This document was prepared in conjunction with work accomplished under Contract No. DE-AC09-96SR18500 with the U.S. Department of Energy.

Geotechnical Engineering at the Savannah River Site and Bechtel

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ABSTRACT: The authors describe two aspects of geotechnical engineering; site characterization utilizing the CPT and recognition of aging as a factor affecting soil properties. These methods were pioneered by Professor Schmertmann and are practiced by the Bechtel Corporation in general and at the Savannah River Site in South Carolina, in particular. This paper describes a general subsurface exploration approach that we have developed over the years. It consists of “phasing” the investigation, employing the principles of the observational method suggested by Peck (1969) and others. In doing so, we have found that the recommendations proposed by Sowers in terms of borehole spacing and exploration cost, are reasonable for developing an investigation program, recognizing that through continuous review the final investigation program will evolve.

At the SRS shallow subsurface soils are of Eocene and Miocene age. It was recognized that the age of these deposits would have a marked effect on their cyclic resistance. A field investigation and laboratory testing program was devised to measure and account for aging as it relates to the cyclic resistance of the site soils. Recently, a panel of experts (Youd et al., 2001) has made recommendations regarding the liquefaction assessment of soils. This paper will address some of those recommendations in the context of re-assessing the liquefaction resistance of the soils at the SRS. It will be shown that, indeed, aging plays a major role in the cyclic resistance of the soils at the SRS, and that aging should be accounted for in liquefaction potential assessments for soils older than Holocene age.

INTRODUCTION

Professor Schmertmann’s contributions to the Geotechnical Engineering profession, spanning over 50 years, have dealt with numerous aspects of soil mechanics important to practicing engineers. His papers, presentations, research reports, and technical discussions published in the ASCE Geotechnical Journals, at ASCE conferences, ASTM Special Technical Publications, at International Conferences, and Public Agency Research Reports cover many aspects of geotechnical engineering, but in particular they relate the application of laboratory and field testing to the strength and compressibility characterization of *in situ* soils. He published guidelines for the interpretation of Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) as early as 1970 (Schmertmann, 1970). His ideas about the potential of these two tests, improved in the subsequent years through lessons learned from additional research and case histories. Although the SPT is giving way to many other *in situ* tests, the CPT and SPT still constitute two of the most important tools for geotechnical site characterization.

For the twenty-fifth Terzaghi Lecture, Professor Schmertmann chose the important topic of aging (Schmertmann, 1991) as it affects soil properties. In his lecture, he elaborated on the impact of aging on soil compressibility, stress-strain characteristics, static and cyclic strength, liquefaction resistance, and on other properties, based on numerous laboratory test results and observations compiled from well-documented case histories. The first part of this paper addresses exploration and the use of the CPT, while the second part of the paper addresses aging of soils and the role that it plays on the dynamic strength of soils.

BACKGROUND

The Savannah River Site (SRS) is located along the Savannah River in the upper portion of the Atlantic Coastal Plain of South Carolina, approximately 160 km upstream of Savannah, Georgia (Figure 1). The SRS occupies about 830 km^2 and is owned by the Department of Energy (DOE). Since its inception in the early 1950s, the SRS has been an integral part of our nation's defense. As a result, several critical facilities have been, and will continue to be, constructed and operated at the SRS. By their nature, these facilities demand the very best we can offer in design and construction and all of the trappings that follow nuclear and defense-related projects, all in an effort to ensure safety during construction, operation, and eventual decommissioning. From a geologic and geotechnical standpoint, the SRS presents a number of interesting challenges. We discuss two of those challenges in this paper; site characterization and how we utilize the CPT; and the effect that age plays in the cyclic strength of soil deposits.

