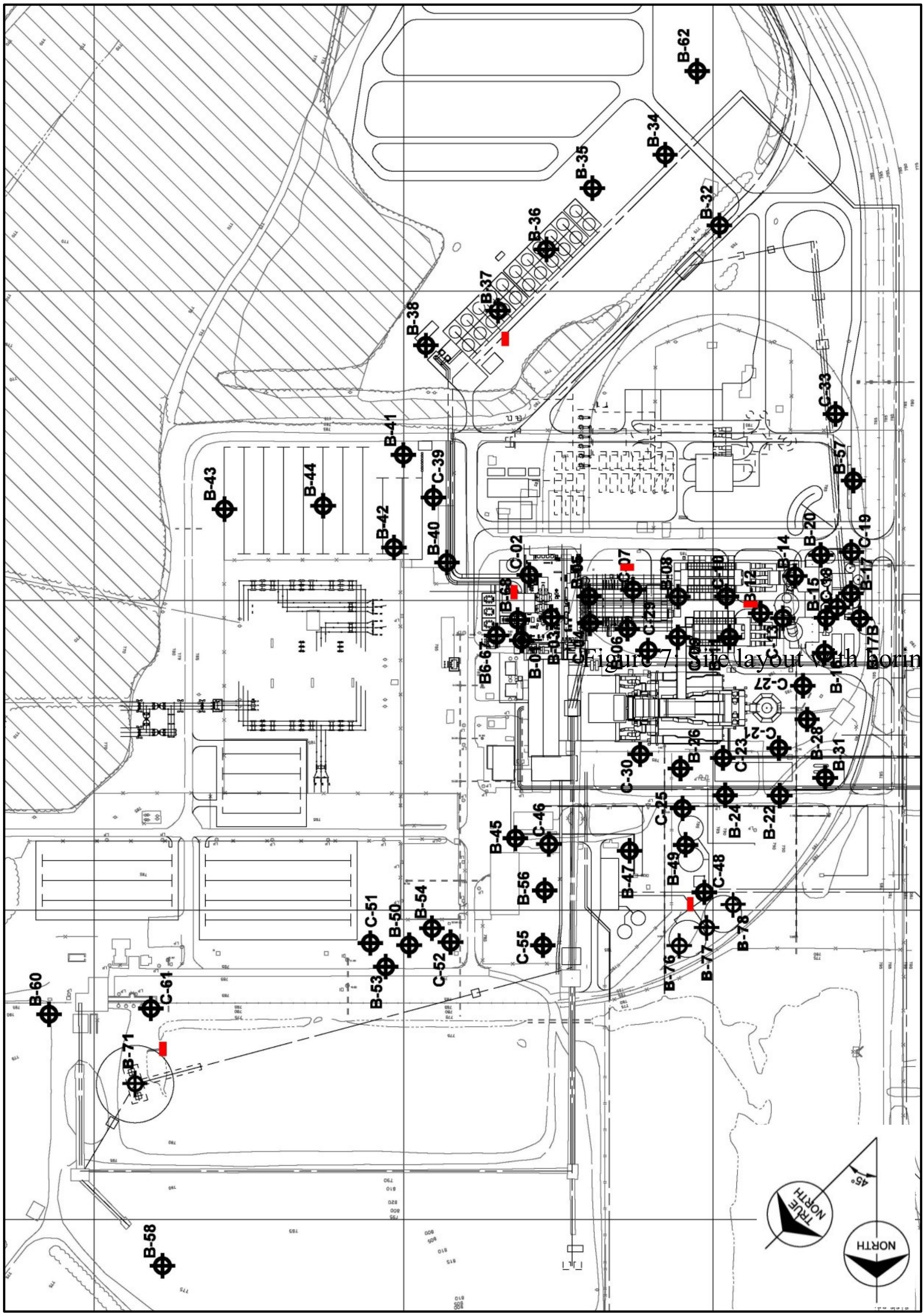


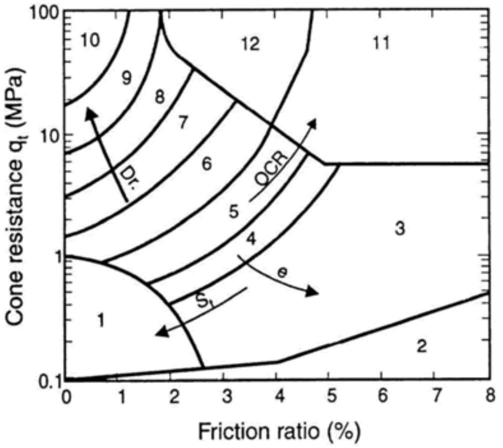
Figure 7: Site layout with boring, CPT sounding, and t

The soil borings in the initial phase of the investigation were completed using truck-mounted Mobile B-57 and CME-55 drill rigs as well as an all-terrain vehicle (ATV) mounted CME-750X drill rig. The soil borings in the second phase included the use of an ATV-mounted Diedrich D-50 drill rig. Drilling methods included a combination of hollow-stem augers and rotary wash drilling. Soil samples were collected at 2.5-foot intervals from the ground surface to a depth of 10 feet and then at 5-foot intervals beyond a depth of 10 feet. Disturbed samples were collected using 1-3/8 inch diameter split-barrel samplers in accordance with ASTM D1586. The drill rigs were all equipped with automatic trip hammers for conducting the standard penetration (SPT) tests. The hammers had calibrated efficiencies of 74, 72, 78, and 69 percent for the Mobile B-57, CME-55, CME-750X, and Diedrich D-50 rigs, respectively. Relatively undisturbed samples were collected using 3-inch diameter thin-walled Shelby tubes in accordance with ASTM D1587. When bedrock was encountered in select borings, NQ2-size (1-7/8 inch inner diameter) rock core was collected.

CPT soundings were advanced using a 20-ton compression type rig equipped with a CPTu system which collects piezometric data in addition to soil strength data. The cone had a tip area of 2.3 square inches and a friction sleeve area of 34.9 square inches. Measurements were collected at 2-inch intervals throughout the full length of each sounding and included tip resistance, q_c, sleeve friction, f_s, dynamic pore water pressure, u, temperature, T, and cone inclination, I. The stratigraphic profile for each sounding was interpreted using the friction ratio, R_f, which is defined as:

$$R_f = 100\left(\frac{f_s}{q_t}\right) \tag{3-1}$$

The soil behavior types (SBT) identified on the CPT sounding logs are based on correlations of R_f with the q_c summarized by Robertson (2010) as shown in Figure 8.



Zone	Soil Behavior Type
1	Sensitive fine grained
2	Organic material
3	Clay
4	Silty Clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silty sand
9	Sand
10	Gravelly sand to sand
11	Very stiff fine grained*
12	Sand to clayey sand*

* *Overconsolidated or cemented*

Figure 8: CPT soil behavior type classification system (Robertson 2010)

The soils encountered in the borings and CPT soundings generally consist of an upper zone of fine to medium sands and silty sands extending to depths of approximately 10 to 15 feet. Using the Unified Soil Classification System (USCS), the sands generally classify as poorly graded sand (SP) and silty sand (SM). Throughout much of the site, a zone of finer grained silts (ML) and clays (CL and CH) are present to a depth of approximately 20 to 25 feet with an average layer thickness of approximately 15 feet. In the area of the cooling tower, which is approximately 10 to 15 feet lower in elevation than the rest of the project site, the silt and clay zone is present beginning at the ground surface. Below the soils transition back to sands which coarsen and have decreasing silt content with depth and are classified as well graded sand (SW) and poorly graded gravel (GP). These granular soils were generally found to be loose to medium dense above a depth of approximately 35 to 40 feet and transitioned to medium dense to dense at greater depths down to the top of bedrock. The bedrock generally consisted of interbedded shale and sandstone with siltstone and limestone encountered in some locations.

Test Pile Program

During the course of the foundation design process, a total of six static compression and six static tension load tests were performed throughout the project site. Tests were performed in the area of the boiler, chimney, powerhouse, coal yard, cooling tower, and water tanks as indicated in Figure 7.

The load tests at the boiler, chimney and powerhouse were performed in August 2006, the load tests at the water tank and coal yard were performed in October and November 2006, and the load test at the cooling tower was performed in July 2007. Each load test was performed on a sacrificial auger cast pile which was not incorporated into the final foundation construction and separate piles were used for the compression and tension load testing at each location. A summary of the test pile configurations, including top elevations, embedment lengths, and tip elevations, is provided in Table 3.

Each of the test piles was installed as a standard 16-inch diameter auger cast pile. Each compression test pile included a full length #10 or #11 center bar. The tension test piles installed at the coal yard, cooling tower, and water tank areas also included full length #10 and #11 center bars. The tension test piles installed at the boiler, chimney, and powerhouse areas included full length #20 center bars. In addition to the center bars, 26-foot long reinforcing cages consisting of 6 - #8 longitudinal bars were installed in the upper portion of each of the test piles.

Table 3. – Test pile configurations.

Pile Location	Pile Type	Grade Elev. (ft.)	Embedment Length (ft.)	Tip Elevation (ft.)
Boiler	Compression	784.6	73.5	711.1
	Tension	784.6	73.0	711.6
Chimney	Both	787.0	75.0	712.0
Powerhouse	Both	782.5	69.0	712.5
Coal Yard	Both	776.9	65.0	711.9
Cooling Tower	Both	781.0	50.0	729.0
Water Tanks	Both	785.0	65.0	720.0

Compression Testing Procedures

Each static compression load test was performed in general accordance with ASTM D1143-81 (1994). Loading generally followed the “Standard Loading Procedure” (Part 5.1) and “Loading in Excess of Standard Test Load” (Part 5.3) with some modifications. Loading was applied in 25-percent increments up to 200 percent of the respective anticipated pile design load, referred to as the standard test load, as described in Part 5.1. The load increments from 25 to 175 percent of the design load were maintained for durations varying from 5 to 20 minutes and the standard test load was maintained for a period ranging from one hour to one and one-half hours. Unloading was then performed in four equal decrements, allowing for 5-minute hold times at each decrement. Each test pile was then reloaded to the standard test load in increments of 50 percent of the design load as recommended in Part 5.3 with each of these increments maintained for 5 to 15 minutes. The applied load was then increased in increments of 10 percent of the design load until the maximum required load was applied (300 percent of the design load) or until failure of the test pile occurred. Each of these increments was maintained for 5 to 15 minutes rather than the 20-minute holds recommended in Part 5.3. The full 300-percent load was held for one hour, unless excess pile head settlement occurred, and then removed in four equal decrements, allowing for 5- to 15-minute hold times at each decrement. The compression test at the powerhouse was terminated following the 10-minute hold at 270 percent of the design load due to excessive pile head settlement.

The compression load test at the coal yard was performed generally as described previously with modifications. The test pile was initially loaded in two small loading/unloading sequences. The first loading/unloading sequence was applied in three increments; 25, 50, and 75 percent of the anticipated pile design load of 125 tons, allowing 10 minutes between load increments, and then unloaded in one decrement. The test pile was then reloaded in two increments; 75 and 100 percent of the proposed design load, allowing 5 and 10 minutes between load increments respectively, then unloaded in one decrement. The test pile was then re-loaded in six increments; 50, 100, 125, 150, 175, and 200 percent of the proposed design load. The duration for the 50 to 175 percent load increments was 5 to 15 minutes. The standard test load of 250 tons (200 percent of the design load) was removed after an approximate one and one-half hour hold time. The unloading sequence was carried out in four decrements; 150, 100, 50, and 0 percent of the working load in general accordance with ASTM D1143, Part 5.1. The test pile was then reloaded to the standard test load in increments of 50 percent of the pile design load, allowing 5 to 10 minutes between load increments. The applied load was then increased in increments of 10 percent of the design load until the maximum required load of 375 tons (300 percent of the design load) had been applied, allowing 10 minutes between load increments. The full 300 percent load was held for 1 hour and then removed in four equal decrements, allowing 5 minutes between decrements.

Tension Testing Procedures

Each tension test load test was performed in general accordance with ASTM D3689-90 (1995). Loading generally followed the "Standard Loading Procedure" (Part 7.2) and "Loading in Excess of 200% of Pile Design Uplift Load" (Part 7.4) with some modifications. Loading was applied in 25-percent increments up to 200 percent of the respective pile design load, referred to as the standard test load, as recommended in Part 7.2. The load increments from 25 to 175 percent of the design load were maintained for durations varying from 5 to 15 minutes and the standard test loads were maintained for periods ranging from one hour to one and one-quarter hours. Unloading was then performed in four equal decrements. Each test pile was then reloaded to the standard test load in increments of 50 percent of the pile design load as recommended in Part 7.4 with each of these increments maintained for 5 to 10 minutes. The applied load was then increased in increments of 10 percent of the design load until the maximum required load was applied (300 percent of the design compression load) or until failure of the test pile occurred. Each of these increments was maintained for 10 to 15 minutes rather than the 20-minute holds recommended in Part 7.4. The full 300-percent load was held for one hour and then removed in four equal decrements, allowing for 5- to 15-minute hold times at each decrement.

Test Pile Instrumentation

Test pile head settlements were measured at each test pile using four independently supported dial gauges, similar to those shown in Figure 9, which were accurate to the nearest 0.001 inch. The dial gauges were mounted at each of the four quadrants of the test pile to allow for detection of eccentric loading.

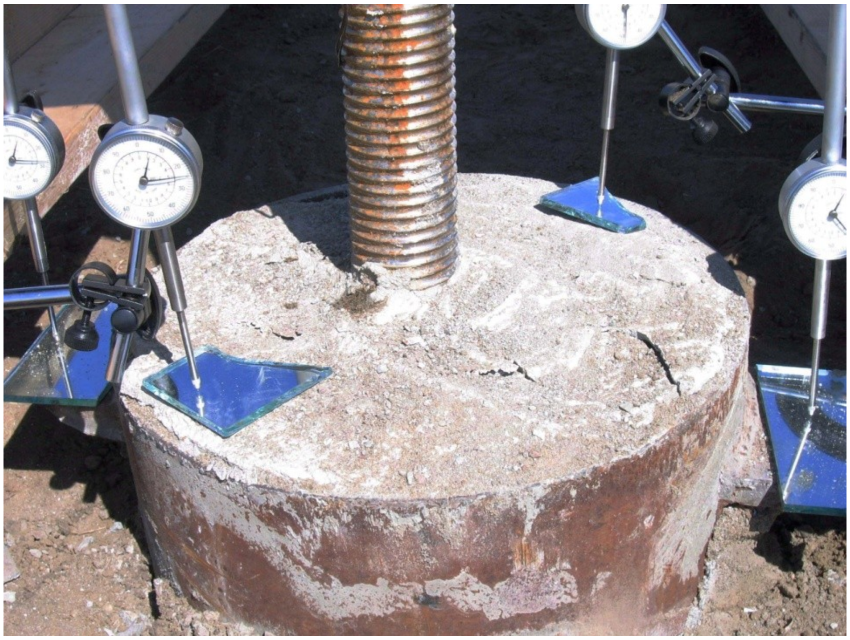


Figure 9: Dial gauges for monitoring pile top movement.

Each of the six compression test piles and three of the tension piles, including those at the coal yard, cooling tower, and water tank, were instrumented with multiple strain gauges. The Geokon Model 4911 “Sister Bar” strain gauges, similar to those shown in Figure 10, were attached to the steel reinforcing center bars at multiple depths throughout each pile. The strain gauges provided a means of monitoring the rate of load transfer in the pile during the load testing. Strain measurements from the sister-bar strain gauges were recorded using a Geokon GK-403 readout box.

Figure 10: Geokon Model 4911 “Sister Bar” strain gauges.

CHAPTER 4

DATA ANALYSIS

As discussed in Chapter 2, the use of the hyperbolic load-displacement relationship to evaluate the non-linear behavior at the soil-structure interface requires the use of a numerical method to solve the governing algebraic equations. A computer program was developed by Roberts (2006) using Mathcad (2002) which utilizes the finite difference methodology to evaluate the hyperbolic soil model. For this research, the program was utilized to back-calculate values for the ultimate shear strength of soil-structure interface, τ_u, and the initial tangent shear modulus of the subgrade reaction at the soil-structure interface, K_{init}. The values of τ_u and K_{init} were adjusted until the theoretical load-settlement curve provided a close approximation of the load-settlement curve developed from each pile load test. The back-calculated values of τ_u and K_{init} were then compared with the SPT N_{60} values collected from nearby soil borings, as well as the q_c values from nearby CPT soundings, to look for trends that would indicate correlations between the field testing data and the soil strengths exhibited by the load testing.

While the soil profile observed in the borings and CPT soundings included some layering of silts and clays, the profile is generally dominated by a mixture of silt, sand, and gravel that generally coarsens with depth. The data provided by the embedded strain gauges generally does not appear to indicate any consistent layering. For model simplicity, this research included the modeling of a single, homogenous layer of soil along the length of the piles and the values of τ_u and K_{init} were each treated as the average value over the full length of pile. Similarly, the SPT N_{60} values and the CPT q_c values from nearby explorations were averaged over the corresponding length of pile penetration for the purposes of evaluating correlations. A summary of the average N_{60} and q_c used for the development of correlations with “t-z” model parameters is included in Table 4 and Table 5.

Table 4 – Summary of field investigation N_{60} values evaluated.

Test Pile Area	Boring	N_{60} Range (bpf)	N_{60} Average (bpf)	N_{60} Average (bpf)	Exploration Tip Elevation (ft.)
Powerhouse	B-01	3 – 49	24	25	697.5
	B-03	7 – 46	26		682.4
	B-67	0 – 43	17		734.6*
	B-68	4 – 44	18		732.9*
Boiler	B-05	4 – 61	26	30	699.6
	B-06	10 – 83	36		682.9
	B-08	4 – 75	28		699.0
Chimney	B-11	5 – 63	28	30	697.7
	B-12	6 – 89	32		682.2
	B-14	4 – 47	23		736.1*
Coal Yard	B-58	5 – 38	18	19	687.7
	B-60	5 – 38	20		736.7*
	B-71	3 – 30	21		709.2
Cooling Tower	B-35	5 – 35	16	19	723.7
	B-36	3 – 47	20		698.2
	B-37	4 – 47	19		723.0
	B-38	4 – 27	19		722.7
Water Tanks	B-49	4 – 35	23	17	765.6*
	B-76	5 – 56	26		735.1*
	B-77	7 – 33	17		699.5
	B-78	2 - 41	20		735.0*

* Boring did not extend to the full depth of the associated test pile.

Table 5 – Summary of field investigation q_c values evaluated.

Test Pile Area	Boring	q_c Range (tsf)	q_c Average (tsf)	q_c Average (tsf)	Exploration Tip Elevation (ft.)
Powerhouse	C-02	3 – 713	167	172	698.7
Boiler	C-04	9 – 527	212	167	700.6
	C-07	6 – 704	148		712.6*
	C-29	6 – 382	155		736.3*
Chimney	C-10	5 – 504	186	167	710.6
	C-13	5 – 409	148		707.7
Coal Yard	C-61	6 – 489	126	126	735.7*
Cooling Tower	No CPT soundings were completed in the near vicinity				
Water Tanks	C-48	5 – 319	126	126	755.5*

* Boring did not extend to the full depth of the associated test pile.

Model Parameters

While the values of τ_u and K_{init} were treated as variables for the load-settlement curve fitting process, the remaining parameters within the model were treated as constants. This includes the values for the non-interaction zones at the top and bottom of the piles, the axial stiffness of the pile, and the tip soil performance including the elastic modulus, ultimate bearing capacity, and Poisson's ratio.

Non-Interaction Zone

For this analysis, a non-interaction zone of 1 foot was included only at the top of the piles for the purpose of curve fitting with the static load test data as described later. The near-surface soils at the site predominantly consisted of cohesionless sands and silts.

While some construction disturbance can be expected in the near surface soils, the use of the auger to maintain a stable hole during the grouting process is expected to limit the amount of disturbance. In addition, seasonal conditions such as frost action and moisture variations would not impact the performance of piles during the relatively short duration between pile installation and performance of the static load testing.

While it has been common practice to also include a non-interaction zone of one diameter at the bottom of drilled shaft and auger cast piles which bear in cohesive soils, the piles considered for this research were terminated in cohesionless sands and gravels. Therefore, a non-interaction zone was not included in the load-settlement model.

Axial Pile Stiffness

The axial stiffness of the piles was calculated based on the 7-day grout strength tests performed on typical 2-inch grout cubes collected during the installation of the test piles. For the coal yard area, the load testing was performed on the 23rd day following test pile installation so the grout strength was estimated using 7-day and 28-day grout cube breaks. A similar approach was used for estimating the grout strength for the water tank area where testing was performed on the 19th day following test pile installation. The compressive strength of the grout was used to calculate the grout modulus of elasticity, E_g. A composite section was evaluated to account for the presence of the steel reinforcement in the test piles. The stiffness of the composite pile section, K_m, was calculated as follows:

$$K_m = \frac{(A_g\, E_g) + (A_s\, E_s)}{A_m} \qquad (4\text{-}1)$$

where, A_g is the area of grout, A_s is the area of steel reinforcement, E_s is the steel modulus of elasticity, and A_m is the total pile area. Due to variations in the quantity of steel reinforcement relative to depth, a weighted average for K_m was used in the analyses.

There is some potential for the grout strengths indicated by laboratory testing to differ from the actual grout strength within the test piles during pile testing. These variations can be attributed to different curing conditions for the test cubes relative to the grout placed within the pile or to time lapses between the date of grout testing and the date of pile testing. Due to this potential for variation of the grout strength, a sensitivity analysis was performed using the data from the Boiler compression test to evaluate the impact on the back-calculated values of τ_u and K_{init}. The data from the Boiler compression test was selected for the sensitivity analysis because it was the first set of data to be used for the load-settlement curve fitting process. As the curve-fitting analyses progressed using the data from other load tests, it was noted that the Boiler compression was one of the data sets that resulted in a close fit with the curves predicted by the "t-z" model. However, for the sensitivity analyses, the quality of fit between the load test data and the "t-z" model was not considered to be as important as the magnitude of variation observed in the predicted load-settlement curves relative to the magnitude of variation to the grout strength. A similar approach was for evaluating the sensitivity of the model to other input variables as discussed later.

With all other model parameters kept as constants, the grout strength was varied from the minimum design grout compressive strength of 5000 pounds per square inch (psi) to the maximum observed laboratory compressive strength of 7020 psi. As exhibited in

Figure 11, the impact of the variation of grout strength has a minimal impact on the predicted load-settlement performance. The analyses indicate that over the range of anticipated grout compressive strengths, the back-calculated value of τ_u varies from 21.7 to 22.5 psi. This is a variation of approximately ±2 percent relative to the value of 22.0 psi back-calculated using the laboratory strength test data for the grout in the compression test pile for the Boiler area. The value of K_{init} was able to be kept constant at 3 psi and maintain a good fit to the load settlement performance of the test pile throughout the range of grout strengths. The potential for variation of the grout compressive strengths, and the affect it has on the axial pile stiffness, is expected to have a negligible impact on the derivation of the soil interaction strength parameters.

Tip Soil Elastic Modulus

The tip soil elastic modulus, E_s, was estimated using correlations with SPT N-values for gravelly sands as found in Bowles (1997). The correlations provided for gravelly sands include:

$$E_s = 1200(N_{60} + 6) \qquad (4\text{-}2)$$

$$E_s = 600(N_{60} + 6) \qquad \text{for } N_{60} \leq 15 \qquad (4\text{-}3)$$

$$E_s = 600(N_{60} + 6) + 2000 \qquad \text{for } N_{60} > 15 \qquad (4\text{-}4)$$

The correlations do not indicate the conditions for which Equation 4-2 should be applied. As the N_{60} for the soils near the tips of the test piles exhibited N-values consistently greater than 15, the results of Equation 4-4 were compared with the results of

Equation 4-2. The resulting value for E_s ranged approximately from 350 to 1300 kips per square foot (ksf) with an average value of approximately 750 ksf.

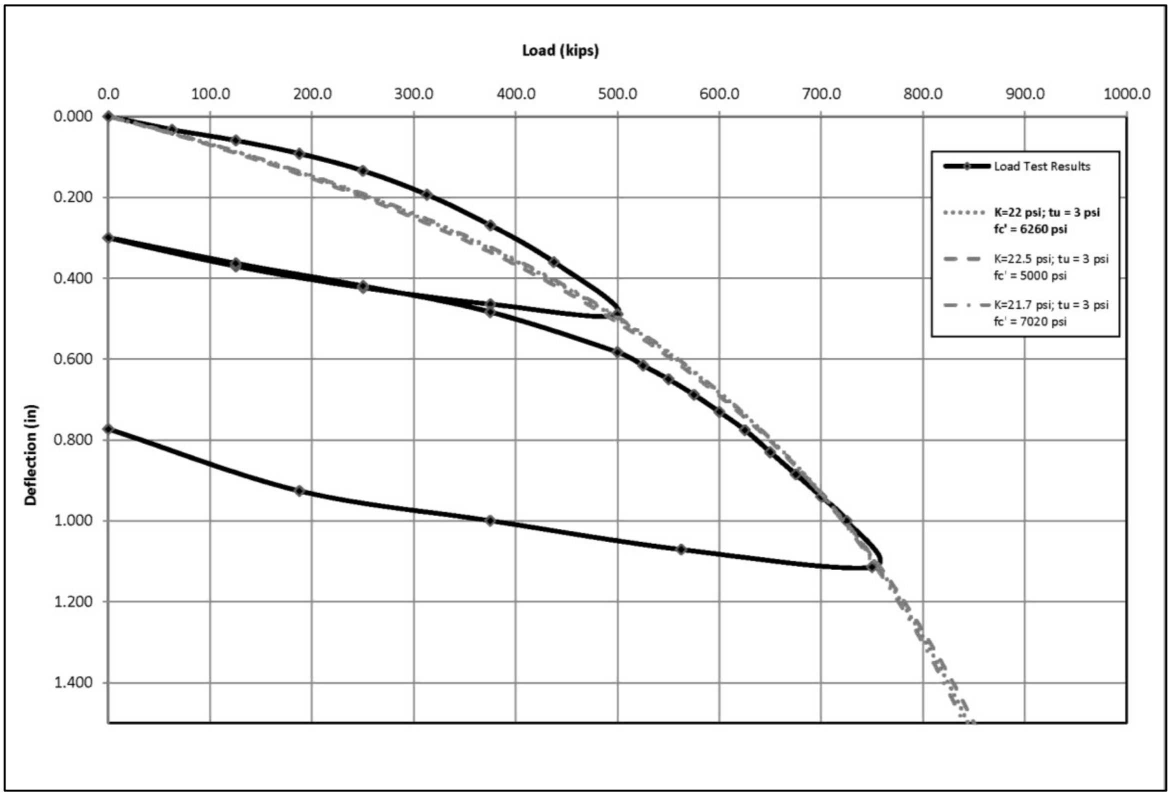


Figure 11: Applied load vs. settlement for grout strength sensitivity analysis using Boiler compression test pile results.

To account for the potential variation of the value of E_s, a sensitivity analysis was performed using the data from the Boiler compression test to evaluate the impact on the back-calculated values of τ_u and K_{init}. With all other parameters kept as constants, the load settlement performance was evaluated at a minimum value 350 ksf and a maximum value of 1300 ksf for E_s. As exhibited in Figure 12, the impact of the variation of tip soil elastic modulus has a minimal impact on the predicted load-settlement performance. The analyses indicate that at the maximum and minimum modeled E_s values, the back-calculated value of τ_u varies from 20.8 to 23.2 psi, respectively. This is a variation of approximately ±5 percent relative to the value of 22.0 psi back-calculated using the average value of 750 ksf. The value of K_{init} was kept constant at 3 psi for this sensitivity analysis. The potential for variation of the tip soil elastic modulus is expected to have a negligible impact on the derivation of the soil interaction strength parameters.

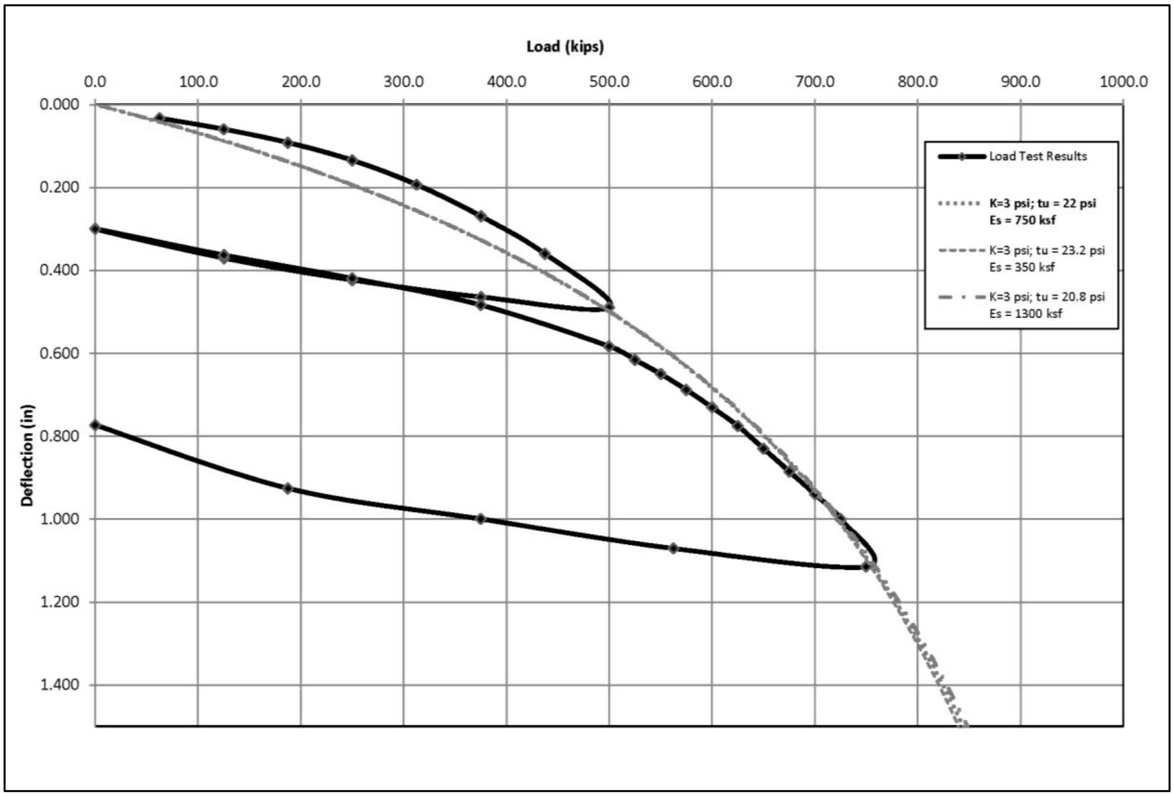


Figure 12: Applied load vs. settlement for tip soil elastic modulus sensitivity analysis using Boiler compression test pile results.

Tip Soil Ultimate Bearing Capacity

The tip soil ultimate bearing capacity, q_t, was estimated using correlations with SPT N-values and with the q_c value from the CPT soundings. Equation 1-10 was used to estimate q_t from the N-values while Equation 1-13 was used to estimate q_t from the CPT soundings. For each correlation, the N-values and the q_t values were averaged over a distance of one diameter above and three diameters below the tips of each pile. Using Equation 1-10, the predicted value of q_t ranges from 20 ksf to 58 ksf with an average value of 38 ksf. Using Equation 1-13, with a k_c value of 0.3 for very dense sands and gravels as shown in Table 1, the predicted value of q_t ranges from 162 ksf to 278 ksf with an average value of 230 ksf.

Data collected from the strain gauges embedded at the tips of the test piles indicate that the bearing pressures developed at the maximum test loads ranged from 29 ksf to 132 ksf with an average value of 85 ksf. At the maximum test loads, the pile top settlements ranged from 0.9 to 1.5 inches. Accounting for elastic shortening of the piles, tip movements were estimated to be on the order of 0.3 to 1.0 inch. Brown et al. (2007) notes that the end bearing component is fully developed at tip displacements on the order of 5 to 10 percent of the pile diameter. For the 16-inch piles used in this research, a pile tip displacement on the order of 0.8 to 1.6 inches would be required for full development of the end bearing capacity. Therefore, the ultimate bearing capacities for the test piles are likely in the higher range of values predicted by the LPC method using the CPT sounding data. The average value of 230 ksf predicted by the LPC method was selected as the basis for analysis for the curve fitting calculations.

Due to the large variation between the predicted values for the ultimate tip bearing relative to the two correlation methods, an analysis was performed using the data from the Boiler compression test to evaluate the sensitivity of the back-calculated values of τ_u and K_{init} relative to variations in the value of q_t. With all other parameters kept as constants, the load-settlement performance was evaluated at a range of value for q_t predicted by the correlations. As exhibited in Figure 13, the impact of the variation of tip soil ultimate bearing capacity has a moderate impact on the predicted load-settlement performance. The analyses indicate that at the maximum and minimum modeled q_t values, the back-calculated value of τ_u varies from 21.8 to 23.8 psi, respectively. This is a variation of approximately minus 1 percent to plus 8 percent relative to the value of 22.0 psi back-calculated using the average value of 230 ksf. The value of K_{init} was kept constant at 3 psi for this sensitivity analysis. While the magnitude of potential variability is higher than for some of the other model parameters evaluated, the strain gauge data provides some justification for using the higher predicted value from the LPC correlations.

Tip Soil Poisson’s Ratio

While correlations to estimate the Poisson’s ratio, μ_s, of soils relative to conventional in-situ testing such as SPT or CPT, there are published typical values based on soil type. For dense sand or gravel, a range of 0.4 to 0.5 is recommended by Arya, O’Neill, and Pincus (1979). A value of 0.3 is recommended for gravel, unless evidence indicates otherwise, by Reese, Isenhower, and Wang (2006). In addition, it is noted that the Poisson's ratio increases to a value of 0.48 to 0.49 in 100 percent saturated soils (U.S. Army Corps of Engineers 1995).

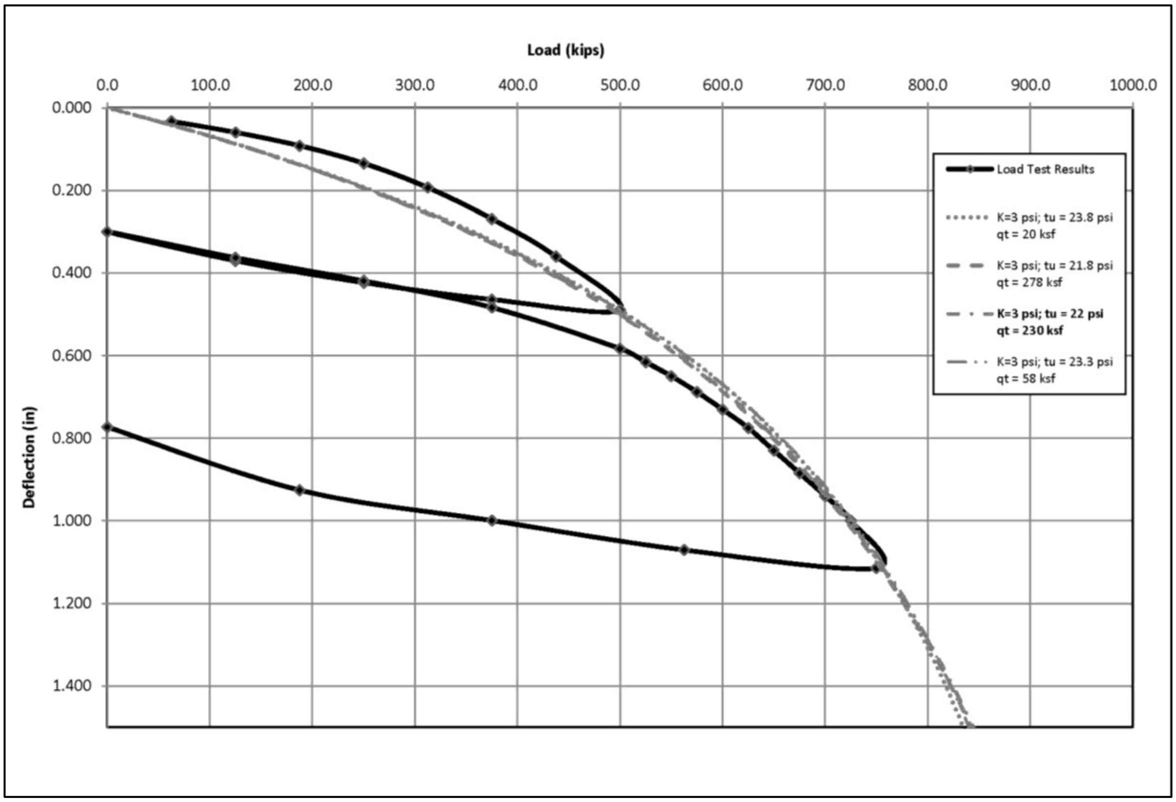


Figure 13: Applied load vs. settlement for tip soil ultimate bearing capacity sensitivity analysis using Boiler compression test pile results.

Similar to the parameters noted above, analyses were performed using the data from the Boiler compression test to evaluate the sensitivity of the back-calculated values of τ_u and K_{init} to variations in the value of Poisson's ratio for the tip soil incorporated into the model. As exhibited in Figure 14, the impact of variations to μ_s for the tips soils has a negligible impact on the predicted load-settlement performance. The analyses indicate that, over the range of potential values suitable for dense sands and gravels, the theoretical load-settlement curves predicted by the model are nearly identical without varying the values of τ_u and K_{init}. With the tip soils being below the groundwater level and thus saturated, a value of 0.49 was used for μ_s in all analyses.

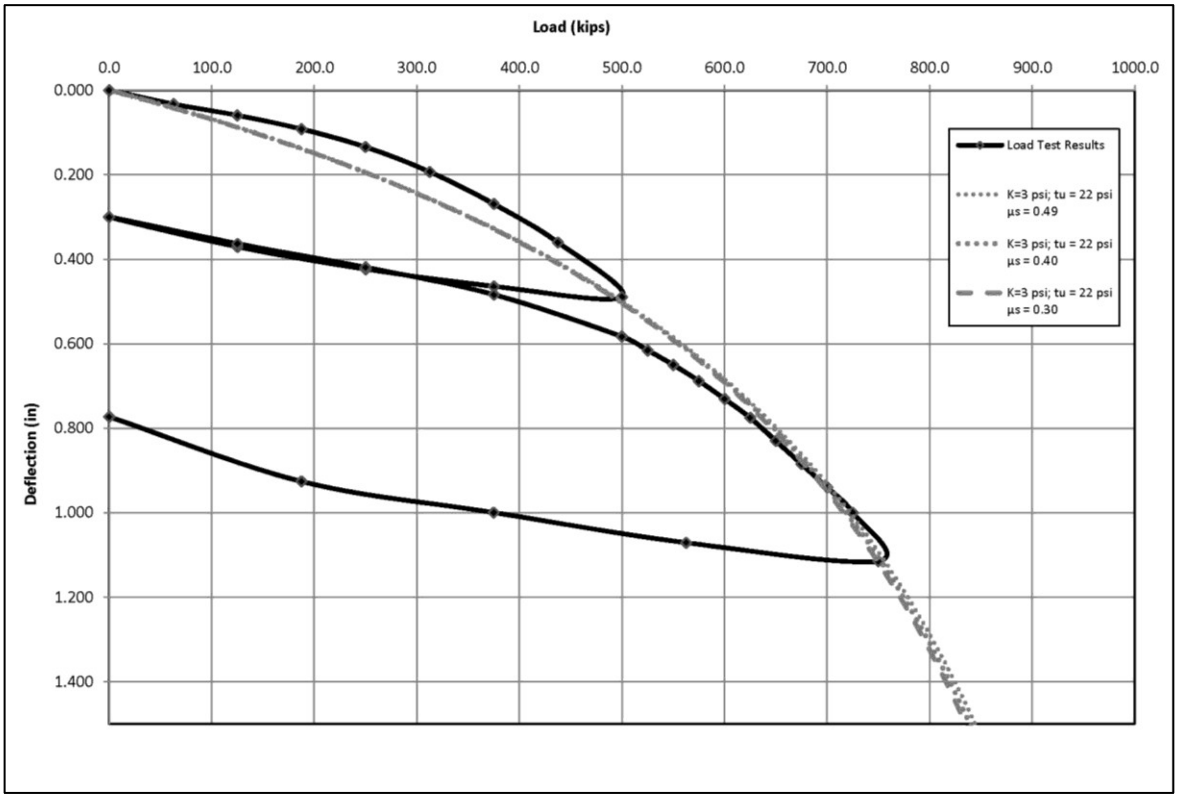


Figure 14: Applied load vs. settlement for tip soil Poisson's ratio sensitivity analysis using Boiler compression test pile results.

Curve Fitting Results

For each of the six static compression and six static tension load tests performed at the project site, the Mathcad model was used to back-calculate average values for the ultimate shear strength of soil-structure interface, τ_u, and the initial tangent shear modulus of the subgrade reaction at the soil-structure interface, K_{init}. The values of τ_u and K_{init} were adjusted until the theoretical load-settlement curve resembled a close approximation to the load-settlement curve produced by the static load testing. While an attempt was made to match the full range of load-settlement data from the static load tests, the data points representing 200 percent of the design load and 300 percent of the design load, or the maximum test load applied to the pile, were considered the key points to match. Since these load increments were maintained the longest during the testing, typically for periods of time ranging from one hour to one and one-half hours, they were considered to be the most representative points to use for the curve fitting. The intermediate load increments were maintained for periods ranging from 5 to 20 minutes.

For the load tests that were instrumented with strain gauge data, a secondary comparison was made between the distributions of load versus depth predicted by the numerical model and with the actual distribution exhibited by the strain gauge data. However, the curve fitting with the load-settlement data was the primary evaluation used to develop the τ_u and K_{init} values.

The results of the load-settlement curve fitting and the load-depth evaluation are grouped by test pile area and exhibited in Figures 15 through 34. For each plot of the load-settlement performance, a series of curves are provided to highlight the sensitivity of the curves to variations of the τ_u and K_{init} values. A summary of the final back-calculated τ_u and K_{init} values is presented in Table 6. In general, the load-settlement curves developed by the “t-z” model present a good match to the curves from the static load testing. However, there are some areas where the curves noticeably deviate from one another over a wide range of applied loads. Examples of these deviations are most apparent in the curves from the Powerhouse tension test and the Coal Yard compression test. Such deviations are considered to be a likely result of the assumption of a single layer soil profile.

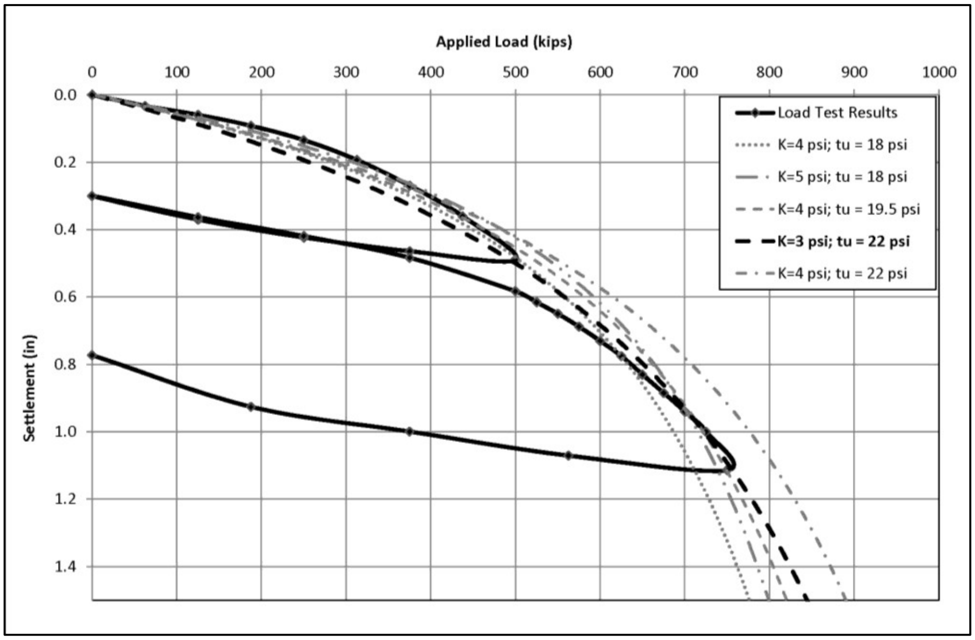


Figure 15: Applied load vs. settlement Boiler compression test pile.

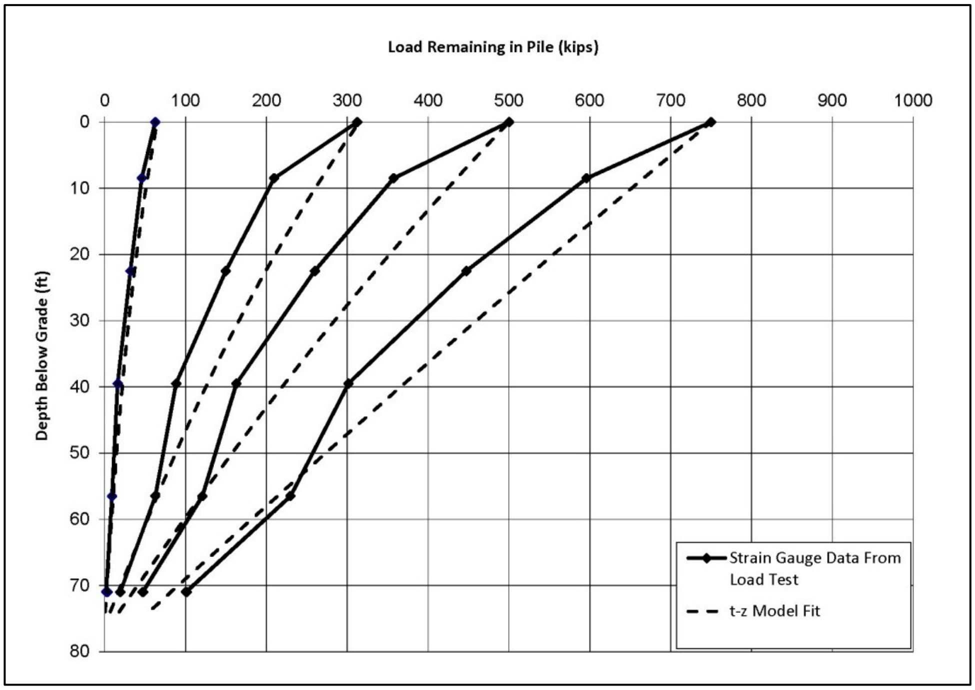


Figure 16: Load remaining in pile vs. depth for Boiler compression test pile.

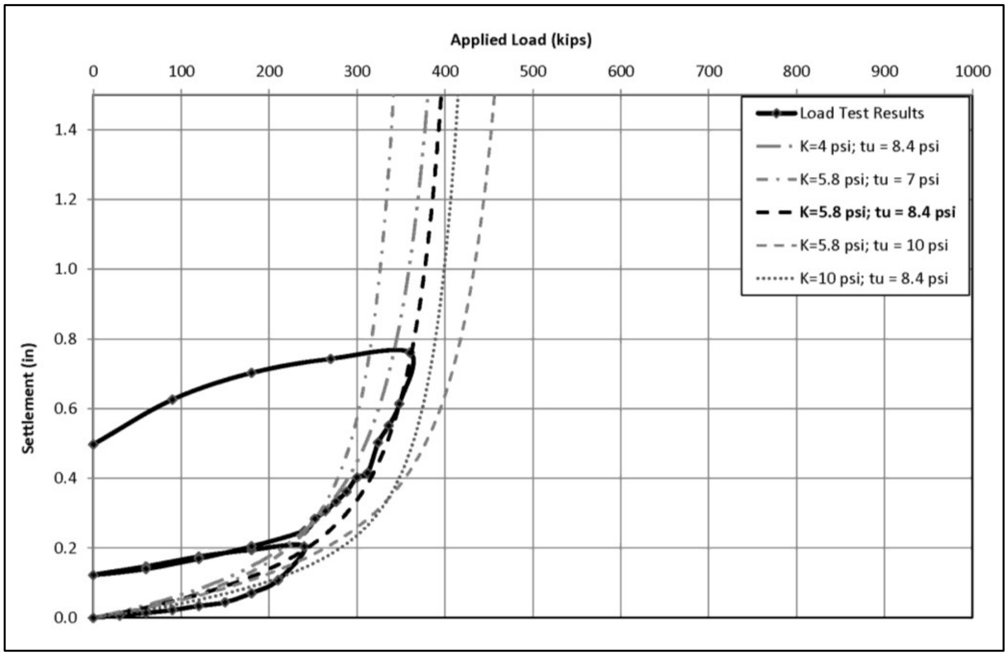


Figure 17: Applied load vs. settlement Boiler tension test pile.

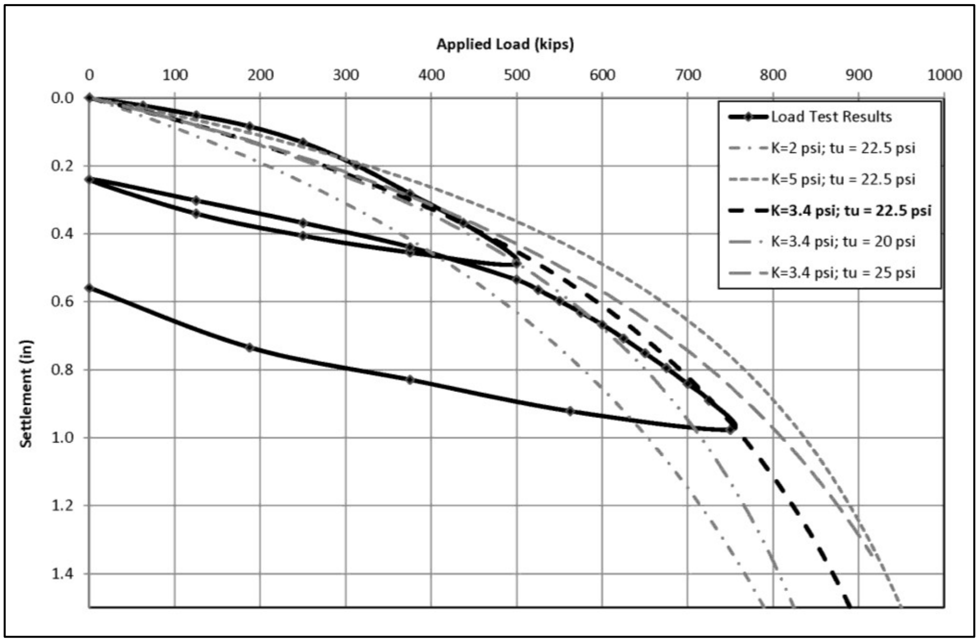


Figure 18: Applied load vs. settlement Chimney compression test pile.

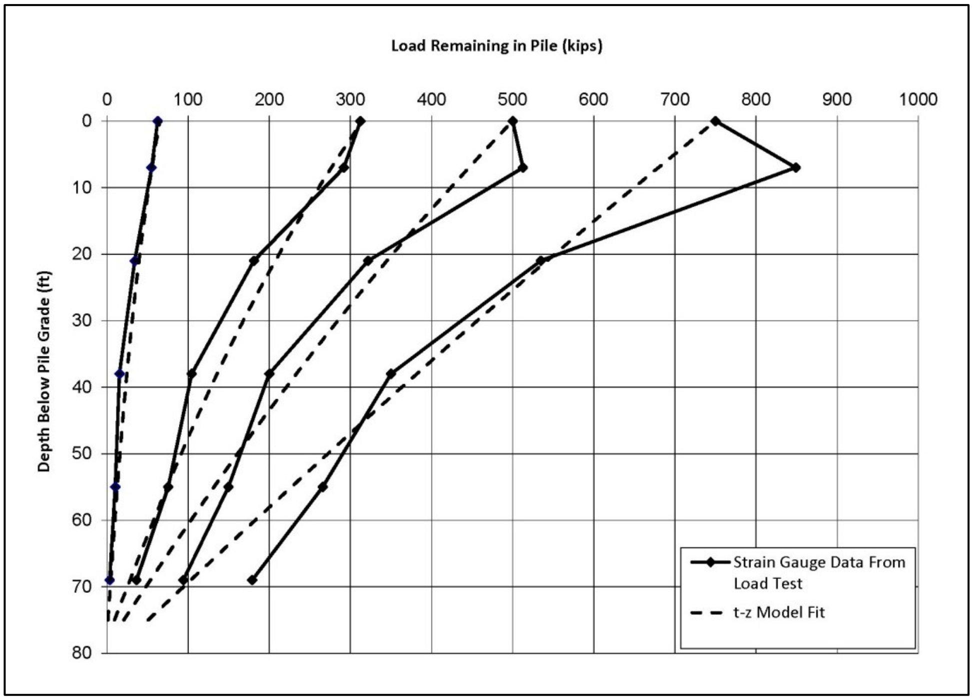


Figure 19: Load remaining in pile vs. depth for Chimney compression test pile.

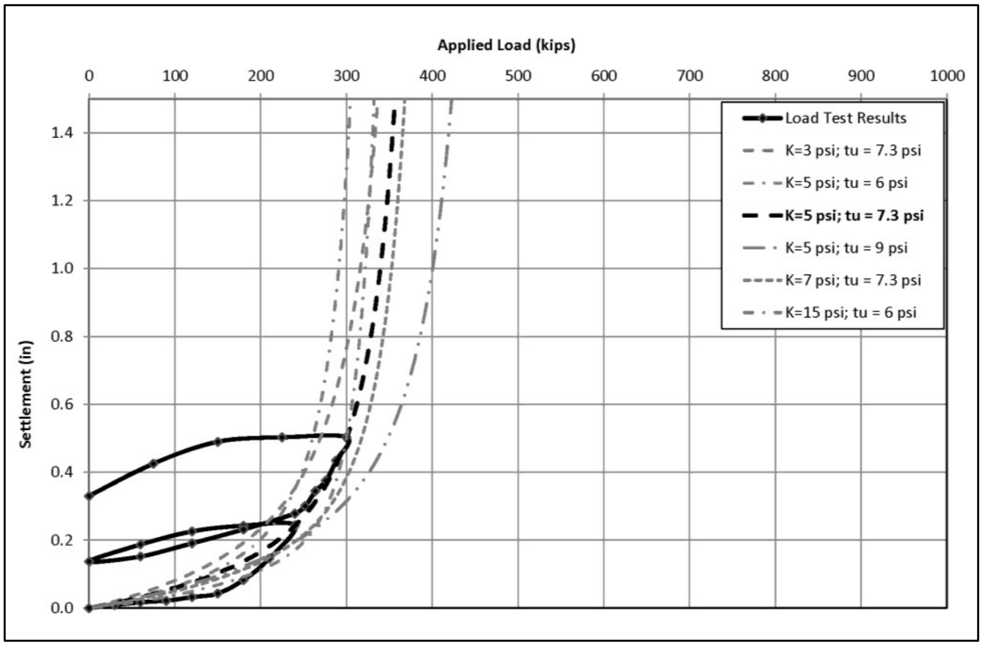


Figure 20: Applied load vs. settlement Chimney tension test pile.

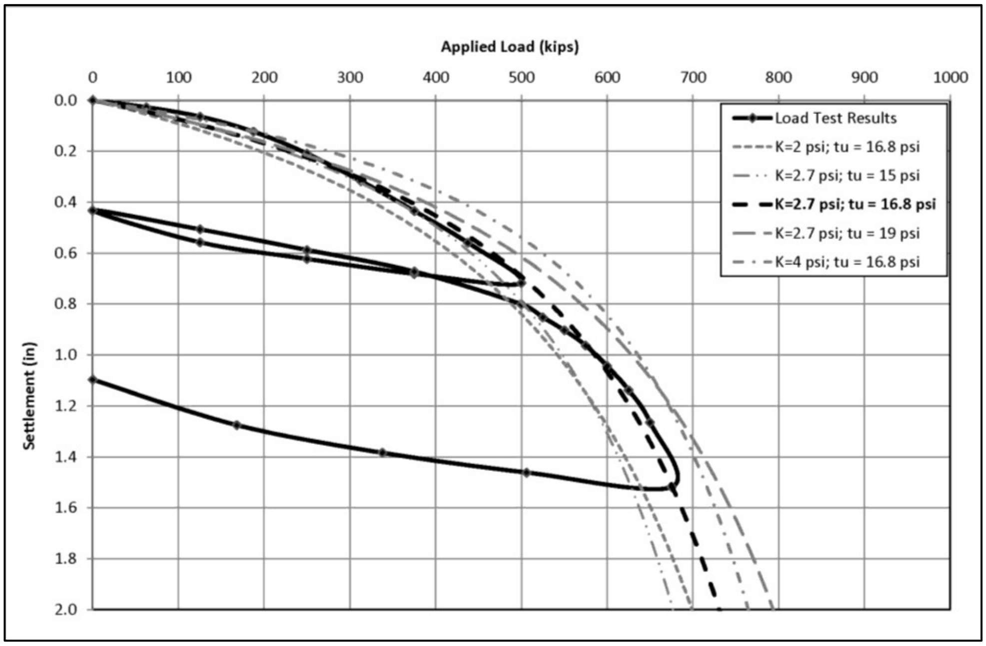


Figure 21: Applied load vs. settlement Powerhouse compression test pile.

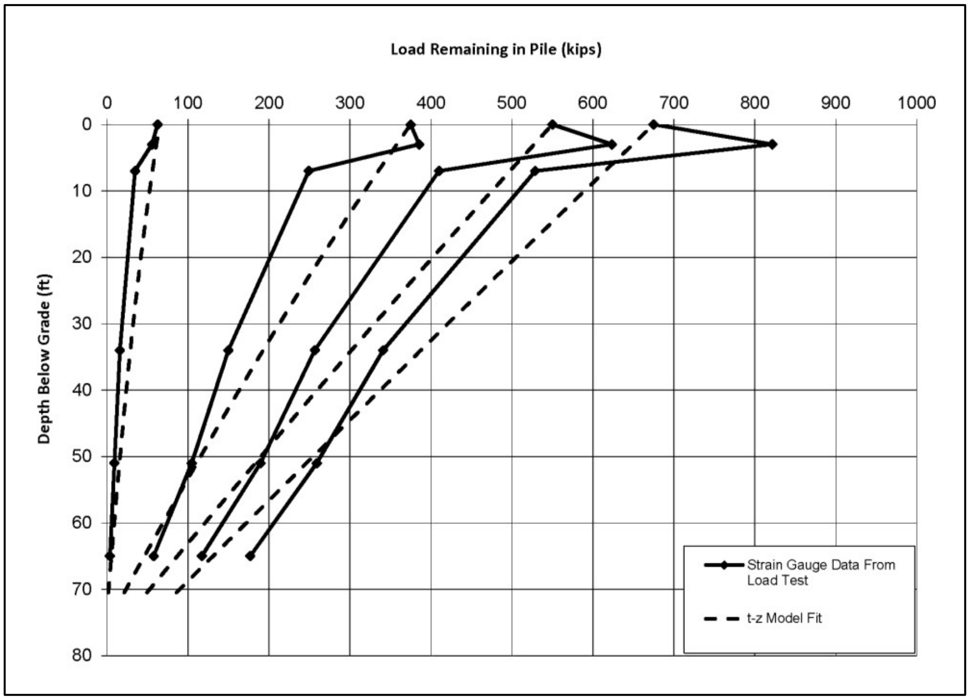


Figure 22: Load remaining in pile vs. depth for Powerhouse compression test pile.

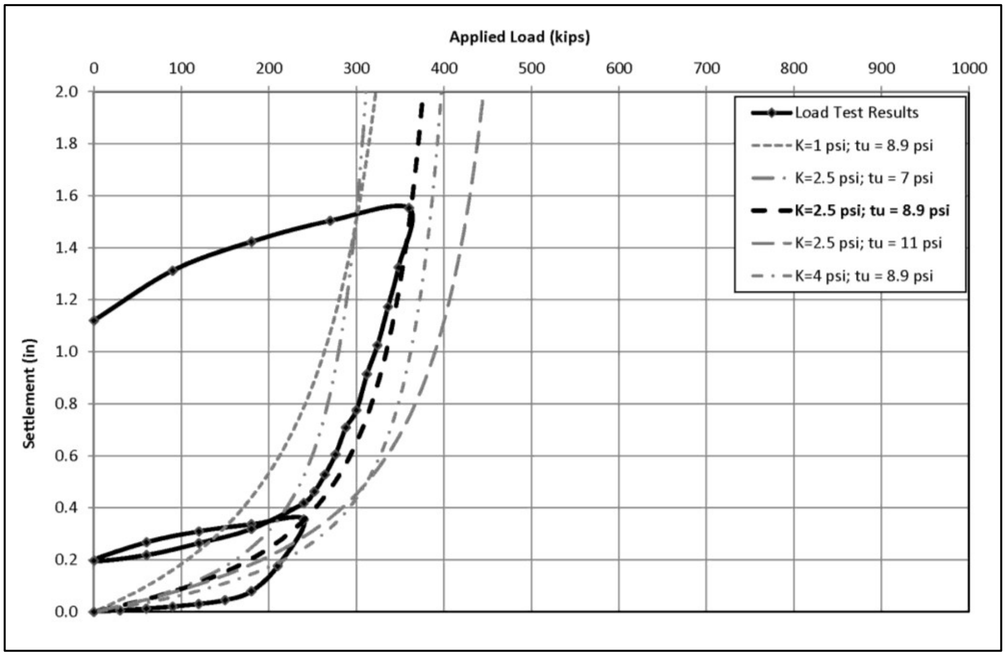


Figure 23: Applied load vs. settlement Powerhouse tension test pile.

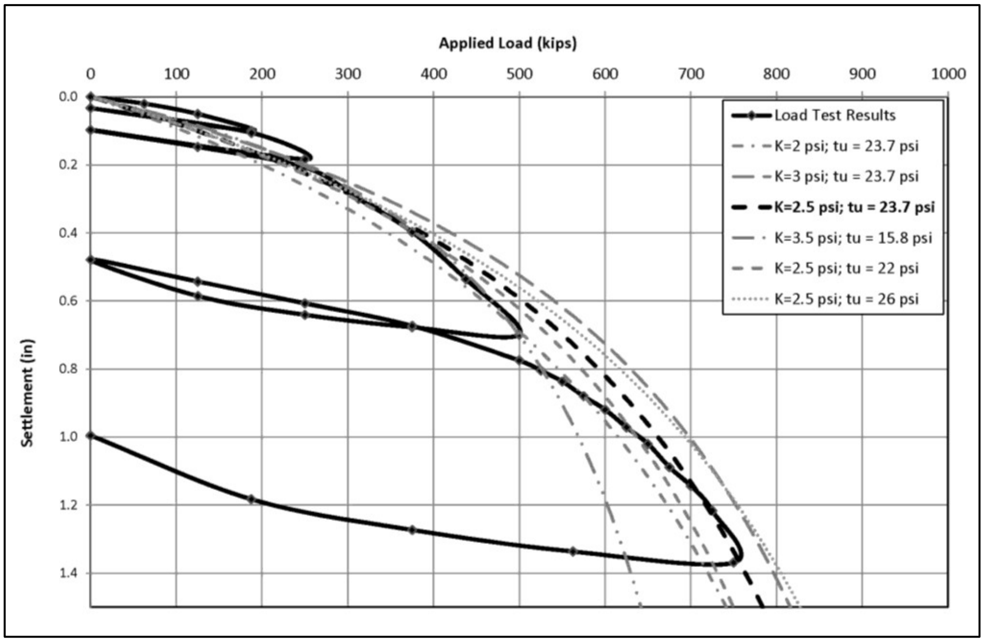


Figure 24: Applied load vs. settlement Coal Yard compression test pile.

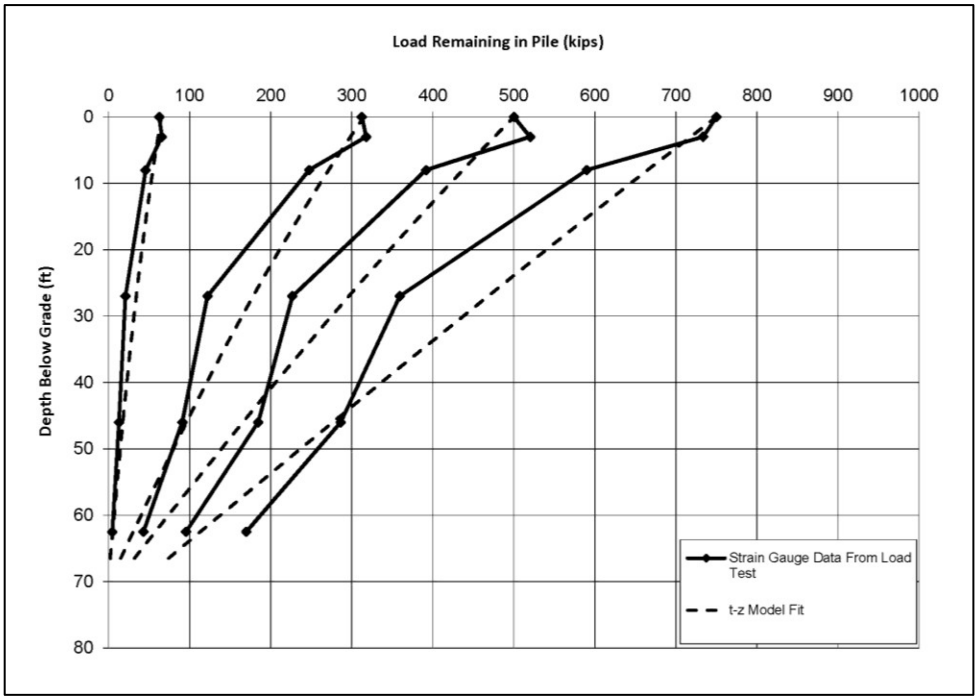


Figure 25: Load remaining in pile vs. depth for Coal Yard compression test pile.

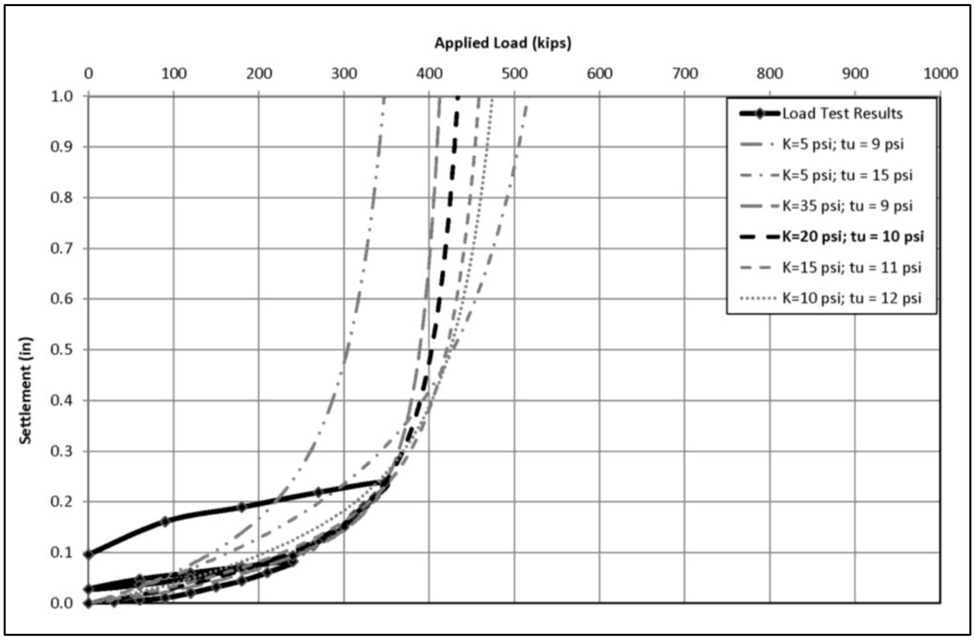


Figure 26: Applied load vs. settlement Coal Yard tension test pile.

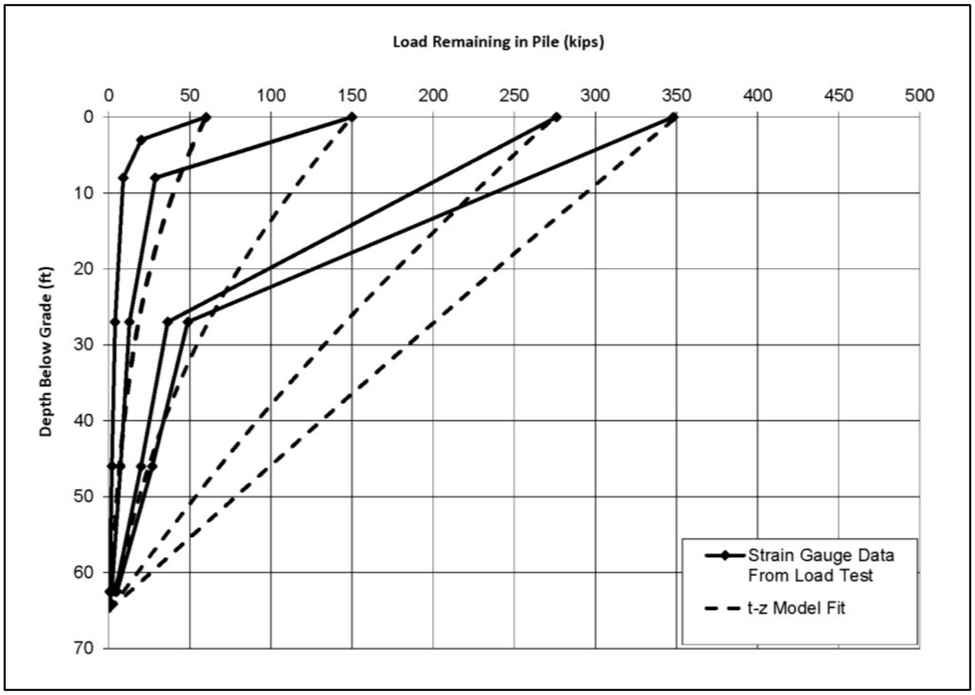


Figure 27: Load remaining in pile vs. depth for Coal Yard tension test pile.

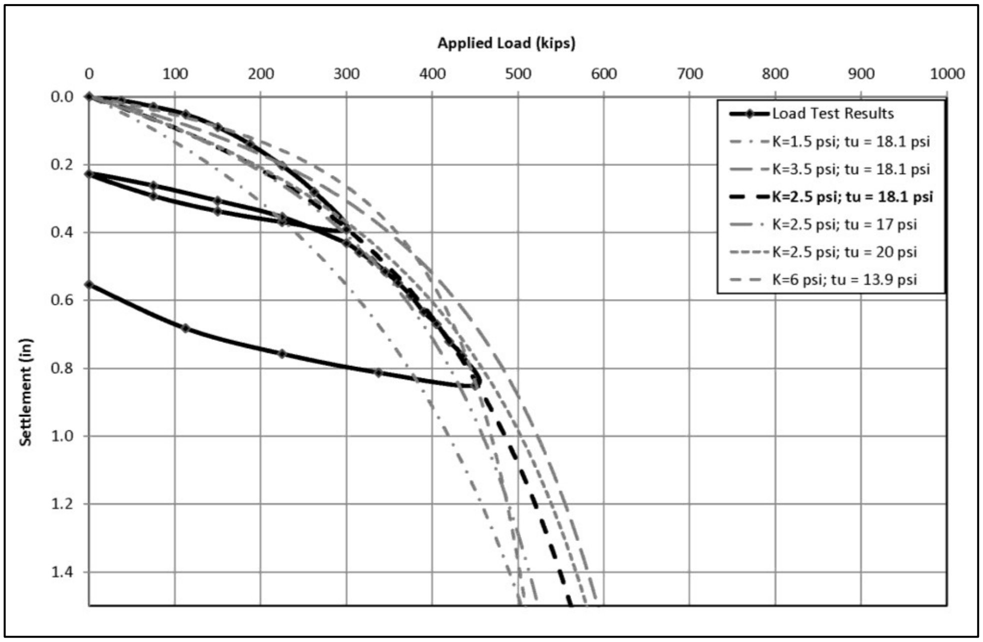


Figure 28: Applied load vs. settlement Cooling Tower compression test pile.

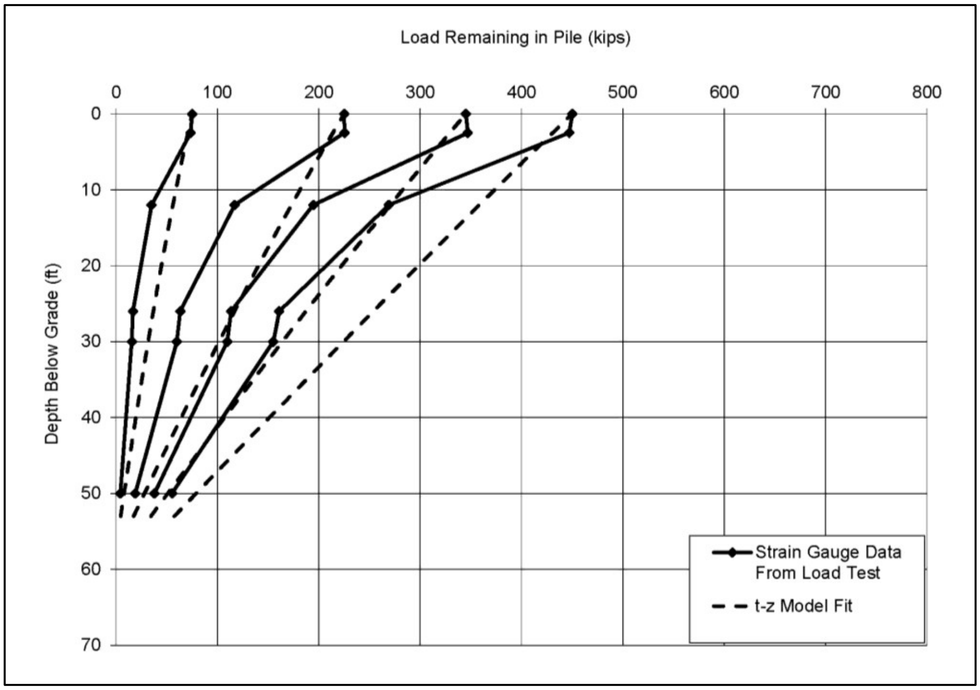


Figure 29: Load remaining in pile vs. depth for Cooling Tower compression test pile.

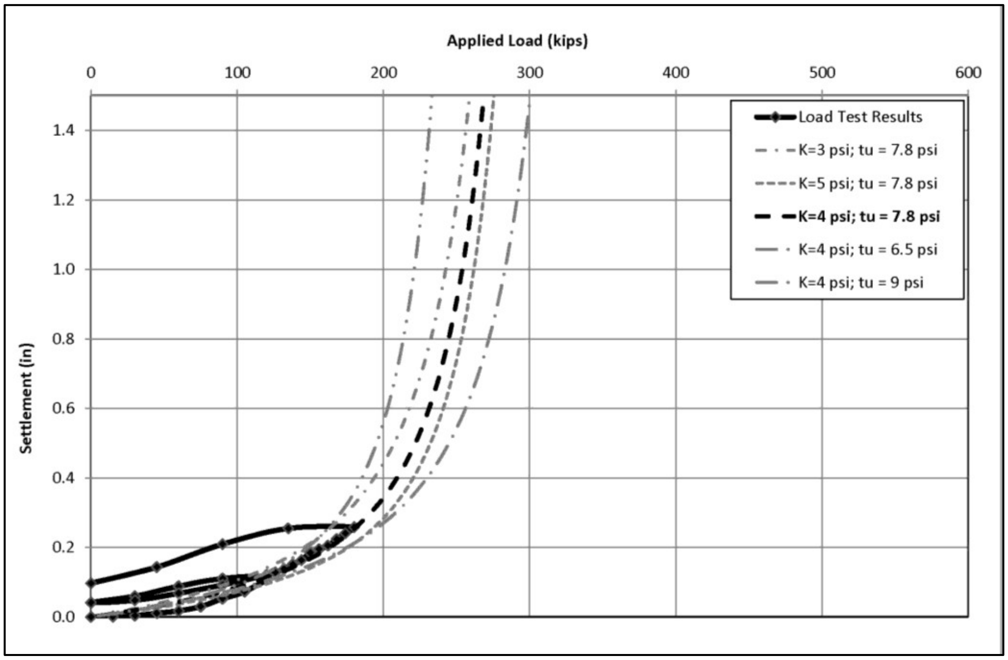


Figure 30: Applied load vs. settlement Cooling Tower tension test pile.

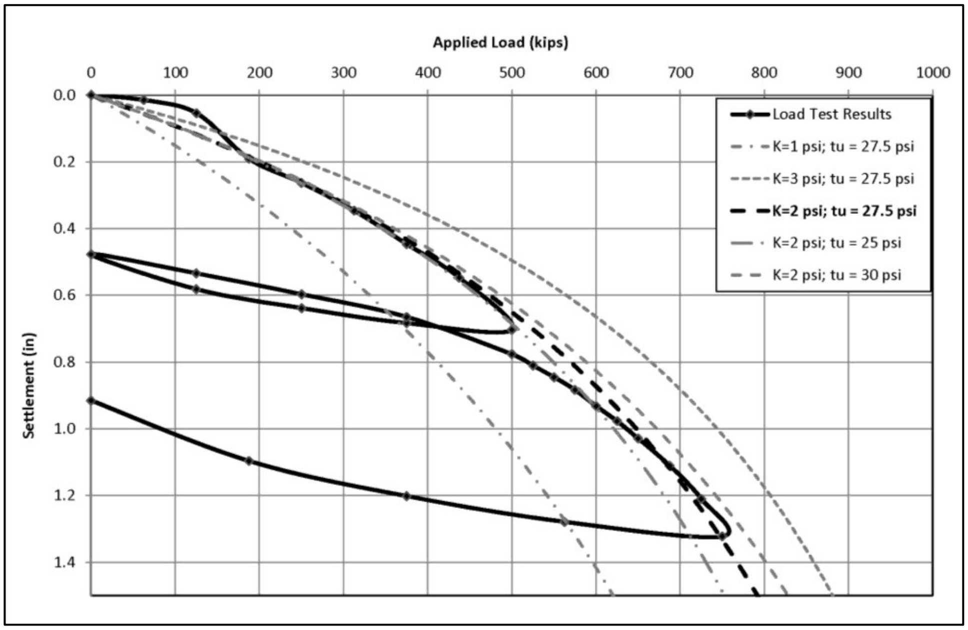


Figure 31: Applied load vs. settlement Water Tank compression test pile.

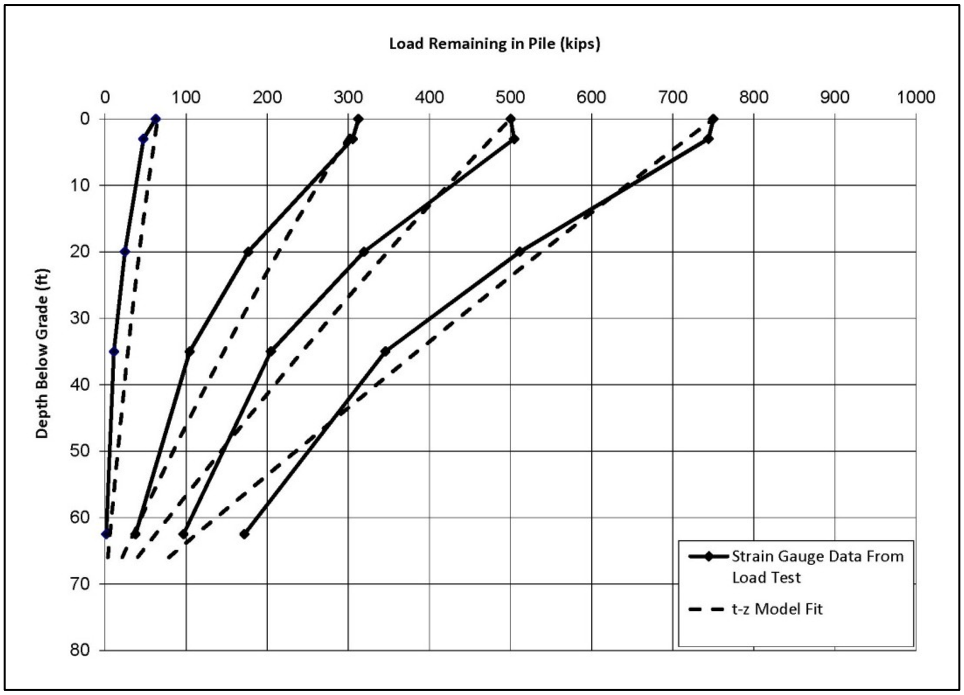


Figure 32: Load remaining in pile vs. depth for Water Tank compression test pile.

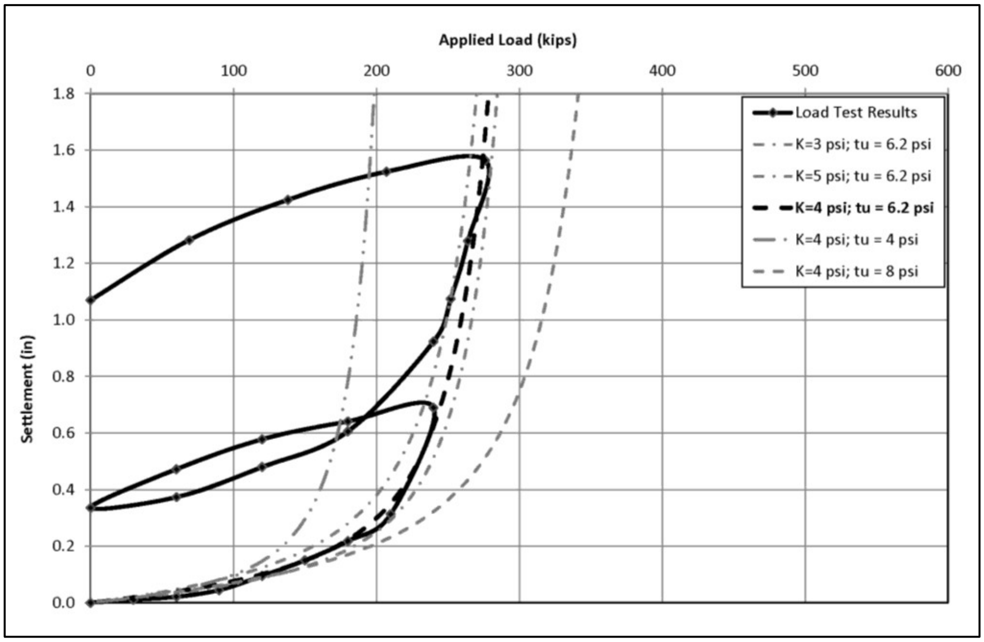


Figure 33: Applied load vs. settlement Water Tank tension test pile.

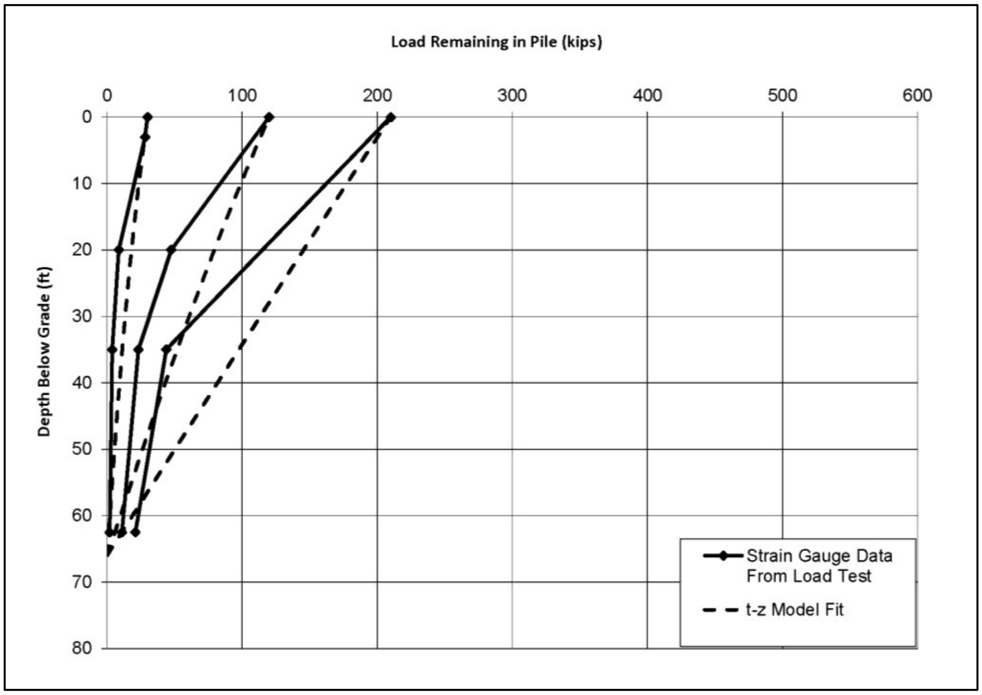


Figure 34: Load remaining in pile vs. depth for Water Tank tension test pile.

Table 6. – Back-calculated “t-z” model parameters.

Pile Location	Pile Type	K_{init} (ksi)	τ_u (psi)	$\tau_{u,tens}$ / $\tau_{u,comp}$
Boiler	Compression	3.0	22.0	0.38
	Tension	5.8	8.4	
Chimney	Compression	3.4	22.5	0.32
	Tension	5.0	7.3	
Powerhouse	Compression	2.7	16.8	0.53
	Tension	2.5	8.9	
Coal Yard	Compression	2.5	23.7	0.42
	Tension	20.0	10.0	
Cooling Tower	Compression	2.5	18.4	0.42
	Tension	4.0	7.8	
Water Tanks	Compression	2.0	27.5	0.23
	Tension	4.0	6.2	

While most of the values of K_{init} fell in a range from 2.0 to 5.0 ksi, the back-calculated value from the Coal Yard tension test (Figure 26) appears to be an outlier with a value of 20.0 ksi. It is unclear at this time whether the higher value of K_{init} is a product of variations in the pile installation or load testing or potentially a product of variations within the back-calculations of the model parameter from that specific test. For the development correlations between the “t-z” model parameters and the field investigation data, the value of K_{init} from the Coal Yard tension test in most cases was not included in the analyses.

It should be noted that the ratio of τ_u from the tension tests relative to the value of τ_u from the compression tests is quite low from this set of load tests. For deep foundations embedded in cohesionless soils, it is somewhat common to apply a reduction factor to the ultimate shear resistance values when evaluating tension capacity. This reduction is due to the potential for a reduction in effective stress in the vicinity of the pile as a result of Poisson's effect. The reduction factor is typically in a range 70 percent and 100 percent of the ultimate shear resistance in compression (Brown et al. 2007). However, the back-calculated values of τ_u from the tensions tests for the current project exhibited an apparent reduction factor ranging from 23 percent to 53 percent. It is unclear at this time whether these notably lower reduction factors are a product of the particular pile installation methods or load testing methods performed on this project or potentially a product of variations within the back-calculations of the model parameter from these load tests.

Correlation of "t-z" Model Parameters with Field Investigation Data

The back-calculated values of K_{init} and τ_u developed from the numerical model were then compared with the SPT N_{60} values collected from conventional soil borings and with the q_c values collected from CPT soundings to evaluate whether suitable correlations could be developed for use on future pile designs. For these comparisons, a single soil layer was assumed for the design profile such that the N_{60} and q_c values from explorations in the near vicinity of each test pile were averaged over the corresponding lengths of pile penetration. For test pile areas where multiple borings or multiple CPT soundings were performed in close proximity to the test pile, a single average value was calculated for N_{60} and q_c as appropriate. These values were previously summarized in Table 4 and Table 5.

Initially, the field investigation data was plotted directly with the back-calculated model parameters. For each model parameter, the values developed from the compression and tension tests were initially plotted separately relative to the field investigation data to identify any trends that were unique to the direction of the axial loading. Where similarities were observed between the results of the compression and tension test correlations, the data sets were combined to develop a single correlation applicable to loading in both axial directions.

Figure 35 and Figure 36 display the relationships for the N_{60} and q_c values, respectively, relative to K_{init} from both the compression and tension tests. The plot of the data generally indicates a linear relationship with the value of K_{init} increasing proportionally with the value of N_{60} or q_c. A simple linear regression correlation is indicated with the relationship forced through an imaginary point at the origin. As noted previously, the back-calculated value from the Coal Yard tension test appears to be an outlier and, while the data point is shown in each figure, that value was not included in the linear regression. While the plotted data does suggest a linear relationship, the sample size is not sufficient to confirm whether the relationship is a statistically accurate prediction of the correlation between the respective values.

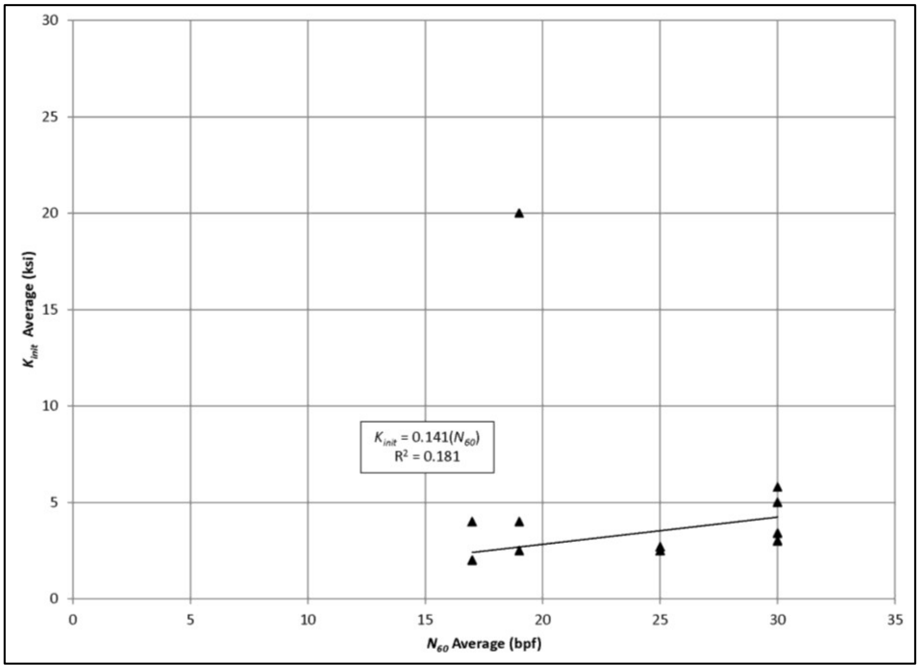


Figure 35: K_{init} vs. N_{60} from compression and tension test piles.

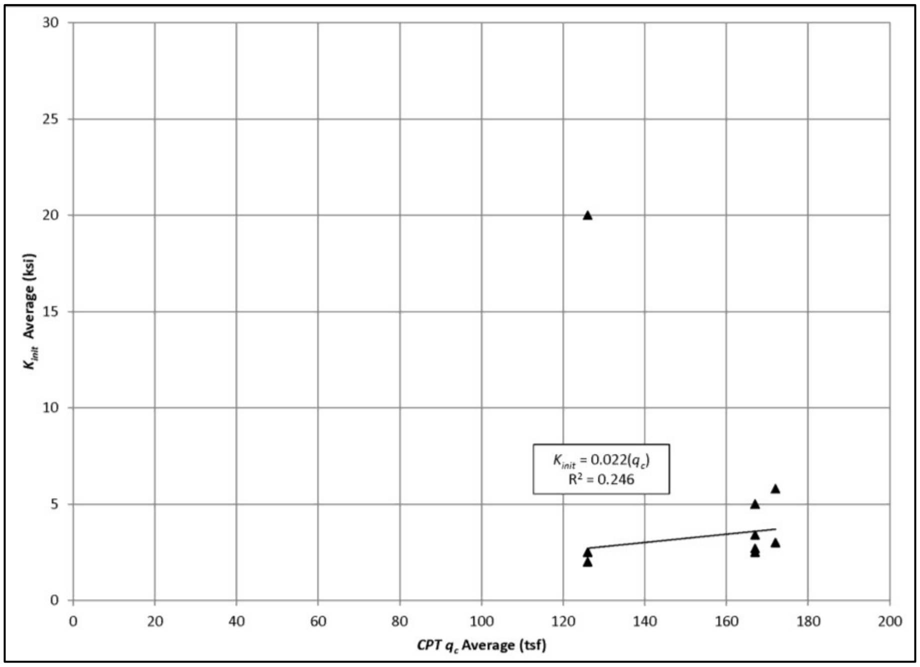


Figure 36: K_{init} vs. q_c from compression and tension test piles.

Figure 37 and Figure 38 display the relationships for the N_{60} and q_c values relative to τ_u from both the compression and tension tests. It was anticipated that a trend would be observed similar to those exhibited in Figures 35 and 36, where the value of τ_u would generally increase with increasing values of N_{60} and q_c. However, the data from both the compression and tension tests displays a higher level of variability and there is no distinct correlation between the back-calculated soil interface parameters and the field investigation data. It is suspected that the apparent lack of correlation between these values may be related to the assumption of a single-layer soil profile.

As noted previously, the pile top settlements from the compression tests ranged from 0.9 to 1.5 inches. Elastic structural deformation of the compression test piles was estimated to range from 0.4 to 0.7 inch with pile tip movements estimated to be in the range of 0.3 to 1.0 inch. The pile top movement exhibited by the tension tests ranged from 0.2 to 1.6 inches. Elastic structural deformation of the tension test piles was estimated to range from 0.1 to 0.3 inch with pile tip movements estimated to be in the range of 0.0 to 1.3 inches. The side resistance component of the pile capacity is fully mobilized with a relatively small amount of axial pile movement, typically less than 0.4 inches. When taking elastic deformation of the pile into consideration, it is likely that full mobilization of the side resistance component did not occur along the lower portions of the piles where the tip movement was estimated to be below 0.4 inch. Where this is the case, the assumption of a single, homogenous soil layer could lead to inaccuracies when averaging the side resistance component over the full length of the test pile.

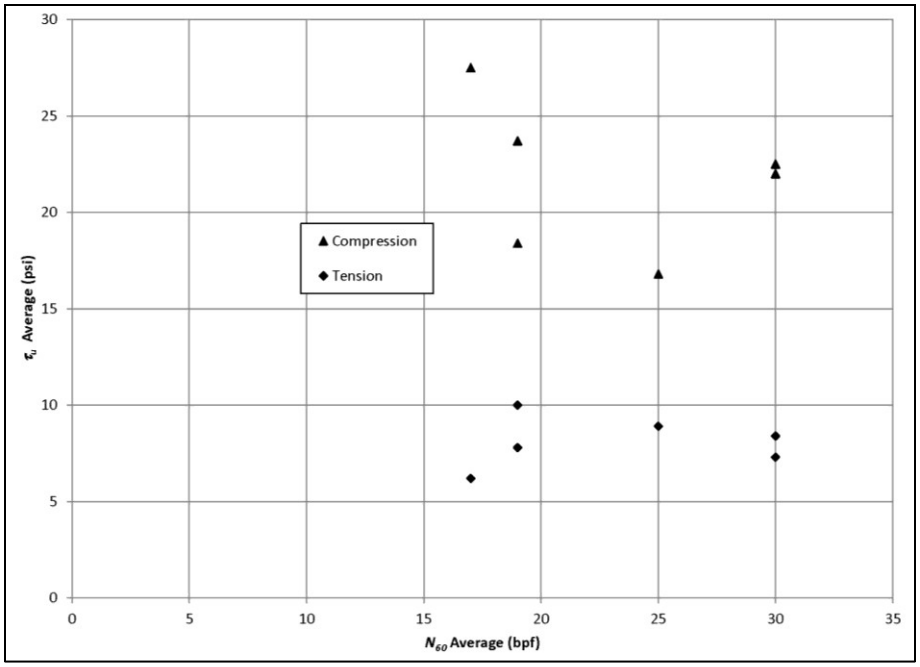


Figure 37: τ_u vs. N_{60} from compression and tension test piles.

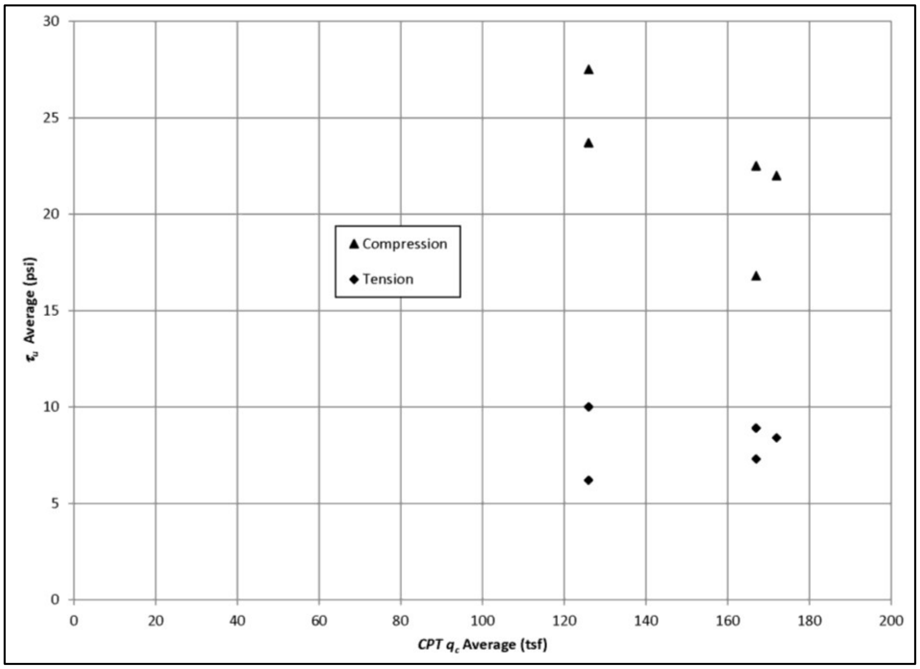


Figure 38: τ_u vs. q_c from compression and tension test piles.

Comparison of LPC Method for Correlating τ_u with q_c

While the overall distribution of the values of τ_u relative to q_c does not reveal a definitive correlation, a closer inspection of the individual relationships was performed to evaluate whether the correlations defined by Equation 1-12 for the LPC Method, along with the α-coefficients in Table 1, might present a suitable relationship when utilizing the numerical methods described herein for the "t-z" model. The values of $\alpha = q_c/\tau_u$ back-calculated from each compression and tension test are summarized in Table 7. The results reveal values for α in the range of 64 to 138 for the compression tests and values in the range from 175 to 318 for the tension tests.

Table 7. – LPC Method α-coefficients back-calculated from load test data.

Pile Location	$\alpha = q_c/\tau_u$ Compression	$\alpha = q_c/\tau_u$ Tension
Boiler	109	284
Chimney	103	318
Powerhouse	138	261
Coal Yard	74	175
Cooling Tower	No CPT Soundings	
Water Tanks	64	282

The α-coefficients developed from the compression tests are generally in line with the values published for the LPC method, previously summarized in Table 1, for soil types including loose silts and sands ($\alpha = 60$), medium dense sands and gravels ($\alpha = 100$), and very dense sands and gravels ($\alpha = 150$). The values of α calculated from the Boiler, Chimney and Powerhouse areas fall in the published range between medium dense to very dense sands and gravels. The test piles in these three areas were the longest of the test piles installed and ranged in length from 69 feet to 75 feet compared with the test piles in the area of Coal Yard, Cooling Tower and Water Tanks which were installed with lengths of 50 to 65 feet. The longer embedment lengths result in deeper penetration of the piles into the sands and gravels at depth which exhibit higher densities. The values of α calculated from the Coal Yard and Water Tanks were 64 and 74, respectively. These values are in the published range between loose and medium sands which may be attributed to their shorter embedment lengths and the resulting stronger influence of the shallower clays, silts and sands on the average value of τ_u. Based on these results, it appears that the published values of α associated with the LPC Method as applied to ACIP piles installed in cohesionless soils, and loaded in compression, are a suitable correlation for developing the τ_u values to be used in the "t-z" model described herein.

As noted previously, the back-calculated values of τ_u for the test piles loaded in tension appear to be uncharacteristically low relative to the values of τ_u calculated for the corresponding compression test piles. In addition, the α-coefficients developed from the tension tests do not appear to exhibit any specific trends relative to the relationship between embedment length and magnitude of α-coefficient as exhibited by the compression test results. As a result, the values of α associated with the LPC Method are not considered to be a suitable correlation for estimating the τ_u values to be used in the "t-z" model.

Correlation of “t-z” Model Parameters with Effective Stress

Due to the lack of suitable correlations when comparing the back-calculated values of K_{init} and τ_u directly with the field investigation data, additional comparisons were performed to evaluate the relationship of the soil model parameters to effective stress. The values of K_{init} and τ_u can both be estimated relative the confining stress of the soil profile. Each soil model value was then plotted against the normalized depth of each test pile. For this evaluation, the normalized depth is defined as the depth of the test pile, *d*, divided by the pile diameter, *D*. Since the soil profile is being modeled as a single layer, the mid-point depth of the test piles is used.

The value of τ_u can be calculated using the β-method described previously in Chapter 1 and utilizing Equation 1-7 which can be rewritten as follows:

$$\beta_{ep} = \frac{\tau_u}{\sigma_v^{'}} \qquad (4\text{-}5)$$

Similarly, the value of K_{init} can be calculated from a relationship developed by Janbu (1963) using the following equation:

$$K_{init} = K_{\mathrm{mod}} \sigma_{atm} \left(\frac{\sigma_v' K_o}{\sigma_{atm}} \right)^x \qquad (4\text{-}6)$$

where, K_o is the at-rest coefficient of earth pressure, K_{mod} is a modulus number, s_{atm} is the atmospheric pressure, and *x* is an exponent describing the rate of variation of K_{init} with respect to σ_v’ K_o. Both *x* and K_{mod} are constants which can be determined experimentally from the results of drained triaxial tests conducted under a variety of confining pressures. However, knowledge of those constants is not required to evaluate the potential for correlations between K_{init} and d/D or σ_v' K_o that can be used for future analyses within the "t-z" numerical model.

The value of σ_v' Equation 4-5 and Equation 4-6 can be calculated using the unit weight of the soil from laboratory test data or from correlations with field investigation data such as the N_{60} values and taking into consideration the influence of groundwater where present. Due to the difficulty in obtaining relatively undisturbed samples of cohesionless soils for laboratory unit weight tests, correlations with N_{60} values are commonly use to estimate the unit weight of sand and gravel soils. Correlations published by Bowles (1997) recommend wet unit weights in the range of 90 to 115 pounds per cubic foot (pcf) for loose sands, 110 to 130 pcf for medium dense sands, and 110 to 140 pcf for dense sands. For soils above the groundwater, a wet unit weight of 120 pcf was estimated for use in calculating the value of σ_v' to be used for calculating the value of β_{ep} for each test pile area.

Figure 39 displays the relationships for β_{ep} relative to d/D from both the compression and tension tests. The data generally indicates decreasing values of β_{ep} relative to increasing normalized depth. The plot includes a regression trendline based on a power function which is typical of correlations for β_{ep} based on trends observed in other data sets. However, the data sample does not cover a wide enough range of normalized depth values to allow for proper statistical analysis.

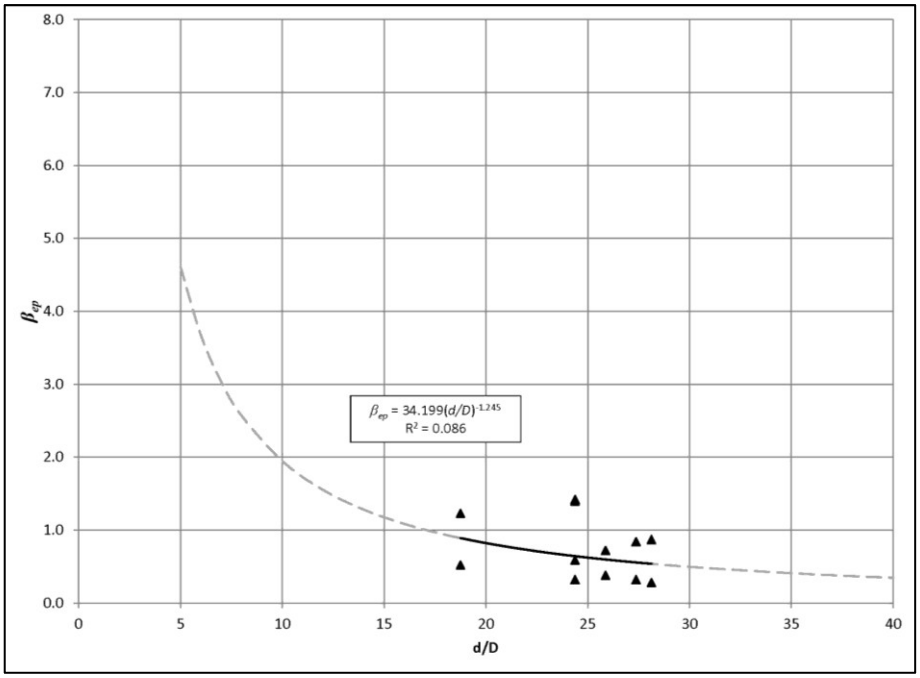


Figure 39: β_{ep} vs. d/D from compression and tension test piles.

As another point of evaluation, the average β_{ep} values calculated from the load test data are compared in Table 8 to the average theoretical β_{ep} values for each associated soil boring calculated using Equation 1-8. While the values back-calculated from the compression tests at the Boiler, Chimney and Powerhouse are within approximately 25 percent of the predicted theoretical values, the remainder of the back-calculated values differ by a factor of approximately two from the predicted theoretical values. Based on these results, the FHWA 1999 "β-method" does not appear to be a suitable correlation for developing the value of τ_u for use in this "t-z" method numerical modeling.

Table 8. – Comparison of back-calculated and theoretical β_{ep} values.

Pile Location	Average β_{ep} Compression	Average β_{ep} Tension	Average β_{ep} Theoretical
Boiler	0.84	0.32	0.67
Chimney	0.87	0.28	0.67
Powerhouse	0.72	0.38	0.69
Coal Yard	1.40	0.59	0.74
Cooling Tower	1.23	0.52	0.86
Water Tanks	1.42	0.32	0.73

Figure 40 displays the relationship for K_{init}/σ_v' relative to normalized depth from both the compression and tension tests. The data appears to show that the value of K_{init}/σ_v' decreases with increasing normalized depth. Similar to the plot for β_{ep} relative to *d/D*, the data sample is too small to allow for proper statistical analysis but a regression trendline based on a power function has been shown.