SITE CHARACTERIZATION

For geotechnical engineers and geologists, site characterization is the most important aspect of our work, for without an accurate depiction of the subsurface conditions and the geology of a site, subsequent analyses are guesswork. In recent times, however, it appears this activity has been receiving less and less attention, or at least it may be taken for granted. We're not sure of the reasons, but we believe one aspect is the ever increasing reliance on modeling, parametric analyses, and statistical inference. While these activities are important and clearly play an integral role in site characterization, they are no substitute for carefully planned and executed subsurface exploration programs. In fact, they should go hand-in-hand.

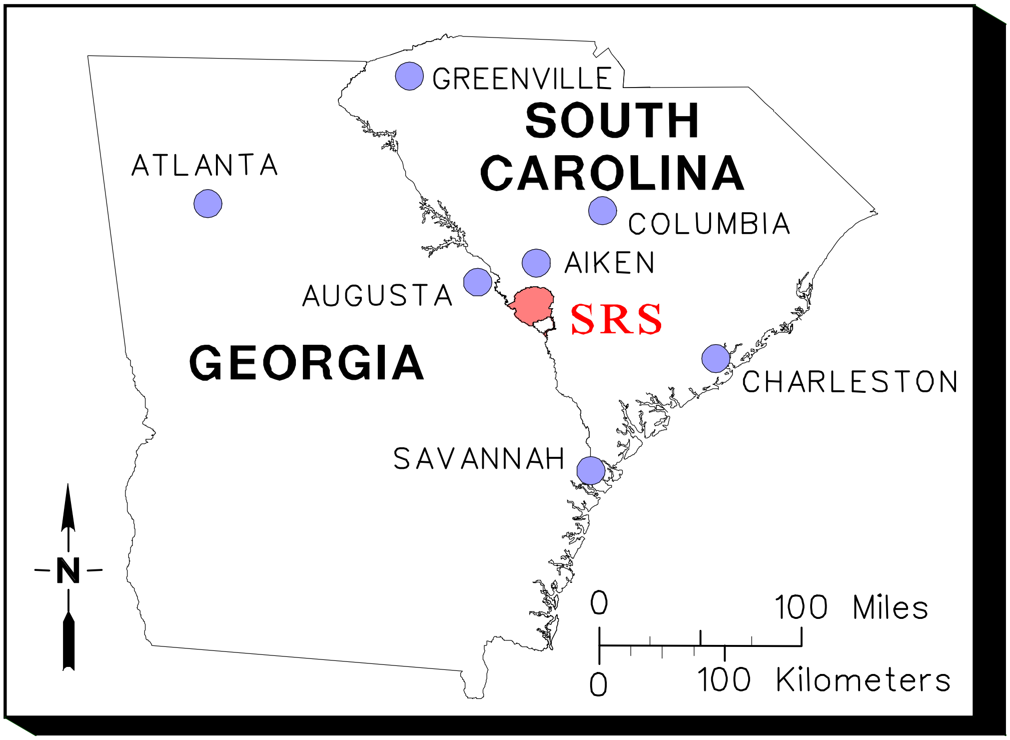

FIG. 1 Savannah River Site and surrounding region

What is site characterization? Gould (1985) described site characterization as "…to describe a site by a statement of its characteristics." Sowers (1979) described it as; "…a program of site investigation that will identify the significant underground conditions and define the variability as far as practical." More recently Baecher and Christian (2003) described site characterization as "a plan of action for obtaining information on site geology and for obtaining estimates of parameters to be used in modeling engineering performance." From our perspective, site characterization is the determination of subsurface conditions by;

- Understanding local/regional geology; (through site visits, geologic mapping and interpretation, and aerial photo interpretations),
- Performing appropriate geophysical surveys, borings, sampling, *in situ* testing (SPT, CPT, DMT, PMT, FV, etc.), and groundwater and piezometric observations,
- Completing appropriate laboratory testing, engineering analyses and modeling,
- Reviewing and interpreting the performance of nearby facilities, and
- Applying individual and collective experience, including site-specific (local) knowledge and general professional judgment.

Additionally, and probably more importantly, our experience is that an effective site exploration program must be flexible and continually reviewed and adjusted in real time as it proceeds. This can and does present challenges with regard to budget and schedule considerations.