Finally, the values of K_{init} are plotted relative to confining stress in the form of $\sigma_v'K_o$ in Figure 41. Based on the relationship observed in the plot of K_{init}/σ_v' relative to normalized depth, it would be expected to see a similar trend with values of K_{init} decreasing relative to increased values of $\sigma_v'K_o$. However, the date plotted suggests an opposite trend of K_{init} increasing relative to higher values of $\sigma_v'K_o$. Once again, this data does not appear to provide a suitable correlation relative to the development of values for K_{init} to be used in this “t-z” numerical model.

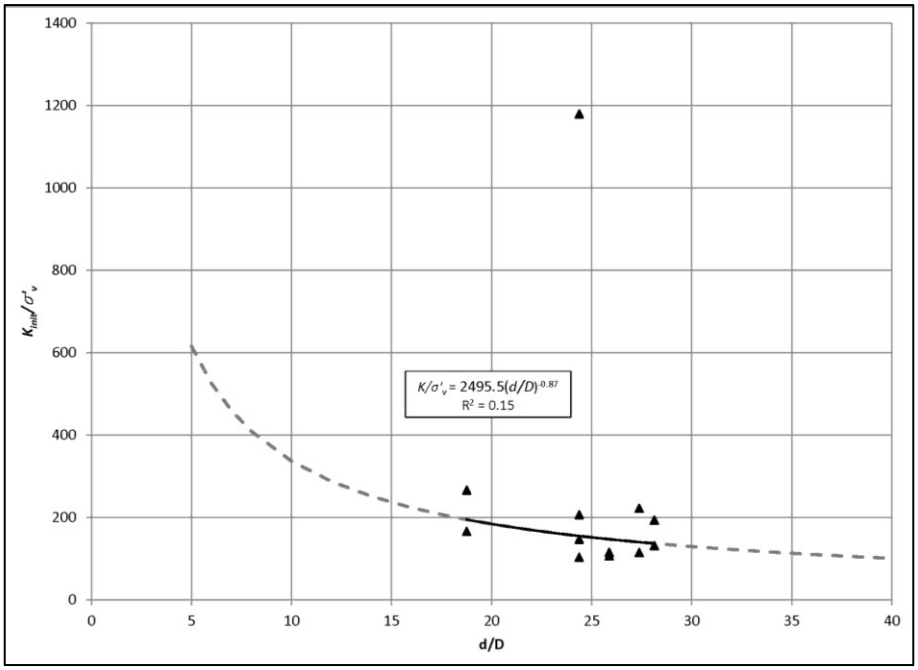


Figure 40: K_{init} vs. d/D from compression and tension test piles.

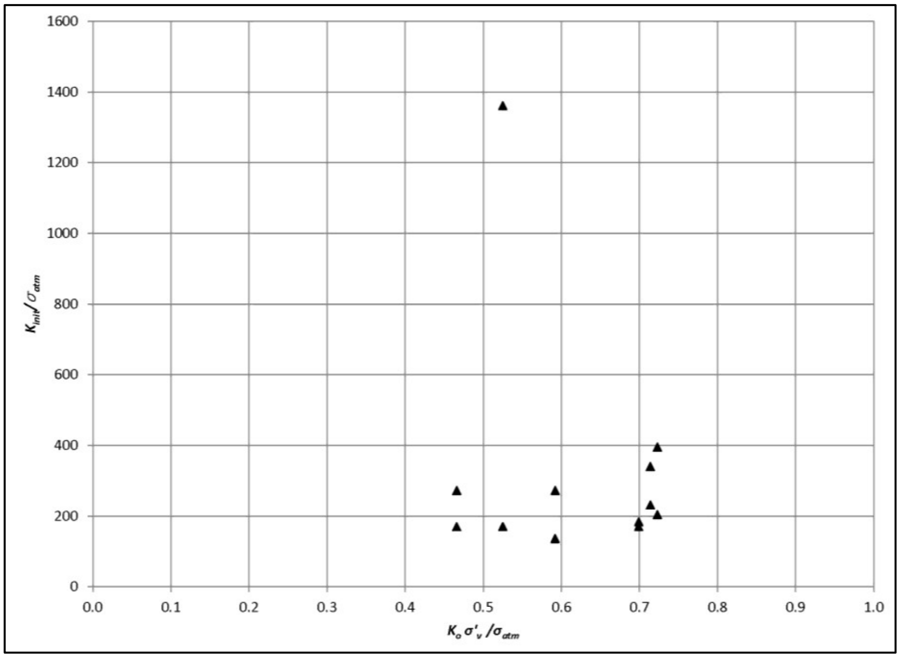


Figure 41: K_{init} vs. $K_o\sigma_v'$ from compression and tension test piles.

The primary source of difficulty in establishing correlations between the model parameters and the effective stress appears to be related to the limited stress range over which the data is plotted. Due to the assumption of a single soil layer, the magnitude of the effective stress, and similarly the magnitude of the normalized depth, presents a limited range over which the relationships can be evaluated.

CHAPTER 5

SUMMARY AND CONCLUSIONS

Project Summary

The purpose of this study was to expand on research previously performed by Roberts (2006) and Misra and Roberts (2006) to develop a reliability-based design methodology for the design and analysis of deep foundations at the service limit state. Specifically, this study focused on the development of "t-z" model parameters for use in service limit state analysis of augered cast-in-place (ACIP) piles. While most ACIP piles are designed based on well-established ultimate limit state methods, the methods for evaluating the service limit state performance, or load-settlement performance under service loads, have not been as thoroughly developed.

The current method most commonly used for evaluating the load-settlement performance of ACIP piles consists of curves based on empirical relationships developed from load tests performed on drilled shafts with a limited range of diameters. Use of those empirical curves methods may not provide an accurate prediction of the load-settlement performance of ACIP piles and load testing is recommended to verify the results of the analyses. While load testing for critical foundations is considered a good practice in general, load testing is often performed as a design verification process rather than for the purposes of optimizing the final foundation design. The development of a theoretical model that can be utilized during the design phase to more accurately evaluate the load-settlement performance of a variety of deep foundation types, including ACIP piles, and which can account for site-specific subsurface conditions, would be beneficial.

For the "t-z" method, the load-displacement behavior evaluated at the pile-soil interface is modeled as a series of springs using either an ideal elasto-plastic model or a hyperbolic model. For this research, the hyperbolic model was selected to better approximate the non-linear load-displacement behavior typically exhibited by deep foundations.

The shape of the hyperbolic load-displacement curves are then defined by a set of four parameters: (1) the initial tangent shear modulus of the subgrade reaction at the soil-structure interface, K_{init}, (2) the ultimate shear strength of soil-structure interface, τ_u, (3) the initial tip soil stiffness, K_{ti}, and (4) tip soil ultimate bearing capacity, q_t. Since the hyperbolic curve cannot be defined by a closed form solution, a finite difference method was used to evaluate the non-linear load-displacement behavior. A Mathcad computer program developed by Roberts (2006) was used to evaluate non-linear performance of the soil-structure interface. The program utilizes the central-difference methodology to solve the algebraic equations which define the load-displacement performance at a series of nodes along the length of the pile. The Mathcad model was used to back-calculate values for K_{init} and τ_u until the theoretical load-settlement curve predicted by the model presented a close approximation to the load-settlement curve developed from full scale load tests.

Data was collected from a series of load tests performed at a project in northwest Missouri, near the city of Weston. A total of 12 load tests were performed for the project, including six static compression and six static tension, on dedicated test piles located throughout the project site. During each load test, pile-top movement was monitored through the use of four dial gauges. In addition, “sister bar” strain gauges were embedded at multiple depths throughout each of the six compression test piles and three of the tension test piles to monitor the rate of load transfer in each pile during testing.

For model simplicity, the soil profile was assumed to consist of a single, homogenous layer with the values of K_{init} and τ_u modeled as average values over the length of each pile. During the curve fitting process, the values of K_{init} and τ_u were treated as variables while the remaining model parameters, including the non-interaction zones, axial stiffness of the pile, and the tip soil performance, were set as constants. Several sensitivity analyses were performed to evaluate the effect of potential variability within those parameters.

When matching the predicted pile performance from the Mathcad model to the data collected from the static load tests, the load-settlement data was the primary evaluation used to back-calculate the τ_u and K_{init} values. The predicted load distribution along the length of the piles, as compared with the data collected from the embedded strain gauges, was used as secondary criteria for comparison. With respect to the load-settlement curve matching, the data points representing 200 percent of the design load and 300 percent of the design load, the maximum test load applied to the pile, were considered the key points to match since those load increments were maintained for the longest time intervals during the testing.

Once average values of τ_u and K_{init} were back-calculated for each of the 12 load tests, they were compared against the N_{60} and q_c values collect from conventional borings and CPT soundings performed in the near vicinity of each test pile. While there is some indication of a linear relationship between K_{init} and N_{60} and q_c, the quantity of data evaluated was not sufficient to identify whether the relationship is a statistically accurate prediction of the correlation between the respective values.

The relationship between τ_u and N_{60} and q_c was much more variable and did not provide any distinct correlation between the back-calculated model parameters and the field investigation data. However, it was noted that when comparing q_c with τ_u, the back-calculated α-coefficients from the compression test data were very similar to values recommended in the LPC Method for hollow auger bored piles. Based on those results, it was suggested that values of α associated with the LPC Method as applied to ACIP piles installed in cohesionless soils, and loaded in compression, are a suitable correlation for developing the τ_u values to be used in this "t-z" model.

Due to overall poor quality of correlation between the back-calculated model parameters and the field investigation data, additional comparisons were performed to evaluate potential relationships between the "t-z" model parameters and effective stress within the soil profile. With the soil profile being modeled as a single layer, the data comparisons were made based on the effective stress at the mid-point depth of the test piles. While the plot of the data relating the model parameters to the effective stress exhibits some grouping, the ability to identify any trends in the correlation is difficult due to the relatively limited range of normalized depth values over which the relationship could be evaluated.

Overall, the assumption of a single, homogenous layer appears to have been an oversimplification of the soil profile for the purposes of developing correlations with the "t-z" model parameters relative to both field investigation data and overburden stress. Consideration of multiple layers within a soil profile would allow for more accurate evaluation of any relationship that might exist between the "t-z" model parameters and associated field investigation data for each respective soil layer. It would also allow for data points over a wider range of normalized depths when evaluating the correlation between the model parameters and overburden stress.

In addition to the use of a multi-layer soil profile, it would also be beneficial to coordinate the location of the field investigation data with the test pile location. For the current study, the distance the test piles to the soil borings and CPT soundings varied from 50 feet to 560 feet (see Table 2). While it is not possible to quantify the impact of increasing distance between the test pile and field exploration locations, the heterogeneous nature of soils makes it more likely to encounter variations over large distances that could lead to poor correlations.

Future Research

The research performed for this study was based on load tests and field investigations from a single project site and did not provide a sufficient quantity of data to identify trends in correlations with proper statistical analysis. Where data from load testing and field investigations are available from other sites, analyses similar to those performed within this study could be used to expand the volume of “t-z” model parameter correlations. The larger volume of correlations can then be evaluated for statistical trends more effectively. As part of any future evaluations to identify trends between “t-z” model parameters and field investigation data, it would also be beneficial to evaluate existing correlations between field investigation data and soil strength parameters, similar to those included in the FHWA 1999 Method or the LPC Method, to establish whether any existing correlations are suitable for use in estimating the “t-z” model parameters as an alternative to developing new correlations.

APPENDIX A

BORING LOGS

Drilling Log

Project Name Iatan		Boring Number	**B-01**
Project No. 41180-3.0209		Page	1 of 6
Ground Elevation 784 ft.msl	Location N 1195268 E 2653722	Total Footage	87.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	87.0 ft.	0.0 ft.	20	0	20.6	3-23-06

Drilling Company Geotechnology, Inc.	Drillers (s) Mike Umfleet, Brian Fingers
Drilling Rig CME -750 ATV	Type of Penetration Test SPT (Auto-Trip Hammer)
Date 3-22-06 To 3-23-06	Field Observer (s) Robert Jaques

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
	Sand and Gravel (FILL)	▩						11:18 Setup; 11:30 Break for lunch and safety meeting.
1	SAND, Fine to Medium Grained, Light Brown, Damp, Medium (SP)	░	7/ 12/ 15		10/ 18	1	SS-1	N = 27 13:26 Calibrating trip hammer. Alex has seen the safety video.
2		░				2		
3		░				3		
4	SAND, Fine to Medium Grained, Light Brown, Damp, Medium, Trace Coarse Sand, Trace Fine Gravel (SP)	░	3/ 11/ 10		14/ 18	4	SS-2	N = 21
5		░				5		
6	SAND, Fine to Medium Grained, Light Brown to Gray, Damp, Medium (SP)	░	3/ 9/ 10		14/ 18	6	SS-3	N = 19
7		░				7		
8		░				8		
9	SAND, Fine Grained, Gray, Wet, Medium (SP)	░	9/ 9/ 10		14/ 18	9	SS-4	N = 19
10		░				10		14:00 Switch to mud rotary.
11		░				11		
12	CLAY, Gray, Wet, Medium to Stiff, High Plasticity, Some Thin Roots (CH)	▨				12		
13		▨				13		
14		▨		1.50 TSF	16/ 18		SS-5	

GEOTECHNICALLOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

	Boring Number **B-01**
Project Name Iatan	Page 2 of 6
Project Number 41180-3.0209	Date 3-22-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
	CLAY, Gray, Wet, Medium to Stiff, High Plasticity, Some Thin Roots (CH)	▨	3/ 3/ 4	1.50 TSF	16/ 18		SS-5	N = 7
15		▨				15		
16		▨		0.75 TSF	19/ 24	16	ST-1	
17		▨				17		
	CLAY, Gray, Wet, Soft, Medium to High Plasticity (CL-CH)	▧						
18		▧				18		
19		▧	1/ 1/ 2	0.25 TSF	18/ 18	19	SS-6	N = 3
20		▧				20		▼
21		▧				21		
22		▧				22		
23		▧				23		
24		▧	1/ 1/ 1	< 0.25 TSF	18/ 18	24	SS-7	N = 2
	SILT, Clayey, Very Fine Grained, Gray, Wet, Very Soft (ML)	▥						
25		▥				25		
26		▥				26		
27	CLAY, Silty, Sandy, Gray, Wet, Very Soft, Medium to High Plasticity, Fine Sand at Bottom of Spoon (CL-CH)	▩				27		
28		▩				28		
29		▩	2/ 3/ 7	< 0.25 TSF	18/ 18	29	SS-8	N = 10
30		▩				30		
31		▩						

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MD.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

	Boring Number **B-01**
Project Name Iatan	Page 3 of 6
Project Number 41180-3.0209	Date 3-22-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
	CLAY, Silty, Sandy, Gray, Wet, Very Soft, Medium to High Plasticity, Fine Sand at Bottom of Spoon (CL-CH)	▨						
32	SAND, Fine Grained, Gray, Wet, Medium (SP)	░				32		
33		░				33		
34		░	5/ 8/ 10		14/ 18	34	SS-9	N = 18
35		░				35		
36		░				36		
	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)	▒						
37		▒				37		
38		▒				38		
39		▒	4/ 7/ 10		13/ 18	39	SS-10	N = 17
40		▒				40		
41		▒				41		
42		▒				42		
43		▒				43		
44	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)	▒	7/ 14/ 15		12/ 18	44	SS-11	N = 29
45		▒				45		
46		▒				46		
47		▒				47		
48		▒						

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

	Boring Number **B-01**
Project Name Iatan	Page 4 of 6
Project Number 41180-3.0209	Date 3-22-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Gray, Wet, Dense, Trace Fine Gravel (SP)							
49			12/ 20/ 18		16/ 18	49	SS-12	N = 38
50						50		
51						51		
52	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Clay and Fine Gravel (SW-SC)					52		
53						53		
54			4/ 5/ 7		10/ 18	54	SS-13	N = 12
55						55		Stop for today. Drained mud lines to prevent freezing. 3-23-06 7:15 On site. Geotech getting fuel for rigs. 8:17 Resume drilling at 55 feet. Hole taking water.
56						56		
57	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Lignite, Trace Fine Gravel (SW)					57		
58						58		
59			6/ 10/ 12		9/ 18	59	SS-14	
60						60		
61						61		
62	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Fine to Medium Gravel (SWG)					62		
63						63		
64			6/ 9/ 11		9/ 18	64	SS-15	N = 20
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

	Boring Number **B-01**
Project Name Iatan	Page 5 of 6
Project Number 41180-3.0209	Date 3-22-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Fine to Medium Gravel (SWG)							
66						66		
	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)							
67						67		
68						68		
69			12/ 14/ 14		10/ 18	69	SS-16	N = 28
70						70		
71						71		Losing circulation. Bit clogged.
72						72		
73						73		
	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)							
74			10/ 12/ 15		9/ 18	74	SS-17	N = 27
75						75		Bit Chattering on Occassional Cobbles.
76						76		
	GRAVEL, Fine to Medium Grained, Sandy, Gray, Wet, Dense, Fine to Coarse Sand (GP)							
77						77		
78						78		
79			20/ 20/ 12		6/ 18	79	SS-18	N = 32
80						80		
81						81		
82								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell

Drilling Log, continued

	Boring Number **B-01**
Project Name Iatan	Page 6 of 6
Project Number 41180-3.0209	Date 3-22-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov./ Advance (Inch)		Sample or Box No.	Remarks
83 84 85 86	GRAVEL, Fine to Medium Grained, Sandy, Gray, Wet, Dense, Fine to Coarse Sand (GP)	⁘				83 84 85 86		Losing circulation. Hole taking water. Drilled past sample point. Bedrock at 87 feet. Fine gravel in drill cuttings. 10:27 Stop at 87 feet. Grouted hole with 6 bags of grout, 50 gallons of water, and 1/2 bag of Quik Gel.
87 88 89 90 91 92 93 94 95 96 97 98 99	Boring Terminated at 87 Feet					87 88 89 90 91 92 93 94 95 96 97 98		

Drilling Log

Project Name Iatan		Boring Number **B-03**
Project No. 41180-3.0209		Page 1 of 7
Ground Elevation 785 ft.msl	Location N 1195147 E 2653707	Total Footage 102.5 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	87.0 ft.	15.5 ft.	20	2	Not Measured	

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Craig Steiner, Shaun Dotson**

Drilling Rig **Mobile B-57**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **3-15-06** To **3-16-06**

Field Observer (s) **Kevin Bolling**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)	Sample or Box No.	Remarks
	SAND; fine to medium grained, light brown, damp, medium (SM).						
1							
2			8/ 12/ 15		16/ 18	SS-1	
3							N=27
4	SAND; fine to medium grained, light brown, damp, dense (SM).		10/ 19/ 18		15/ 18	SS-2	
5							N=37
6	SAND; fine to medium grained, brown, damp, medium (SP).						
7			10/ 11/ 15		18/ 18	SS-3	
8	SILT; fine sand, trace clay, gray, wet, medium, trace plasticity (ML).						N=26
9			10/ 8/ 8		16/ 18	SS-4	
10							N=16
11							
12							
13							
14				2.0 TSF	18/ 18	SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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Drilling Log, continued

	Boring Number **B-03**
Project Name **Iatan**	Page **2 of 7**
Project Number **41180-3.0209**	Date **3-15-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	CLAY; brownish gray, stiff to very stiff, highly plastic (CH).	▨	3/ 5/ 6	2.0 TSF	18/ 18		SS-5	
15		▨				15		N=11
16		▨		2.25 TSF	24/ 24	16	ST-1	
17	CLAY; trace silt, brownish gray, very stiff, highly plastic (CH).	▨				17		
18		▨				18		
19	CLAY; brownish gray, stiff, highly plastic (CH).	▨	2/ 3/ 3	1.5 TSF	16/ 18	19	SS-6	
20		▨				20		N=6
21		▨				21		Switch to rotary wash.
22	SILT; trace clay, trace fine sand, gray, wet, medium, nonplastic (ML).	▥				22		
23		▥				23		
24		▥	7/ 8/ 8		14/ 18	24	SS-7	
25		▥				25		N=16
26		▥				26		
27	SAND; fine grained, silt, gray, wet, medium (SP).	░				27		
28		░				28		
29		░	6/ 6/ 7		16/ 18	29	SS-8	
30		░				30		N=13
31		░						

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

	Boring Number **B-03**
Project Name **Iatan**	Page **3 of 7**
Project Number **41180-3.0209**	Date **3-15-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; medium to coarse grained, trace fine gravel, trace lignite, gray, wet, medium (SP).							
32						32		
33						33		
34			6/ 8/ 8		11/ 18	34	SS-9	
35						35		N=16
36						36		
37						37		
38						38		
39	SAND; medium to coarse grained, gray, wet, medium, with trace gravel (SP).		7/ 7/ 14		12/ 18	39	SS-10	
40						40		N=21
41						41		
42						42		
43						43		
44	SAND; fine to medium grained, trace coarse sand, trace lignite, gray, wet, medium to dense (SP).		15/ 15/ 15		13/ 18	44	SS-11	
45						45		N=30
46						46		
47						47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

	Boring Number **B-03**
Project Name **Iatan**	Page **4 of 7**
Project Number **41180-3.0209**	Date **3-15-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; medium grained, gray, wet, medium (SP).							
49			14/ 12/ 15		12/ 18	49	SS-12	
50						50		N=27
51						51		
	SAND; fine to coarse grained, trace fines, gray, wet, medium (SW).							
52						52		
53						53		
54			8/ 10/ 15		13/ 18	54	SS-13	
55						55		N=25
56						56		
	SAND; medium to coarse grained, gray, wet, medium, with trace fine sand (SP).							
57						57		
58						58		
59			12/ 13/ 13		12/ 18	59	SS-14	
60						60		N=26
61						61		
	SAND; coarse grained, with fine gravel, brownish gray, wet, medium (SW).							
62						62		
63						63		
64			11/ 10/ 11			64	SS-15	
65								N=21

Drilling Log, continued

Boring Number **B-03**

Project Name **Iatan**

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Project Number 41180-3.0209

Date 3-15-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; coarse grained, with fine gravel, brownish gray, wet, medium (SW).							
66						66		
67						67		
68						68		
69	SAND; medium to coarse grained, trace fine sand and gravel, brownish gray, wet, medium (SW).		11/ 10/ 10		12/ 18	69	SS-16	
70						70		N=20
71						71		
72						72		
73						73		
74	SAND; coarse grained, with gravel, gray, wet, medium (SW).		11/ 10/ 9		11/ 18	74	SS-17	
75						75		N=19
76						76		
77	GRAVEL; with coarse sand, gray, wet, medium (GP).					77		
78						78		
79			18/ 14/ 10		6/ 18	79	SS-18	
80						80		N= 24
81						81		Stop at 16:45 on 3/15/06.
82								

Drilling Log, continued

	Boring Number **B-03**
Project Name **Iatan**	Page **6 of 7**
Project Number **41180-3.0209**	Date **3-15-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; coarse grained, gravel, gray, wet, medium (SW).	░						
83						83		
84			21/ 15/ 12		11/ 18	84	SS-19	
85						85		N=27
86						86		
87	LIMESTONE; gray, fresh, moderately strong. SHALE; gray, fresh, weak, fissile.	▦ ▤				87		
88						88		
89	SANDSTONE; gray, fresh, strong, laminated with shale.	░		RQD= 67%	40/ 42	89	Run#1	
90						90		
91	SANDSTONE; fine grained, gray, fresh, strong; interbedded with SHALE; fissile, gray, fresh, weak.	▤				91		
92						92		
93	SANDSTONE; fine grained, gray, fresh, strong, with shell fragments.	░		RQD= 35%	58/ 60	93	Run#2	
94						94		
95						95		
96						96		
97				RQD= 8%	59/ 60	97	Run#3	
98						98		
99								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued

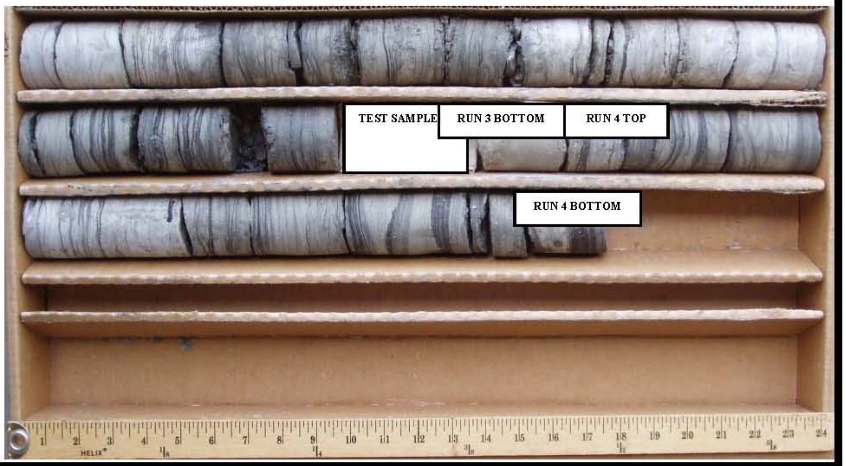
	Boring Number **B-03**
Project Name **Iatan**	Page **7 of 7**
Project Number **41180-3.0209**	Date **3-15-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SANDSTONE; fine grained, gray, fresh, strong, with shell fragments.	⁙		RQD= 8%	59/ 60		Run#3	
100						100		
101				RQD= 42%	24/ 24	101	Run#4	
102						102		
103						103		
104	Boring Terminated at 102.5 Feet.					104		
105						105		
106						106		
107						107		
108						108		
109						109		
110						110		
111						111		
112						112		
113						113		
114						114		
115						115		
116								

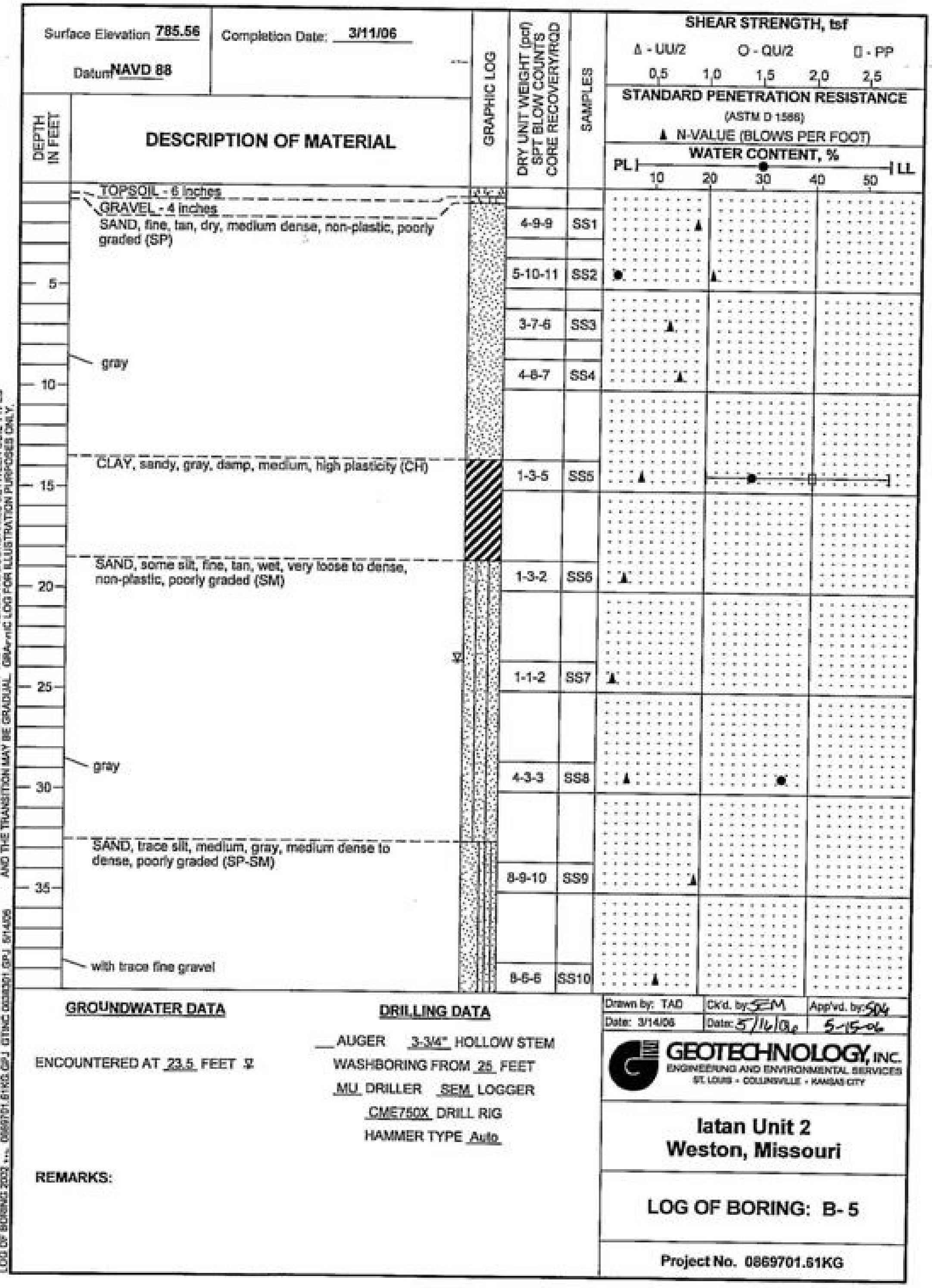
B-3 **IATAN UNIT 2 PROJECT** **B&McD Project 41180**

Run No.	Depth (ft)
Run 1	87.0 to 90.5
Run 2	90.5 to 95.5
Run 3	95.5 to Continued

B-3 **IATAN UNIT 2 PROJECT** **B&McD Project 41180**



Run No.	Depth (ft)
Run 3	Continued to 100.5
Run 4	100.5 to 102.5



Surface Elevation **785.56**　Completion Date: **3/11/06**

Datum **NAVD 88**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL - 6 inches GRAVEL - 4 inches SAND, fine, tan, dry, medium dense, non-plastic, poorly graded (SP)	4-9-9	SS1
5		5-10-11	SS2
		3-7-6	SS3
10	gray	4-8-7	SS4
15	CLAY, sandy, gray, damp, medium, high plasticity (CH)	1-3-5	SS5
20	SAND, some silt, fine, tan, wet, very loose to dense, non-plastic, poorly graded (SM)	1-3-2	SS6
25		1-1-2	SS7
30	gray	4-3-3	SS8
35	SAND, trace silt, medium, gray, medium dense to dense, poorly graded (SP-SM)	8-9-10	SS9
	with trace fine gravel	8-6-6	SS10

NOTE: STRATIFICATION LINES REPRESENT THE OXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 23.5 FEET ☿

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDG
Date: 3/14/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B- 5

Project No. 0869701.61KG

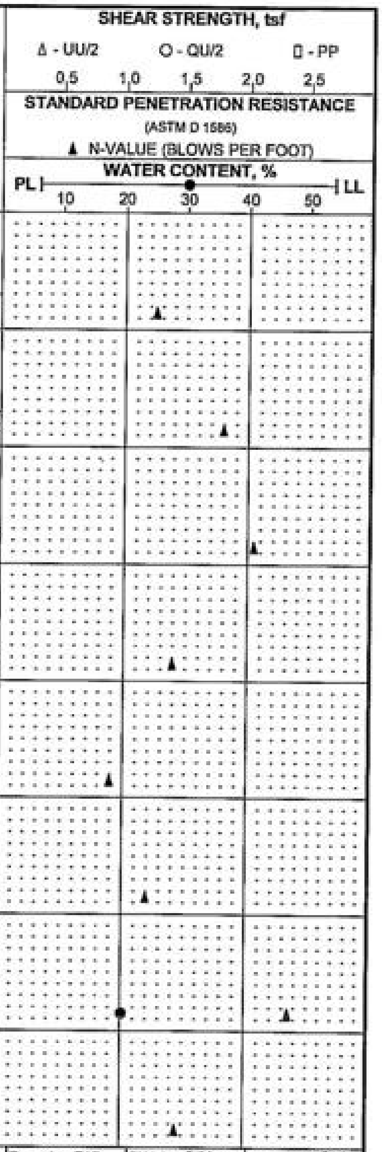
LOG OF BORING 2003 ... 0869701.61KG.GPJ GTINC 0308301.GPJ 5/14/06

Surface Elevation **785.56**

Datum **NAVD 88**

Completion Date: **3/11/06**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, medium, gray, medium dense to dense, poorly graded (SP-SM) *(continued)*			
45			10-15-10	SS11
50			10-13-23	SS12
55			15-20-21	SS13
	coarse			
60			13-14-14	SS14
65			0-6-12	SS15
	medium-coarse, gray			
70			7-7-17	SS16
75			17-22-25	SS17
	coarse		9-14-15	SS18



NOTE: STRATIFICATION LINES REPRESENT THE OXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 23.5 FEET ∇

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SFM	App'vd. by: SOG
Date: 3/14/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B- 5

Project No. 0869701.61KG

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0828301.GPJ 5/14/06

Surface Elevation 785.56

Datum NAVD 88

Completion Date: 3/11/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, medium, gray, medium dense to dense, poorly graded (SP-SM) *(continued)*			
	with gravel and limestone fragments		10-16-12	SS19
85				
	Boring terminated at roller bit refusal at 86 feet			
90				
95				
100				
105				
110				
115				

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - PP

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

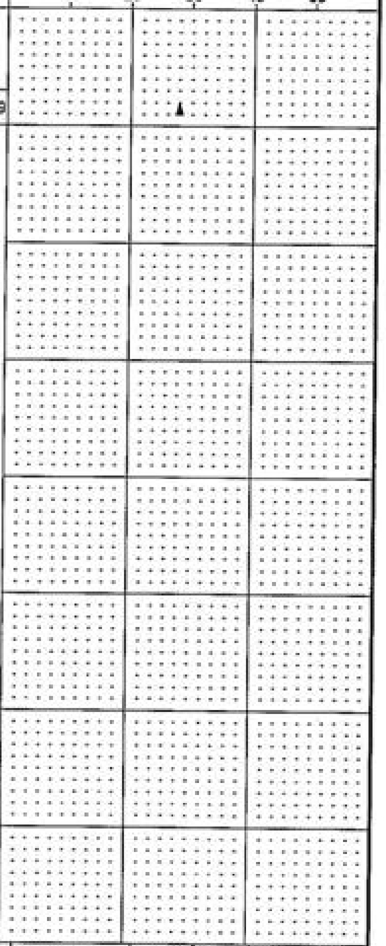
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢——●——⊣ LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 23.5 FEET ∇

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM

WASHBORING FROM 25 FEET

MU DRILLER SEM LOGGER

CME750X DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SOC
Date: 3/14/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.

ENGINEERING AND ENVIRONMENTAL SERVICES

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Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B- 5

Project No. 0869701.61KG

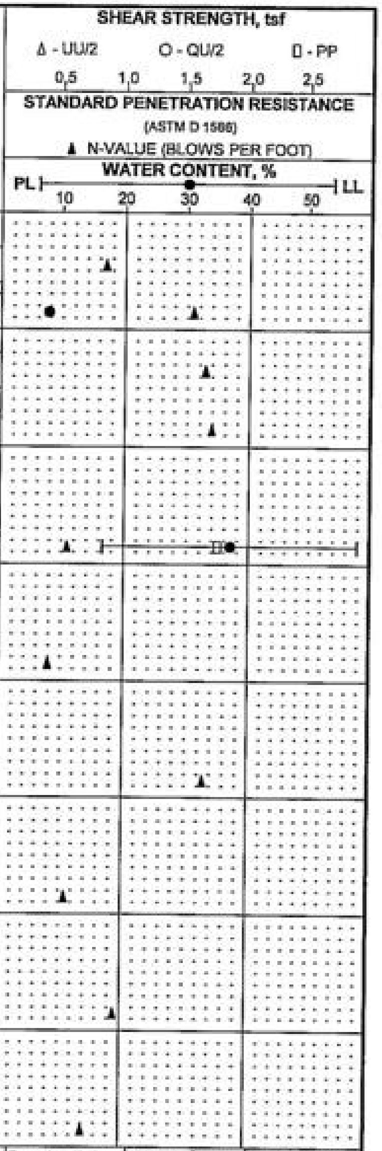
LOG OF BORING 2002 Vn. 0869701.61KG.GPJ GTINC 0638301.GPJ 5/14/06

Surface Elevation 786.44

Datum NAVD 88

Completion Date: 4/1/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL - 12 inches			
	CLAY, sandy, brown, damp, very stiff to hard, high plasticity (CH)		6-8-9	SS1
			5-12-19	SS2
5	SAND, fine to medium, gray, damp, medium dense to dense, non-plastic, poorly graded (SP)		6-16-17	SS3
10			14-16-18	SS4
	CLAY, black, damp, stiff, high plasticity (CH)			
15			3-4-7	SS5
20	SAND, some silt, fine to medium, gray, damp, loose to dense, non-plastic (SM)		5-3-5	SS6
			10-15-18	SS7
25	SAND, coarse, gray, wet, medium dense to dense, non-plastic, poorly graded (SP)			
	▽			
30			1-4-7	SS8
35			12-10-9	SS9
			8-9-5	SS10



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 28 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 30 FEET
CS DRILLER RB LOGGER
Mobile B57 DRILL RIG
HAMMER TYPE Auto

REMARKS: **Auger refusal at 88 feet**

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SB
Date: 4/3/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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Iatan Unit 2
Weston, Missouri

LOG OF BORING: B- 6

Project No. 0869701.61KG

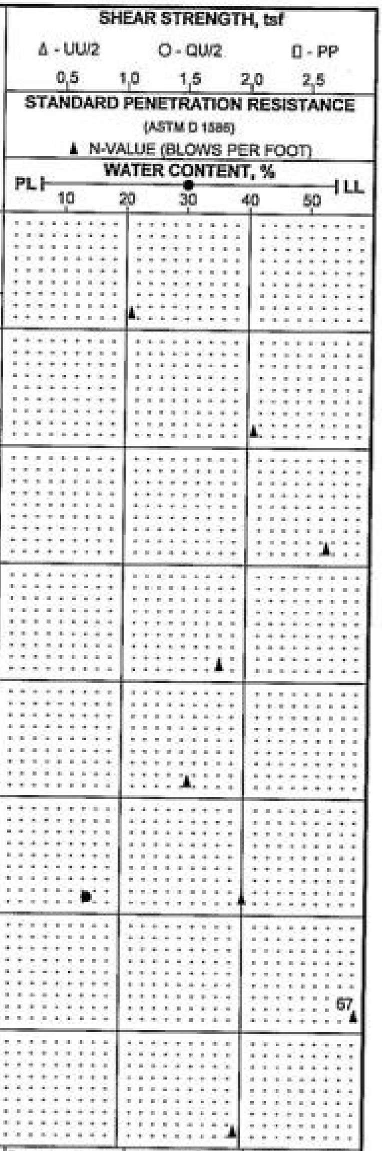
LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0636301.GPJ 5/14/06

Surface Elevation **786.44**

Datum **NAVD 88**

Completion Date: **4/1/06**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, coarse, gray, wet, medium dense to dense, non-plastic, poorly graded (SP) *(continued)*		
45	trace clay	13-9-12	SS11
50		22-19-22	SS12
55	fine, black	22-28-25	SS13
60		18-20-16	SS14
65		8-15-16	SS15
70	medium to coarse	20-21-19	SS16
75		22-28-39	SS17
		17-19-20	SS18



NOTE: STRATIFICATION LINES REPRESENT TH. ROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2002 w/L 0869701.61KG.GPJ GTINC 0809001.GPJ 5/14/06

GROUNDWATER DATA

ENCOUNTERED AT 28 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM

WASHBORING FROM 30 FEET

CS DRILLER RB LOGGER

Mobile B57 DRILL RIG

HAMMER TYPE Auto

REMARKS: Auger refusal at 88 feet

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDG
Date: 4/3/06	Date: 5/15/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B- 6

Project No. 0869701.61KG

Surface Elevation **786.44**

Datum **NAVD 88**

Completion Date: **4/1/06**

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, coarse, gray, wet, medium dense to dense, non-plastic, poorly graded (SP) *(continued)*			
85			15-16-17	SS19
	LIMESTONE, shaly, gray, slightly weathered, moderately strong to strong			
90	SHALE, gray, slightly weathered grading to fresh, weak to moderately strong, limy with limestone seams		65% 37%	NQ1
			98% 55%	NQ2
95				
			98% 80%	NQ3
100				
			100% 62%	NQ4
	Boring terminated at 103.5 feet			
105				
110				
115				

SHEAR STRENGTH, tsf

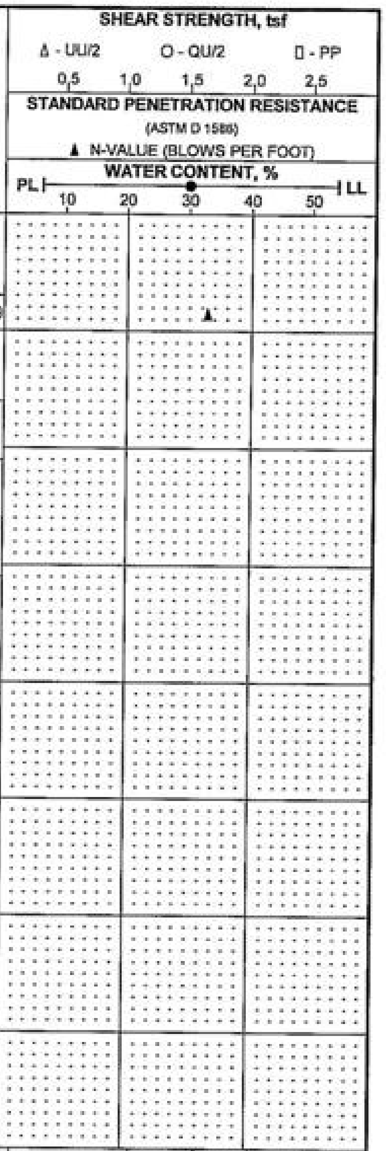
Δ - UU/2 O - QU/2 ▯ - PP

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %



NOTE: STRATIFICATION LINES REPRESENT TH ROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0638301.GPJ 5/14/06

GROUNDWATER DATA

ENCOUNTERED AT 28 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 30 FEET
CS DRILLER RB LOGGER
Mobile B57 DRILL RIG
HAMMER TYPE Auto

REMARKS: Auger refusal at 88 feet

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SOG
Date: 4/3/06	Date: 5/15/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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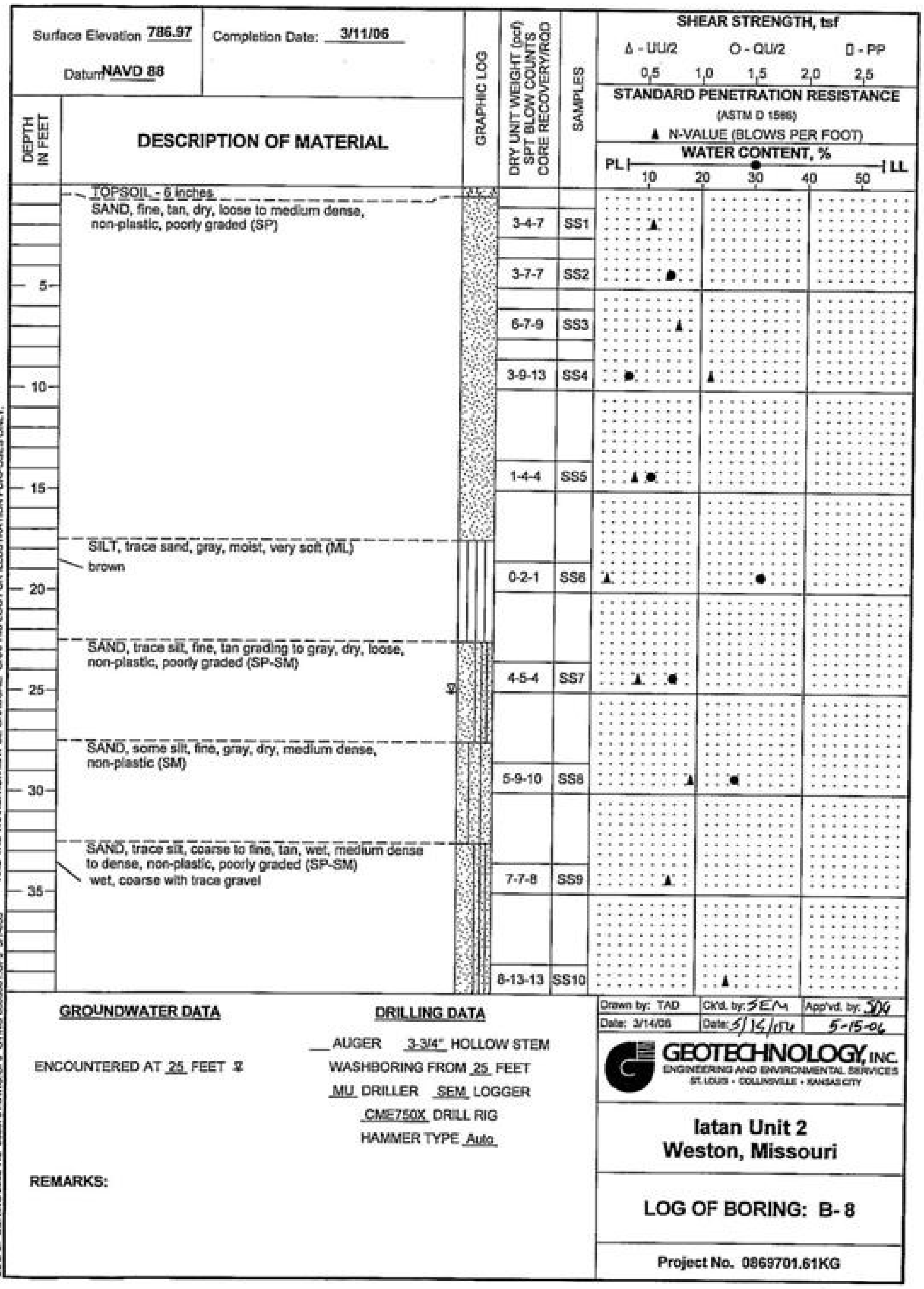
Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B- 6

Project No. 0869701.61KG



Surface Elevation 786.97

Datum NAVD 88

Completion Date: 3/11/06

DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
TOPSOIL - 6 inches SAND, fine, tan, dry, loose to medium dense, non-plastic, poorly graded (SP)	3-4-7	SS1
	3-7-7	SS2
	6-7-9	SS3
	3-9-13	SS4
	1-4-4	SS5
SILT, trace sand, gray, moist, very soft (ML) brown	0-2-1	SS6
SAND, trace silt, fine, tan grading to gray, dry, loose, non-plastic, poorly graded (SP-SM)	4-5-4	SS7
SAND, some silt, fine, gray, dry, medium dense, non-plastic (SM)	5-9-10	SS8
SAND, trace silt, coarse to fine, tan, wet, medium dense to dense, non-plastic, poorly graded (SP-SM) wet, coarse with trace gravel	7-7-8	SS9
	8-13-13	SS10

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 25 FEET ⊻

DRILLING DATA

___ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDY
Date: 3/14/06	Date: 5/15/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B- 8

Project No. 0869701.61KG

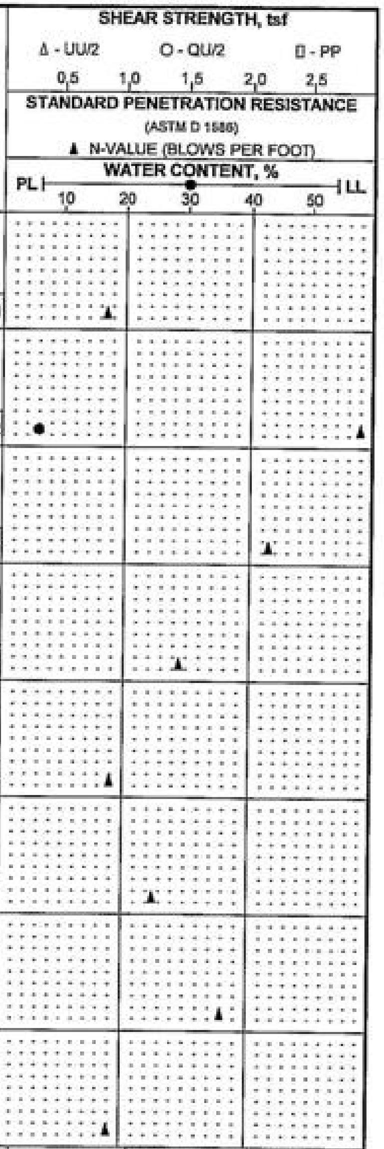
LOG OF BORING 2002 Ver. 0869701.61KG.GPJ GTINC 0608301.GPJ 5/14/06

Surface Elevation 786.97

Datum NAVD 88

Completion Date: 3/11/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, coarse to fine, tan, wet, medium dense to dense, non-plastic, poorly graded (SP-SM) *(continued)*		
		10-9-8	SS11
45			
		15-23-35	SS12
50			
		17-20-23	SS13
55			
		10-12-17	SS14
60			
	SAND, medium to coarse, gray, wet, medium dense to dense, non-plastic, poorly graded (SP)		
	coarse	7-8-10	SS15
65			
		11-11-14	SS16
70			
	with trace gravel	15-16-20	SS17
75			
		9-9-9	SS18



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 25 FEET ⊻

DRILLING DATA

__AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SOG
Date: 3/14/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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Iatan Unit 2
Weston, Missouri

CONTINUATION OF LOG OF BORING: B- 8

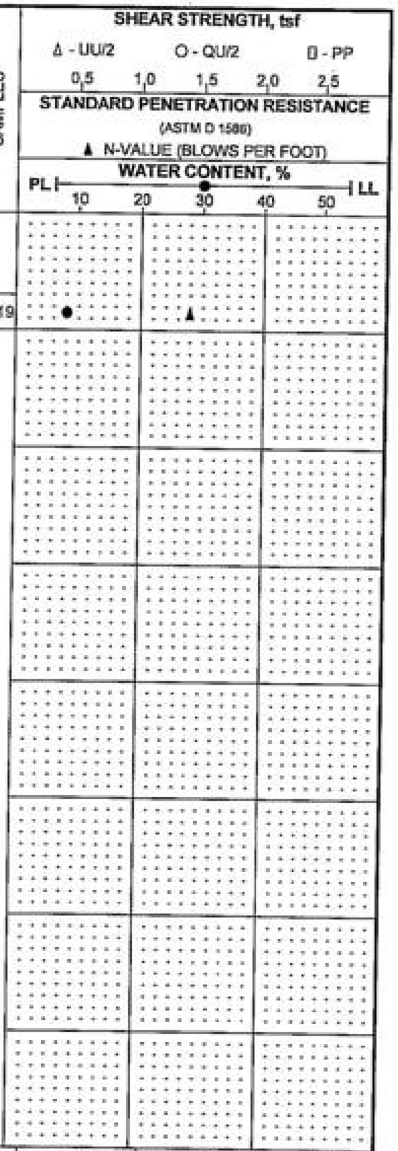
Project No. 0869701.61KG

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0636301.GPJ 5/14/06

Surface Elevation 786.97 | Completion Date: 3/11/06

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, medium to coarse, gray, wet, medium dense to dense, non-plastic, poorly graded (SP) *(continued)*			
85			12-15-13	SS19
	Boring terminated at roller bit refusal at 88 feet			
90				
95				
100				
105				
110				
115				



NOTE: STRATIFICATION LINES REPRESENT TH ROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GR IC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 25 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

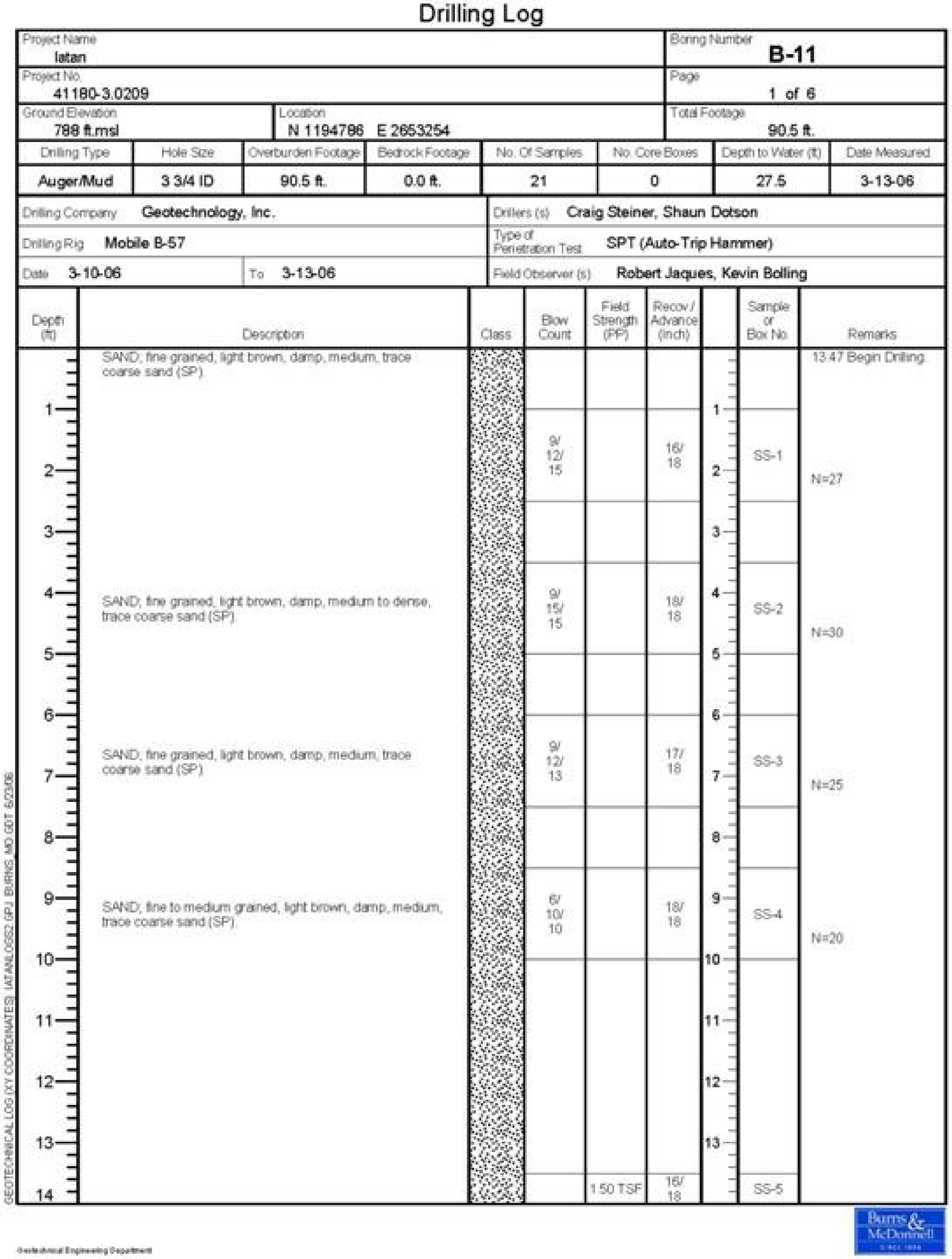
Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDG
Date: 3/14/06	Date: 5/16/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF LOG OF BORING: B- 8

Project No. 0869701.61KG



Drilling Log

Project Name Iatan						Boring Number **B-11**	
Project No. 41180-3.0209						Page 1 of 6	
Ground Elevation 788 ft.msl		Location N 1194786 E 2653254				Total Footage 90.5 ft.	
Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	90.5 ft.	0.0 ft.	21	0	27.5	3-13-06

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Craig Steiner, Shaun Dotson**

Drilling Rig **Mobile B-57**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **3-10-06** To **3-13-06**

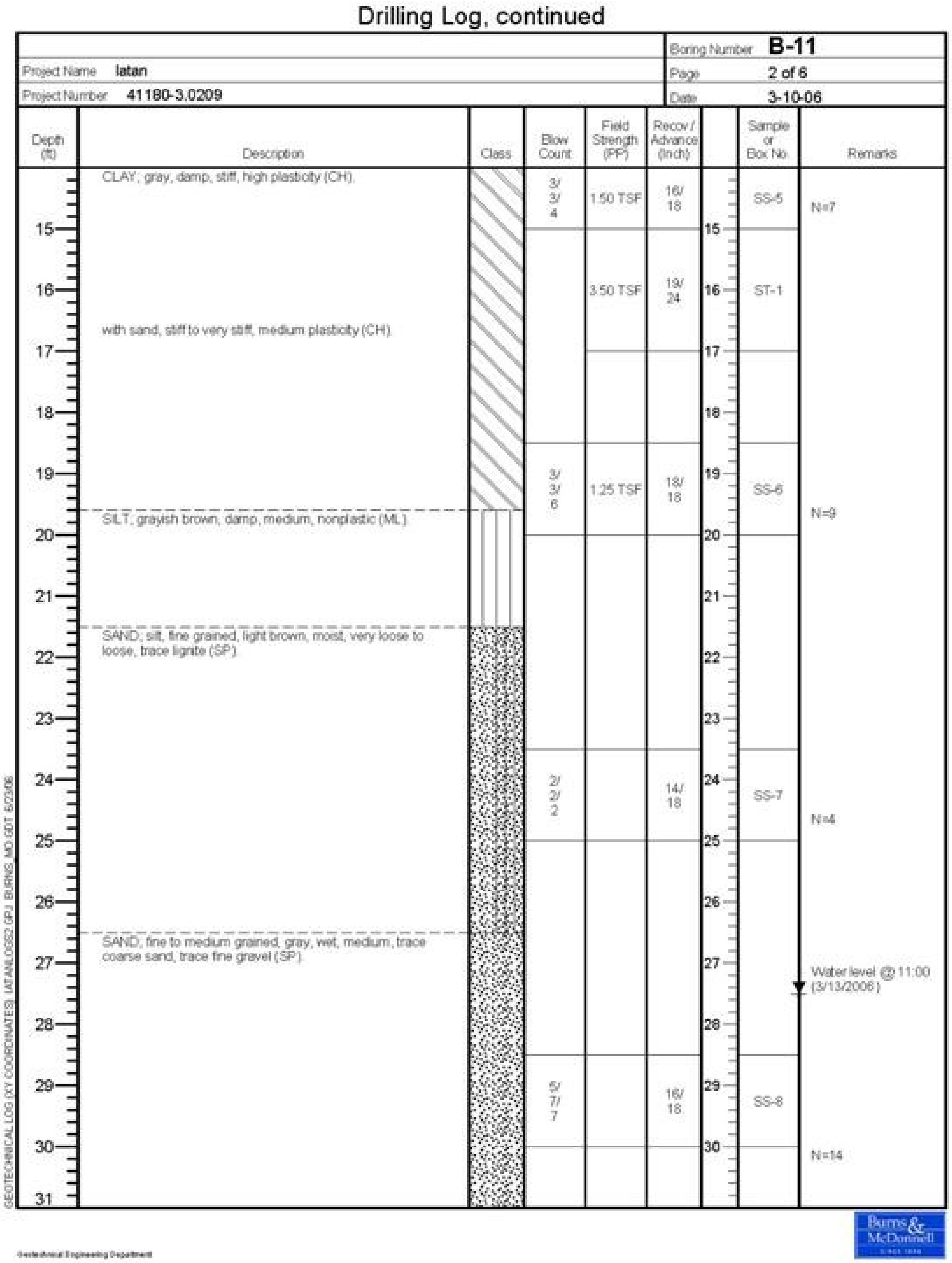
Field Observer (s) **Robert Jaques, Kevin Bolling**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
0	SAND; fine grained, light brown, damp, medium, trace coarse sand (SP).							13:47 Begin Drilling.
1			9/ 12/ 15		16/ 18	1	SS-1	N=27
2						2		
3						3		
4	SAND; fine grained, light brown, damp, medium to dense, trace coarse sand (SP).		9/ 15/ 15		18/ 18	4	SS-2	N=30
5						5		
6	SAND; fine grained, light brown, damp, medium, trace coarse sand (SP).		9/ 12/ 13		17/ 18	6	SS-3	N=25
7						7		
8						8		
9	SAND; fine to medium grained, light brown, damp, medium, trace coarse sand (SP).		6/ 10/ 10		18/ 18	9	SS-4	N=20
10						10		
11						11		
12						12		
13						13		
14				1.50 TSF	16/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 5/23/06

Geotechnical Engineering Department

Burns & McDonnell



Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-11**
Page	2 of 6
Date	3-10-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)	Sample or Box No.	Remarks
14–15	CLAY; gray, damp, stiff, high plasticity (CH).	▨	3/ 3/ 4	1.50 TSF	16/ 18	SS-5	N=7
15–17	with sand, stiff to very stiff, medium plasticity (CH).	▨		3.50 TSF	19/ 24	ST-1	
17–18.5		▨					
18.5–20	SILT; grayish brown, damp, medium, nonplastic (ML).	▨	3/ 3/ 6	1.25 TSF	18/ 18	SS-6	N=9
20–23.5	SAND; silt, fine grained, light brown, moist, very loose to loose, trace lignite (SP).						
23.5–25			2/ 2/ 2		14/ 18	SS-7	N=4
25–28.5	SAND; fine to medium grained, gray, wet, medium, trace coarse sand, trace fine gravel (SP).						▼ Water level @ 11:00 (3/13/2006)
28.5–30			5/ 7/ 7		16/ 18	SS-8	N=14
30–31							

Drilling Log, continued

	Boring Number **B-11**
Project Name **Iatan**	Page **3 of 6**
Project Number **41180-3.0209**	Date **3-10-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; fine to coarse grained, gray, wet, medium, trace fine gravel (SW).							Switch to mud rotary at 30 ft.
32						32		
33						33		
34			4/ 6/ 5		12/ 18	34	SS-9	
35						35		N=11
36						36		
37						37		
38						38		
39	SAND; fine to coarse grained, gray, wet, medium, with fine gravel (SW).		5/ 9/ 15		14/ 18	39	SS-10	N=24
40						40		
41						41		
42						42		
43						43		
44	SAND; fine to medium grained, gray, wet, medium, some coarase sand (SW).		5/ 15/ 13		15/ 18	44	SS-11	N=28
45						45		
46						46		
47	SAND; fine gravel, fine to coarse grained, gray, wet, dense (SWG).					47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

		Boring Number	**B-11**
Project Name	**Iatan**	Page	4 of 6
Project Number	41180-3.0209	Date	3-10-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; fine gravel, fine to coarse grained, gray, wet, dense (SWG).		15/ 24/ 17		18/ 18		SS-12	
49	SAND; fine grained, gray, wet, dense (SP).					49		N=41
50						50		
51						51		
	SAND; fine to coarse grained, gray, wet, medium, trace fine gravel (SW).							
52						52		
53			9/ 9/ 9		14/ 18	53	SS-13	
54						54		N=18
55						55		
56						56		
	SAND; fine grained, gray, wet, dense, laminated (SP).							
57						57		
58			9/ 15/ 16		11/ 18	58	SS-14	
59						59		N=31
60						60		
61						61		
	SAND; fine to coarse grained, gray, wet, medium (SW).							
62						62		
63			18/ 15/ 6		10/ 18	63	SS-15	
64						64		N= 21
65								16:25 Stop for today (3/10/2006).

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 5/23/06

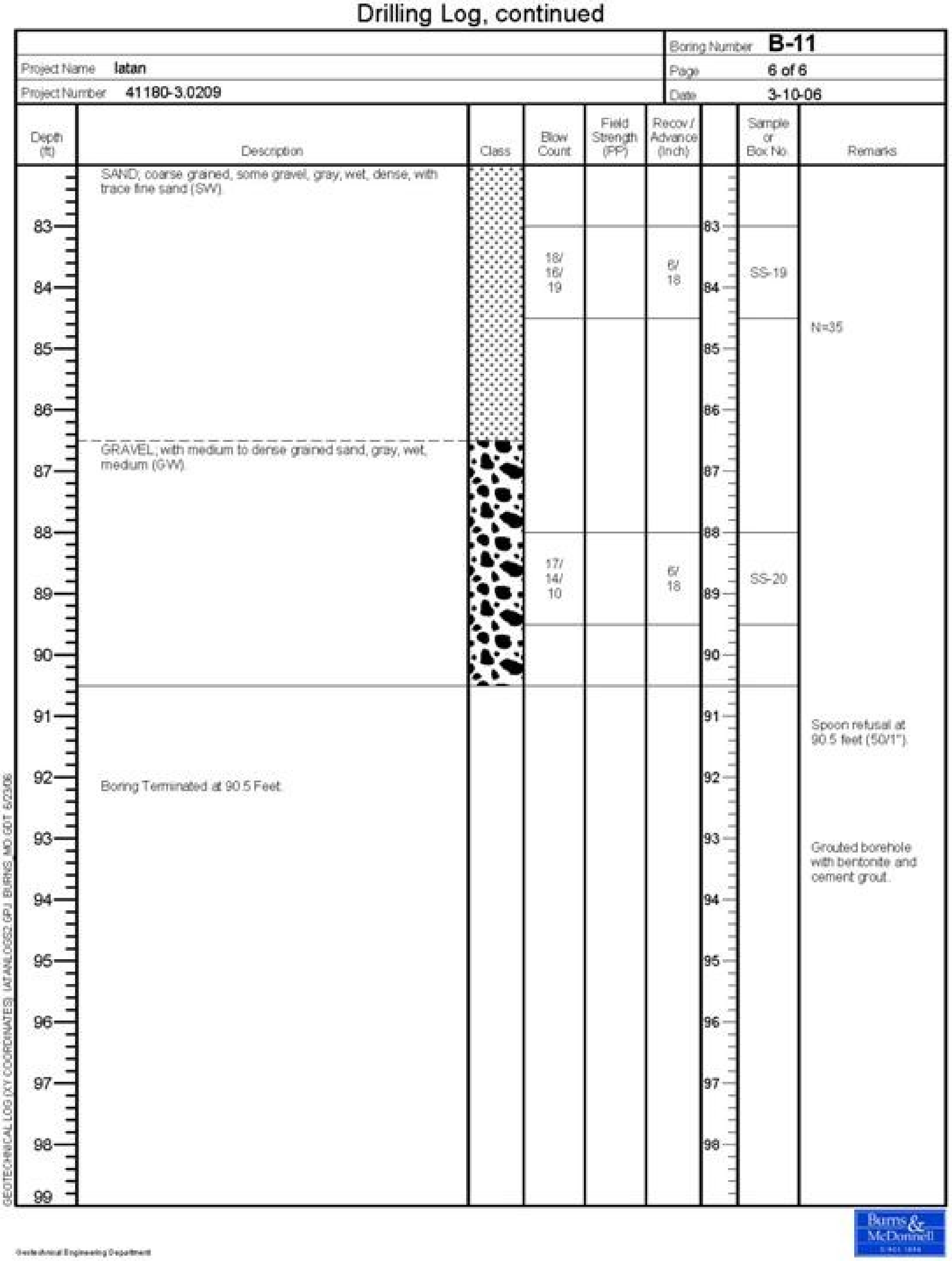
Drilling Log, continued

Project Name	Iatan	Boring Number	**B-11**
Project Number	41180-3.0209	Page	5 of 6
		Date	3-10-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; fine to coarse grained, gray, wet, medium (SW).							11:10 Begin on 3/13/2006.
66						66		
67						67		
68						68		
69	SAND; fine to coarse grained, gray, wet, dense to very dense (SW).		19/ 23/ 28		18/ 18	69	SS-16	
70						70		N=51
71						71		
72						72		
73	SAND; fine to coarse grained, gray, wet, medium, with coarse gravel zones (SW).					73		
74			15/ 12/ 7		9/ 18	74	SS-17	
75						75		N=19
76						76		
77	SAND; some gravel, fine to medium grained, gray, wet, medium to dense, gravel is fine grained (SW)					77		
78						78		
79			11/ 16/ 15		13/ 18	79	SS-18	
80						80		N=31
81						81		
82								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Drilling Log, continued



Project Name **Iatan**

Project Number **41180-3.0209**

Boring Number **B-11**

Page **6 of 6**

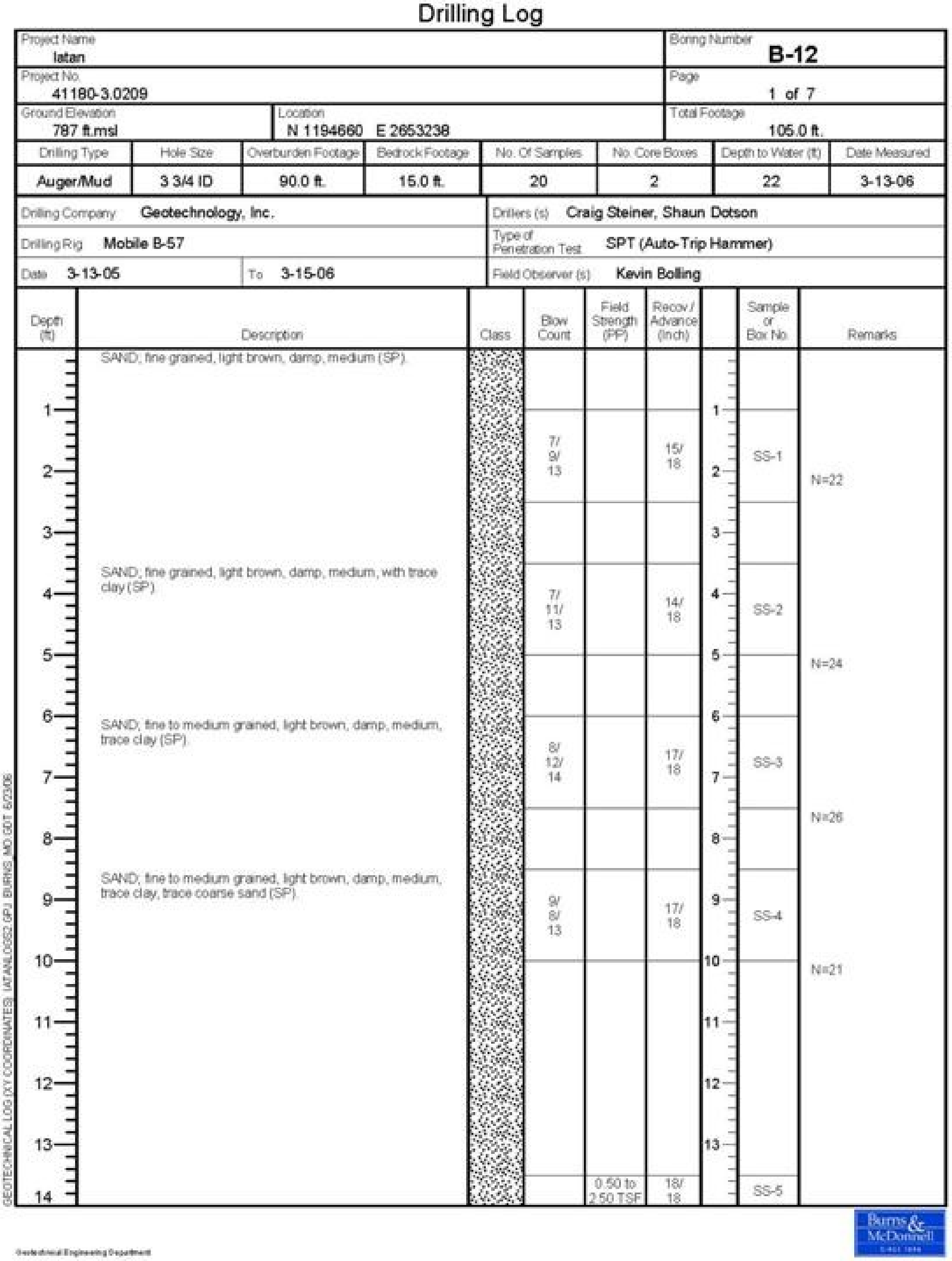
Date **3-10-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; coarse grained, some gravel, gray, wet, dense, with trace fine sand (SW).							
83 84			18/ 16/ 19		6/ 18	83 84	SS-19	N=35
85 86						85 86		
87	GRAVEL; with medium to dense grained sand, gray, wet, medium (GW).					87		
88 89			17/ 14/ 10		6/ 18	88 89	SS-20	
90						90		
91						91		Spoon refusal at 90.5 feet (50/1").
92	Boring Terminated at 90.5 Feet.					92		
93 94 95 96 97 98 99						93 94 95 96 97 98		Grouted borehole with bentonite and cement grout.

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell SINCE 1898



Drilling Log

Project Name Iatan		Boring Number **B-12**
Project No. 41180-3.0209		Page 1 of 7
Ground Elevation 787 ft.msl	Location N 1194660 E 2653238	Total Footage 105.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	90.0 ft.	15.0 ft.	20	2	22	3-13-06

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Craig Steiner, Shaun Dotson**

Drilling Rig **Mobile B-57**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **3-13-05** To **3-15-06**

Field Observer (s) **Kevin Bolling**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; fine grained, light brown, damp, medium (SP).	▨						
1 2		▨	7/ 9/ 13		15/ 18	1 2	SS-1	N=22
3		▨				3		
4	SAND; fine grained, light brown, damp, medium, with trace clay (SP).	▨	7/ 11/ 13		14/ 18	4	SS-2	
5		▨				5		N=24
6 7	SAND; fine to medium grained, light brown, damp, medium, trace clay (SP).	▨	8/ 12/ 14		17/ 18	6 7	SS-3	
8		▨				8		N=26
9	SAND; fine to medium grained, light brown, damp, medium, trace clay, trace coarse sand (SP).	▨	9/ 8/ 13		17/ 18	9	SS-4	
10 11 12 13		▨				10 11 12 13		N=21
14		▨		0.50 to 2.50 TSF	18/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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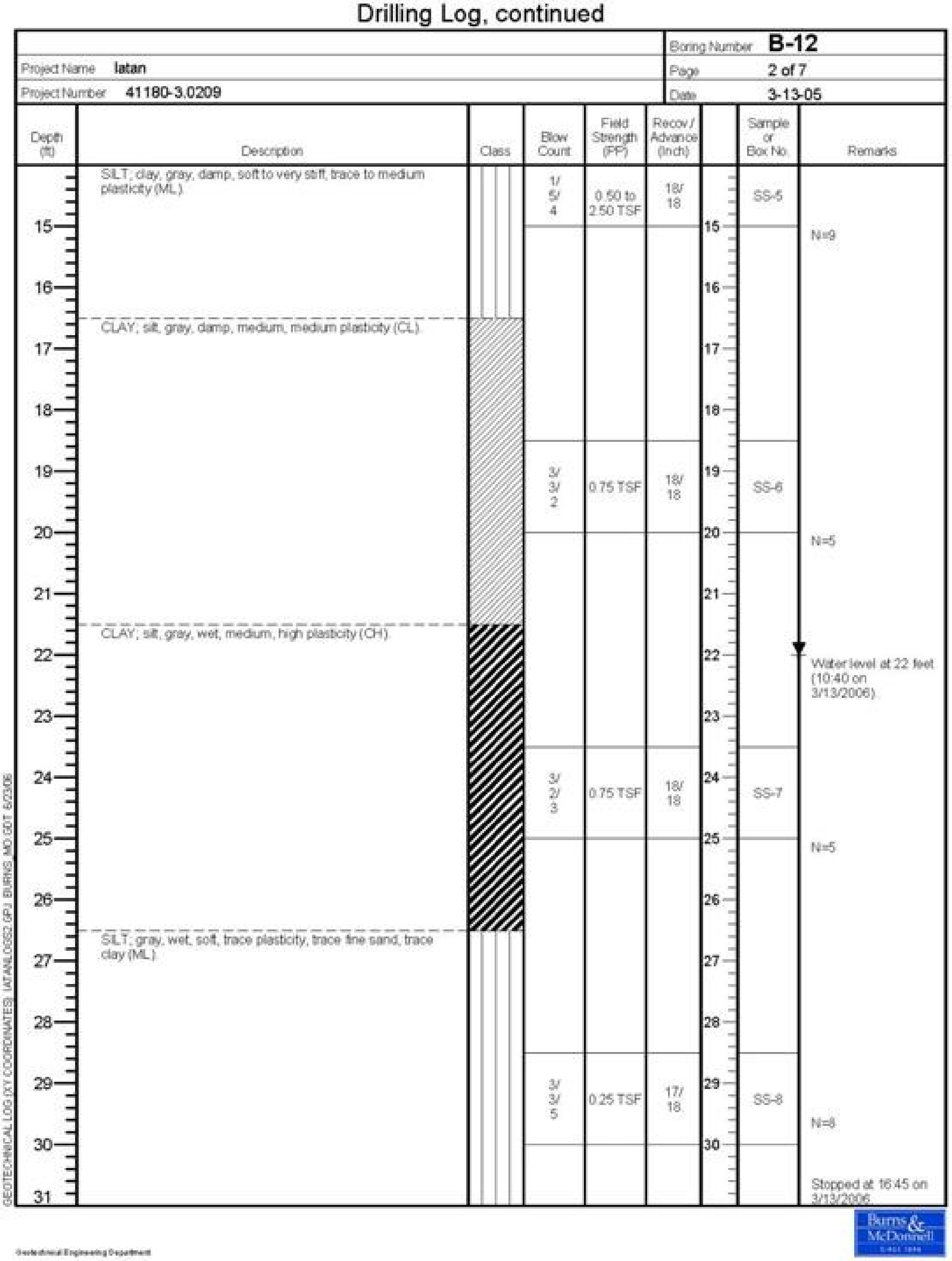

Drilling Log, continued

Boring Number **B-12**
Project Name **Iatan**
Page **2 of 7**
Project Number **41180-3.0209**
Date **3-13-05**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SILT; clay, gray, damp, soft to very stiff, trace to medium plasticity (ML).		1/ 5/ 4	0.50 to 2.50 TSF	18/ 18		SS-5	
15						15		N=9
16						16		
	CLAY; silt, gray, damp, medium, medium plasticity (CL).							
17						17		
18						18		
19			3/ 3/ 2	0.75 TSF	18/ 18	19	SS-6	
20						20		N=5
21						21		
	CLAY; silt, gray, wet, medium, high plasticity (CH).							
22						22		Water level at 22 feet (10:40 on 3/13/2006).
23						23		
24			3/ 2/ 3	0.75 TSF	18/ 18	24	SS-7	
25						25		N=5
26						26		
	SILT; gray, wet, soft, trace plasticity, trace fine sand, trace clay (ML).							
27						27		
28						28		
29			3/ 3/ 5	0.25 TSF	17/ 18	29	SS-8	
30						30		N=8
31								Stopped at 16:45 on 3/13/2006.

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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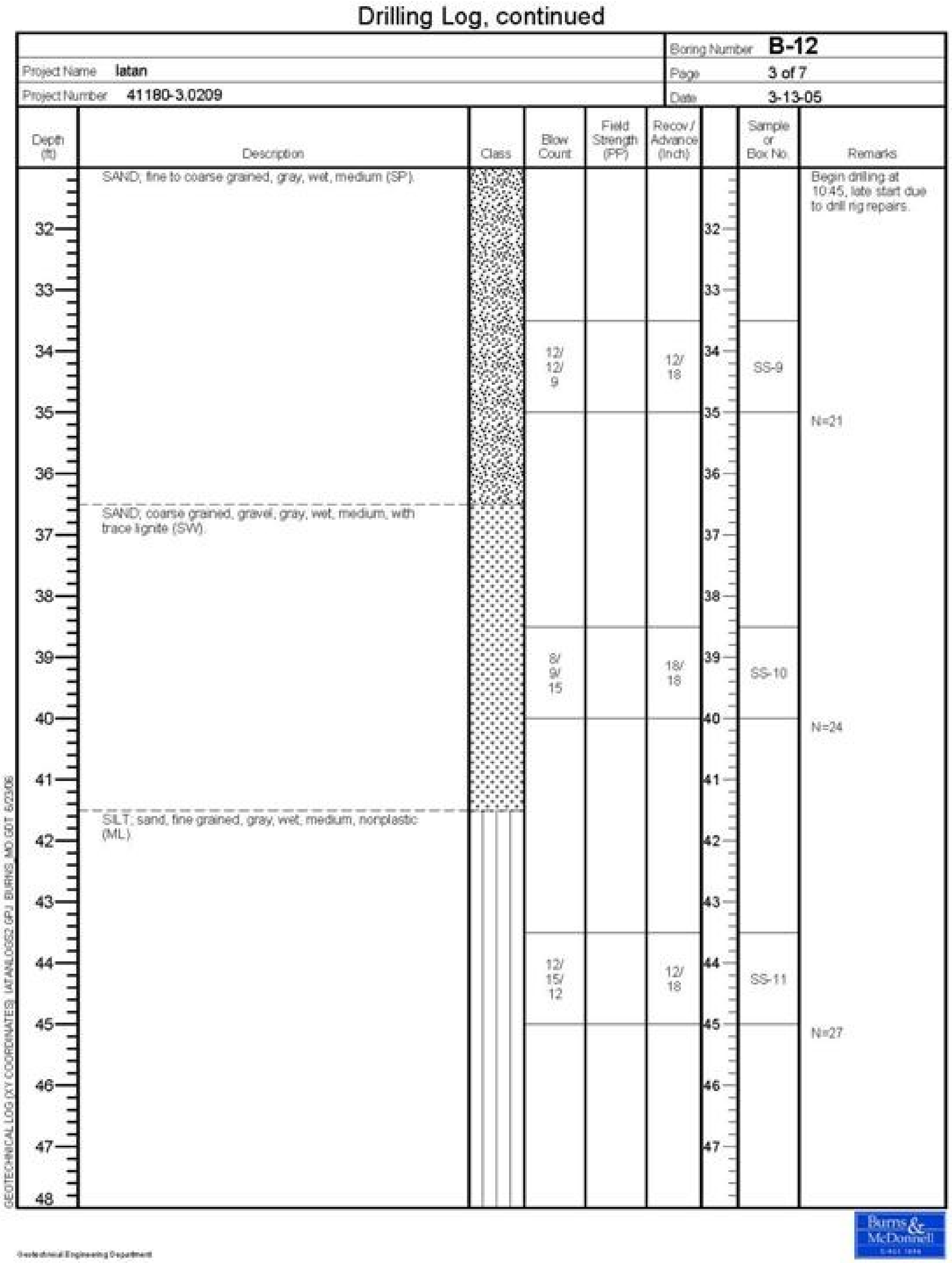


Drilling Log, continued

Project Name **Iatan**
Project Number **41180-3.0209**
Boring Number **B-12**
Page **3 of 7**
Date **3-13-05**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; fine to coarse grained, gray, wet, medium (SP).							Begin drilling at 10:45, late start due to drill rig repairs.
32						32		
33						33		
34			12/ 12/ 9		12/ 18	34	SS-9	
35						35		N=21
36						36		
37	SAND; coarse grained, gravel, gray, wet, medium, with trace lignite (SW).					37		
38						38		
39			8/ 9/ 15		18/ 18	39	SS-10	
40						40		N=24
41						41		
42	SILT; sand, fine grained, gray, wet, medium, nonplastic (ML).					42		
43						43		
44			12/ 15/ 12		12/ 18	44	SS-11	
45						45		N=27
46						46		
47						47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

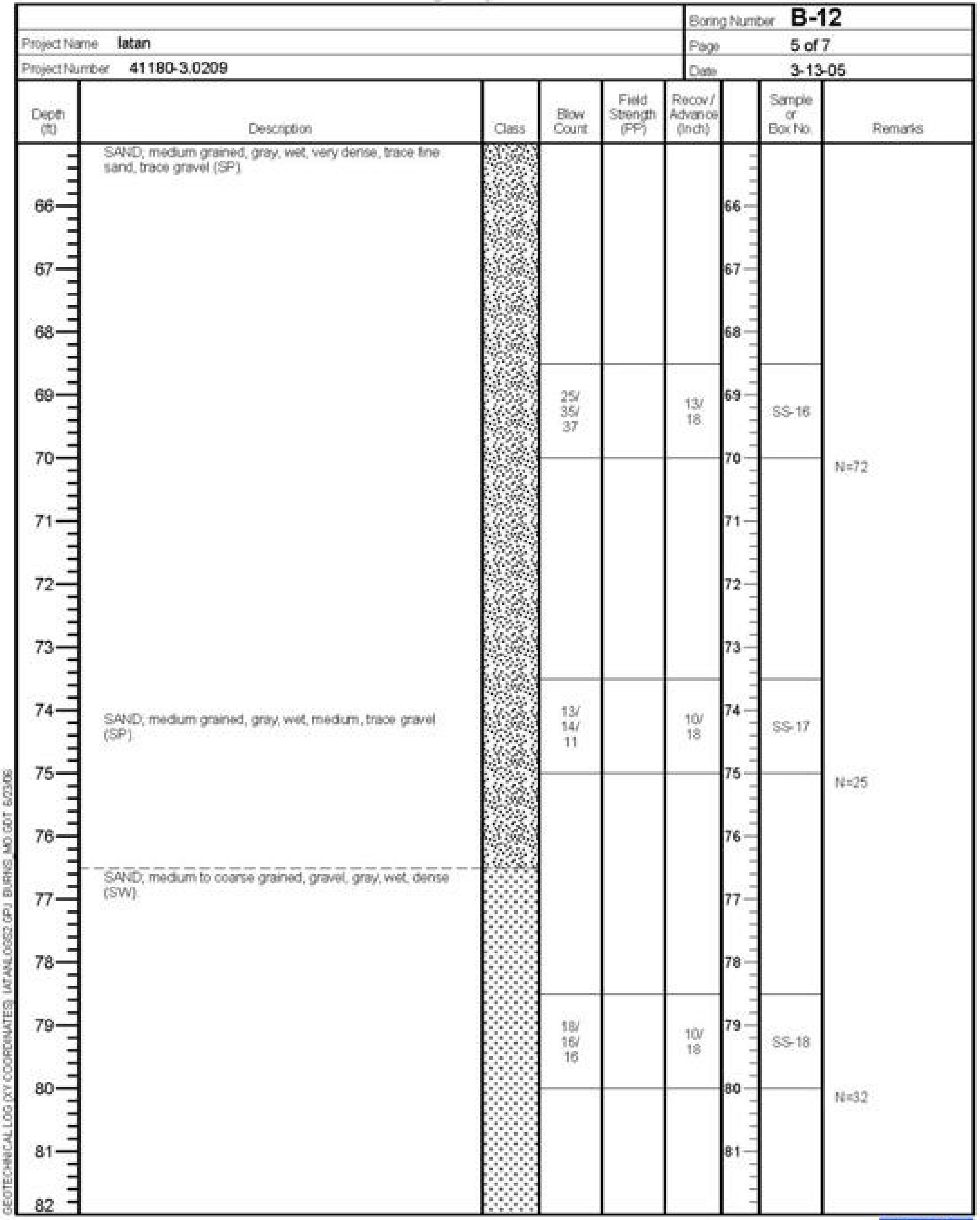
Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-12**
Page	4 of 7
Date	3-13-05

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SILT; fine sand, gray, wet, medium (ML).							
49			10/ 11/ 15		16/ 18	49	SS-12	
50						50		N=26
51						51		
52						52		
53						53		
54	SILT; trace coarse sand, gray, wet, very dense, nonplastic (ML).		16/ 19/ 33		16/ 18	54	SS-13	
55						55		N=52
56						56		
57	SAND; fine to coarse grained, gray, wet, medium, trace coarse gravel (SW).					57		
58						58		
59			16/ 7/ 10		15/ 18	59	SS-14	
60						60		N=17
61						61		
62						62		
63	SAND; fine to coarse grained, gray, wet, very dense (SW).					63		
64			17/ 25/ 30		16/ 18	64	SS-15	N=55
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MD.GDT 6/23/06

Drilling Log, continued

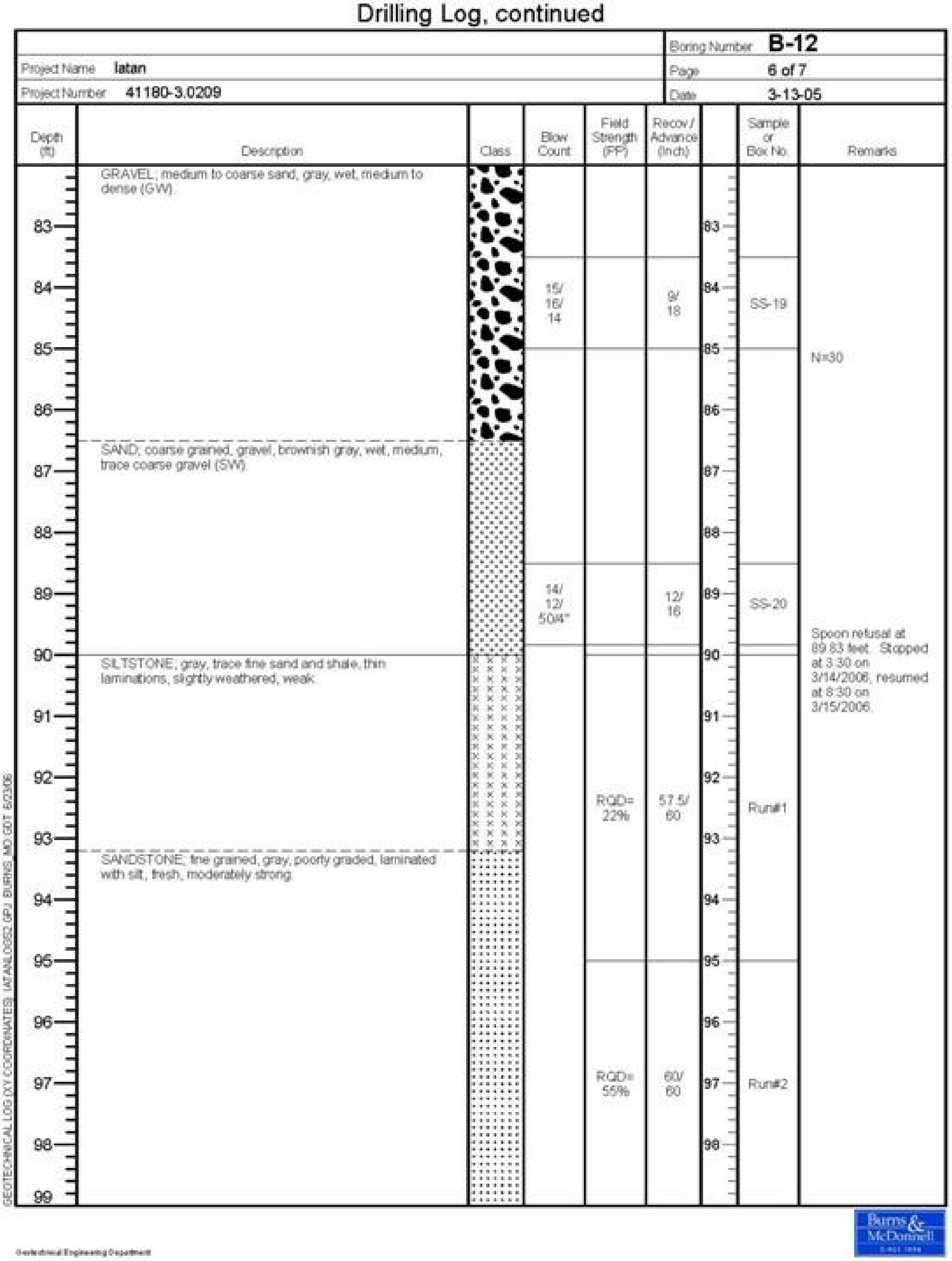


Project Name	**Iatan**
Project Number	**41180-3.0209**
Boring Number	**B-12**
Page	5 of 7
Date	3-13-05

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND; medium grained, gray, wet, very dense, trace fine sand, trace gravel (SP).							
66						66		
67						67		
68						68		
69			25/ 35/ 37		13/ 18	69	SS-16	
70						70		N=72
71						71		
72						72		
73						73		
74	SAND; medium grained, gray, wet, medium, trace gravel (SP).		13/ 14/ 11		10/ 18	74	SS-17	
75						75		N=25
76						76		
77	SAND; medium to coarse grained, gravel, gray, wet, dense (SW).					77		
78						78		
79			18/ 16/ 16		10/ 18	79	SS-18	
80						80		N=32
81						81		
82								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



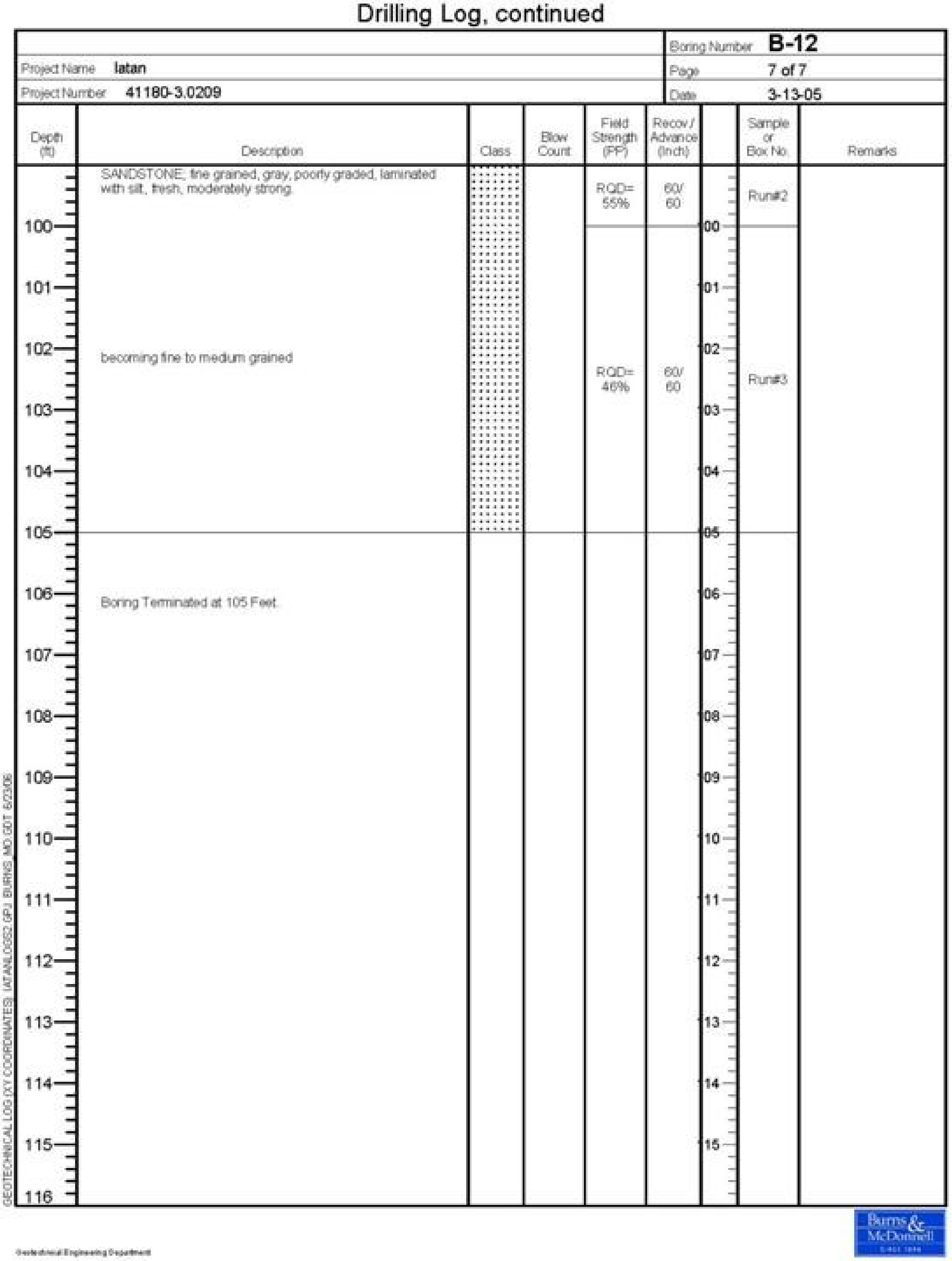
Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-12**
Page	6 of 7
Date	3-13-05

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	GRAVEL; medium to coarse sand, gray, wet, medium to dense (GW).							
83						83		
84			15/ 16/ 14		9/ 18	84	SS-19	
85						85		N=30
86						86		
87	SAND; coarse grained, gravel, brownish gray, wet, medium, trace coarse gravel (SW).					87		
88						88		
89			14/ 12/ 50/4"		12/ 16	89	SS-20	Spoon refusal at 89.83 feet. Stopped at 3:30 on 3/14/2006, resumed at 8:30 on 3/15/2006.
90	SILTSTONE; gray, trace fine sand and shale, thin laminations, slightly weathered, weak.					90		
91						91		
92				RQD= 22%	57.5/ 60	92	Run#1	
93	SANDSTONE; fine grained, gray, poorly graded, laminated with silt, fresh, moderately strong.					93		
94						94		
95						95		
96						96		
97				RQD= 55%	60/ 60	97	Run#2	
98						98		
99								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



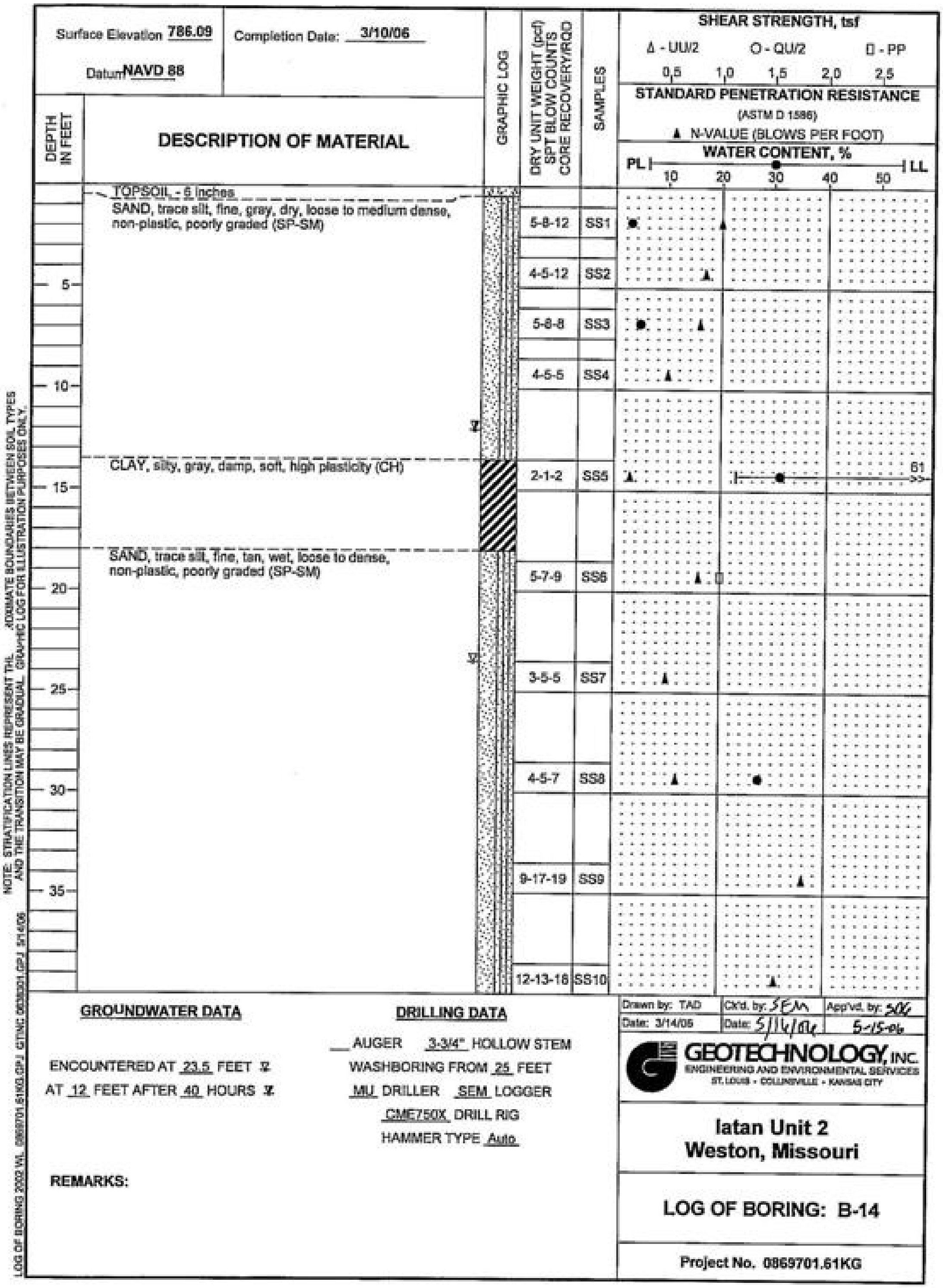
Drilling Log, continued

		Boring Number	**B-12**
Project Name	**Iatan**	Page	7 of 7
Project Number	41180-3.0209	Date	3-13-05

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SANDSTONE; fine grained, gray, poorly graded, laminated with silt, fresh, moderately strong.	░		RQD= 55%	60/ 60		Run#2	
100		░				00		
101		░				01		
102	becoming fine to medium grained	░		RQD= 46%	60/ 60	02	Run#3	
103		░				03		
104		░				04		
105						05		
106	Boring Terminated at 105 Feet.					06		
107						07		
108						08		
109						09		
110						10		
111						11		
112						12		
113						13		
114						14		
115						15		
116								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 5/23/06

Geotechnical Engineering Department



Surface Elevation 786.09

Datum NAVD 88

Completion Date: 3/10/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL - 6 inches		
	SAND, trace silt, fine, gray, dry, loose to medium dense, non-plastic, poorly graded (SP-SM)	5-8-12	SS1
5		4-5-12	SS2
		5-8-8	SS3
10		4-5-5	SS4
	CLAY, silty, gray, damp, soft, high plasticity (CH)	2-1-2	SS5
15			
	SAND, trace silt, fine, tan, wet, loose to dense, non-plastic, poorly graded (SP-SM)	5-7-9	SS6
20			
25		3-5-5	SS7
30		4-5-7	SS8
35		9-17-19	SS9
		12-13-16	SS10

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 23.5 FEET ▽

AT 12 FEET AFTER 40 HOURS ▼

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM

WASHBORING FROM 25 FEET

MU DRILLER SEM LOGGER

CME750X DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SKG
Date: 3/14/06	Date: 5/14/06	5-15-06

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-14

Project No. 0869701.61KG

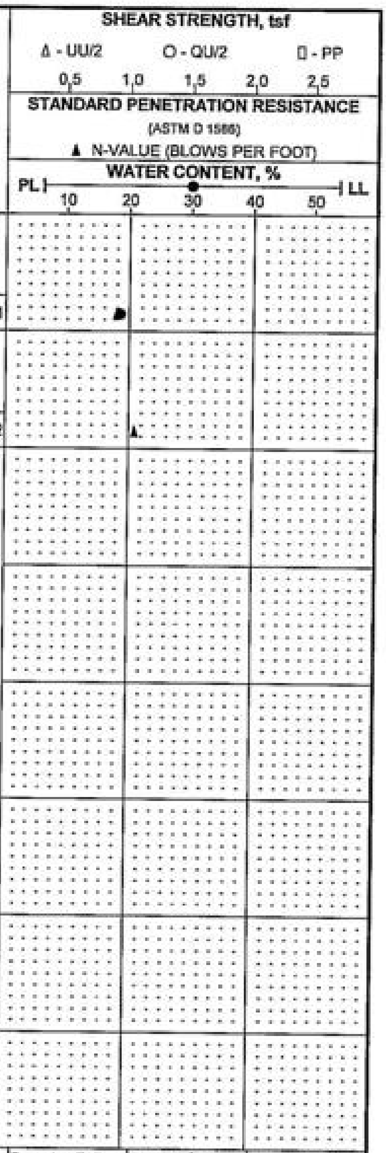
LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0636001.GPJ 5/14/06

Surface Elevation 786.09

Datum NAVD 88

Completion Date: 3/10/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, fine, tan, wet, loose to dense, non-plastic, poorly graded (SP-SM) *(continued)*			
	coarse, trace gravel			
45			6-8-10	SS11
			8-9-12	SS12
50	Boring terminated at 50 feet			
55				
60				
65				
70				
75				



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 23.5 FEET ⊻
AT 12 FEET AFTER 40 HOURS ⊻

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 25 FEET
MU DRILLER SEM LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SOG
Date: 3/14/06	Date: 5/15/06	5-15-06

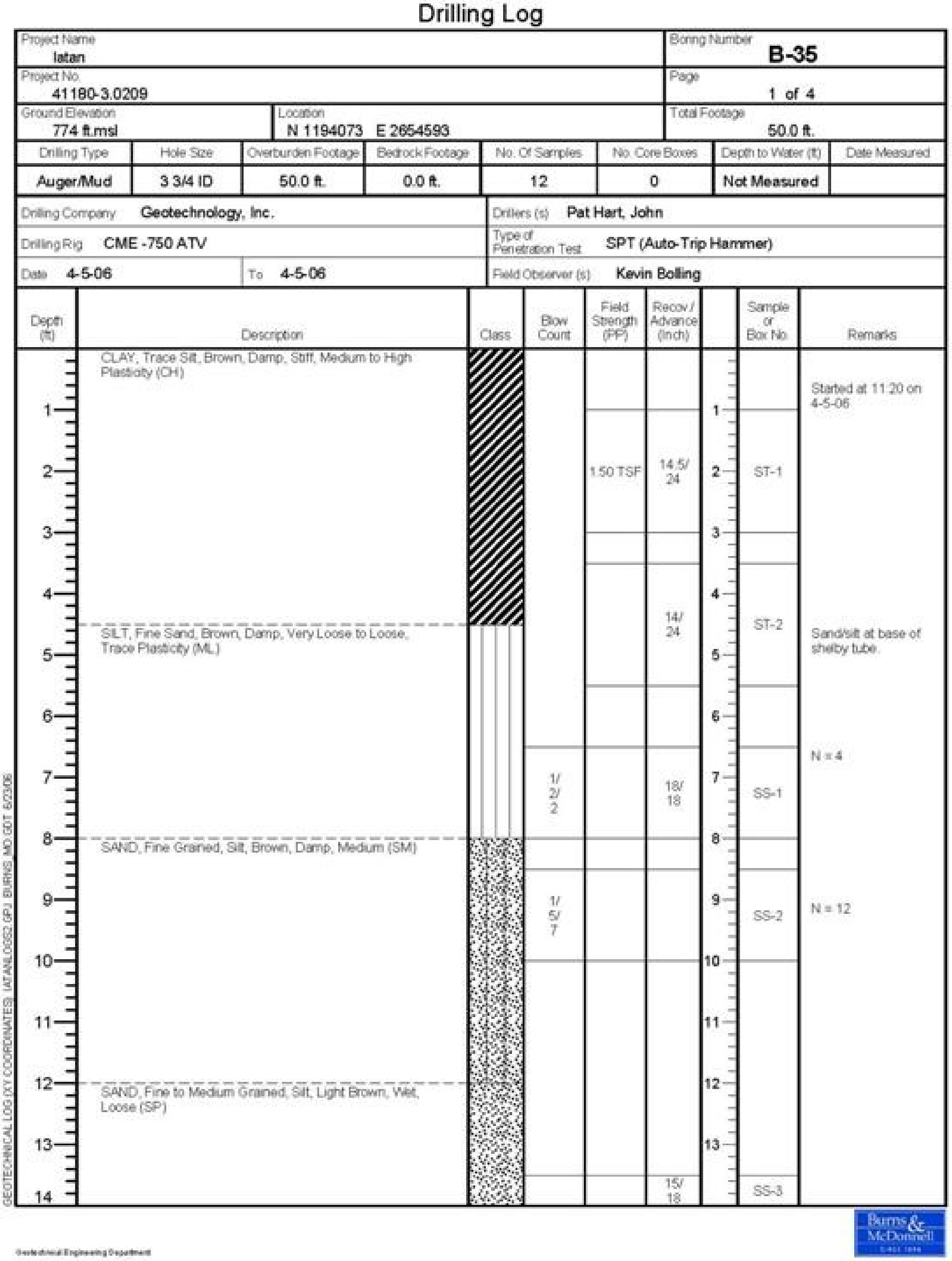
GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B-14

Project No. 0869701.61KG

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0606301.GPJ 5/14/06



Drilling Log

Project Name: Iatan
Boring Number: **B-35**
Project No.: 41180-3.0209
Page: 1 of 4
Ground Elevation: 774 ft.msl
Location: N 1194073 E 2654593
Total Footage: 50.0 ft.