The final objective is to predict the future performance of the proposed facility. As means to this end, we must understand; the geology, the groundwater conditions, the physical, mechanical, and dynamic properties of the affected strata, and the performance of existing facilities. In order to meet these objectives the characterization program needs to be well planned and communicated with the project team and the customer, and it must include two key components. First, the quality of the characterization data must be assessed continuously. Are the data adequate and accurate? Second, the data need to be interpreted and analyzed on a near real-time basis. What answers are suggested by the data? Do they make sense, and do we need additional information? In addition, all exploration programs have some uncertainty attached to the results; it's unavoidable. The question is how to keep uncertainty to a minimum given constraints such as budget and schedule.

Level of Effort

In developing an exploration program the scope is invariably reduced to cost. Historically our experience indicates that most non-geotechnical professionals will attempt to limit this expenditure, not because the cost isn't justified, but simply to "manage" the project. This so-called "low cost/high speed mentality" may be fine on some or even most projects, but it can be a recipe for disaster.

We clearly endorse project management principles and the need to manage the effort; as geotechnical professionals we also recognize the need for flexible investigation programs that take into account actual site-specific conditions and the inherent uncertainty we have in all of our site investigations. Therefore, it is incumbent upon the geoscience professional to ensure this philosophy is clearly understood by the decision makers on the project and the client; this is precisely where tools such as the CPT are invaluable.

At the SRS the cost of a routine boring to 50 m depth with split spoon sampling every 1.5 m is about three times the cost and takes four times longer than a seismic piezocone to the same depth (Note: It is the opinion of the authors that except for the most routine projects, if the CPT is to be utilized, it should be the seismic piezocone [$SCPT_u$] rather than the conventional CPT). Thus, at any stage of the investigation program, and for less time and money, much more stratigraphic detail can be obtained utilizing CPT technology as a first choice over conventional drilling and sampling methods. We do not advocate the abandonment of traditional borings as a technique for subsurface exploration. In fact, and as will be discussed in the next section, CPT technology should be combined with drilling and sampling (and other *in situ* testing) for a highly effective exploration program.

In our experience, on most major and critical projects the initial budget is normally at issue. Although heavily scrutinized, budgets generally are given to perform a scope of work, albeit ill-defined at the beginning of a project. Rather it is a combination of schedule, (having enough time to complete the initial program and evaluate the results), and/or revisions to the program based on actual conditions encountered (scope changes), that present the greatest challenge to investigation programs. In other words, changes to the original scope, even though they may be fully warranted, are difficult to get approved. Therefore, we are obligated to communicate risk and uncertainty, common to every program, to the decision makers. In this case risk can be in terms of money and time to complete a program, and/or technical risk if a program is not fully implemented or if it is cut short.

Risk and uncertainty can't be alleviated, but they can be managed. A discussion about risk and uncertainty is far beyond the scope of this paper; however based on experience, we can factor in actual results (cost and scope) for like projects and ensure that the project under consideration is not an outlier in terms of the proposed level of effort. For example, Figure 2 shows results of the number of borings/CPTs by facility area and hazard category for projects with which the authors have been involved, as well as results from familiar case histories. The hazard category is somewhat subjective and is not based on any hard and fast criteria; rather it is more qualitative based on the authors' experience. The results show considerable scatter in terms of hazard, but there is a distinct trend in terms of size; the larger the facility the more exploration. These results are not unlike the suggestions of Sowers (1979) in the size range of about 1,000 to 100,000 m^2 for dams/dikes, multistory buildings, and manufacturing plants.