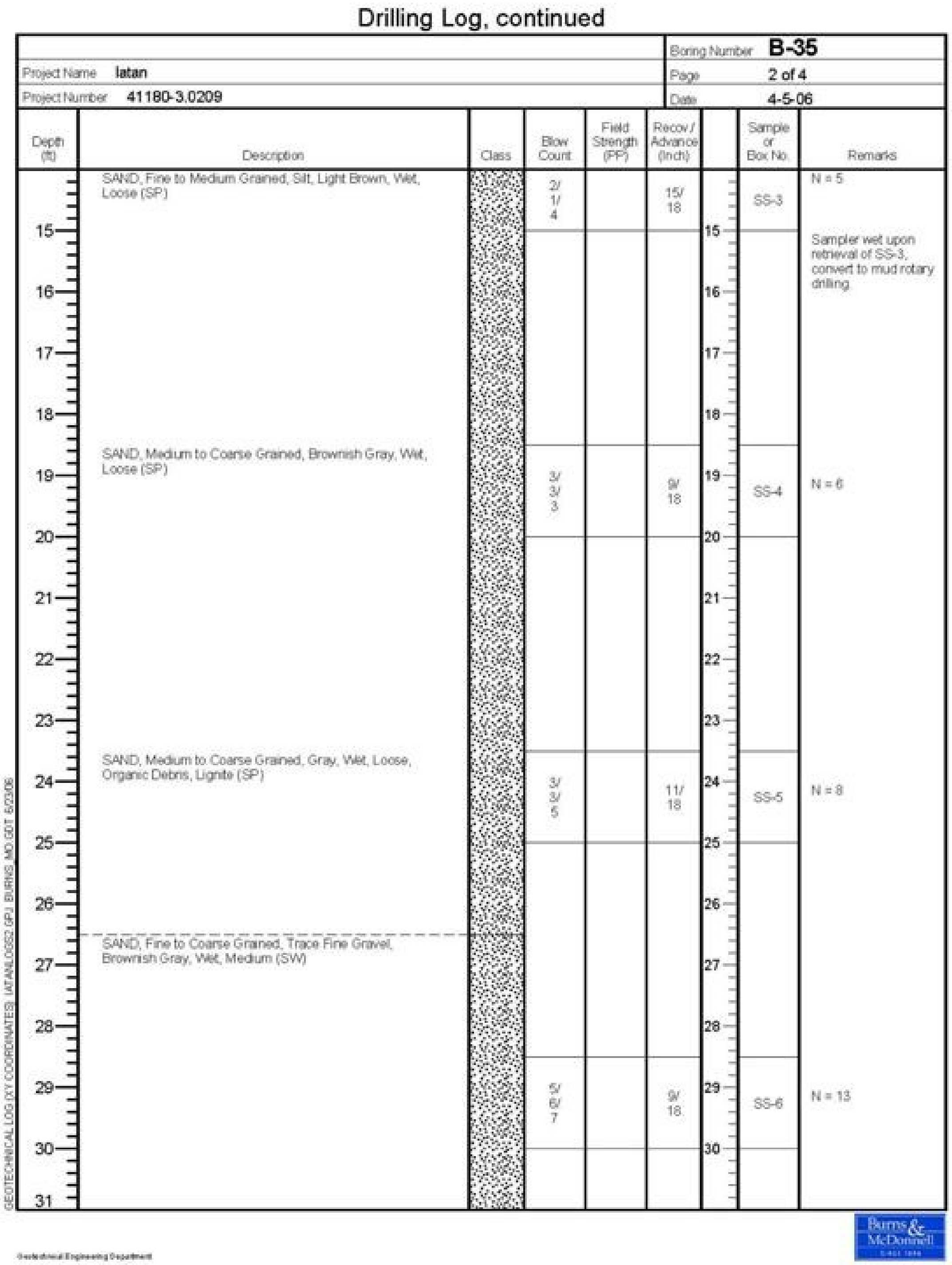
Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	50.0 ft.	0.0 ft.	12	0	Not Measured	

Drilling Company: Geotechnology, Inc.
Drillers (s): Pat Hart, John
Drilling Rig: CME -750 ATV
Type of Penetration Test: SPT (Auto-Trip Hammer)
Date: 4-5-06 To: 4-5-06
Field Observer (s): Kevin Bolling

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
0	CLAY, Trace Silt, Brown, Damp, Stiff, Medium to High Plasticity (CH)	▨						Started at 11:20 on 4-5-06
2				1.50 TSF	14.5/ 24	2	ST-1	
4.5	SILT, Fine Sand, Brown, Damp, Very Loose to Loose, Trace Plasticity (ML)	▥			14/ 24		ST-2	Sand/silt at base of shelby tube.
7			1/ 2/ 2		18/ 18	7	SS-1	N = 4
8	SAND, Fine Grained, Silt, Brown, Damp, Medium (SM)	░				8		
9			1/ 5/ 7			9	SS-2	N = 12
12	SAND, Fine to Medium Grained, Silt, Light Brown, Wet, Loose (SP)	░				12		
14					15/ 18		SS-3	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log, continued

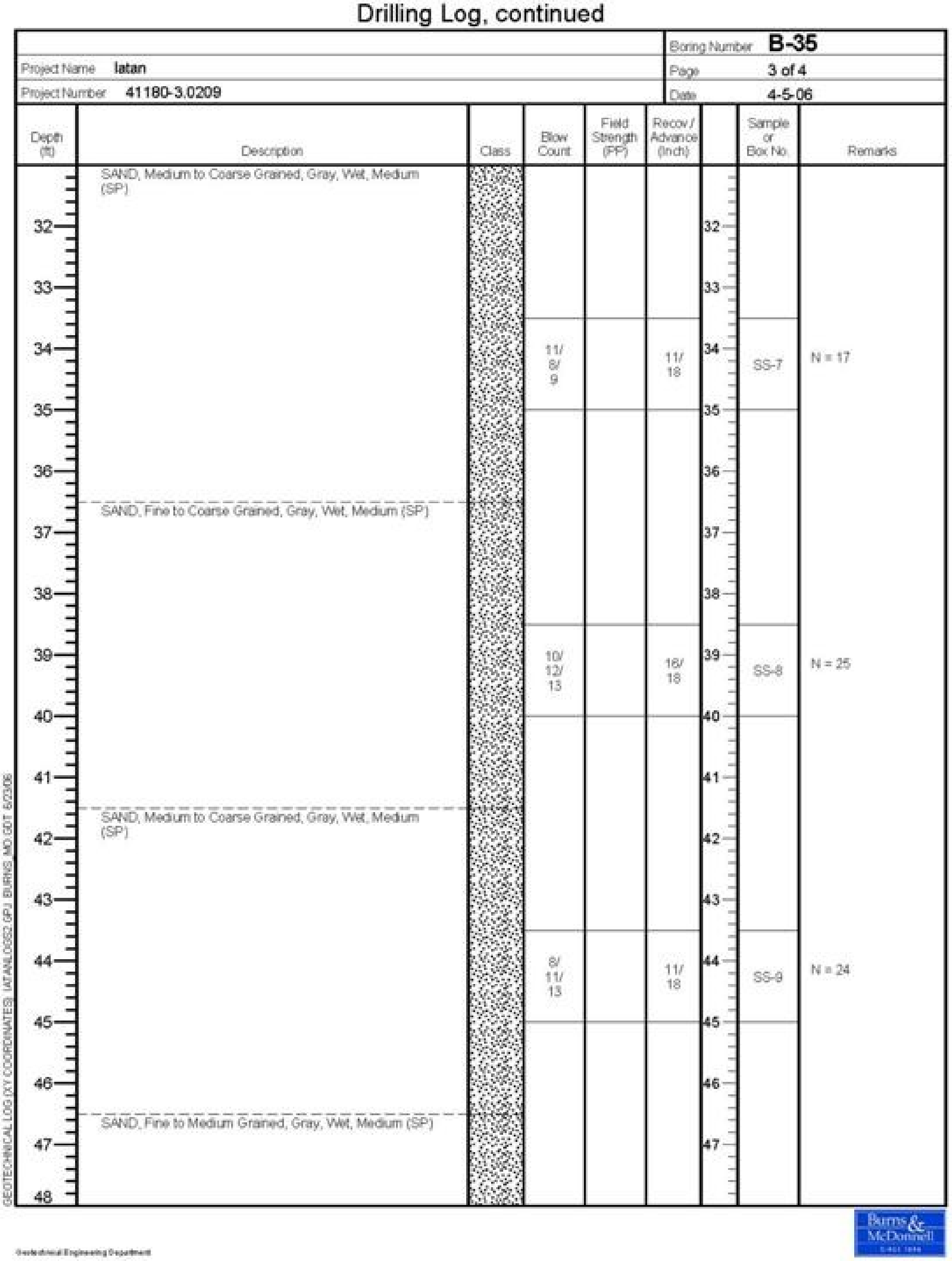
		Boring Number	**B-35**
Project Name	**Iatan**	Page	2 of 4
Project Number	**41180-3.0209**	Date	4-5-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Silt, Light Brown, Wet, Loose (SP)		2/ 1/ 4		15/ 18		SS-3	N = 5
15						15		Sampler wet upon retrieval of SS-3, convert to mud rotary drilling.
16						16		
17						17		
18	SAND, Medium to Coarse Grained, Brownish Gray, Wet, Loose (SP)					18		
19			3/ 3/ 3		9/ 18	19	SS-4	N = 6
20						20		
21						21		
22						22		
23	SAND, Medium to Coarse Grained, Gray, Wet, Loose, Organic Debris, Lignite (SP)					23		
24			3/ 3/ 5		11/ 18	24	SS-5	N = 8
25						25		
26	SAND, Fine to Coarse Grained, Trace Fine Gravel, Brownish Gray, Wet, Medium (SW)					26		
27						27		
28						28		
29			5/ 6/ 7		9/ 18	29	SS-6	N = 13
30						30		
31								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell
SINCE 1898



Drilling Log, continued

		Boring Number	**B-35**
Project Name	Iatan	Page	3 of 4
Project Number	41180-3.0209	Date	4-5-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Medium to Coarse Grained, Gray, Wet, Medium (SP)							
32						32		
33						33		
34			11/ 8/ 9		11/ 18	34	SS-7	N = 17
35						35		
36						36		
	SAND, Fine to Coarse Grained, Gray, Wet, Medium (SP)							
37						37		
38						38		
39			10/ 12/ 13		16/ 18	39	SS-8	N = 25
40						40		
41						41		
	SAND, Medium to Coarse Grained, Gray, Wet, Medium (SP)							
42						42		
43						43		
44			8/ 11/ 13		11/ 18	44	SS-9	N = 24
45						45		
46						46		
	SAND, Fine to Medium Grained, Gray, Wet, Medium (SP)							
47						47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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Drilling Log, continued

		Boring Number	**B-35**
Project Name	**Iatan**	Page	4 of 4
Project Number	41180-3.0209	Date	4-5-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Gray, Wet, Medium (SP)	▨						
49		▨	11/ 13/ 14		13/ 18	49	SS-10	N = 27
50	Boring Terminated at 50 Feet					50		15:00 End at 50 feet bgs.
51						51		
52						52		
53						53		
54						54		
55						55		
56						56		
57						57		
58						58		
59						59		
60						60		
61						61		
62						62		
63						63		
64						64		
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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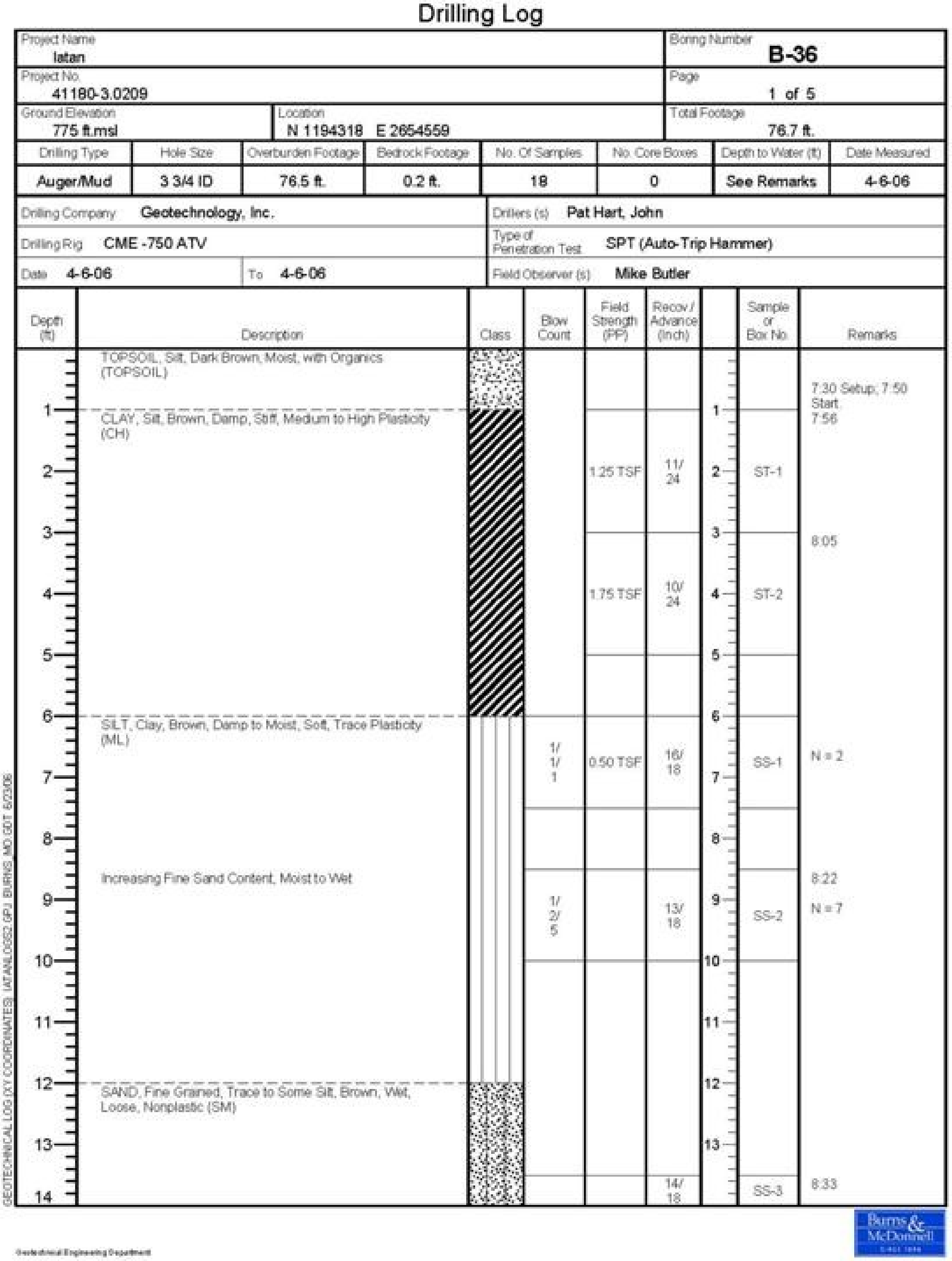


Drilling Log

Project Name: Iatan	Boring Number: **B-36**
Project No.: 41180-3.0209	Page: 1 of 5
Ground Elevation: 775 ft.msl Location: N 1194318 E 2654559	Total Footage: 76.7 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	76.5 ft.	0.2 ft.	18	0	See Remarks	4-6-06

Drilling Company: **Geotechnology, Inc.**

Drillers (s): **Pat Hart, John**

Drilling Rig: **CME -750 ATV**

Type of Penetration Test: **SPT (Auto-Trip Hammer)**

Date: **4-6-06** To: **4-6-06**

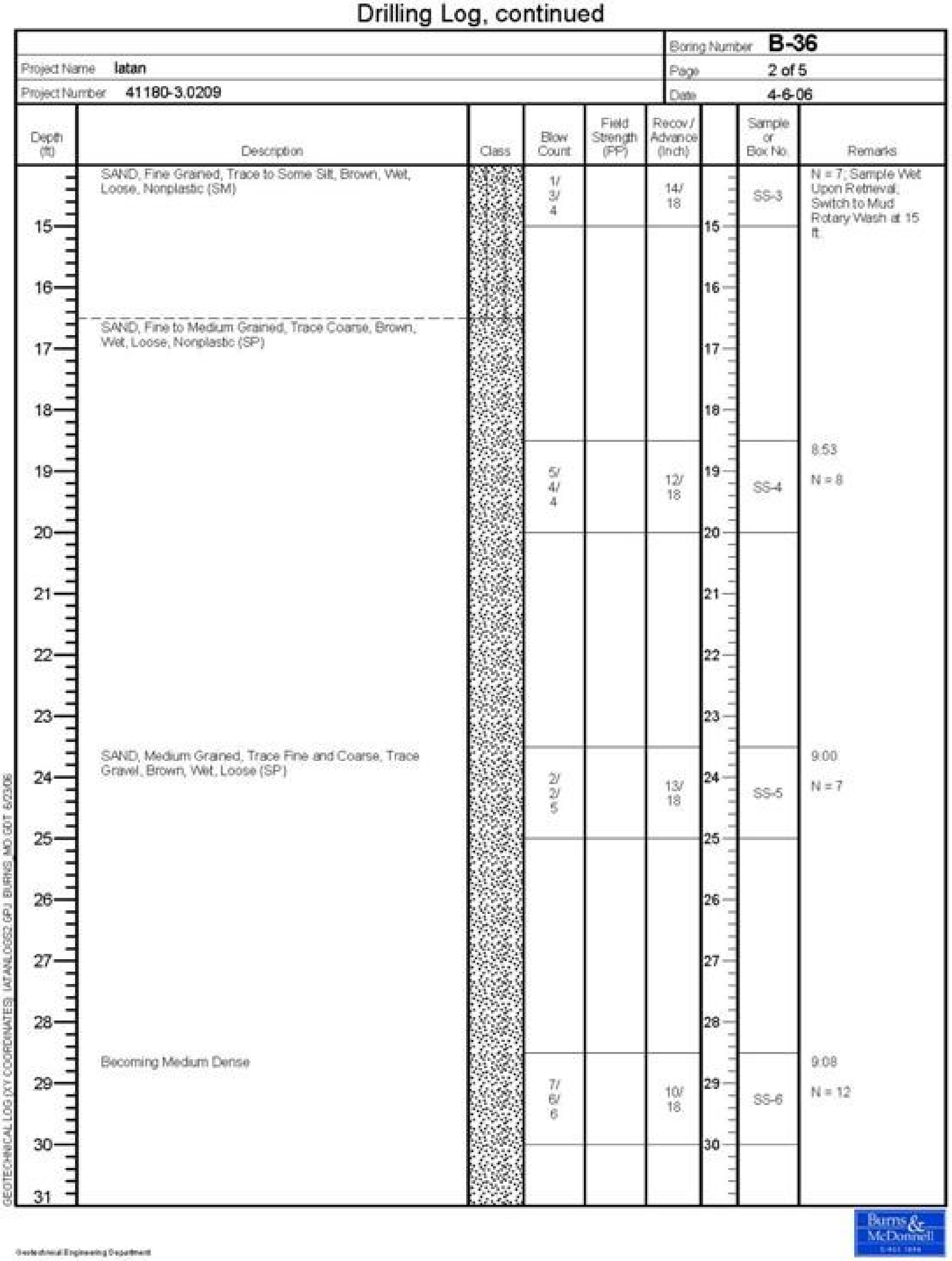
Field Observer (s): **Mike Butler**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	TOPSOIL, Silt, Dark Brown, Moist, with Organics (TOPSOIL)							7:30 Setup; 7:50 Start 7:56
1	CLAY, Silt, Brown, Damp, Stiff, Medium to High Plasticity (CH)					1		
2				1.25 TSF	11/ 24	2	ST-1	
3						3		8:05
4				1.75 TSF	10/ 24	4	ST-2	
5						5		
6	SILT, Clay, Brown, Damp to Moist, Soft, Trace Plasticity (ML)		1/ 1/ 1	0.50 TSF	16/ 18	6	SS-1	N = 2
7						7		
8	Increasing Fine Sand Content, Moist to Wet					8		8:22
9			1/ 2/ 5		13/ 18	9	SS-2	N = 7
10						10		
11						11		
12	SAND, Fine Grained, Trace to Some Silt, Brown, Wet, Loose, Nonplastic (SM)					12		
13					14/ 18	13	SS-3	8:33
14								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell



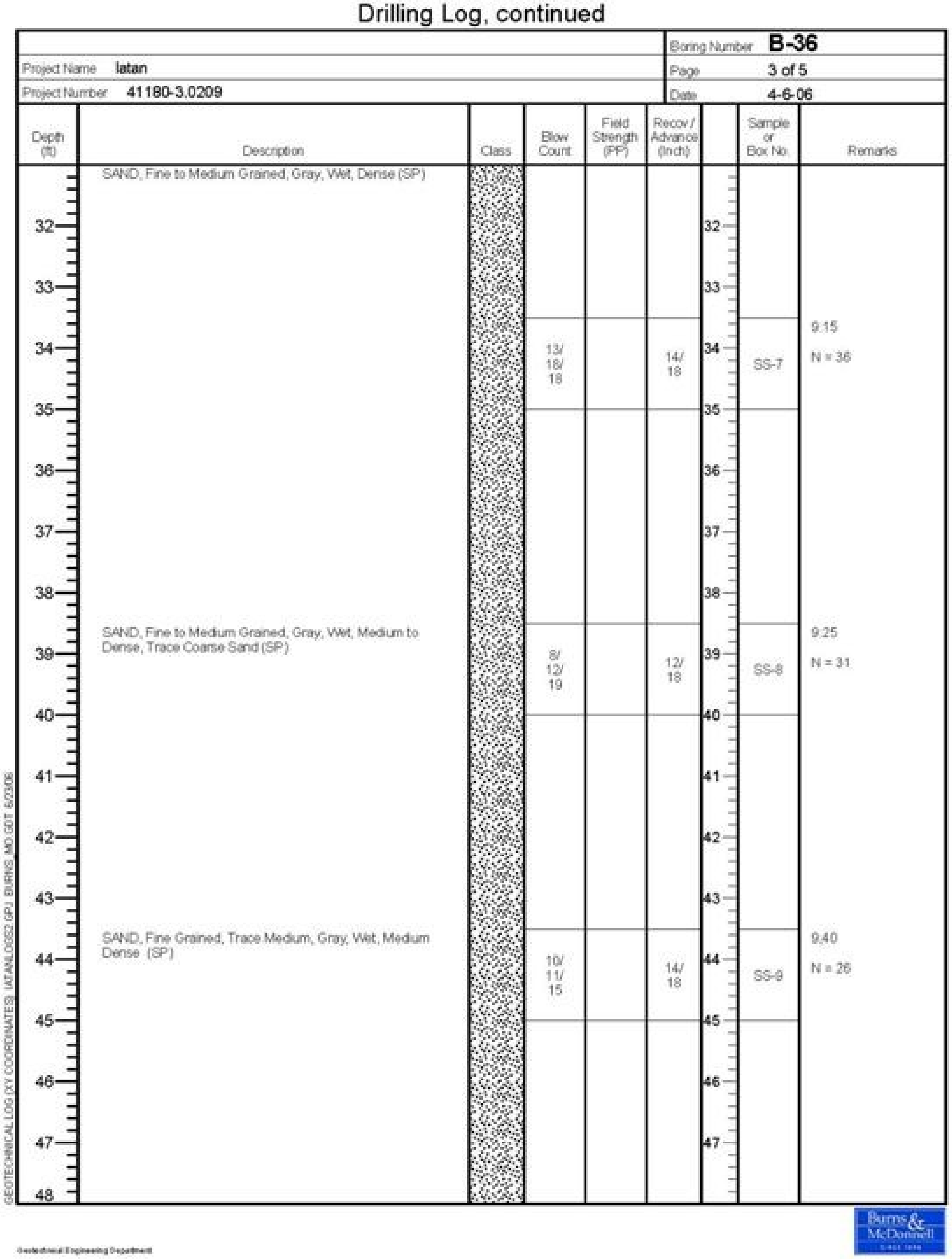
Drilling Log, continued

Boring Number **B-36**
Project Name **Iatan**
Page 2 of 5
Project Number 41180-3.0209
Date 4-6-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Trace to Some Silt, Brown, Wet, Loose, Nonplastic (SM)		1/ 3/ 4		14/ 18		SS-3	N = 7; Sample Wet Upon Retrieval; Switch to Mud Rotary Wash at 15 ft.
	SAND, Fine to Medium Grained, Trace Coarse, Brown, Wet, Loose, Nonplastic (SP)							
			5/ 4/ 4		12/ 18		SS-4	8:53 N = 8
	SAND, Medium Grained, Trace Fine and Coarse, Trace Gravel, Brown, Wet, Loose (SP)		2/ 2/ 5		13/ 18		SS-5	9:00 N = 7
	Becoming Medium Dense		7/ 6/ 6		10/ 18		SS-6	9:08 N = 12

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log, continued

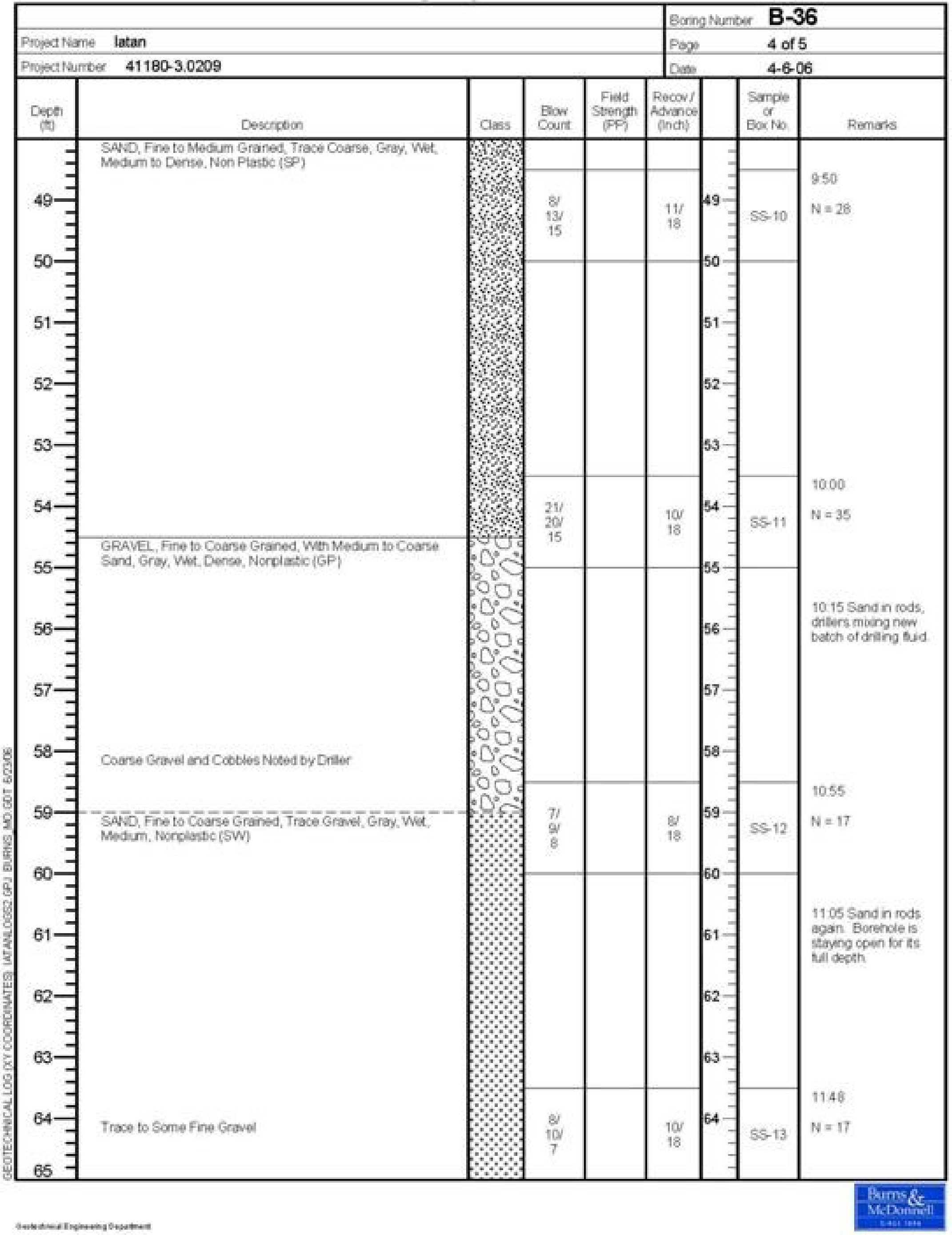
Boring Number **B-36**
Project Name **Iatan**
Page 3 of 5
Project Number 41180-3.0209
Date 4-6-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Gray, Wet, Dense (SP)							
32						32		
33						33		
34			13/ 18/ 18		14/ 18	34	SS-7	9:15 N = 36
35						35		
36						36		
37						37		
38						38		
39	SAND, Fine to Medium Grained, Gray, Wet, Medium to Dense, Trace Coarse Sand (SP)		8/ 12/ 19		12/ 18	39	SS-8	9:25 N = 31
40						40		
41						41		
42						42		
43						43		
44	SAND, Fine Grained, Trace Medium, Gray, Wet, Medium Dense (SP)		10/ 11/ 15		14/ 18	44	SS-9	9:40 N = 26
45						45		
46						46		
47						47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

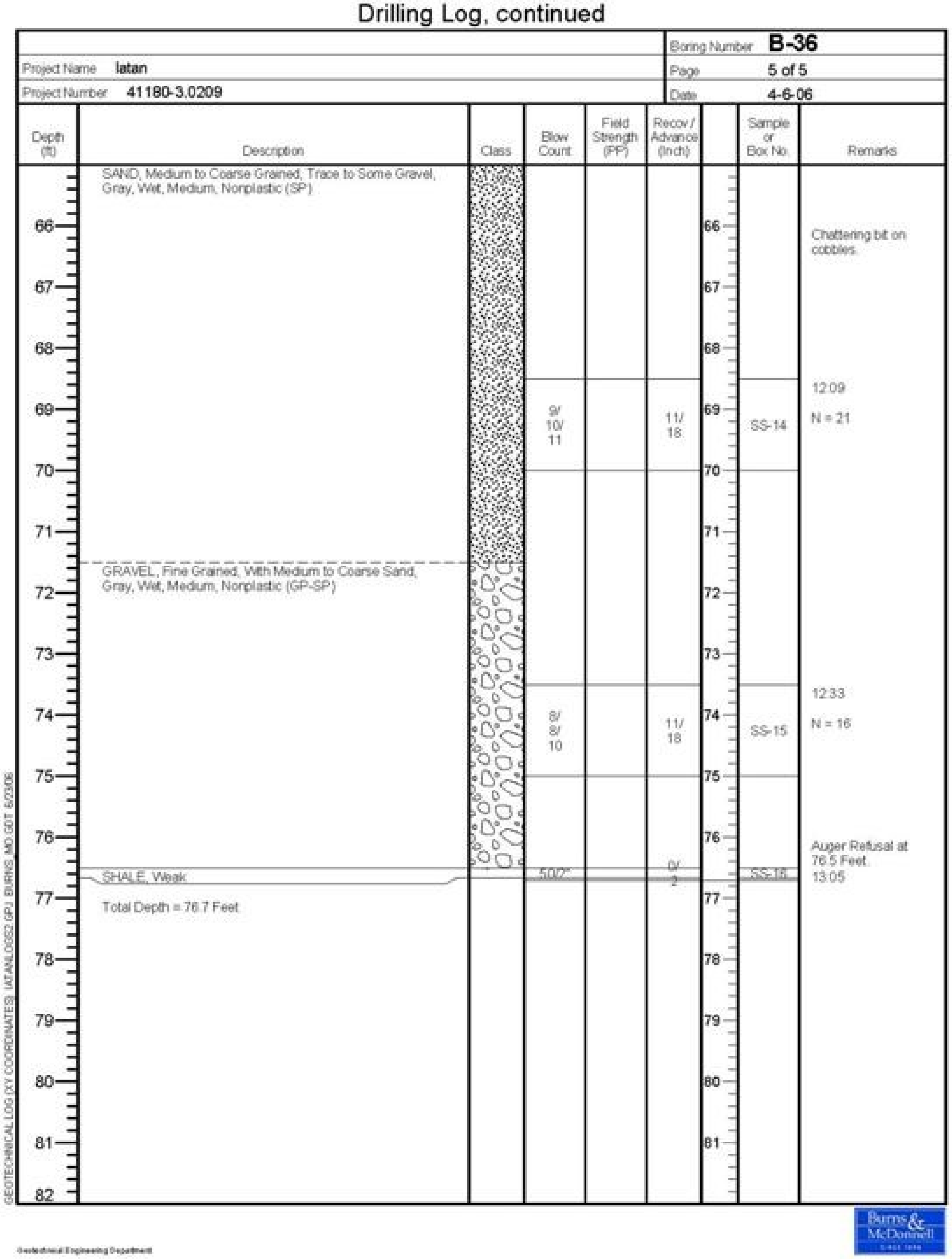


Project Name	Iatan	Boring Number	**B-36**
		Page	4 of 5
Project Number	41180-3.0209	Date	4-6-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Trace Coarse, Gray, Wet, Medium to Dense, Non Plastic (SP)							
49			8/ 13/ 15		11/ 18	49	SS-10	9:50 N = 28
50						50		
51						51		
52						52		
53						53		
54			21/ 20/ 15		10/ 18	54	SS-11	10:00 N = 35
	GRAVEL, Fine to Coarse Grained, With Medium to Coarse Sand, Gray, Wet, Dense, Nonplastic (GP)							
55						55		
56						56		10:15 Sand in rods, drillers mixing new batch of drilling fluid.
57						57		
58	Coarse Gravel and Cobbles Noted by Driller					58		
59	SAND, Fine to Coarse Grained, Trace Gravel, Gray, Wet, Medium, Nonplastic (SW)		7/ 9/ 8		8/ 18	59	SS-12	10:55 N = 17
60						60		
61						61		11:05 Sand in rods again. Borehole is staying open for its full depth.
62						62		
63						63		
64	Trace to Some Fine Gravel		8/ 10/ 7		10/ 18	64	SS-13	11:48 N = 17
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log, continued

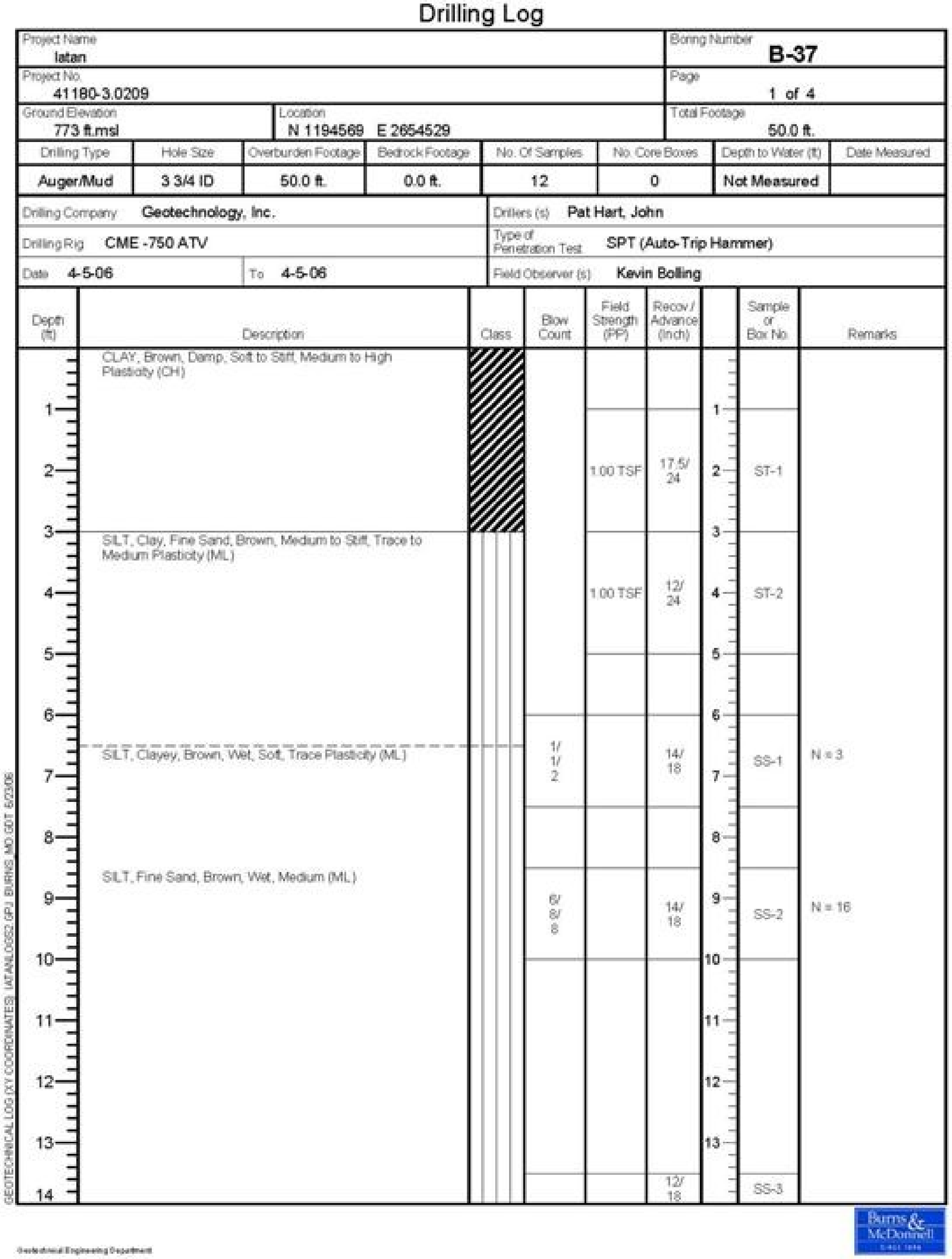
Project Name **Iatan**
Project Number **41180-3.0209**

Boring Number **B-36**
Page **5 of 5**
Date **4-6-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Medium to Coarse Grained, Trace to Some Gravel, Gray, Wet, Medium, Nonplastic (SP)							
66						66		Chattering bit on cobbles.
67						67		
68						68		12:09
69			9/ 10/ 11		11/ 18	69	SS-14	N = 21
70						70		
71						71		
72	GRAVEL, Fine Grained, With Medium to Coarse Sand, Gray, Wet, Medium, Nonplastic (GP-SP)					72		
73						73		12:33
74			8/ 8/ 10		11/ 18	74	SS-15	N = 16
75						75		
76						76		Auger Refusal at 76.5 Feet.
	SHALE, Weak		50/2"		0/ 2		SS-16	13:05
77	Total Depth = 76.7 Feet					77		
78						78		
79						79		
80						80		
81						81		
82								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log

Project Name Iatan			Boring Number **B-37**
Project No. 41180-3.0209			Page 1 of 4
Ground Elevation 773 ft.msl	Location N 1194569 E 2654529		Total Footage 50.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	50.0 ft.	0.0 ft.	12	0	Not Measured	

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Pat Hart, John**

Drilling Rig **CME -750 ATV**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **4-5-06** To **4-5-06**

Field Observer (s) **Kevin Bolling**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
0	CLAY, Brown, Damp, Soft to Stiff, Medium to High Plasticity (CH)	▨						
2				1.00 TSF	17.5/ 24	2	ST-1	
3	SILT, Clay, Fine Sand, Brown, Medium to Stiff, Trace to Medium Plasticity (ML)							
4				1.00 TSF	12/ 24	4	ST-2	
6.5	SILT, Clayey, Brown, Wet, Soft, Trace Plasticity (ML)		1/ 1/ 2		14/ 18	7	SS-1	N = 3
8.7	SILT, Fine Sand, Brown, Wet, Medium (ML)							
9			6/ 8/ 8		14/ 18	9	SS-2	N = 16
14					12/ 18		SS-3	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

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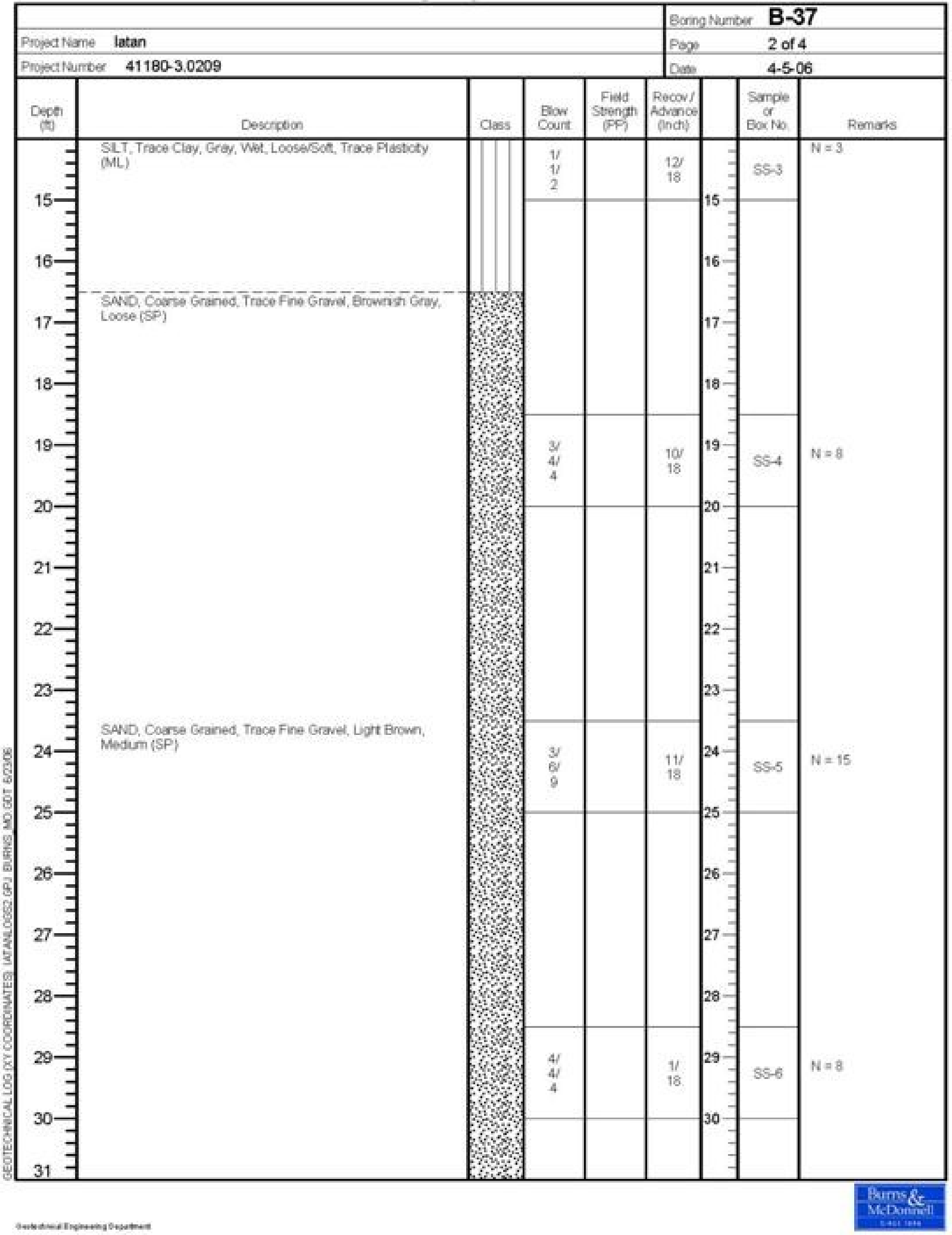
Drilling Log, continued

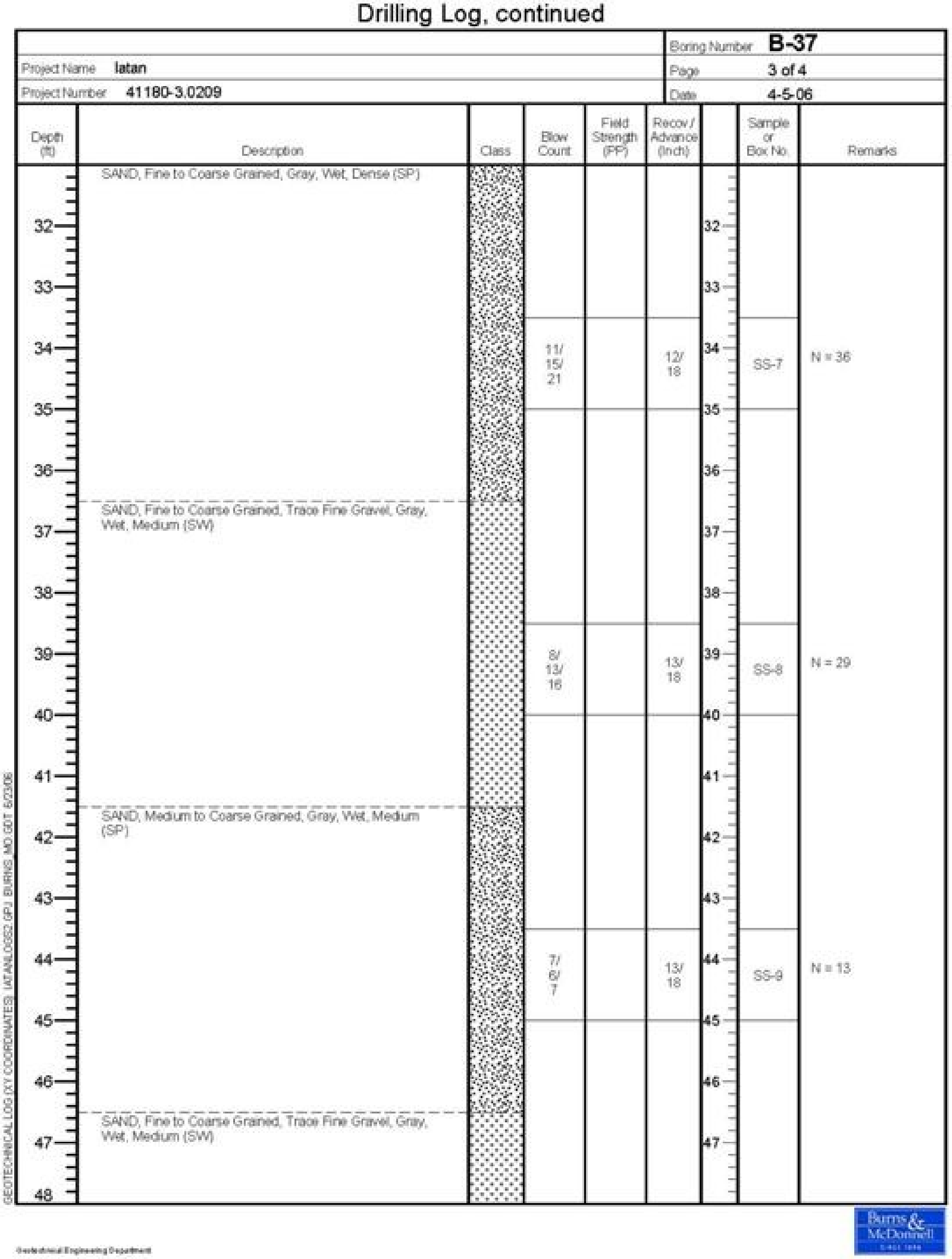


Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-37**
Page	2 of 4
Date	4-5-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SILT, Trace Clay, Gray, Wet, Loose/Soft, Trace Plasticity (ML)		1/ 1/ 2		12/ 18		SS-3	N = 3
	SAND, Coarse Grained, Trace Fine Gravel, Brownish Gray, Loose (SP)							
			3/ 4/ 4		10/ 18		SS-4	N = 8
	SAND, Coarse Grained, Trace Fine Gravel, Light Brown, Medium (SP)		3/ 6/ 9		11/ 18		SS-5	N = 15
			4/ 4/ 4		1/ 18		SS-6	N = 8

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log, continued

Boring Number **B-37**
Project Name Iatan
Page 3 of 4
Project Number 41180-3.0209
Date 4-5-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Coarse Grained, Gray, Wet, Dense (SP)							
34			11/ 15/ 21		12/ 18	34	SS-7	N = 36
	SAND, Fine to Coarse Grained, Trace Fine Gravel, Gray, Wet, Medium (SW)							
39			8/ 13/ 16		13/ 18	39	SS-8	N = 29
	SAND, Medium to Coarse Grained, Gray, Wet, Medium (SP)							
44			7/ 6/ 7		13/ 18	44	SS-9	N = 13
	SAND, Fine to Coarse Grained, Trace Fine Gravel, Gray, Wet, Medium (SW)							

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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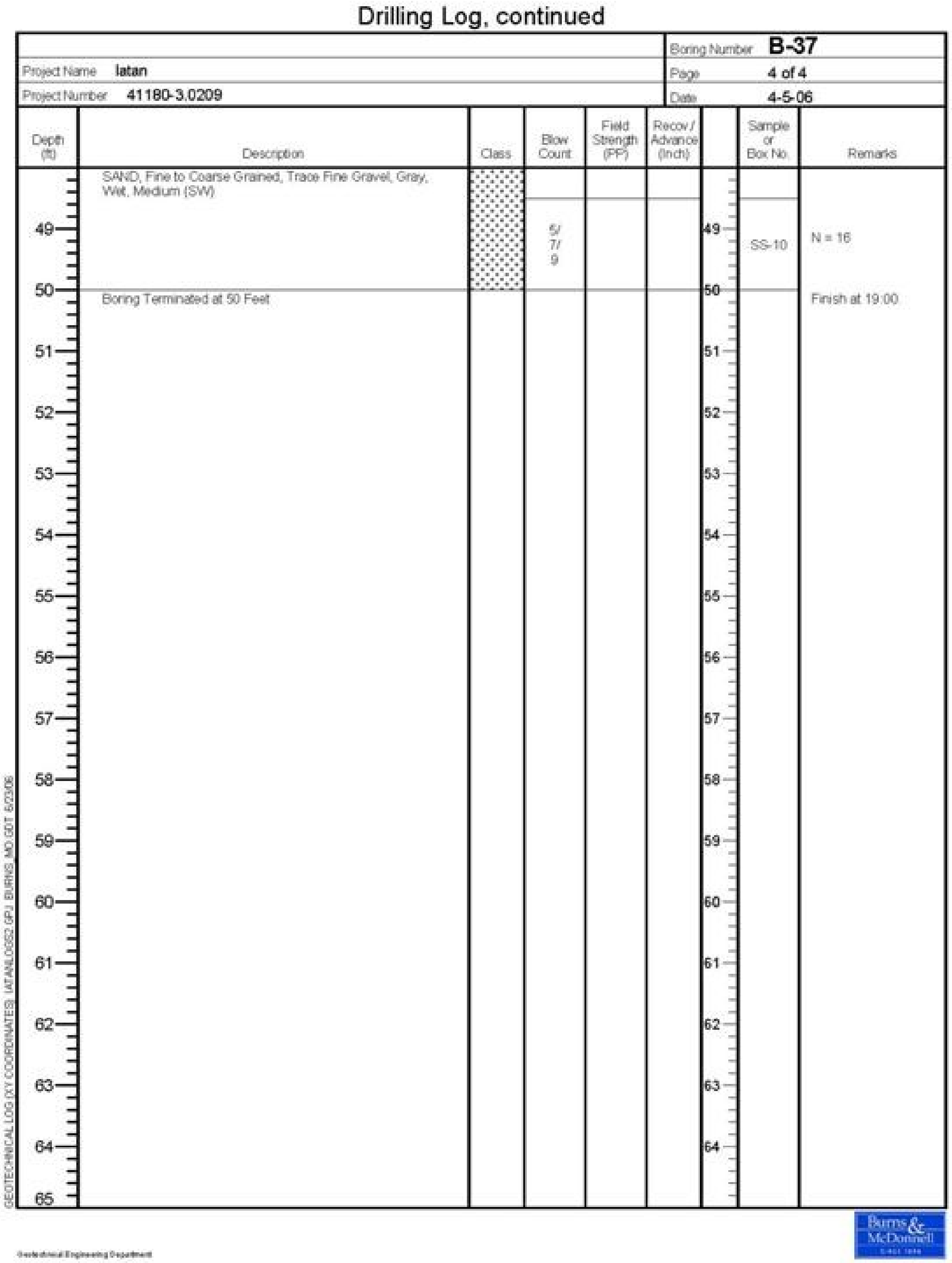

Drilling Log, continued

Project Name **Iatan**

Project Number **41180-3.0209**

Boring Number **B-37**

Page **4 of 4**

Date **4-5-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Coarse Grained, Trace Fine Gravel, Gray, Wet, Medium (SW)	░						
49		░	5/ 7/ 9			49	SS-10	N = 16
50	Boring Terminated at 50 Feet					50		Finish at 19:00
51						51		
52						52		
53						53		
54						54		
55						55		
56						56		
57						57		
58						58		
59						59		
60						60		
61						61		
62						62		
63						63		
64						64		
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 5/23/06

Geotechnical Engineering Department

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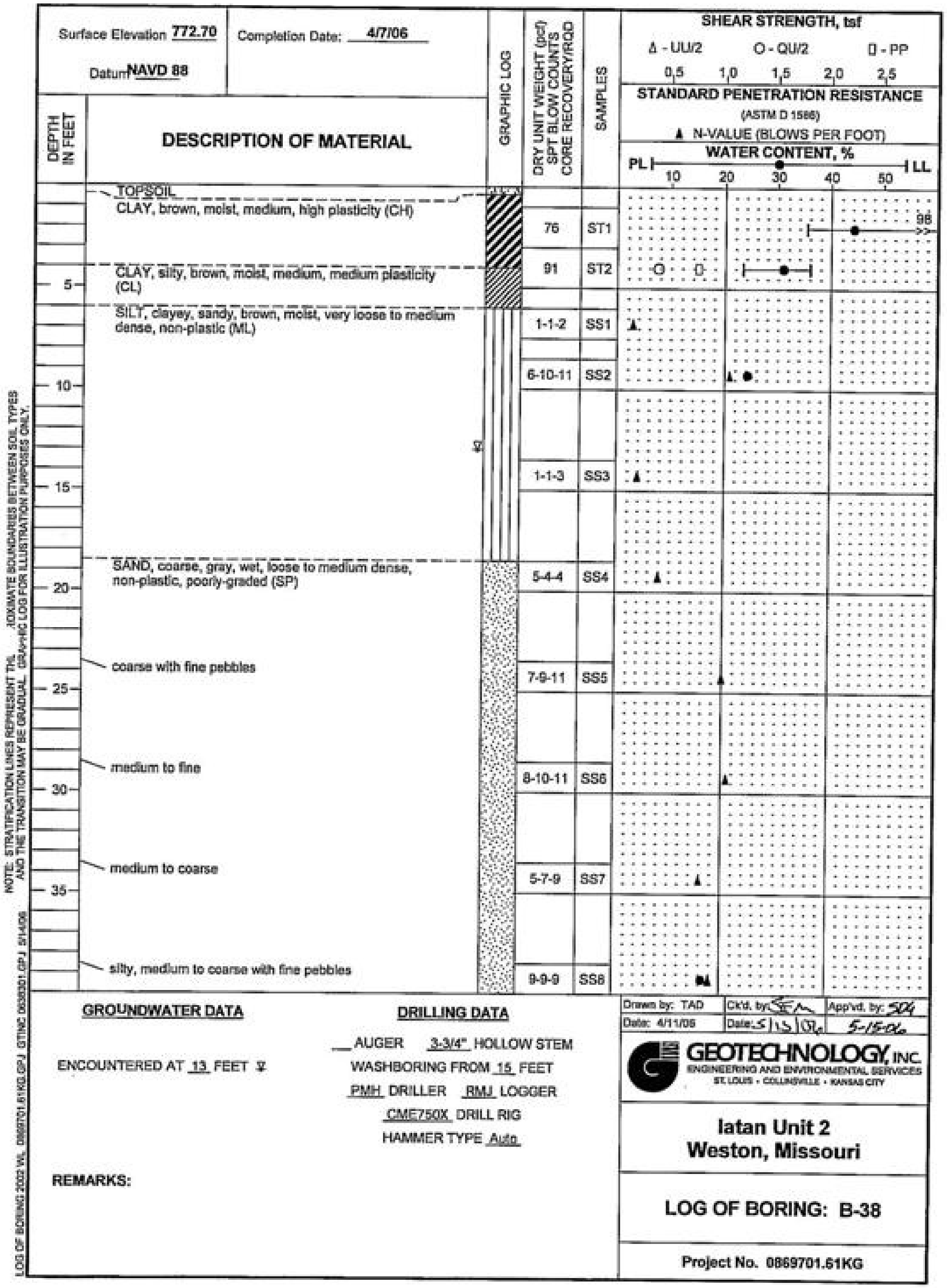


Surface Elevation 772.70

Datum NAVD 88

Completion Date: 4/7/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL		
	CLAY, brown, moist, medium, high plasticity (CH)	76	ST1
5	CLAY, silty, brown, moist, medium, medium plasticity (CL)	91	ST2
	SILT, clayey, sandy, brown, moist, very loose to medium dense, non-plastic (ML)	1-1-2	SS1
10		6-10-11	SS2
15		1-1-3	SS3
20	SAND, coarse, gray, wet, loose to medium dense, non-plastic, poorly-graded (SP)	5-4-4	SS4
25	coarse with fine pebbles	7-9-11	SS5
30	medium to fine	8-10-11	SS6
35	medium to coarse	5-7-9	SS7
	silty, medium to coarse with fine pebbles	9-9-9	SS8

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 13 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
PMH DRILLER RMJ LOGGER
CME750X DRILL RIG
HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDG
Date: 4/11/06	Date: 5/15/06	5-15-06

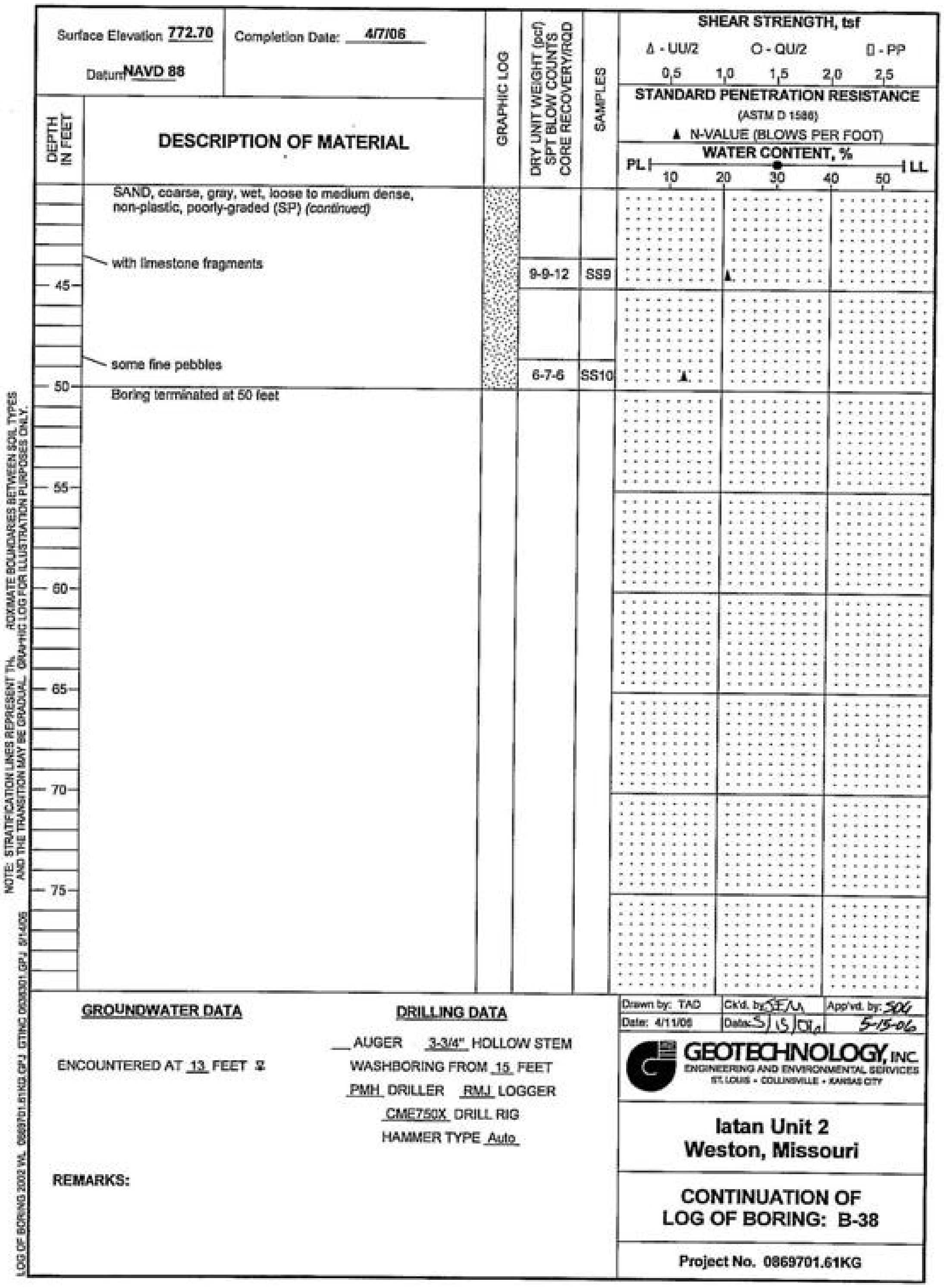
GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-38

Project No. 0869701.61KG

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0636301.GPJ 5/14/06



Surface Elevation 772.70

Datum NAVD 88

Completion Date: 4/7/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, coarse, gray, wet, loose to medium dense, non-plastic, poorly-graded (SP) *(continued)*			
	with limestone fragments			
45			9-9-12	SS9
	some fine pebbles			
50			6-7-6	SS10
50	Boring terminated at 50 feet			
55				
60				
65				
70				
75				

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 ▯ - PP

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ├──●──┤ LL

10 20 30 40 50

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 13 FEET ▽

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

PMH DRILLER RMJ LOGGER

CME750X DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: TAD	Ck'd. by: SEM	App'vd. by: SDG
Date: 4/11/06	Date: 5/15/06	5-15-06

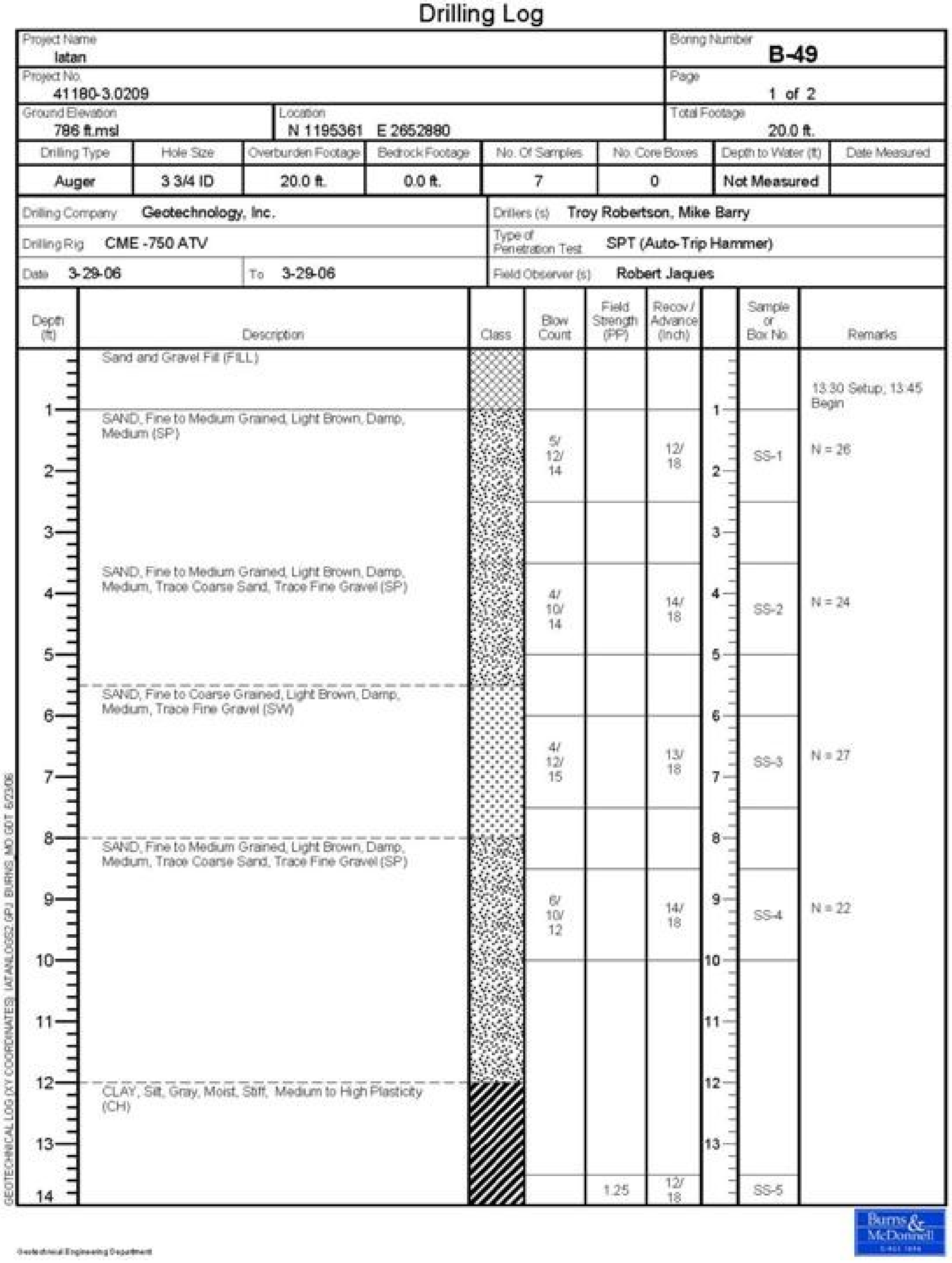
GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B-38

Project No. 0869701.61KG

LOG OF BORING 2002 WL 0869701.61KG.GPJ GTINC 0606301.GPJ 8/14/06



Drilling Log

Project Name	Iatan
Boring Number	**B-49**
Project No.	41180-3.0209
Page	1 of 2
Ground Elevation	786 ft.msl
Location	N 1195361 E 2652880
Total Footage	20.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger	3 3/4 ID	20.0 ft.	0.0 ft.	7	0	Not Measured	

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Troy Robertson, Mike Barry**

Drilling Rig **CME -750 ATV**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **3-29-06** To **3-29-06**

Field Observer (s) **Robert Jaques**

Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)	Sample or Box No.	Remarks
Sand and Gravel Fill (FILL)						13:30 Setup; 13:45 Begin
SAND, Fine to Medium Grained, Light Brown, Damp, Medium (SP)		5/ 12/ 14		12/ 18	SS-1	N = 26
SAND, Fine to Medium Grained, Light Brown, Damp, Medium, Trace Coarse Sand, Trace Fine Gravel (SP)		4/ 10/ 14		14/ 18	SS-2	N = 24
SAND, Fine to Coarse Grained, Light Brown, Damp, Medium, Trace Fine Gravel (SW)		4/ 12/ 15		13/ 18	SS-3	N = 27
SAND, Fine to Medium Grained, Light Brown, Damp, Medium, Trace Coarse Sand, Trace Fine Gravel (SP)		6/ 10/ 12		14/ 18	SS-4	N = 22
CLAY, Silt, Gray, Moist, Stiff, Medium to High Plasticity (CH)			1.25	12/ 18	SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

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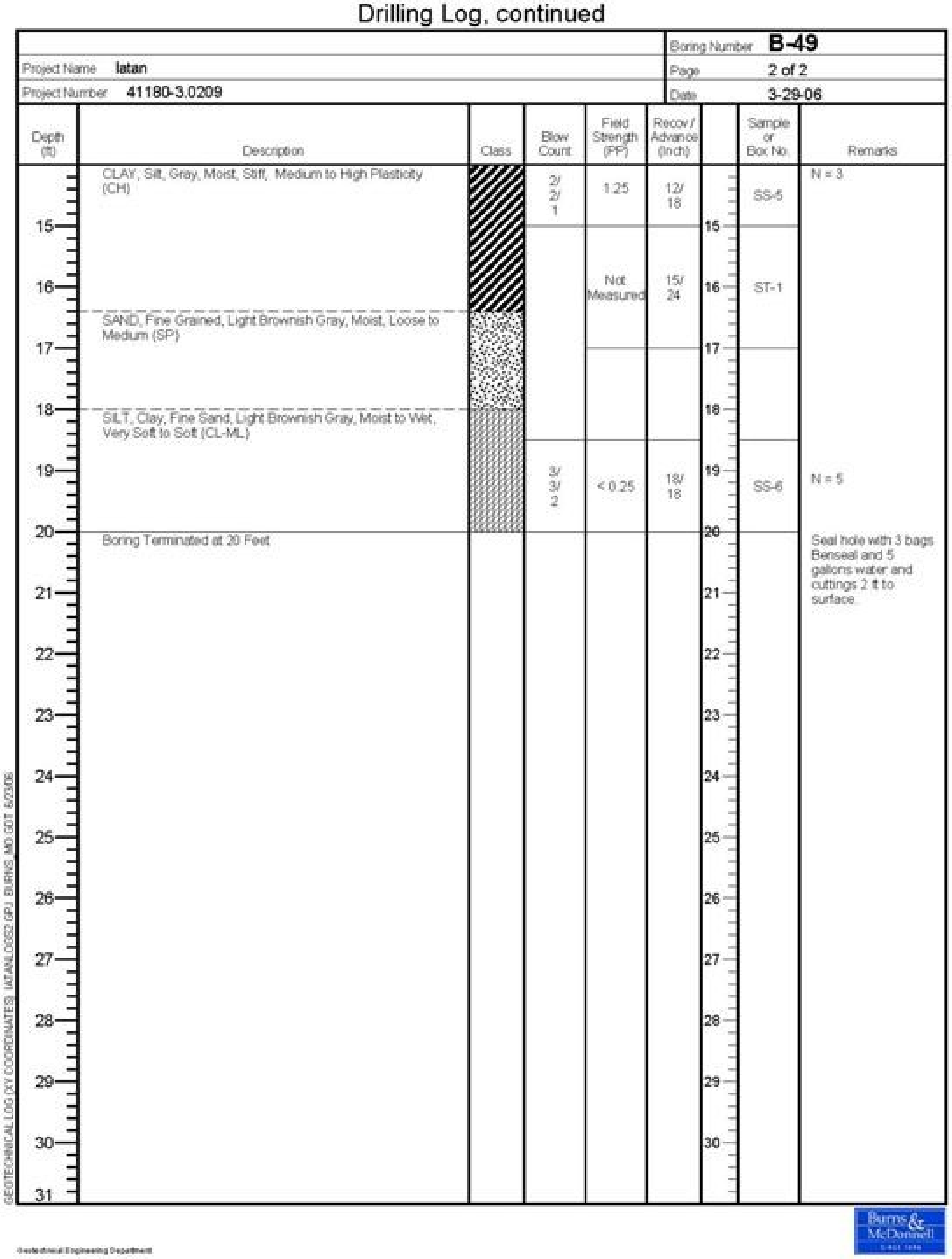


Drilling Log, continued

Project Name **Iatan**
Project Number **41180-3.0209**

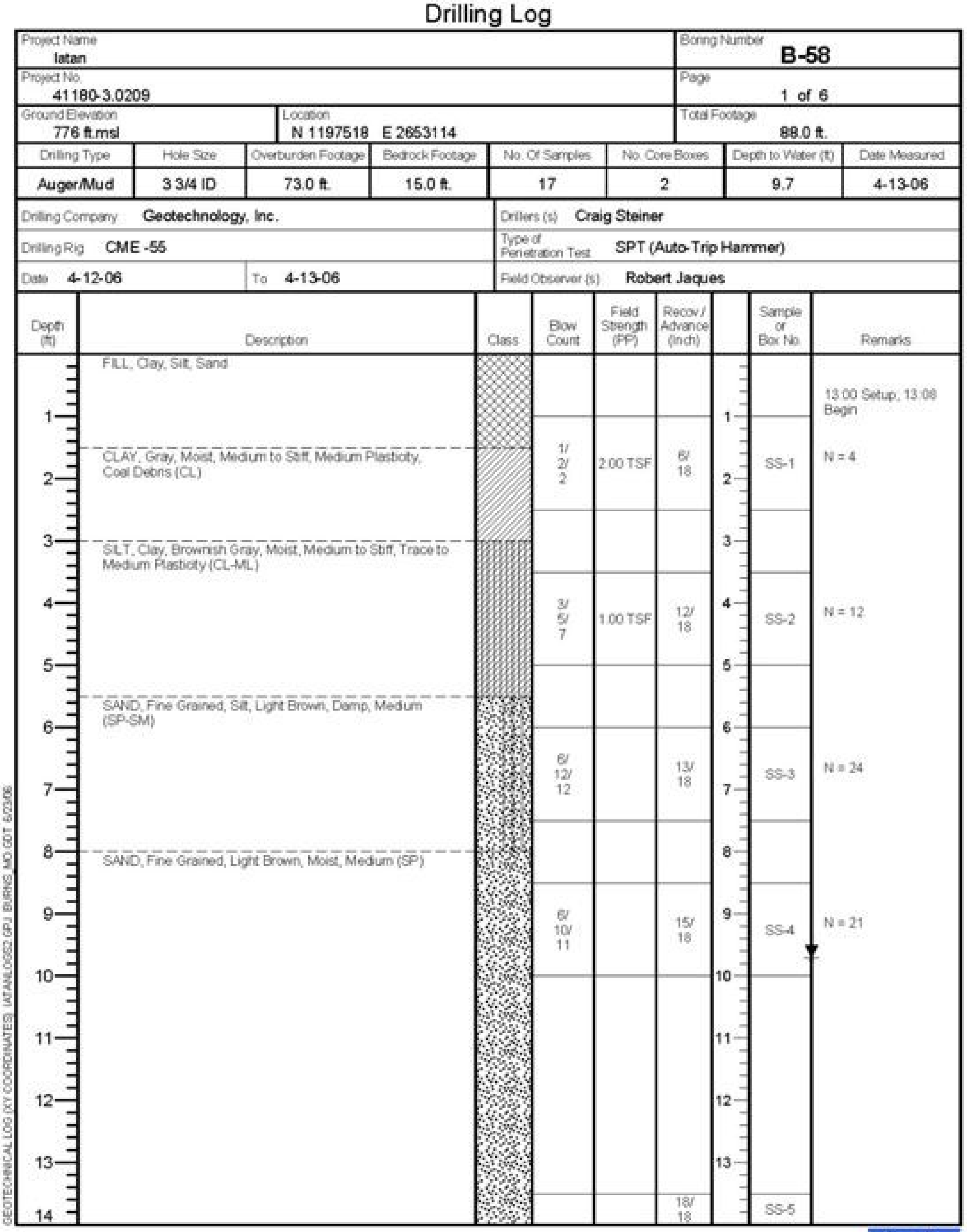
Boring Number **B-49**
Page **2 of 2**
Date **3-29-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	CLAY, Silt, Gray, Moist, Stiff, Medium to High Plasticity (CH)	▨	2/ 2/ 1	1.25	12/ 18		SS-5	N = 3
15		▨				15		
16		▨		Not Measured	15/ 24	16	ST-1	
	SAND, Fine Grained, Light Brownish Gray, Moist, Loose to Medium (SP)	░						
17		░				17		
18	SILT, Clay, Fine Sand, Light Brownish Gray, Moist to Wet, Very Soft to Soft (CL-ML)	▒				18		
19		▒	3/ 3/ 2	< 0.25	18/ 18	19	SS-6	N = 5
20	Boring Terminated at 20 Feet					20		Seal hole with 3 bags Benseal and 5 gallons water and cuttings 2 ft to surface.
21						21		
22						22		
23						23		
24						24		
25						25		
26						26		
27						27		
28						28		
29						29		
30						30		
31								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log



Project Name Iatan		Boring Number **B-58**
Project No. 41180-3.0209		Page 1 of 6
Ground Elevation 776 ft.msl	Location N 1197518 E 2653114	Total Footage 88.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	73.0 ft.	15.0 ft.	17	2	9.7	4-13-06

Drilling Company **Geotechnology, Inc.**

Drillers (s) **Craig Steiner**

Drilling Rig **CME -55**

Type of Penetration Test **SPT (Auto-Trip Hammer)**

Date **4-12-06** To **4-13-06**

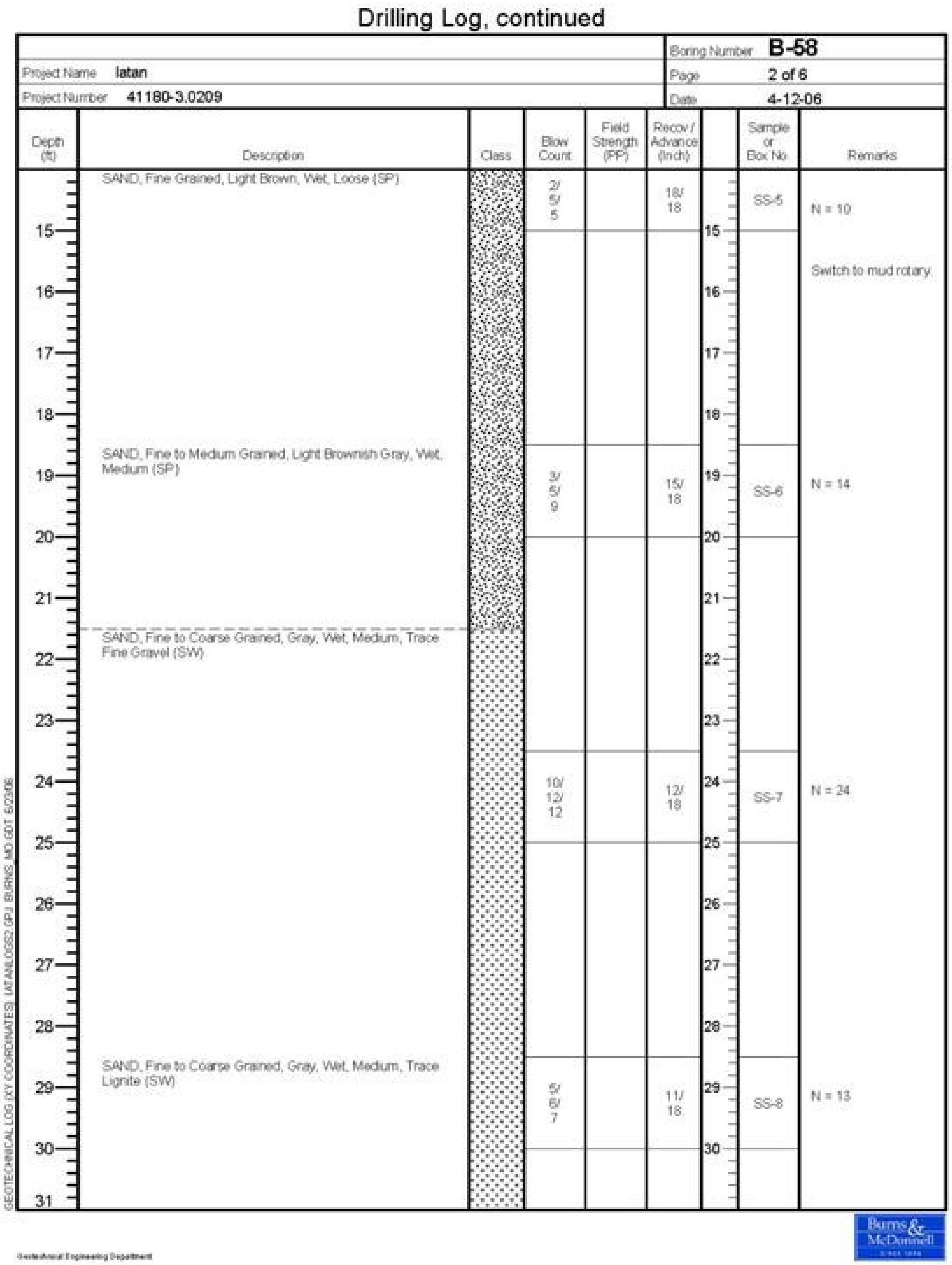
Field Observer (s) **Robert Jaques**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	FILL, Clay, Silt, Sand							13:00 Setup, 13:08 Begin
1						1		
2	CLAY, Gray, Moist, Medium to Stiff, Medium Plasticity, Coal Debris (CL)		1/ 2/ 2	2.00 TSF	6/ 18	2	SS-1	N = 4
3	SILT, Clay, Brownish Gray, Moist, Medium to Stiff, Trace to Medium Plasticity (CL-ML)					3		
4			3/ 5/ 7	1.00 TSF	12/ 18	4	SS-2	N = 12
5						5		
6	SAND, Fine Grained, Silt, Light Brown, Damp, Medium (SP-SM)					6		
7			6/ 12/ 12		13/ 18	7	SS-3	N = 24
8	SAND, Fine Grained, Light Brown, Moist, Medium (SP)					8		
9			6/ 10/ 11		15/ 18	9	SS-4	N = 21
10						10		
11						11		
12						12		
13						13		
14					18/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MD.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell



Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	B-58
Page	2 of 6
Date	4-12-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Light Brown, Wet, Loose (SP)		2/ 5/ 5		18/ 18		SS-5	N = 10
15						15		Switch to mud rotary.
16						16		
17						17		
18						18		
19	SAND, Fine to Medium Grained, Light Brownish Gray, Wet, Medium (SP)		3/ 5/ 9		15/ 18	19	SS-6	N = 14
20						20		
21						21		
22	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)					22		
23						23		
24			10/ 12/ 12		12/ 18	24	SS-7	N = 24
25						25		
26						26		
27						27		
28						28		
29	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Lignite (SW)		5/ 6/ 7		11/ 18	29	SS-8	N = 13
30						30		
31								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

Project Name	Iatan	Boring Number	**B-58**
Project Number	41180-3.0209	Page	3 of 6
		Date	4-12-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
31	SAND, Fine to Medium Grained, Gray, Wet, Loose (SP)							
34			6/ 5/ 3		15/ 18	34	SS-9	N = 8 Trace clay at base of SS-9 sample
35	CLAY, Gray, Wet, Medium, Medium Plasticity (CL)				10/ 24	35	ST-1	
36	SAND, Fine to Coarse Grained, Gray, Wet, Medium (SW)					36		
39	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)		5/ 7/ 8		12/ 18	39	SS-10	N = 15
41.5	SAND, Fine to Medium Grained, Gray, Wet, Medium (SP)							
44			10/ 14/ 12		11/ 18	44	SS-11	N = 26
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

	Boring Number **B-58**
Project Name **Iatan**	Page **4 of 6**
Project Number **41180-3.0209**	Date **4-12-06**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Medium Grained, Gray, Wet, Dense (SP)							
49			12/ 16/ 16		14/ 18	49	SS-12	N = 32
50						50		
51						51		
	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)							
52						52		
53						53		
54			5/ 7/ 7		11/ 18	54	SS-13	N = 14
55						55		
56						56		
	SAND, Coarse Grained, Gravel, Gray, Wet, Medium (SPG)							
57						57		
58						58		
59			5/ 7/ 6		10/ 18	59	SS-14	N = 13
60						60		
61						61		
62						62		
63						63		
64	SAND, Medium to Coarse Grained, Gray, Wet, Medium, Fine Gravel (SPG)		6/ 12/ 12		11/ 18	64	SS-15	N = 24
65								

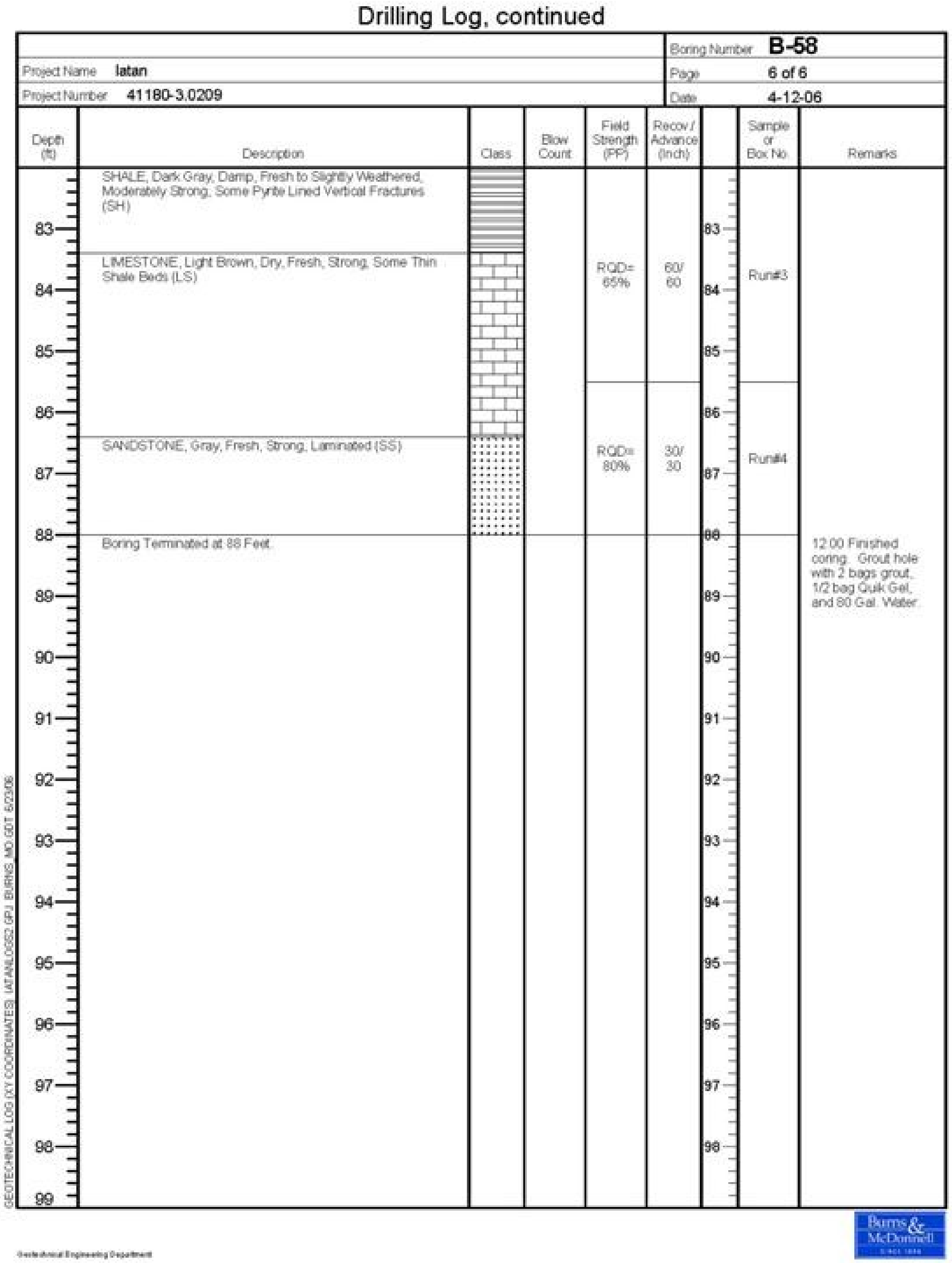
GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Burns & McDonnell

Drilling Log, continued

Boring Number	**B-58**
Project Name	**Iatan**
Page	5 of 6
Project Number	41180-3.0209
Date	4-12-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Coarse Grained, Gravel, Gray, Wet, Medium (SPG)							16:00 Stop for today.
66						66		
67	SAND, Fine to Medium Grained, Gray, Wet, Medium, Trace Fine Gravel (SP)					67		
68						68		
69			8/ 12/ 13		9/ 18	69	SS-16	N = 25
70						70		
71						71		
72						72		
73	LIMESTONE, Light Brown to Gray, Dry, Fresh, Strong, Fossiliferous, Some Thin Shale Beds (LS)					73		
74				RQD= 16%	23/ 30	74	Run#1	8:39 Hard drilling at 73 ft. Set casing to core.
75						75		
76						76		
77						77		
78	SHALE, Dark Gray, Damp, Fresh to Slightly Weathered, Moderately Strong, Some Pyrite Lined Vertical Fractures (SH)			RQD= 65%	59/ 60	78	Run#2	
79						79		Core barrel jammed.
80						80		
81				RQD= 65%	60/ 60	81	Run#3	
82								



Drilling Log, continued

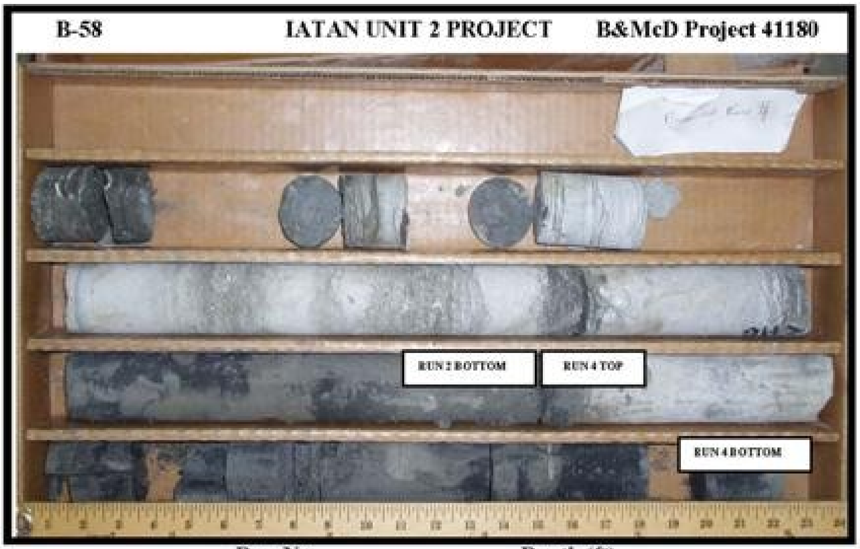
Boring Number **B-58**
Project Name **Iatan**
Page 6 of 6
Project Number 41180-3.0209
Date 4-12-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SHALE, Dark Gray, Damp, Fresh to Slightly Weathered, Moderately Strong, Some Pyrite Lined Vertical Fractures (SH)	▤						
	LIMESTONE, Light Brown, Dry, Fresh, Strong, Some Thin Shale Beds (LS)	▦		RQD= 65%	60/ 60		Run#3	
	SANDSTONE, Gray, Fresh, Strong, Laminated (SS)	⠿		RQD= 80%	30/ 30		Run#4	
	Boring Terminated at 88 Feet.							12:00 Finished coring. Grout hole with 2 bags grout, 1/2 bag Quik Gel, and 80 Gal. Water.

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Run No.	Depth (ft)
Run 1	73.0 to 75.5
Run 2	75.5 to 80.5
Run 3	80.5 to Continued



Run No.	Depth (ft)
Run 3	Continued to 85.5
Run 4	85.5 to 88.0

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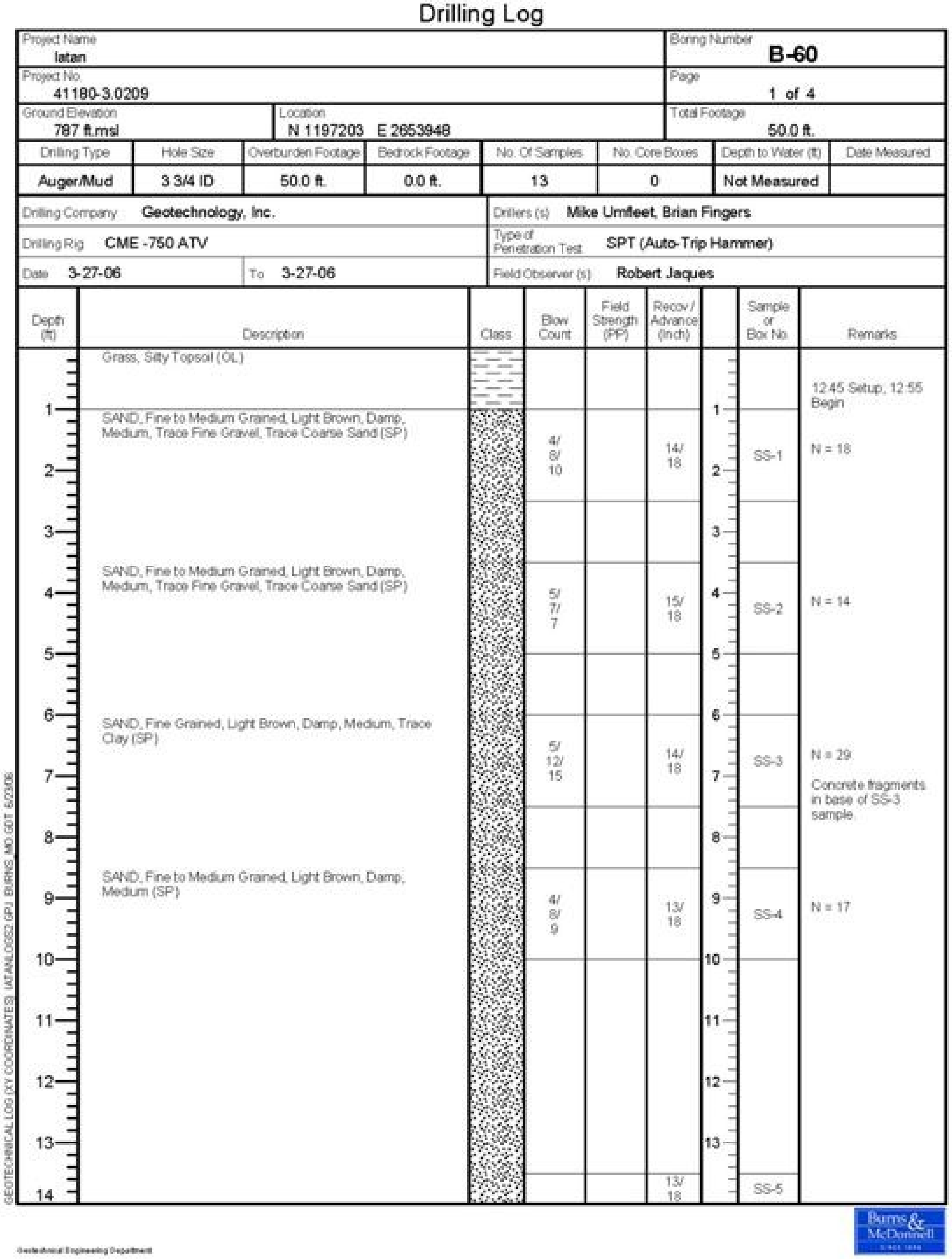


Drilling Log

Project Name **Iatan**						Boring Number **B-60**	
Project No. **41180-3.0209**						Page **1 of 4**	
Ground Elevation **787 ft.msl**		Location **N 1197203 E 2653948**				Total Footage **50.0 ft.**	
Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	**3 3/4 ID**	**50.0 ft.**	**0.0 ft.**	**13**	**0**	**Not Measured**	
Drilling Company **Geotechnology, Inc.**				Drillers (s) **Mike Umfleet, Brian Fingers**			
Drilling Rig **CME -750 ATV**				Type of Penetration Test **SPT (Auto-Trip Hammer)**			
Date **3-27-06**	To **3-27-06**			Field Observer (s) **Robert Jaques**			

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
0–1	Grass, Silty Topsoil (OL)							12:45 Setup, 12:55 Begin
1–2.5	SAND, Fine to Medium Grained, Light Brown, Damp, Medium, Trace Fine Gravel, Trace Coarse Sand (SP)		4/ 8/ 10		14/ 18		SS-1	N = 18
3.5–5	SAND, Fine to Medium Grained, Light Brown, Damp, Medium, Trace Fine Gravel, Trace Coarse Sand (SP)		5/ 7/ 7		15/ 18		SS-2	N = 14
6–7.5	SAND, Fine Grained, Light Brown, Damp, Medium, Trace Clay (SP)		5/ 12/ 15		14/ 18		SS-3	N = 29 Concrete fragments in base of SS-3 sample.
8.5–10	SAND, Fine to Medium Grained, Light Brown, Damp, Medium (SP)		4/ 8/ 9		13/ 18		SS-4	N = 17
13.5–14					13/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

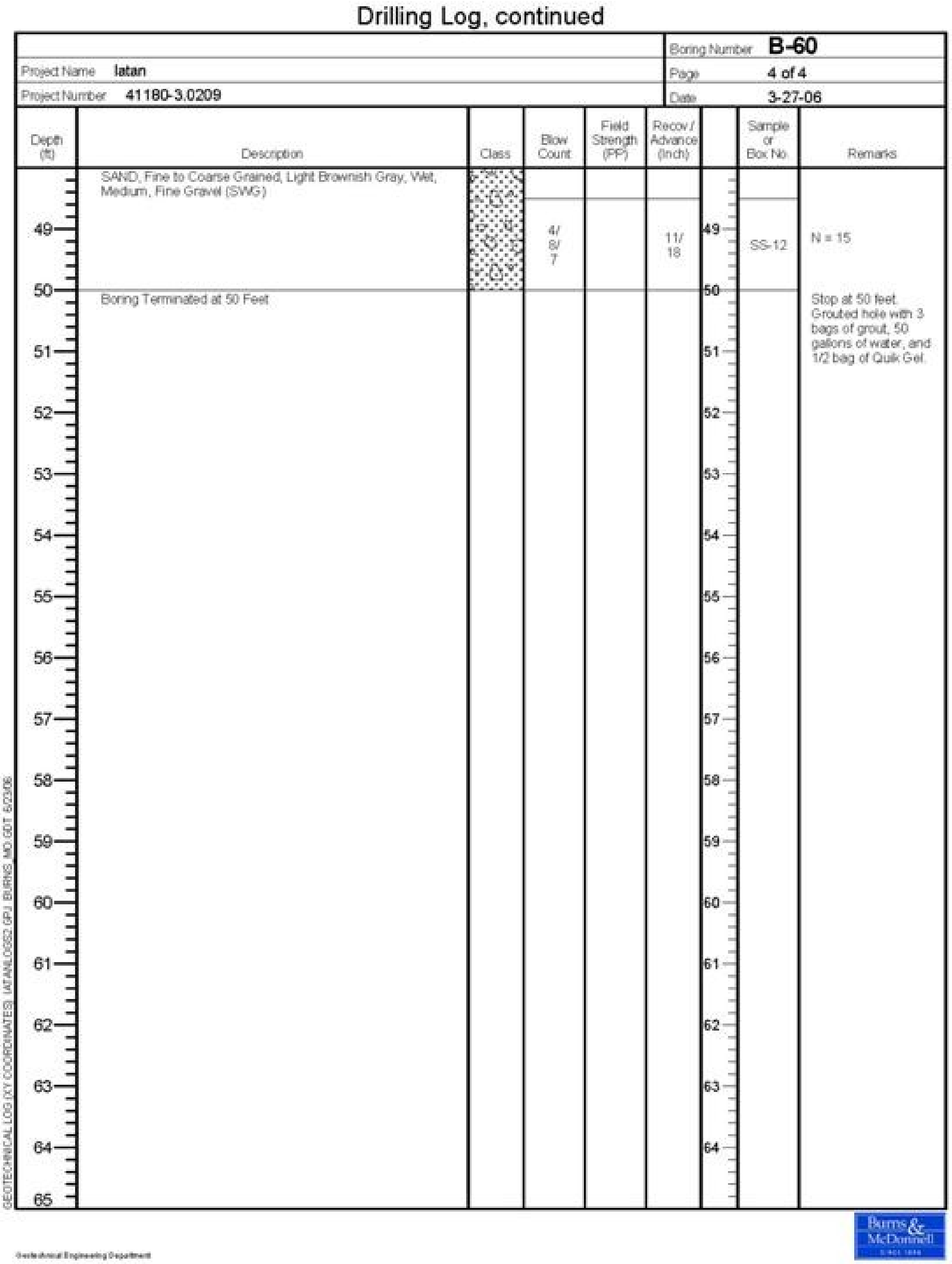
		Boring Number	**B-60**
Project Name	Iatan	Page	2 of 4
Project Number	41180-3.0209	Date	3-27-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Light Brown, Damp, Loose (SP)		2/ 3/ 3		13/ 18		SS-5	N = 6
15						15		
16						16		
	SILT, Clayey, Brownish Gray, Moist, Soft to Medium, Trace to Medium Plasticity (CL-ML)							
17						17		
18						18		
19			1/ 2/ 2	0.50 TSF	17/ 18	19	SS-6	N = 4
20	SILT, Clayey, Brownish Gray, Moist, Medium to Stiff, Trace to Medium Plasticity CL-ML)					20		
21				1.00 TSF	21/ 24	21	ST-1	
22						22		
23	SAND, Fine Grained, Brownish Gray, Moist to Wet, Medium (SP)					23		
24			4/ 9/ 14		16/ 18	24	SS-7	N = 23
25						25		Switch to mud rotary.
26						26		
27						27		
28						28		
	SAND, Fine Grained, Brownish Gray, Wet, Medium (SP)							
29			5/ 10/ 14		13/ 18	29	SS-8	N = 24
30						30		
31								

Drilling Log, continued

Project Name	Iatan	Boring Number	**B-60**
Project Number	41180-3.0209	Page	3 of 4
		Date	3-27-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Brownish Gray, Moist to Wet, Medium (SP)							
32						32		
	SAND, Fine to Coarse Grained, Light Brownish Gray, Wet, Medium (SW)							
33						33		
34			5/ 10/ 10		10/ 18	34	SS-9	N = 20
35						35		
36						36		
37						37		
38						38		
39	SAND, Fine to Coarse Grained, Light Brown, Wet, Medium, Trace Fine Gravel (SW)		4/ 6/ 7		10/ 18	39	SS-10	N = 13
40						40		
41						41		
	SAND, Fine to Coarse Grained, Light Brown, Wet, Loose, Fine Gravel (SWG)							
42						42		
43						43		
44			5/ 5/ 5		10/ 18	44	SS-11	N = 10
45						45		
46						46		
47						47		
48								



Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-60**
Page	4 of 4
Date	3-27-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Coarse Grained, Light Brownish Gray, Wet, Medium, Fine Gravel (SWG)	▨						
49		▨	4/ 8/ 7		11/ 18	49	SS-12	N = 15
50	Boring Terminated at 50 Feet					50		Stop at 50 feet. Grouted hole with 3 bags of grout, 50 gallons of water, and 1/2 bag of Quik Gel.
51						51		
52						52		
53						53		
54						54		
55						55		
56						56		
57						57		
58						58		
59						59		
60						60		
61						61		
62						62		
63						63		
64						64		
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell

Drilling Log

Project Name Iatan			Boring Number **B-67**
Project No. 41180-3.0209			Page 1 of 4
Ground Elevation 785 ft.msl	Location N 1195314 E 2653791		Total Footage 50.0 ft.

Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	50.0 ft.	0.0 ft.	12	0	Not Measured	

Drilling Company **Geotechnology, Inc.**	Drillers (s) **Mike Umfleet, Brian Fingers**
Drilling Rig **CME -750 ATV**	Type of Penetration Test **SPT (Auto-Trip Hammer)**
Date **3-26-06** To **3-26-06**	Field Observer (s) **Robert Jaques**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	Sand and Gravel Fill (FILL)	▩						7:26 Setup
1	SAND, Fine Grained, Light Brown, Damp, Medium (SP)	░				1		
2		░	6/ 6/ 7		13/ 18	2	SS-1	N = 13
3		░				3		
4	SAND, Fine Grained, Light Brown, Damp, Medium (SP)	░	2/ 5/ 7		11/ 18	4	SS-2	N = 12
5		░				5		
6	SAND, Fine Grained, Gray, Damp, Loose (SP)	░				6		
7		░	3/ 5/ 5		14/ 18	7	SS-3	N = 10
8		░				8		
9	SAND, Fine Grained, Gray, Moist, Medium (SP)	░	3/ 8/ 3		13/ 18	9	SS-4	N = 11
10		░				10		
11		░				11		
12	CLAY, Gray, Wet, Stiff, High Plasticity (CH)	▨				12		
13		▨				13		
14		▨		1.50 TSF	11/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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Drilling Log, continued

Project Name	**Iatan**	Boring Number	**B-67**
		Page	2 of 4
Project Number	41180-3.0209	Date	3-26-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	CLAY, Gray, Wet, Stiff, High Plasticity (CH)	▨	1/ 2/ 2	1.50 TSF	11/ 18		SS-5	N = 4
15		▨				15		
16		▨				16		
17	CLAY, Silty, Gray, Wet, Medium to Stiff, Medium Plasticity, Some Fine Sand (CL)	▧				17		
18		▧				18		
19		▧		1.25 TSF	24/ 24	19	ST-1	
20		▧				20		
21		▧				21		
22	SILT, Clayey, Gray, Wet, Loose (ML)	▥				22		
23		▥				23		
24		▥	2/ 3/ 7		18/ 18	24	SS-6	N = 10
25		▥				25		Switch to mud rotary.
26		▥				26		
27	SAND, Fine Grained, Gray, Wet, Medium (SP)	░				27		
28		░				28		
29		░	4/ 5/ 7		10/ 18	29	SS-7	N = 12
30		░				30		
31		░						

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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Drilling Log, continued

	Boring Number	**B-67**
Project Name **Iatan**	Page	3 of 4
Project Number **41180-3.0209**	Date	3-26-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Gray, Wet, Medium (SP)							
32						32		
33	SAND, Fine to Medium Grained, Gray, Wet, Medium (SP)					33		
34			4/ 5/ 11		11/ 18	34	SS-8	N = 16
35						35		
36	SAND, Fine to Coarse Grained, Gray, Wet, Very Loose to Loose (SW)					36		
37						37		
38						38		
39			1/ 1/ 3		12/ 18	39	SS-9	N = 4
40						40		
41	SAND, Fine Grained, Gray, Wet, Medium to Dense (SP)					41		
42						42		
43						43		
44			6/ 12/ 17		12/ 18	44	SS-10	N = 29
45						45		
46						46		
47						47		
48								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Drilling Log, continued

Boring Number	**B-67**
Project Name	**Iatan**
Page	4 of 4
Project Number	41180-3.0209
Date	3-26-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine Grained, Gray, Wet, Medium to Dense (SP)	▨						
49		▨	11/ 18/ 15		12/ 18	49	SS-11	N = 33
50	Boring Terminated at 50 Feet					50		Stop at 50 feet. Grouted hole with 2 bags of grout, 60 gallons of water, and 1/2 bag of Quik Gel.
51						51		
52						52		
53						53		
54						54		
55						55		
56						56		
57						57		
58						58		
59						59		
60						60		
61						61		
62						62		
63						63		
64						64		
65								

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

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Drilling Log

Project Name Iatan		Boring Number **B-68**
Project No. 41180-3.0209		Page 1 of 4
Ground Elevation 783 ft.msl	Location N 1195231 E 2653778	Total Footage 50.0 ft.

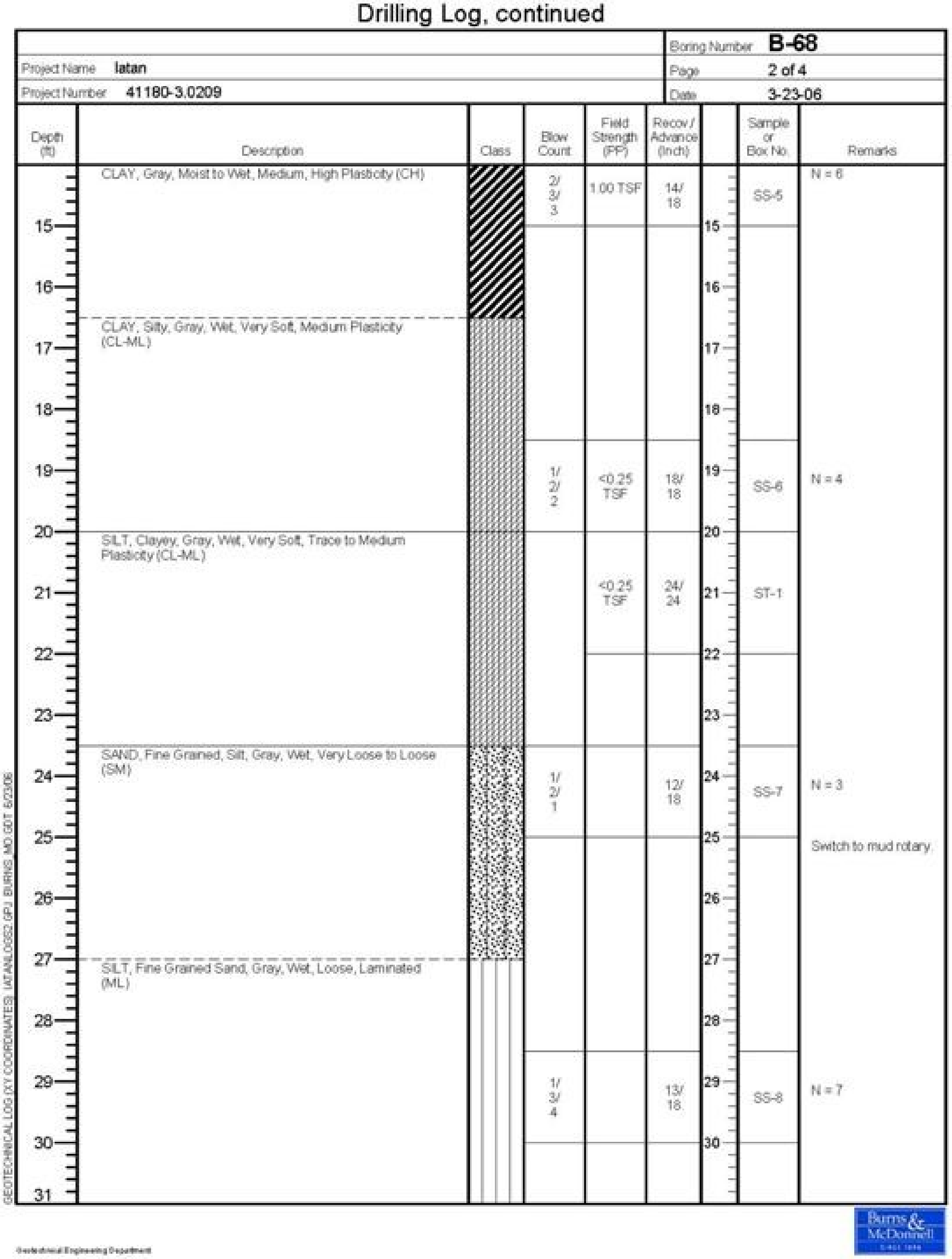
Drilling Type	Hole Size	Overburden Footage	Bedrock Footage	No. Of Samples	No. Core Boxes	Depth to Water (ft)	Date Measured
Auger/Mud	3 3/4 ID	50.0 ft.	0.0 ft.	13	0	Not Measured	

Drilling Company **Geotechnology, Inc.**	Drillers (s) **Mike Umfleet, Brian Fingers**
Drilling Rig **CME -750 ATV**	Type of Penetration Test **SPT (Auto-Trip Hammer)**
Date **3-23-06** To **3-24-06**	Field Observer (s) **Robert Jaques**

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
0–1	Grass, Silty Topsoil (OL)	▤						15:40 Setup, 15:50 Begin
1–2.5	SAND, Fine Grained, Light Brown, Damp, Medium (SP)	▒	3/ 7/ 7		11/ 18		SS-1	N = 14
2.5–3.5		▒						
3.5–5	SAND, Fine Grained, Light Brown, Damp, Medium (SP)	▒	3/ 5/ 8		10/ 18		SS-2	N = 13
5–6		▒						Stop for today. 3-24-06 7:25 Setup. 7:56 Resume
6–7.5	SAND, Fine Grained, Light Brown, Damp, Medium (SP)	▒	3/ 5/ 6		14/ 18		SS-3	N = 11
7.5–8.5		▒						
8.5–10	SAND, Fine Grained,Gray, Moist, Medium (SP-SM)	▒	3/ 6/ 7		14/ 18		SS-4	N = 13
10–12		▒						
12–13.5	CLAY, Gray, Moist to Wet, Medium, High Plasticity (CH)	▨						
13.5–14		▨		1.00 TSF	14/ 18		SS-5	

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department



Drilling Log, continued

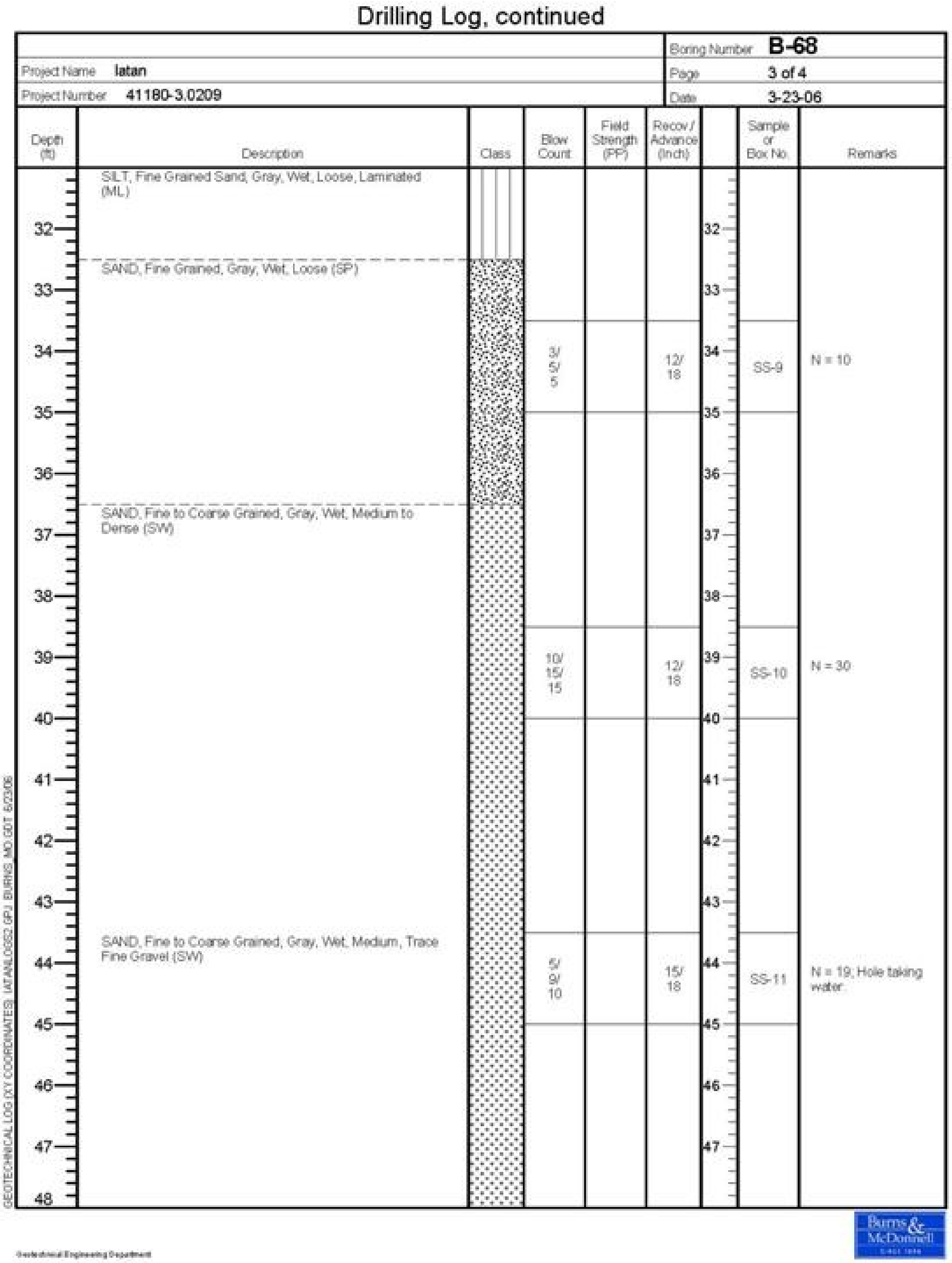
Project Name	Iatan	Boring Number	**B-68**
Project Number	41180-3.0209	Page	2 of 4
		Date	3-23-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	CLAY, Gray, Moist to Wet, Medium, High Plasticity (CH)		2/ 3/ 3	1.00 TSF	14/ 18		SS-5	N = 6
15 16						15 16		
17 18	CLAY, Silty, Gray, Wet, Very Soft, Medium Plasticity (CL-ML)					17 18		
19			1/ 2/ 2	<0.25 TSF	18/ 18	19	SS-6	N = 4
20 21	SILT, Clayey, Gray, Wet, Very Soft, Trace to Medium Plasticity (CL-ML)			<0.25 TSF	24/ 24	20 21	ST-1	
22 23						22 23		
24	SAND, Fine Grained, Silt, Gray, Wet, Very Loose to Loose (SM)		1/ 2/ 1		12/ 18	24	SS-7	N = 3
25 26						25 26		Switch to mud rotary.
27 28	SILT, Fine Grained Sand, Gray, Wet, Loose, Laminated (ML)					27 28		
29			1/ 3/ 4		13/ 18	29	SS-8	N = 7
30 31						30		

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell



Drilling Log, continued

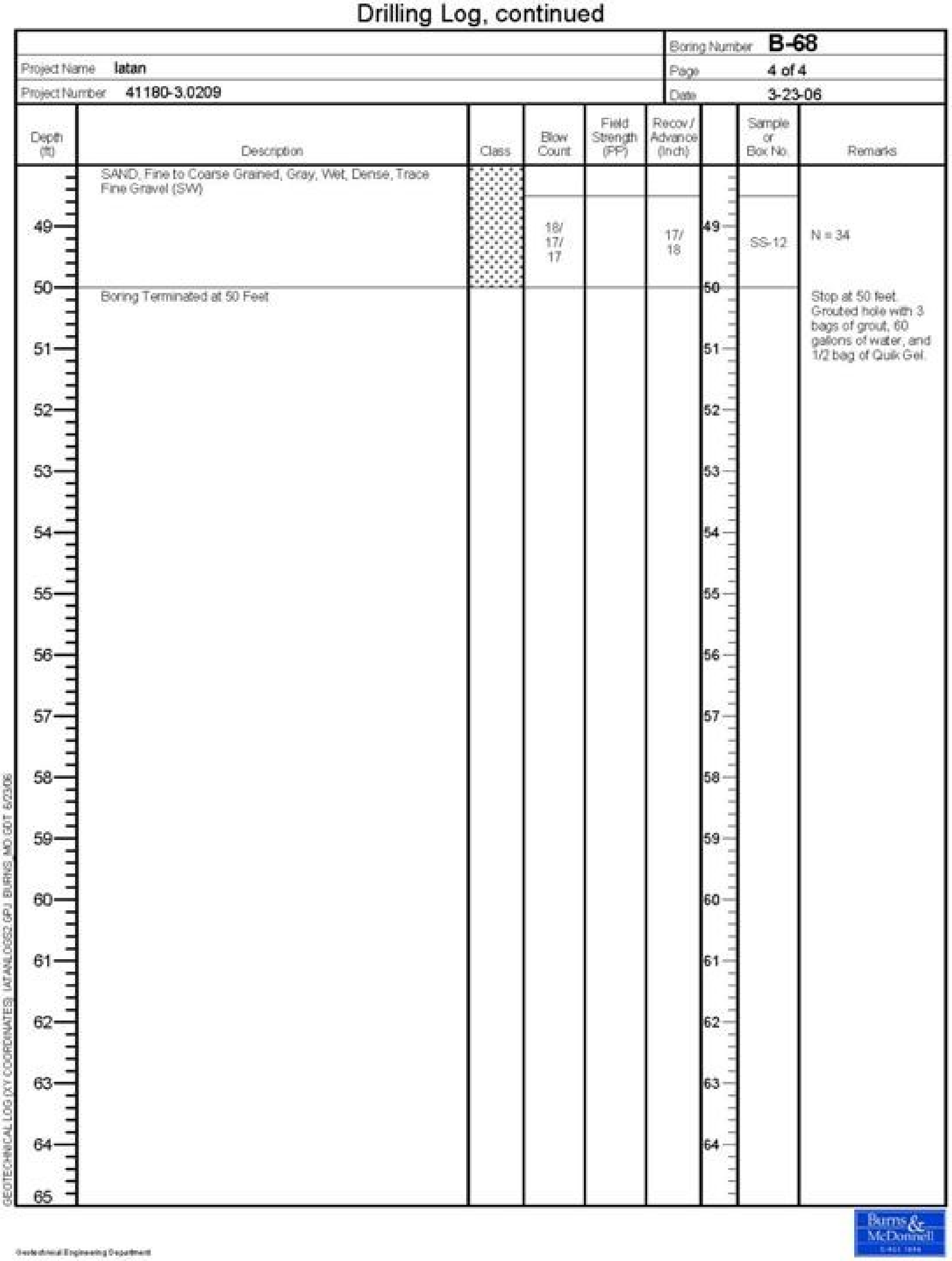
Project Name	Iatan	Boring Number	**B-68**
Project Number	41180-3.0209	Page	3 of 4
		Date	3-23-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
31	SILT, Fine Grained Sand, Gray, Wet, Loose, Laminated (ML)							
32.5	SAND, Fine Grained, Gray, Wet, Loose (SP)							
33.5–35			3/ 5/ 5		12/ 18		SS-9	N = 10
36.5	SAND, Fine to Coarse Grained, Gray, Wet, Medium to Dense (SW)							
38.5–40			10/ 15/ 15		12/ 18		SS-10	N = 30
43.5–45	SAND, Fine to Coarse Grained, Gray, Wet, Medium, Trace Fine Gravel (SW)		5/ 9/ 10		15/ 18		SS-11	N = 19; Hole taking water.

GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Burns & McDonnell SINCE 1898



Drilling Log, continued

Project Name	Iatan
Project Number	41180-3.0209
Boring Number	**B-68**
Page	4 of 4
Date	3-23-06

Depth (ft)	Description	Class	Blow Count	Field Strength (PP)	Recov / Advance (Inch)		Sample or Box No.	Remarks
	SAND, Fine to Coarse Grained, Gray, Wet, Dense, Trace Fine Gravel (SW)	▨						
49		▨	18/ 17/ 17		17/ 18	49	SS-12	N = 34
50	Boring Terminated at 50 Feet					50		Stop at 50 feet. Grouted hole with 3 bags of grout, 60 gallons of water, and 1/2 bag of Quik Gel.
51						51		
52						52		
53						53		
54						54		
55						55		
56						56		
57						57		
58						58		
59						59		
60						60		
61						61		
62						62		
63						63		
64						64		
65								

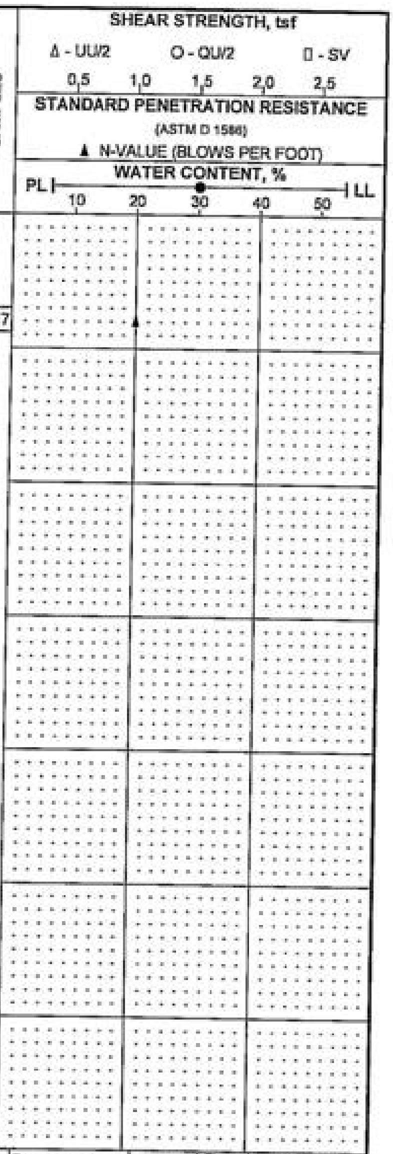
GEOTECHNICAL LOG (XY COORDINATES) IATANLOGS2.GPJ BURNS_MO.GDT 6/23/06

Geotechnical Engineering Department

Surface Elevation 785.65 Completion Date: 11/9/06

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	GRAVEL, sandy, fine , gray, medium dense (GS) *(continued)*			
	SAND, gravelly, coarse, gray, medium dense		8-9-11	SS17
75				
	Auger refusal, Boring Terminated			
80				
85				
90				
95				
100				



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Water was not encountered before wash boring. Offset 5' East. Boring caved to 6 feet at 24 hours.
Northing: 2653593.31 Easing: 1197163.15

Drawn by: VS	Ck'd. by: LEW	App'vd. by: SDV
Date: 11/10/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS - COLLINSVILLE - KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF LOG OF BORING: B-71

Project No. 0869701.61KG

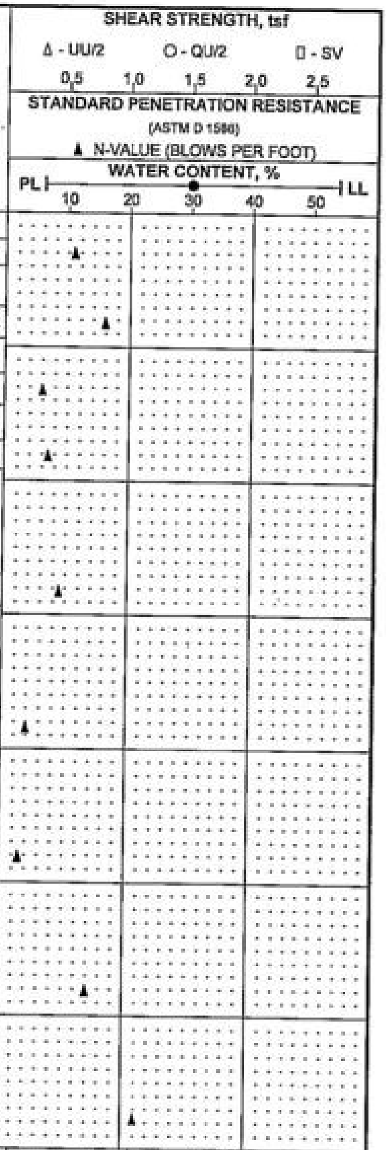
LOG OF BORING 2002 869701.61KG.GPJ GTINC 0636001.GPJ 2/28/07

Surface Elevation 785.65

Datum NAVD 88

Completion Date: 11/9/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	FILL, sand and coal, black		
	SAND, fine to medium, brown, medium dense (SP)	2-5-6	SS1
		2-8-8	SS2
5	trace coal		
	loose		
		2-2-4	SS3
		2-3-4	SS4
10			
	SILT, brown, firm, low plasticity (ML)	3-4-5	SS5
15			
	gray, soft		
		1-2-2	SS6
20			
	trace sand		
		1-1-2	SS7
25			
	SILT, sandy, gray stiff		
		5-6-8	SS8
30	SAND, trace clay, fine to coarse, medium dense (SP)		
		6-9-13	SS9



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Water was not encountered before wash boring. Offset 5' East.
Boring caved to 6 feet at 24 hours.
Northing: 2663593.31 Easing: 1197163.15

Drawn by: VS	Ck'd. by: LEW	App'vd. by: SDG
Date: 11/10/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-71

Project No. 0869701.61KG

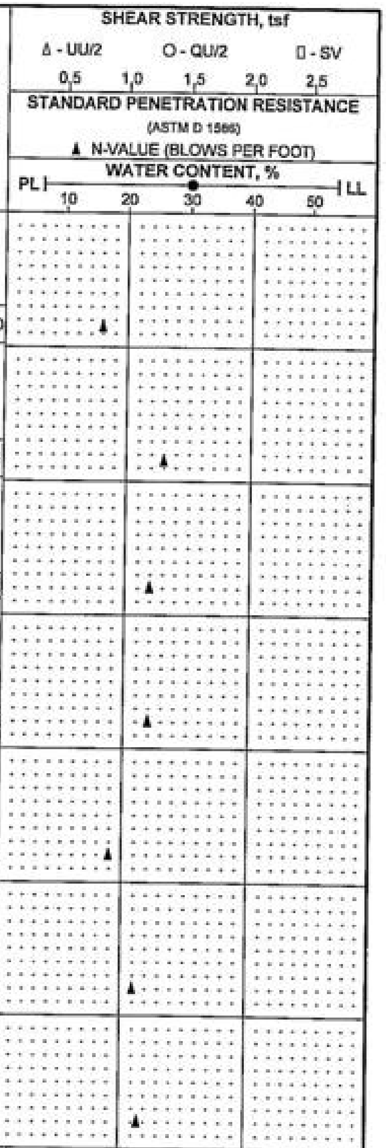
LOG OF BORING 2002 0869701.61KG.GPJ GTINC 0606301.GPJ 2/26/07

Surface Elevation 785.65

Datum NAVD 88

Completion Date: 11/9/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace clay, fine to coarse, medium dense (SP) *(continued)*		
		8-8-8	SS10
40			
	SAND, silty, fine to coarse,mdelum dense (SP)		
		8-12-14	SS11
45			
	SAND, trace silt, fine to medium, medium dense (SP)		
		6-12-12	SS12
50			
		10-12-12	SS13
55			
		7-8-10	SS14
60	fine to coarse		
		8-9-13	SS15
65	GRAVEL, sandy, fine , gray, medium dense (GS)		
		8-10-13	SS16



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Water was not encountered before wash boring. Offset 5' East.
Boring caved to 6 feet at 24 hours.
Northing: 2653593.31 Easing: 1197163.15

Drawn by: VS	Ck'd. by: LEO	App'vd. by: SOG
Date: 11/10/06	Date: 3-4-07	Date: 3-20-07

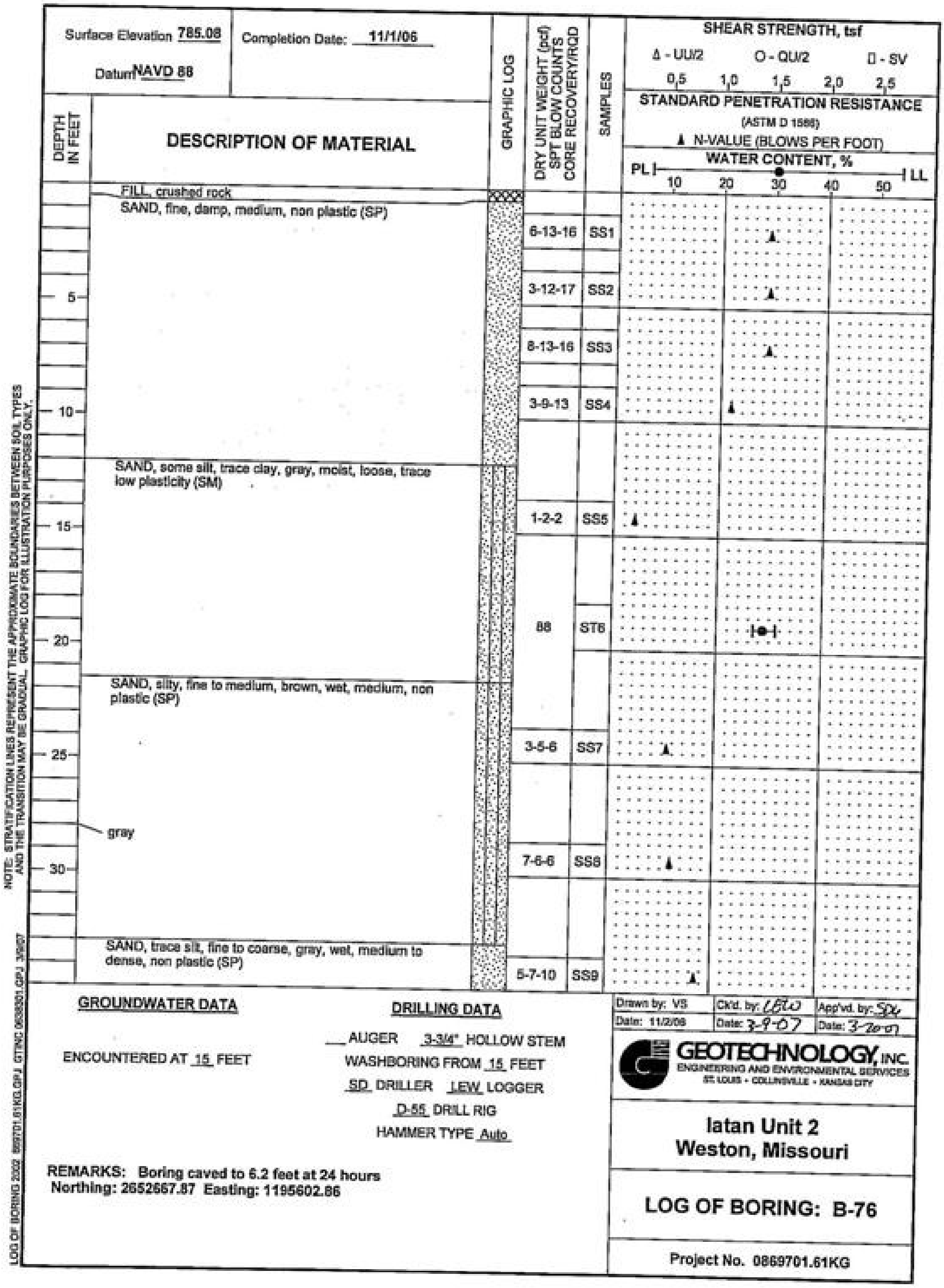
GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF LOG OF BORING: B-71

Project No. 0869701.61KG

LOG OF BORING 2002 869701.61KG.GPJ GTINC 0636301.GPJ 3/28/07



Surface Elevation 785.08 Completion Date: 11/1/06

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
0	FILL, crushed rock		
	SAND, fine, damp, medium, non plastic (SP)	6-13-16	SS1
5		3-12-17	SS2
		8-13-16	SS3
10		3-9-13	SS4
	SAND, some silt, trace clay, gray, moist, loose, trace low plasticity (SM)		
15		1-2-2	SS5
20		88	ST6
	SAND, silty, fine to medium, brown, wet, medium, non plastic (SP)		
25		3-5-6	SS7
	gray		
30		7-6-6	SS8
	SAND, trace silt, fine to coarse, gray, wet, medium to dense, non plastic (SP)	5-7-10	SS9

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-55 DRILL RIG
HAMMER TYPE Auto

REMARKS: Boring caved to 6.2 feet at 24 hours
Northing: 2652667.87 Easting: 1195602.86

Drawn by: VS	Ck'd. by: LBW	App'vd. by: SDW
Date: 11/2/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-76

Project No. 0869701.61KG

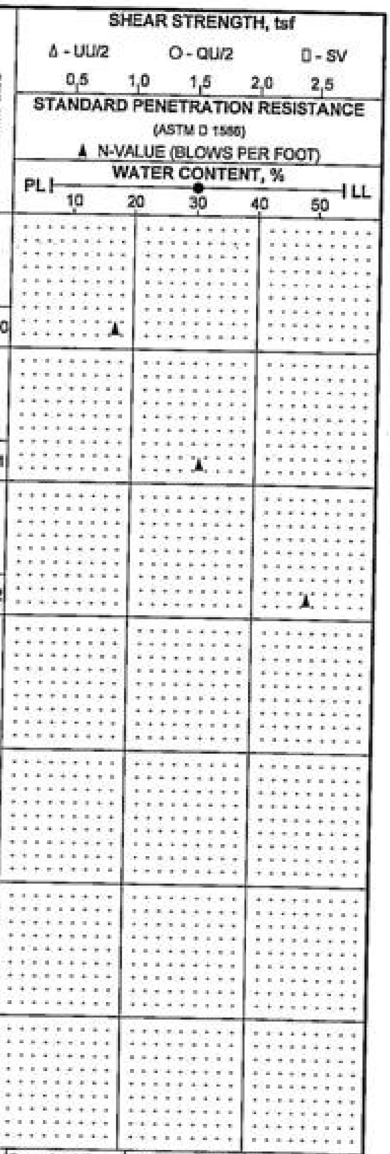
LOG OF BORING 2002 0869701.61KG.GPJ GTINC 0608301.GPJ 3/8/07

Surface Elevation 785.08

Datum NAVD 88

Completion Date: 11/1/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, fine to coarse, gray, wet, medium to dense, non plastic (SP) *(continued)*			
40			5-7-10	SS10
45			8-14-17	SS11
50			19-23-26	SS12
	Boring terminated at 50 feet			
55				
60				
65				



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-55 DRILL RIG
HAMMER TYPE Auto

REMARKS: Boring caved to 6.2 feet at 24 hours
Northing: 2652667.87 Easting: 1195502.86

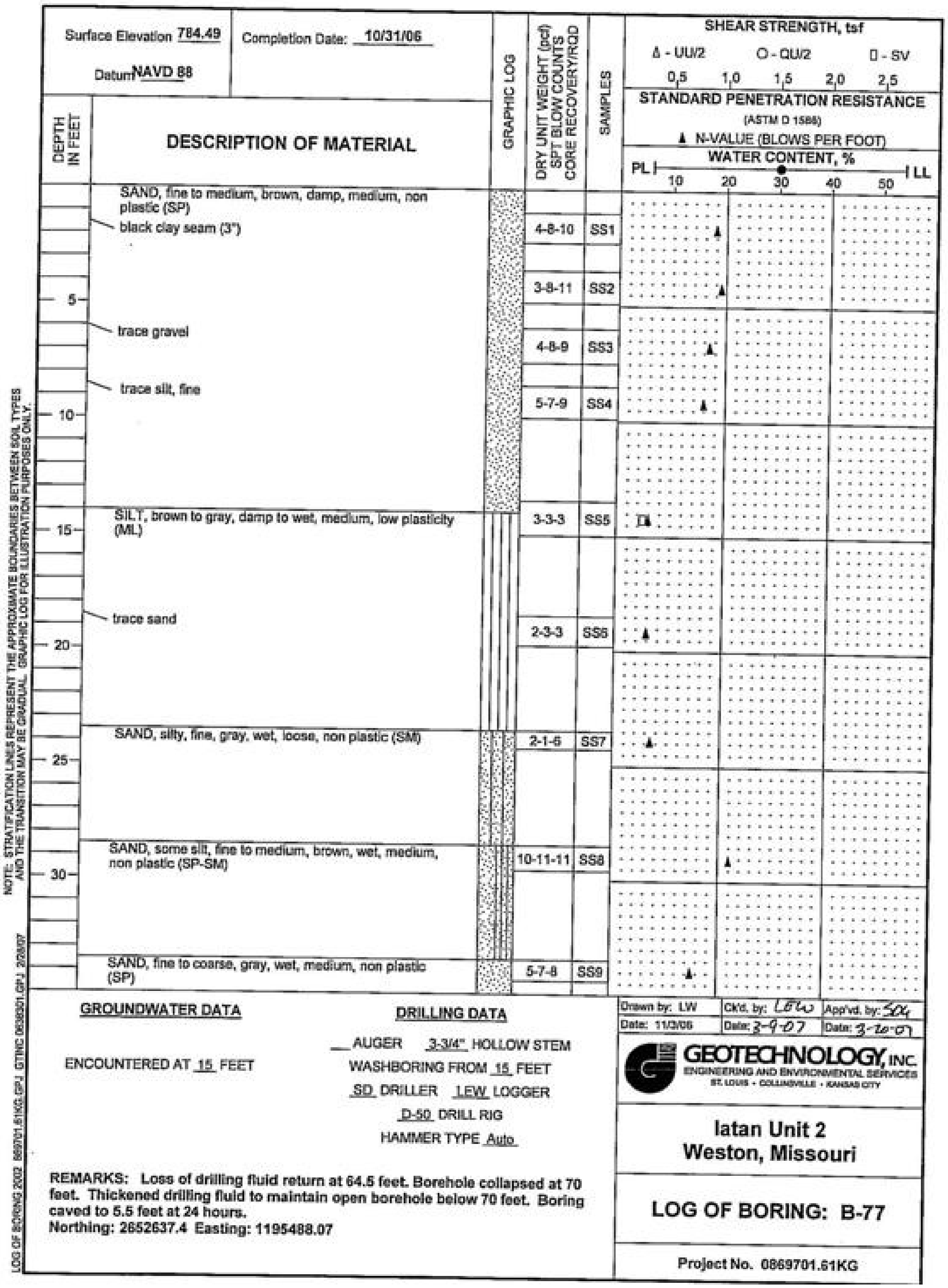
Drawn by: VS	Ck'd. by: LEW	App'vd. by: SDK
Date: 11/2/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B-76

Project No. 0869701.61KG



Surface Elevation 784.49 Completion Date: 10/31/06

Datum NAVD 88

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, fine to medium, brown, damp, medium, non plastic (SP)		
	black clay seam (3")	4-8-10	SS1
5		3-8-11	SS2
	trace gravel	4-8-9	SS3
10	trace silt, fine	5-7-9	SS4
15	SILT, brown to gray, damp to wet, medium, low plasticity (ML)	3-3-3	SS5
20	trace sand	2-3-3	SS6
25	SAND, silty, fine, gray, wet, loose, non plastic (SM)	2-1-6	SS7
30	SAND, some silt, fine to medium, brown, wet, medium, non plastic (SP-SM)	10-11-11	SS8
	SAND, fine to coarse, gray, wet, medium, non plastic (SP)	5-7-8	SS9

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Loss of drilling fluid return at 64.5 feet. Borehole collapsed at 70 feet. Thickened drilling fluid to maintain open borehole below 70 feet. Boring caved to 5.5 feet at 24 hours.
Northing: 2652637.4 Easting: 1195488.07

Drawn by: LW	Ck'd. by: LGW	App'vd. by: SCK
Date: 11/3/06	Date: 3-9-07	Date: 3-20-07

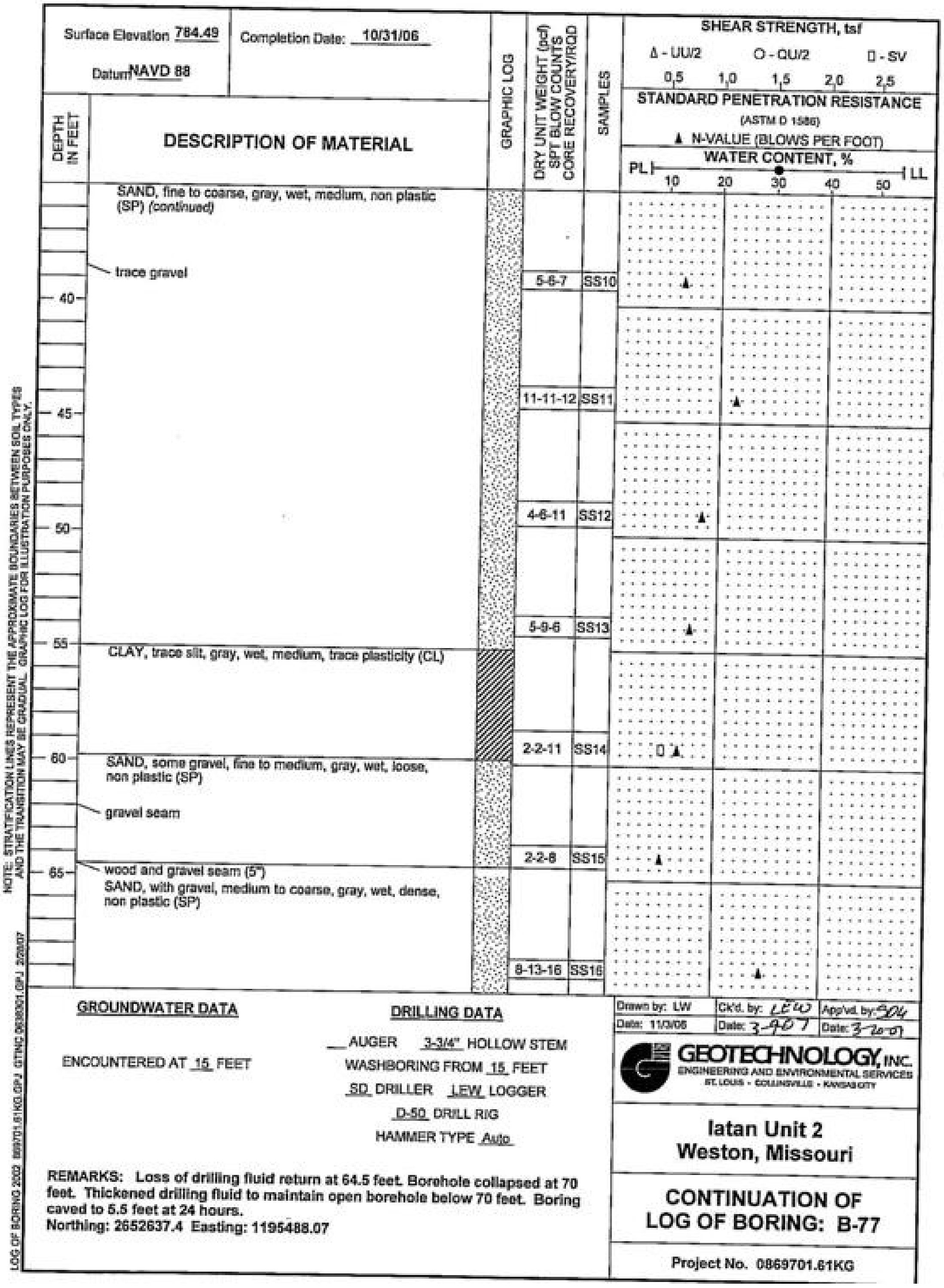
GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-77

Project No. 0869701.61KG

LOG OF BORING 2002 869701.61KG.GPJ GTINC 0636301.GPJ 2/28/07



Surface Elevation 784.49

Datum NAVD 88

Completion Date: 10/31/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, fine to coarse, gray, wet, medium, non plastic (SP) *(continued)*		
	trace gravel	5-6-7	SS10
40			
		11-11-12	SS11
45			
		4-6-11	SS12
50			
		5-9-6	SS13
55	CLAY, trace silt, gray, wet, medium, trace plasticity (CL)		
		2-2-11	SS14
60	SAND, some gravel, fine to medium, gray, wet, loose, non plastic (SP)		
	gravel seam		
		2-2-8	SS15
65	wood and gravel seam (5") SAND, with gravel, medium to coarse, gray, wet, dense, non plastic (SP)		
		8-13-16	SS16

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER LEW LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Loss of drilling fluid return at 64.5 feet. Borehole collapsed at 70 feet. Thickened drilling fluid to maintain open borehole below 70 feet. Boring caved to 5.5 feet at 24 hours.
Northing: 2652637.4 Easting: 1195488.07

Drawn by: LW	Ck'd. by: LEW	App'vd. by: SDW
Date: 11/3/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS - COLLINSVILLE - KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF
LOG OF BORING: B-77

Project No. 0869701.61KG

LOG OF BORING 2002 0869701.61KG.GPJ GTINC 0808001.GPJ 2/20/07

Surface Elevation 784.49

Datum NAVD 88

Completion Date: 10/31/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	SPT BLOW COUNTS	SAMPLES
	SAND, with gravel, medium to coarse, gray, wet, dense, non plastic (SP) *(continued)*		
75	GRAVEL, with sand, fine, gray, wet, medium, non plastic (GP)	11-13-11	SS17
80		5-9-9	SS18
85	SAND, some gravel, medium to coarse, brown, wet, medium, non plastic (SP)	15-13-13	SS19
	Boring terminated at 85 feet.		
90			
95			
100			

GRAPHIC LOG

DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

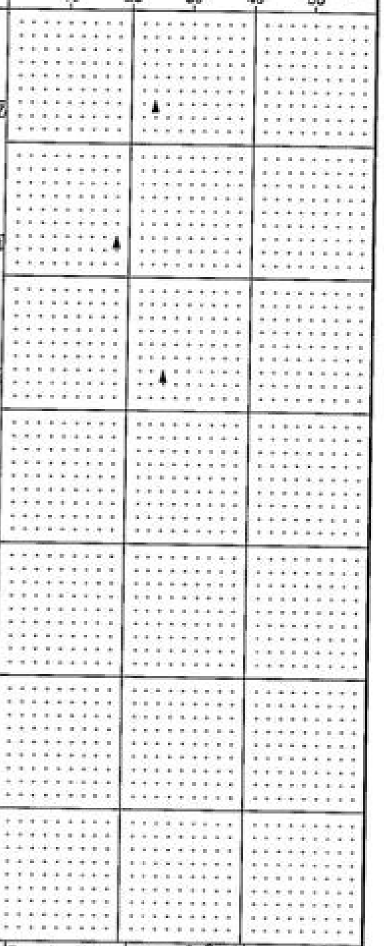
STANDARD PENETRATION RESISTANCE (ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢——●——⊣ LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2002 869701.61KG.GPJ GTINC 0630301.GPJ 2/28/07

GROUNDWATER DATA

ENCOUNTERED AT 15 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

SD DRILLER LEW LOGGER

D-50 DRILL RIG

HAMMER TYPE Auto

REMARKS: Loss of drilling fluid return at 64.5 feet. Borehole collapsed at 70 feet. Thickened drilling fluid to maintain open borehole below 70 feet. Boring caved to 5.5 feet at 24 hours.
Northing: 2652637.4 Easting: 1195488.07

Drawn by: LW	Ck'd. by: 3-4-07	App'vd. by: SDX
Date: 11/3/06	Date: [illegible]	Date: 3-2007

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

Iatan Unit 2
Weston, Missouri

CONTINUATION OF LOG OF BORING: B-77

Project No. 0869701.61KG

Surface Elevation 784.95

Datum NAVD 88

Completion Date: 10/27/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	TOPSOIL			
	CLAY, sandy, dark brown, moist, trace plasticity (CLS)			
	SAND, fine, brown, damp, medium, non plastic (SP)		3-6-9	SS1
5			4-9-10	SS2
			4-9-9	SS3
			3-4-6	SS4
10	loose			
			3-5-6	SS5
15	CLAY, sandy, gray, damp to wet, stiff, medium plastic (CL)			
	soft trace plasticity trace wood fragments			
20			2-1-1	SS6
	medium		4-2-3	SS7
25				
			5-9-9	SS8
30	SAND, trace silt, fine to coarse, gray, wet, medium, non plastic (SP)			
			10-13-10	SS9

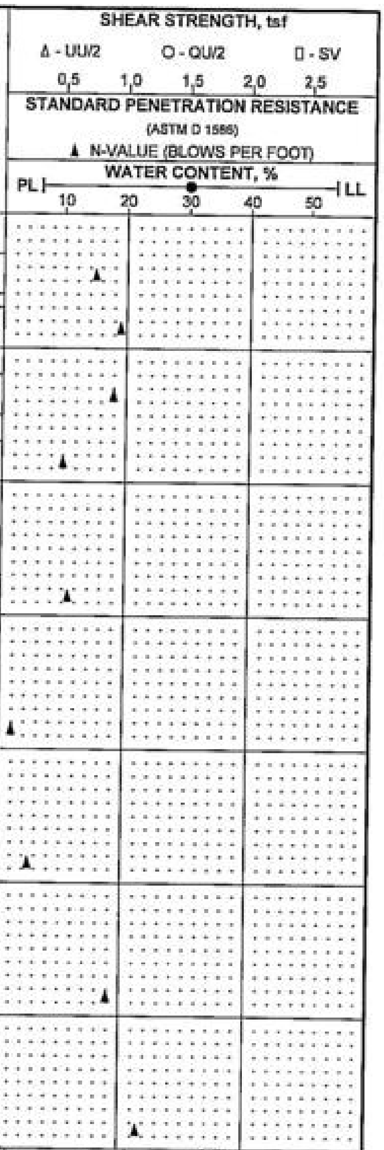
SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

STANDARD PENETRATION RESISTANCE (ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 14 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER JT LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Loss of circulation of wash water from 22.0 to 26.0 feet in the mud pit. Boring caved to 6.4 feet at 24 hours.
Northing: 2652632.07 Easting: 1195386.76

Drawn by: VS	Ck'd. by: LEW	App'vd. by: SDL
Date: 10/30/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS · COLLINSVILLE · KANSAS CITY

Iatan Unit 2
Weston, Missouri

LOG OF BORING: B-78

Project No. 0869701.61KG

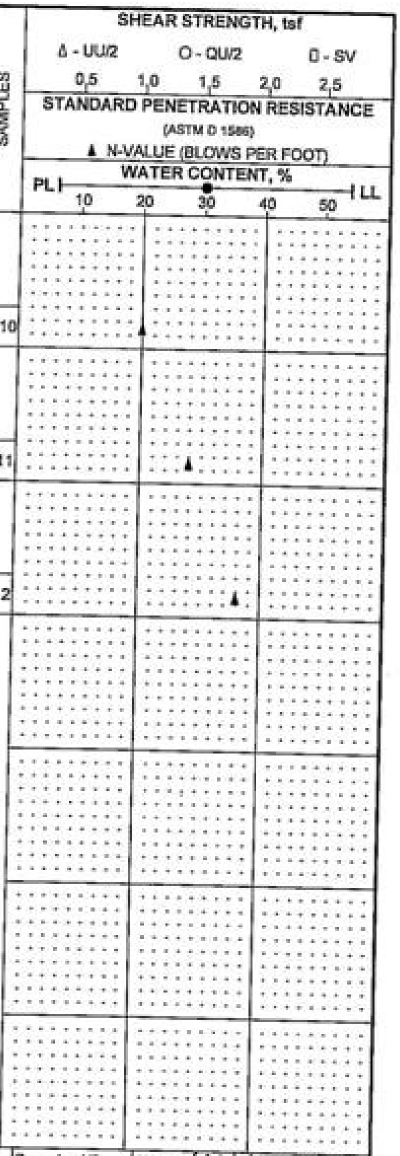
LOG OF BORING 2002 0869701.61KG.GPJ GTINC 0869301.GPJ 2/28/07

Surface Elevation 784.95

Datum NAVD 88

Completion Date: 10/27/06

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	SAND, trace silt, fine to coarse, gray, wet, medium, non plastic (SP) *(continued)*			
40			6-8-12	SS10
45			9-13-15	SS11
50	Boring terminated at 50 feet.		16-20-16	SS12
55				
60				
65				



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 14 FEET

DRILLING DATA

__ AUGER 3-3/4" HOLLOW STEM
WASHBORING FROM 15 FEET
SD DRILLER JT LOGGER
D-50 DRILL RIG
HAMMER TYPE Auto

REMARKS: Loss of circulation of wash water from 22.0 to 26.0 feet in the mud pit. Boring caved to 6.4 feet at 24 hours. Northing: 2652632.07 Easting: 1195386.76

Drawn by: VS	Ck'd. by: LEW	App'vd. by: SDG
Date: 10/30/06	Date: 3-9-07	Date: 3-20-07

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
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Iatan Unit 2
Weston, Missouri

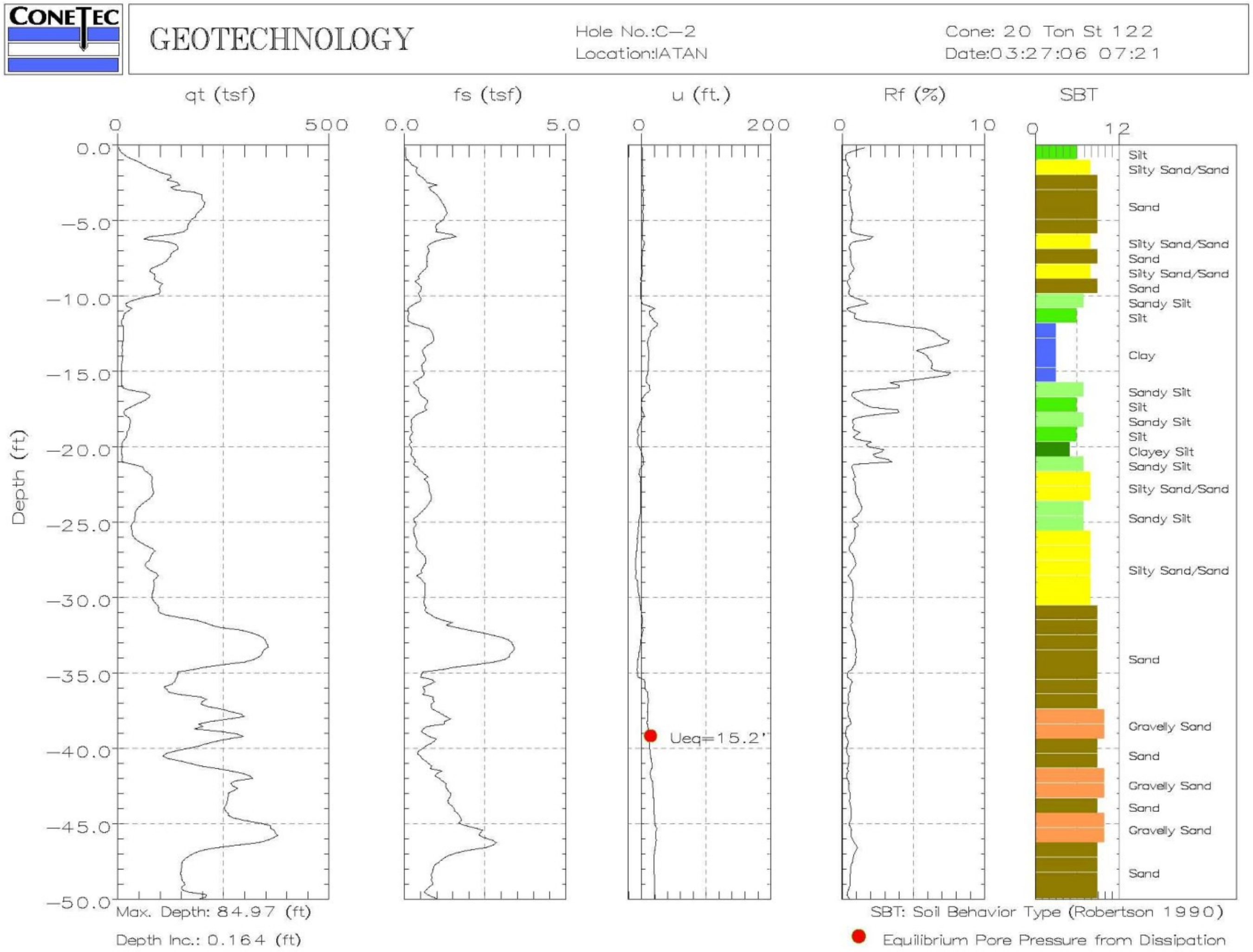
CONTINUATION OF LOG OF BORING: B-78

Project No. 0869701.61KG

LOG OF BORING 2002 869701.61KG.GPJ GTINC 0638301.GPJ 2/26/07

APPENDIX B

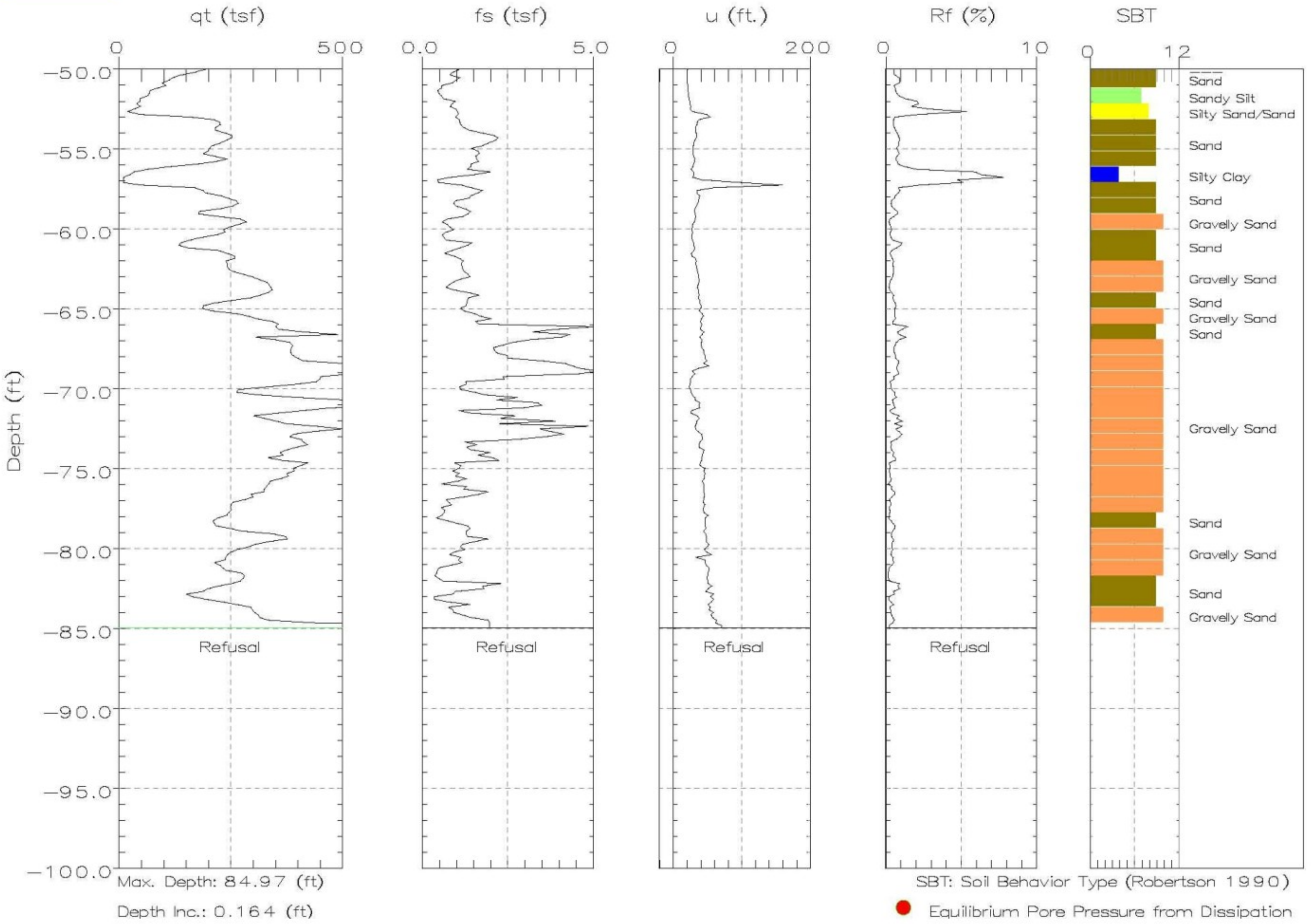
CPT SOUNDING LOGS


CONETEC
GEOTECHNOLOGY
Hole No.:C–2
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 07:21
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Rf (%)
0
10
SBT
0
12
Depth (ft)
0.0
–5.0
–10.0
–15.0
–20.0
–25.0
–30.0
–35.0
–40.0
–45.0
–50.0
Ueq=15.2'
Silt
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Sandy Silt
Silt
Clay
Sandy Silt
Silt
Sandy Silt
Silt
Clayey Silt
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
Sand
Max. Depth: 84.97 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

CONETEC GEOTECHNOLOGY

Hole No.:C–2
Location:IATAN

Cone: 20 Ton St 122
Date:03:27:06 07:21



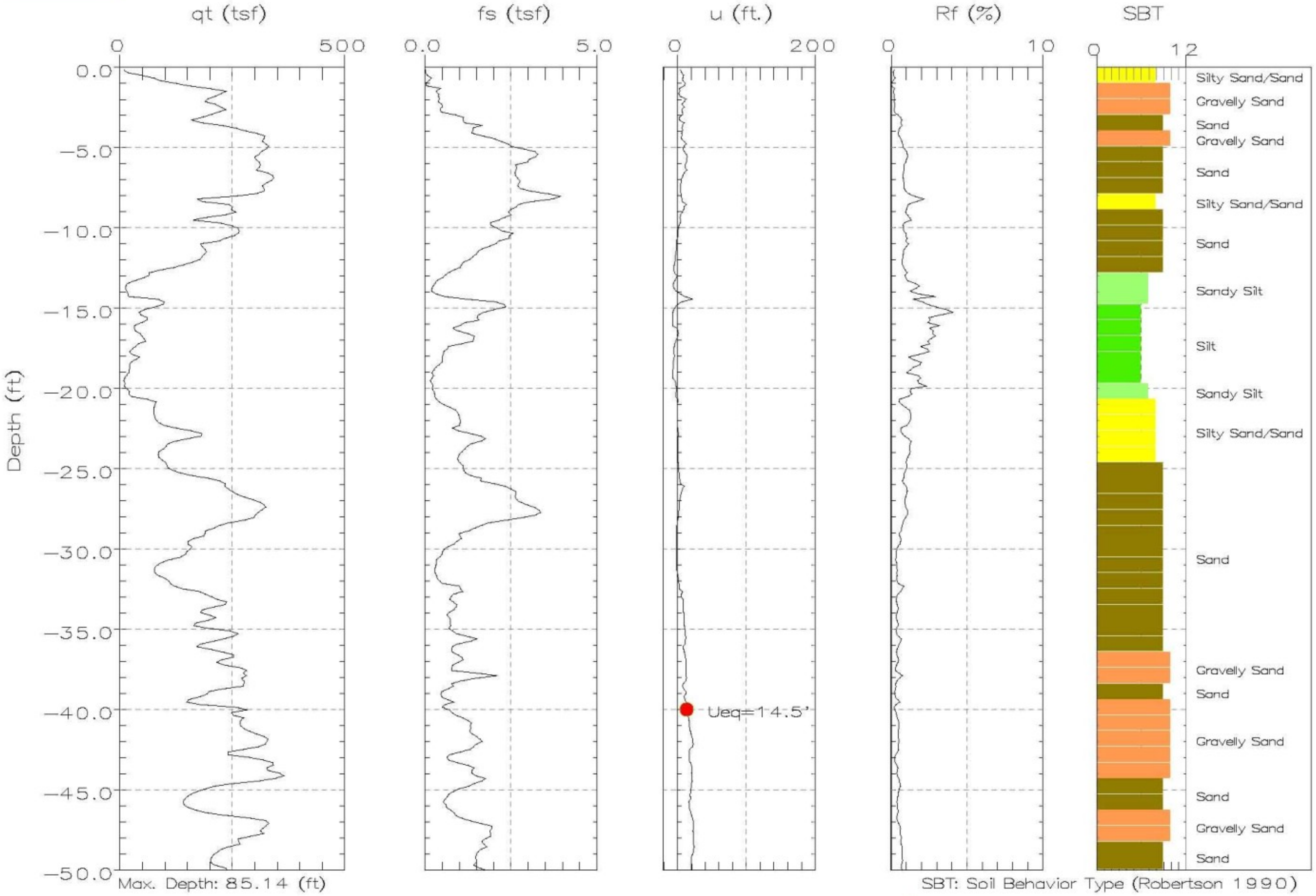
Max. Depth: 84.97 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

CONETEC **GEOTECHNOLOGY**

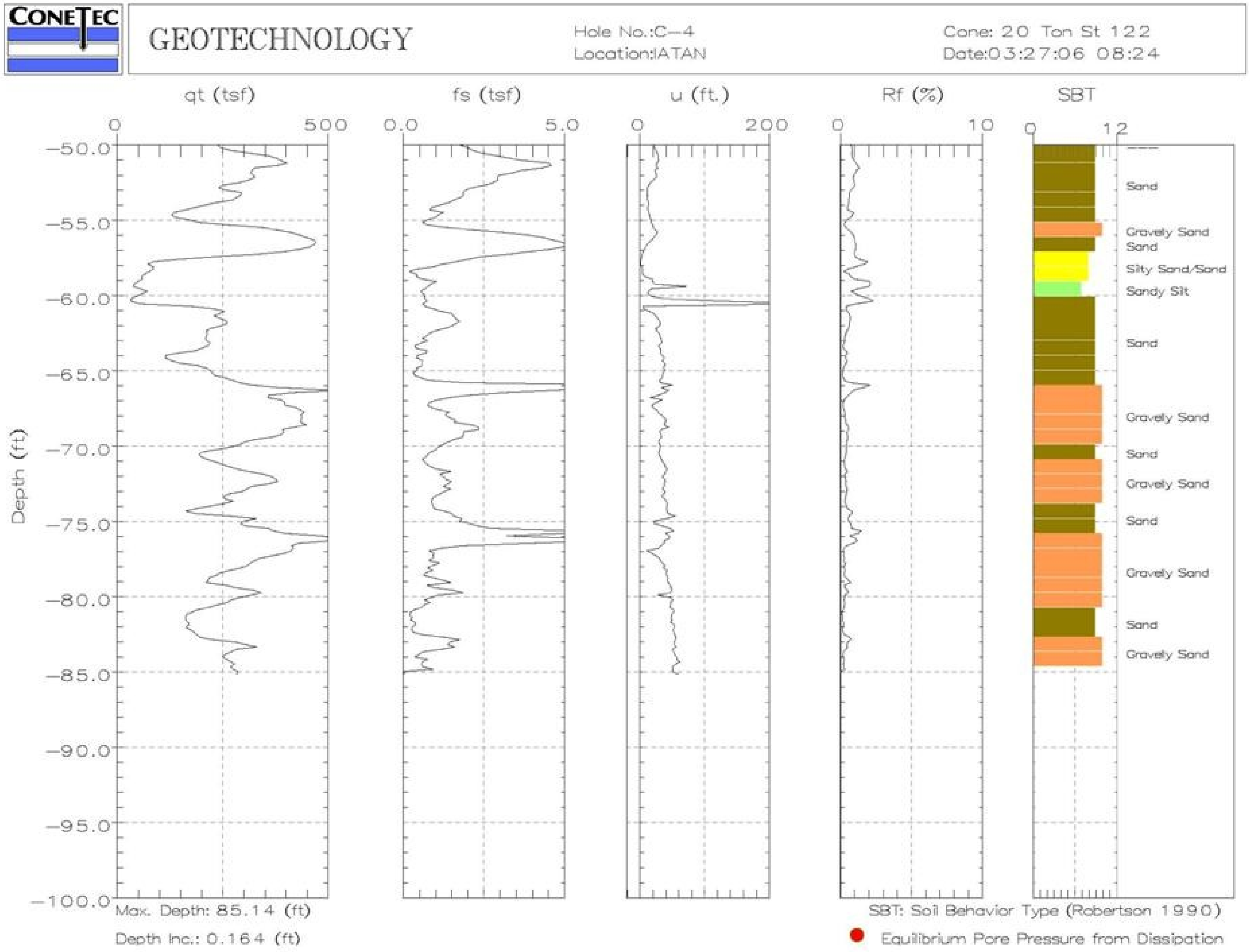
Hole No.:C–4
Location:IATAN

Cone: 20 Ton St 122
Date:03:27:06 08:24



Max. Depth: 85.14 (ft)
Depth Inc.: 0.164 (ft)

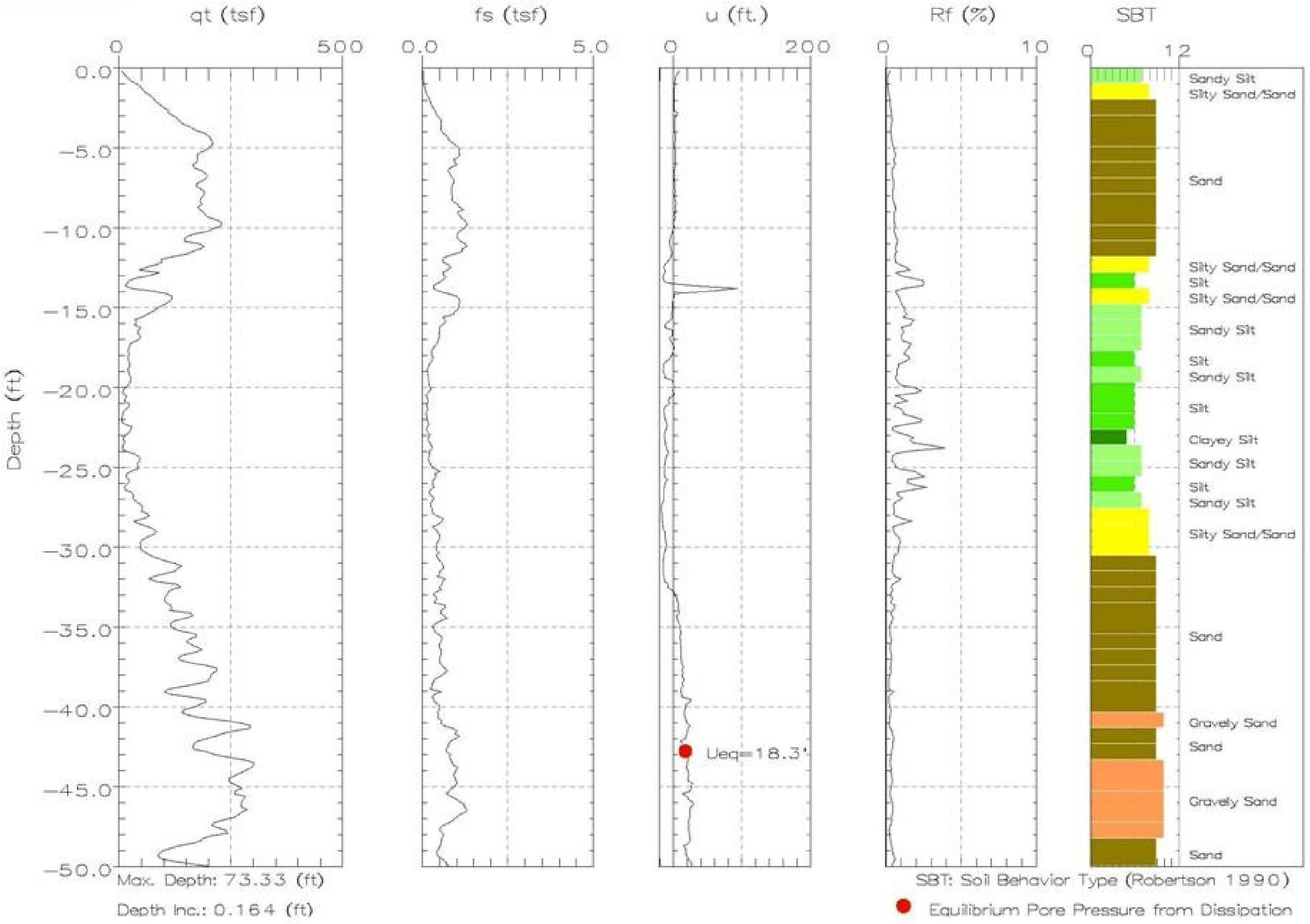
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

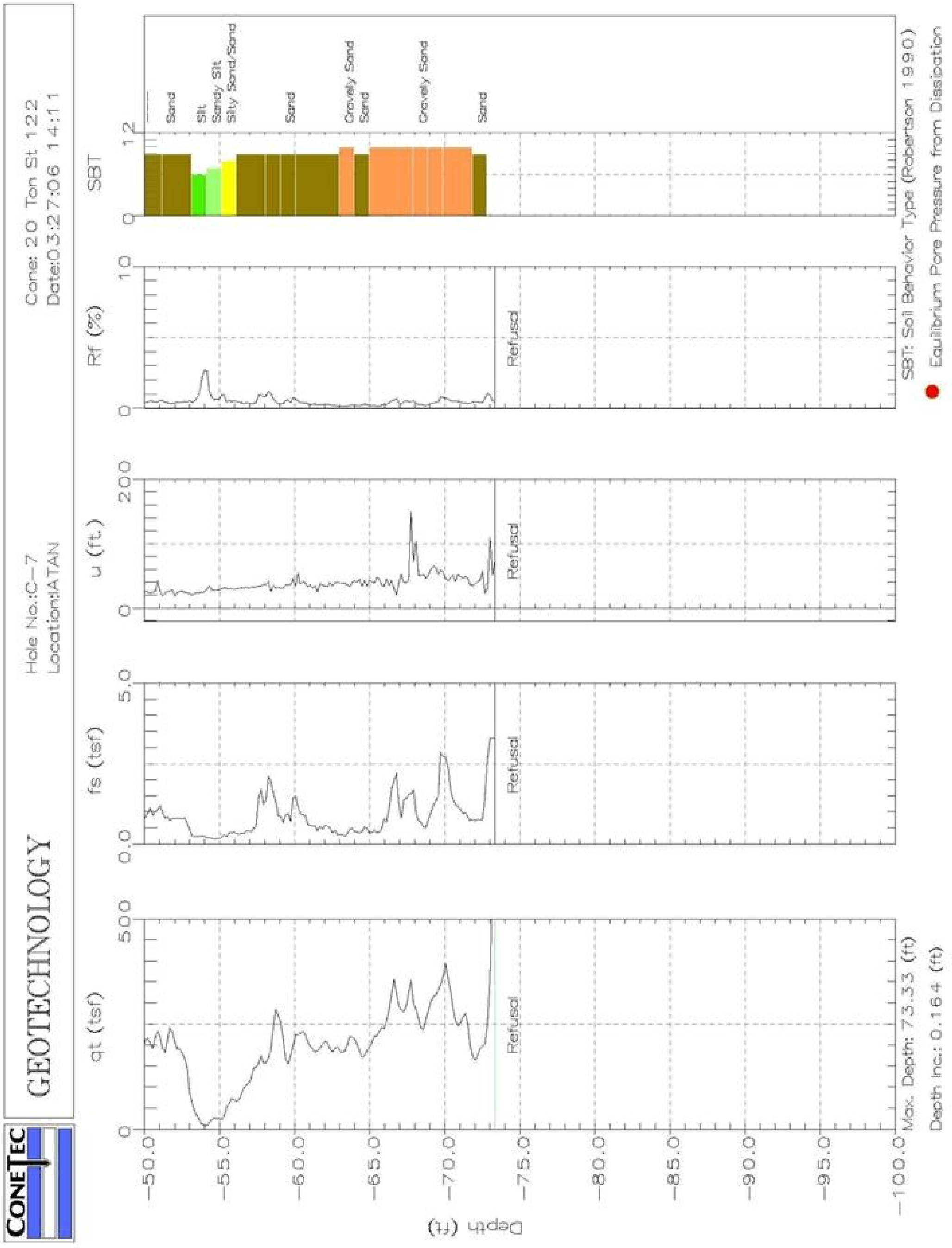

CONETEC
GEOTECHNOLOGY
Hole No.:C-4
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 08:24
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Rf (%)
0
10
SBT
0
12
Depth (ft)
-50.0
-55.0
-60.0
-65.0
-70.0
-75.0
-80.0
-85.0
-90.0
-95.0
-100.0
Sand
Gravely Sand
Sand
Silty Sand/Sand
Sandy Silt
Sand
Gravely Sand
Sand
Gravely Sand
Sand
Gravely Sand
Sand
Gravely Sand
Max. Depth: 85.14 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

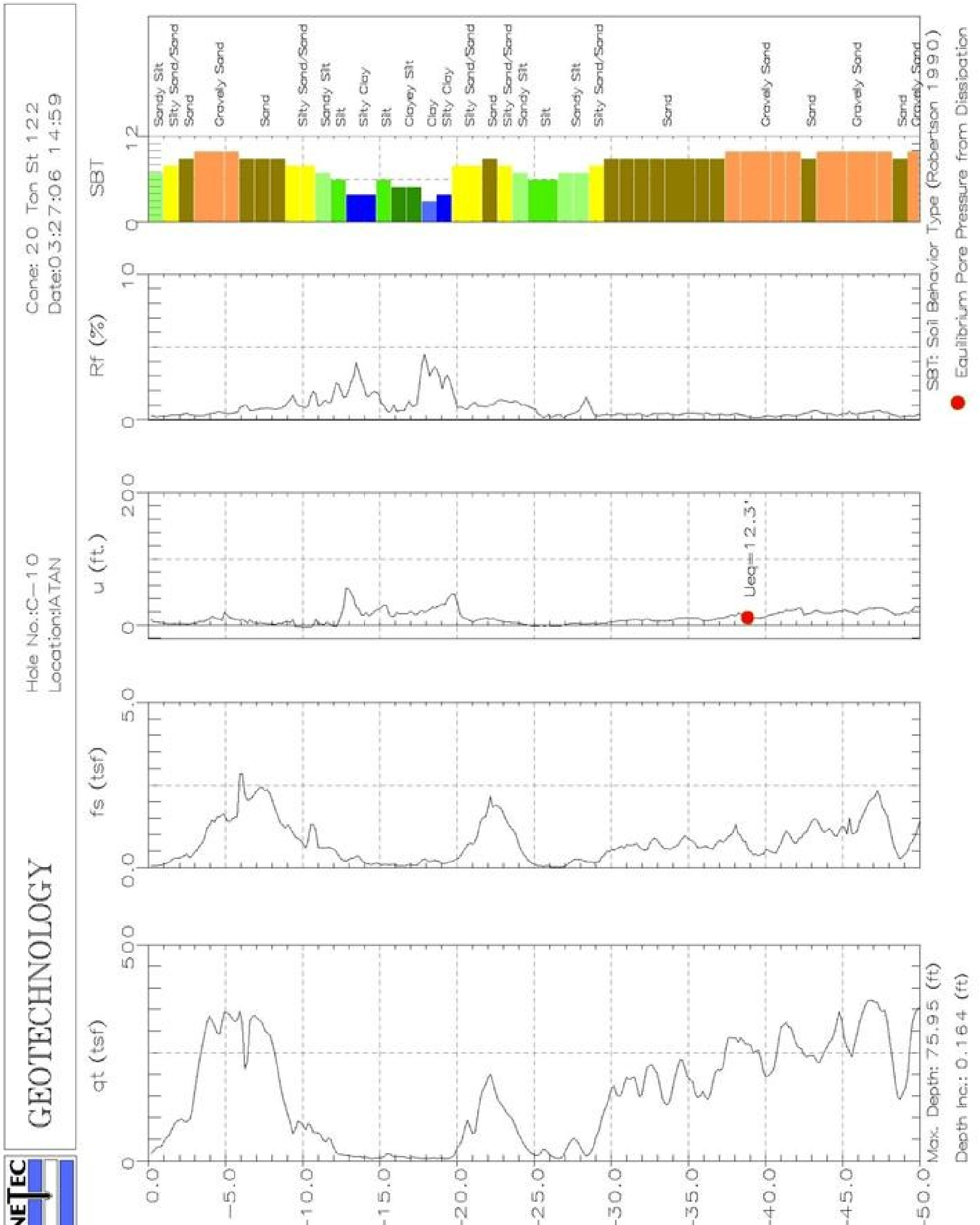
CONETEC GEOTECHNOLOGY

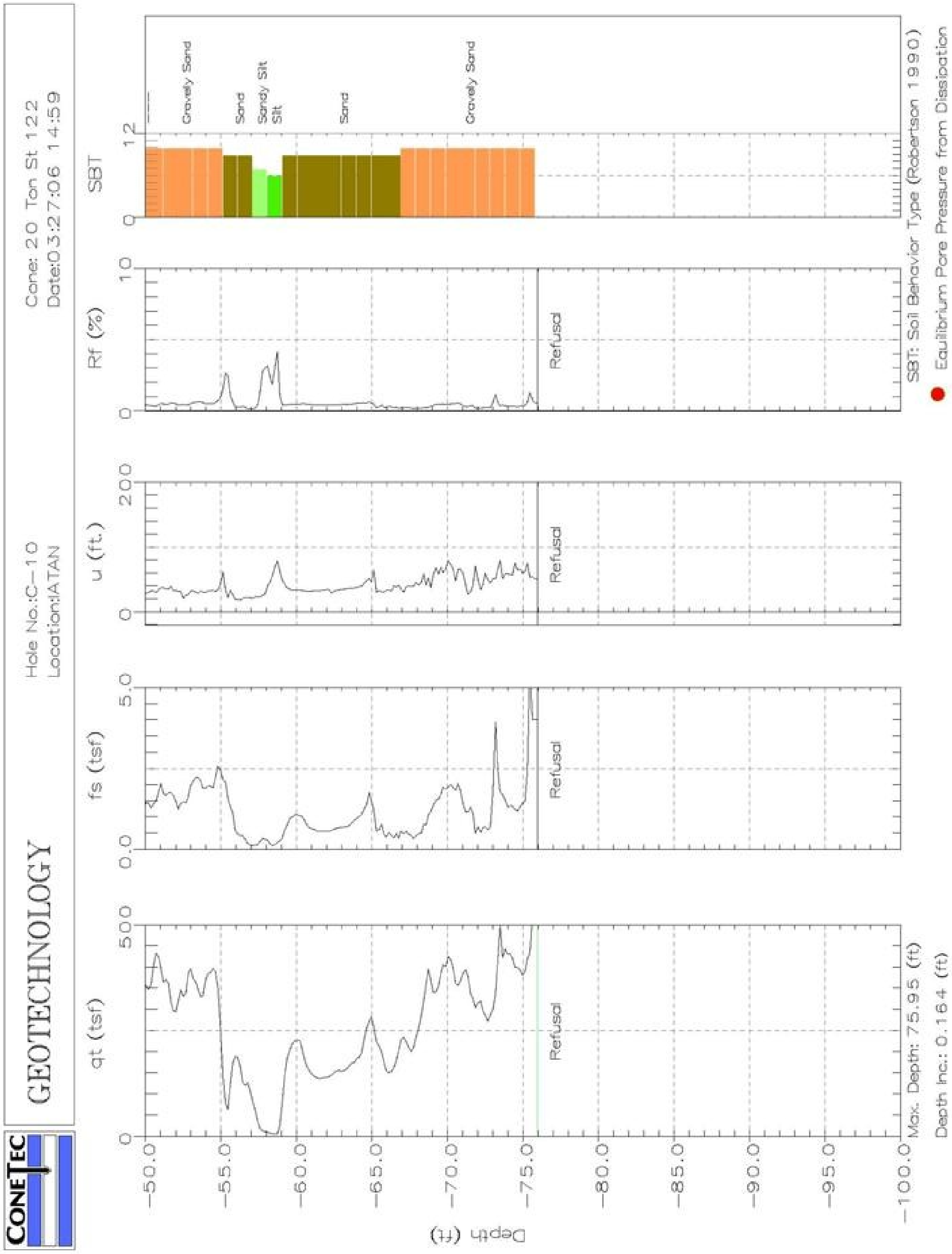
Hole No.:C-7
Location:IATAN

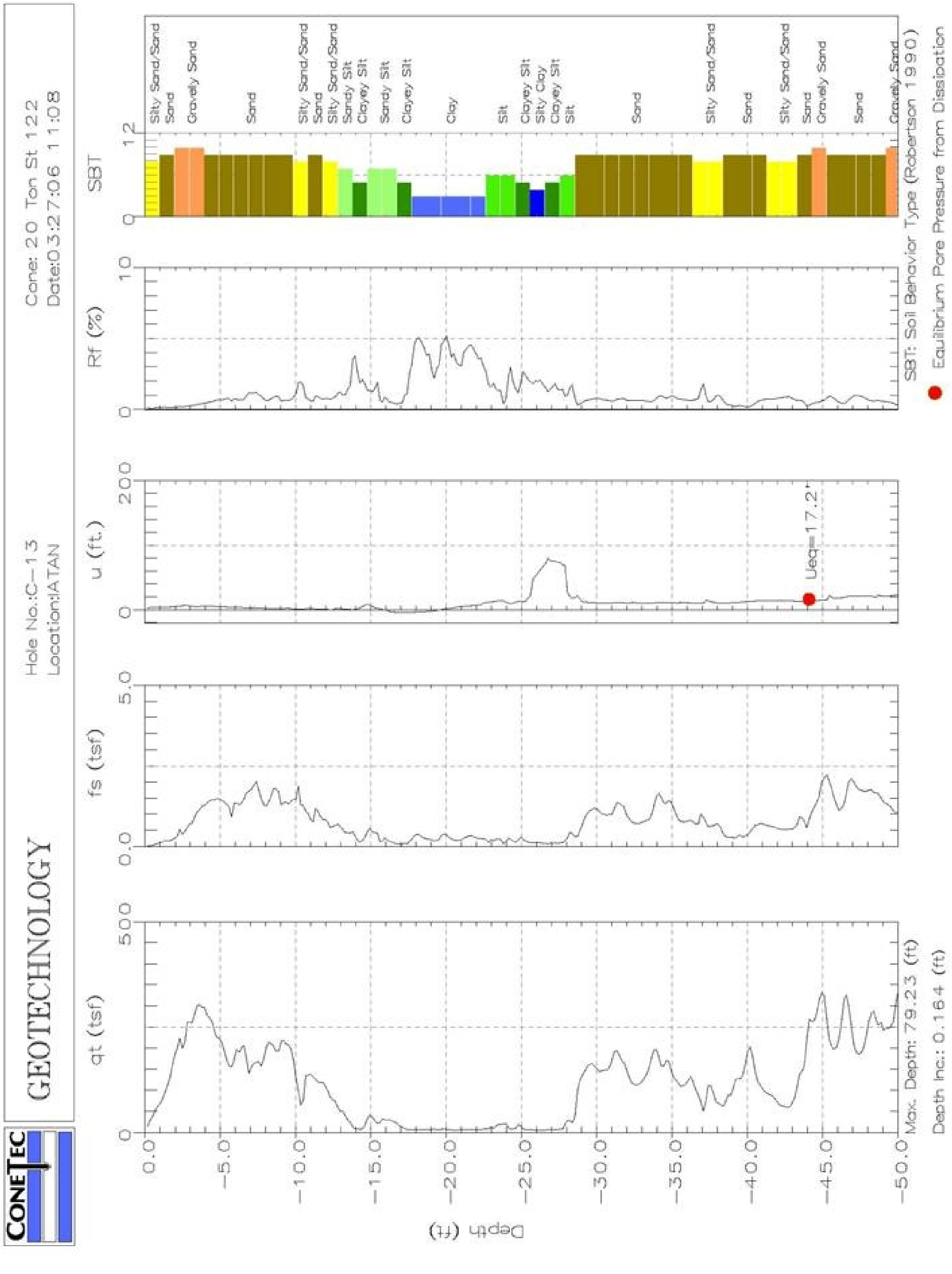
Cone: 20 Ton St 122
Date:03:27:06 14:11

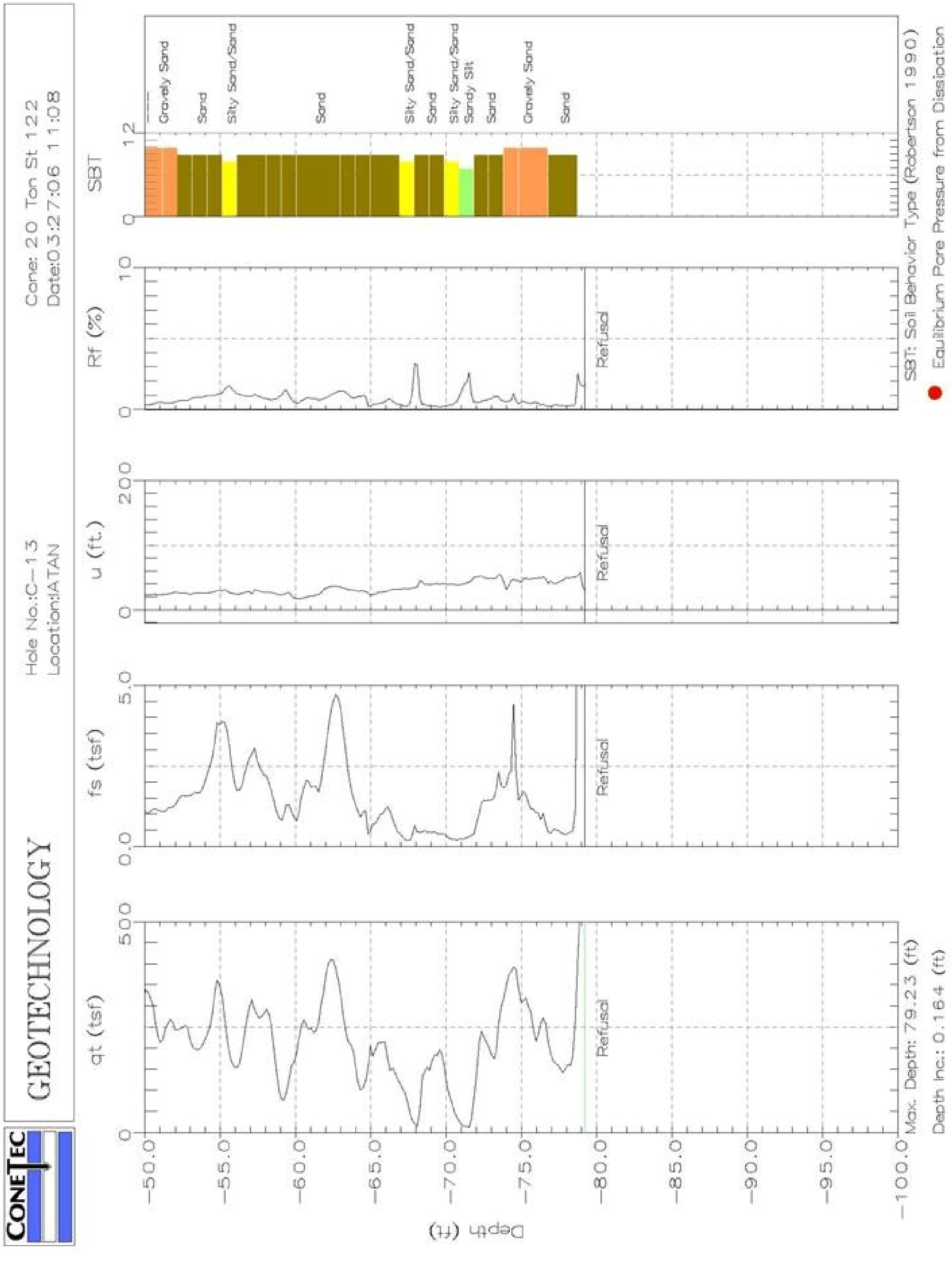


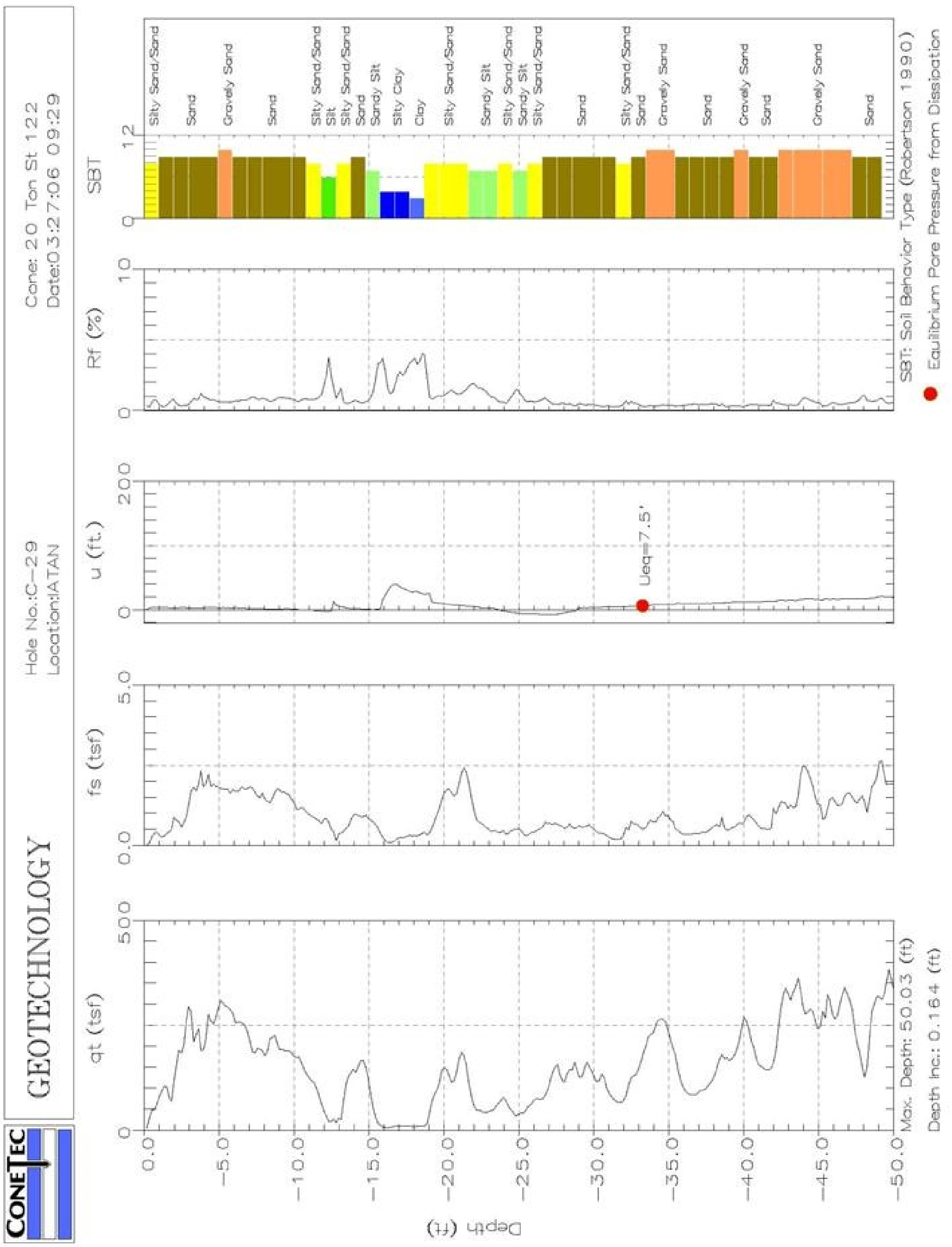

ConeTec
GEOTECHNOLOGY
Hole No.:C-7
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 14:11
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Rf (%)
0
10
SBT
0
12
Depth (ft)
-50.0
-55.0
-60.0
-65.0
-70.0
-75.0
-80.0
-85.0
-90.0
-95.0
-100.0
Refusal
Sand
Silt
Sandy Silt
Silty Sand/Sand
Sand
Gravely Sand
Sand
Gravely Sand
Sand
Max. Depth: 73.33 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

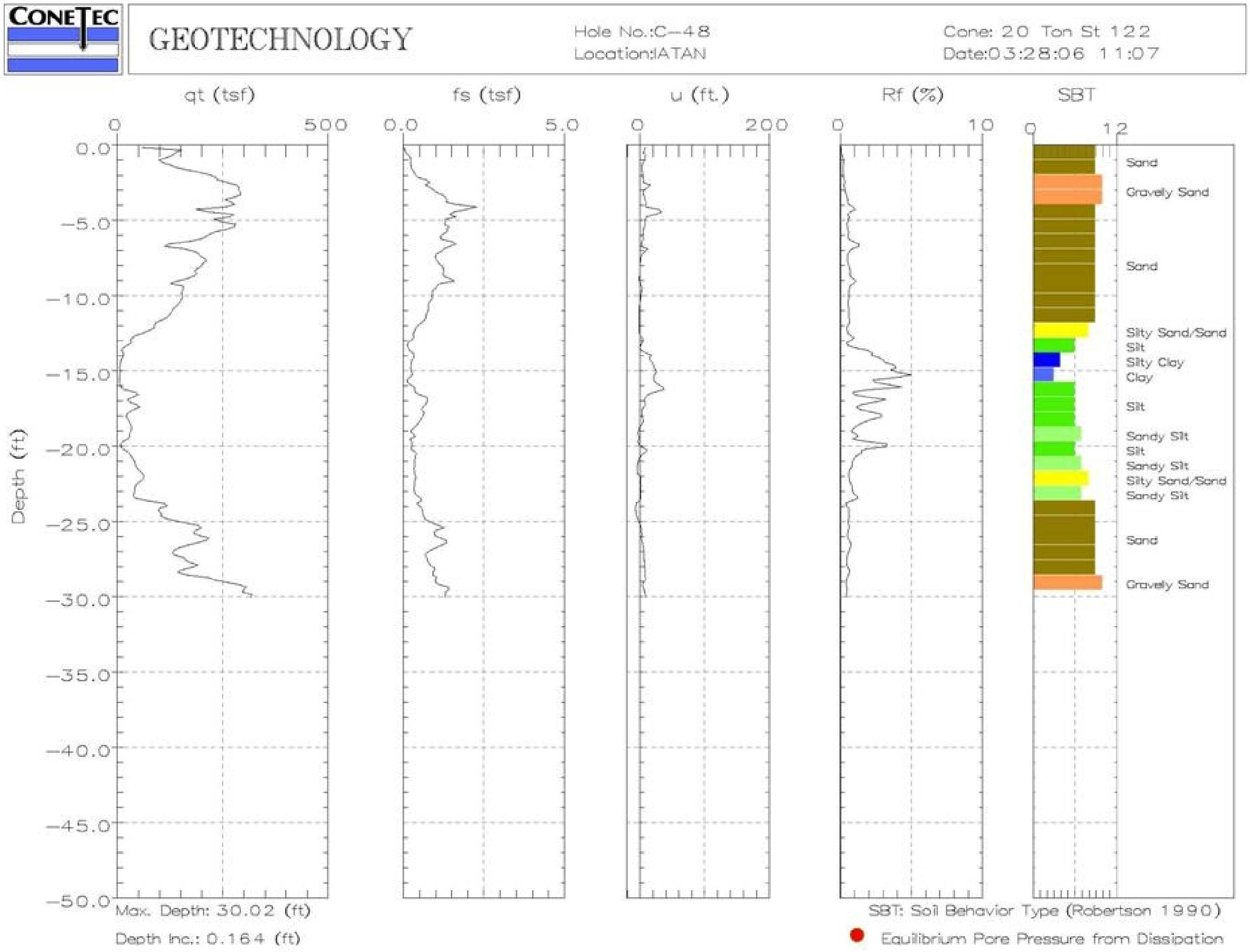

ConeTec
GEOTECHNOLOGY
Hole No.:C-10
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 14:59
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Ueq=12.3'
Rf (%)
0
10
SBT
0
12
Sandy Silt
Silty Sand/Sand
Sand
Gravely Sand
Sand
Silty Sand/Sand
Sandy Silt
Silt
Silty Clay
Silt
Clayey Silt
Clay
Silty Clay
Silty Sand/Sand
Sand
Silty Sand/Sand
Sandy Silt
Silt
Sandy Silt
Silty Sand/Sand
Sand
Gravely Sand
Sand
Gravely Sand
Sand
Gravely Sand
Depth (ft)
0.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
-35.0
-40.0
-45.0
-50.0
Max. Depth: 75.95 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation


CONETEC
GEOTECHNOLOGY
Hole No.:C-10
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 14:59
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Rf (%)
0
10
SBT
0
12
Depth (ft)
-50.0
-55.0
-60.0
-65.0
-70.0
-75.0
-80.0
-85.0
-90.0
-95.0
-100.0
Refusal
Refusal
Refusal
Refusal
Gravelly Sand
Sand
Sandy Silt
Silt
Sand
Gravelly Sand
Max. Depth: 75.95 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation


ConeTec
GEOTECHNOLOGY
Hole No.:C-13
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 11:08
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Ueq=17.2'
Rf (%)
0
10
SBT
0
12
Depth (ft)
0.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
-35.0
-40.0
-45.0
-50.0
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sandy Silt
Clayey Silt
Sandy Silt
Clayey Silt
Clay
Silt
Clayey Silt
Silty Clay
Clayey Silt
Silt
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
Max. Depth: 79.23 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
Equilibrium Pore Pressure from Dissipation


CONETEC
GEOTECHNOLOGY
Hole No.:C-13
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 11:08
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Rf (%)
0
10
SBT
0
12
Depth (ft)
-50.0
-55.0
-60.0
-65.0
-70.0
-75.0
-80.0
-85.0
-90.0
-95.0
-100.0
Refusal
Refusal
Refusal
Refusal
Gravely Sand
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sandy Silt
Sand
Gravely Sand
Sand
Max. Depth: 79.23 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation


CONETEC
GEOTECHNOLOGY
Hole No.:C-29
Location:IATAN
Cone: 20 Ton St 122
Date:03:27:06 09:29
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Ueq=7.5'
Rf (%)
0
10
SBT
0
12
Depth (ft)
0.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
-35.0
-40.0
-45.0
-50.0
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Silty Sand/Sand
Silt
Silty Sand/Sand
Sand
Sandy Silt
Silty Clay
Clay
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Sand
Silty Sand/Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
Sand
Gravelly Sand
Sand
Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
Equilibrium Pore Pressure from Dissipation



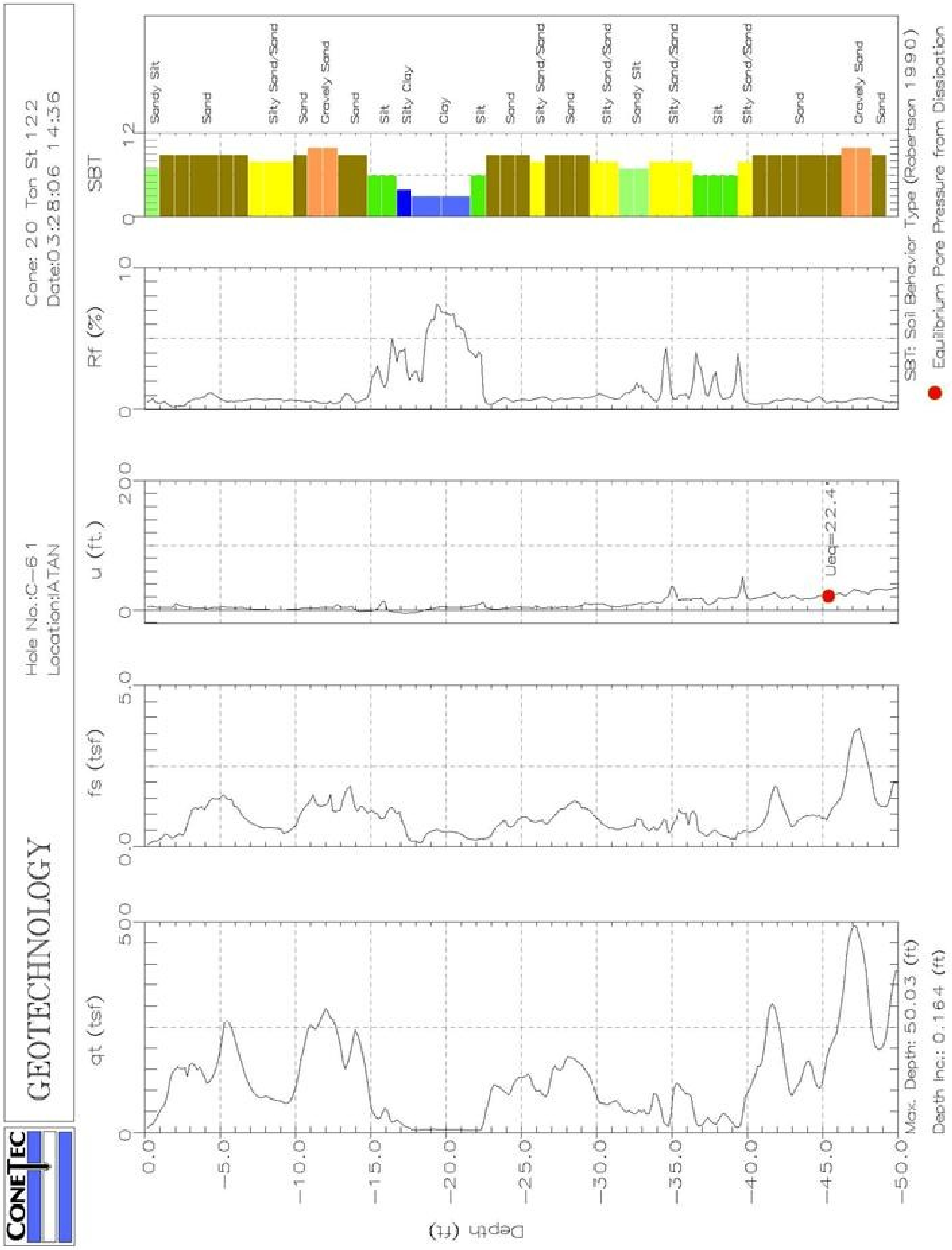
GEOTECHNOLOGY

Hole No.:C-48
Location:IATAN

Cone: 20 Ton St 122
Date:03:28:06 11:07

Max. Depth: 30.02 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation


CONETEC
GEOTECHNOLOGY
Hole No.:C-61
Location:IATAN
Cone: 20 Ton St 122
Date:03:28:06 14:36
qt (tsf)
0
500
fs (tsf)
0.0
5.0
u (ft.)
0
200
Ueq=22.4'
Rf (%)
0
10
SBT
0
12
Depth (ft)
0.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
-35.0
-40.0
-45.0
-50.0
Sandy Silt
Sand
Silty Sand/Sand
Sand
Gravely Sand
Sand
Silt
Silty Clay
Clay
Silt
Sand
Silty Sand/Sand
Sand
Silty Sand/Sand
Sandy Silt
Silty Sand/Sand
Silt
Silty Sand/Sand
Sand
Gravely Sand
Sand
Max. Depth: 50.03 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)
● Equilibrium Pore Pressure from Dissipation

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VITA

Bradley Scott Gardner was born on June 22, 1972, in Dallas, Texas. His family moved to the Kansas City area in 1973 and he grew up in Liberty, Missouri. He attended elementary and secondary schools in the Liberty Public School District and graduated from Liberty High School in 1990. He attended the University of Missouri – Rolla in Rolla, Missouri and graduated with his Bachelor of Science in Geological Engineering in December 1994.

Mr. Gardner began his career as an environmental engineer with ABB-Environmental Services (ABB-ES) in Tallahassee, Florida. During his two years with ABB-ES, he served as the Field Operations Leader for on-going site characterization and remediation work at the Marine Corps Logistics Base in Albany, Georgia.

In June of 1996, he met his future wife, Kimberly, at a wedding and by the end of that year, he had decided to relocate back to the Kansas City area. Mr. Gardner went to work for HNTB Corporation in January 1997 where he began a new phase of his career focusing on geotechnical engineering. In early 1999 they moved to Greeley, Colorado, and Mr. Gardner spent two years working as a geotechnical engineer with Rocky Mountain Consultants in Longmont, Colorado.

In September of 2001, Mr. Gardner and his wife returned to Kansas City and he went to work for Burns & McDonnell where he is currently still employed as an Associate Geotechnical Engineer. Upon completion of his degree requirements, Mr. Gardner plans to continue working with Burns & McDonnell.