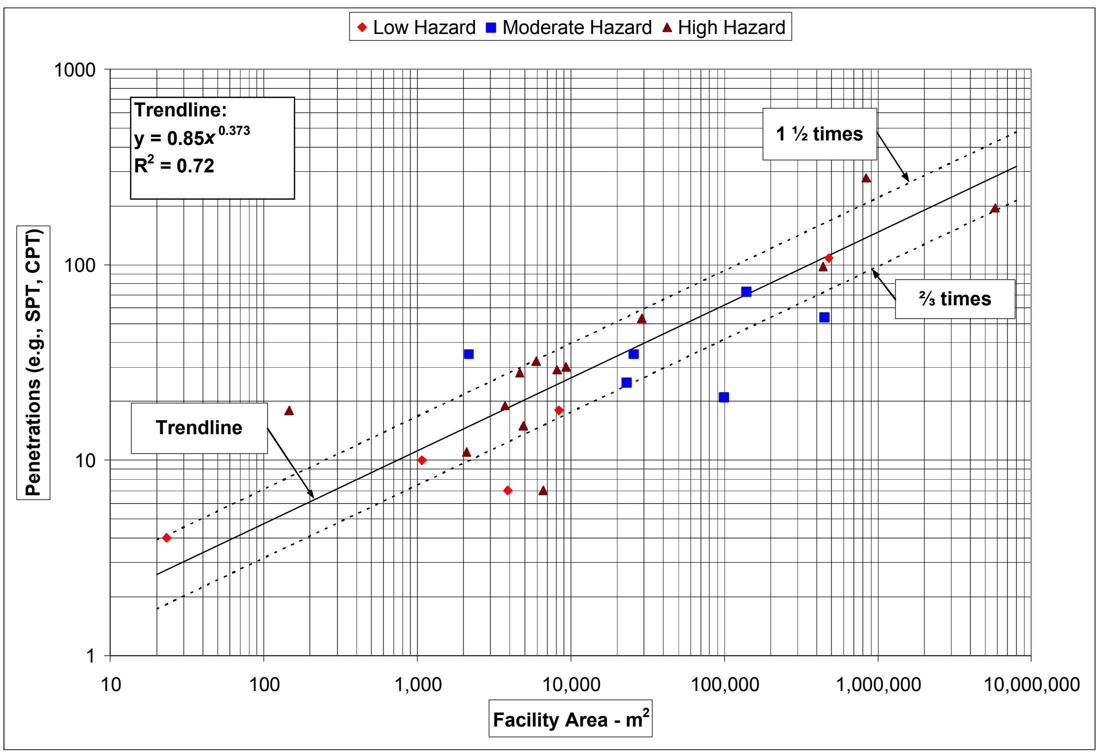

FIG. 2 Penetrations per square meter for various projects by hazard category

In the same way, Figure 3 depicts geotechnical cost in relation to the total estimated cost (TEC) of a particular project. The projects shown are those with which the authors have been involved or are projects found in the literature where reasonable cost information is available. The projects are categorized by focus on transportation, power, nuclear fuel handling, and liquefied natural gas (LNG). While the scatter is significant, trends are still obvious; the higher the estimated cost, the lower the geotechnical effort on a percent basis. The projects shown have an average geotechnical expenditure of approximately 0.6% for the range of TEC shown. The large differences result mostly from actual site conditions and the geologic variability associated with, particularly, the transportation projects, which traverse great distances and involve widely varying geologic and site conditions.

The results are not unlike other published cost information. For example, Sowers reports that for "an adequate investigation (including laboratory testing and geotechnical engineering)" the cost ranges from 0.05 to 0.2% of the TEC, but for critical facilities or facilities with unusual site or subsurface conditions the cost could increase to 0.5 to 1%. A range of site investigation costs as a function of TEC was reported by Sara (1994) for tunnels (0.3-2%), dams (0.3-1.6%), bridges (0.3-1.8%), roads (0.2-1.5%), and buildings (0.2-0.5%). Littlejohn, et al. (1994) report that for building projects in the United Kingdom the expenditure for site investigations ranges from 0.1 to 0.3% of TEC, however they also report that the perception of the respective clients was that the site investigation was, on average, five times higher. We're not sure how to interpret this discordance other than an apparent lack of communication coupled with scope growth.

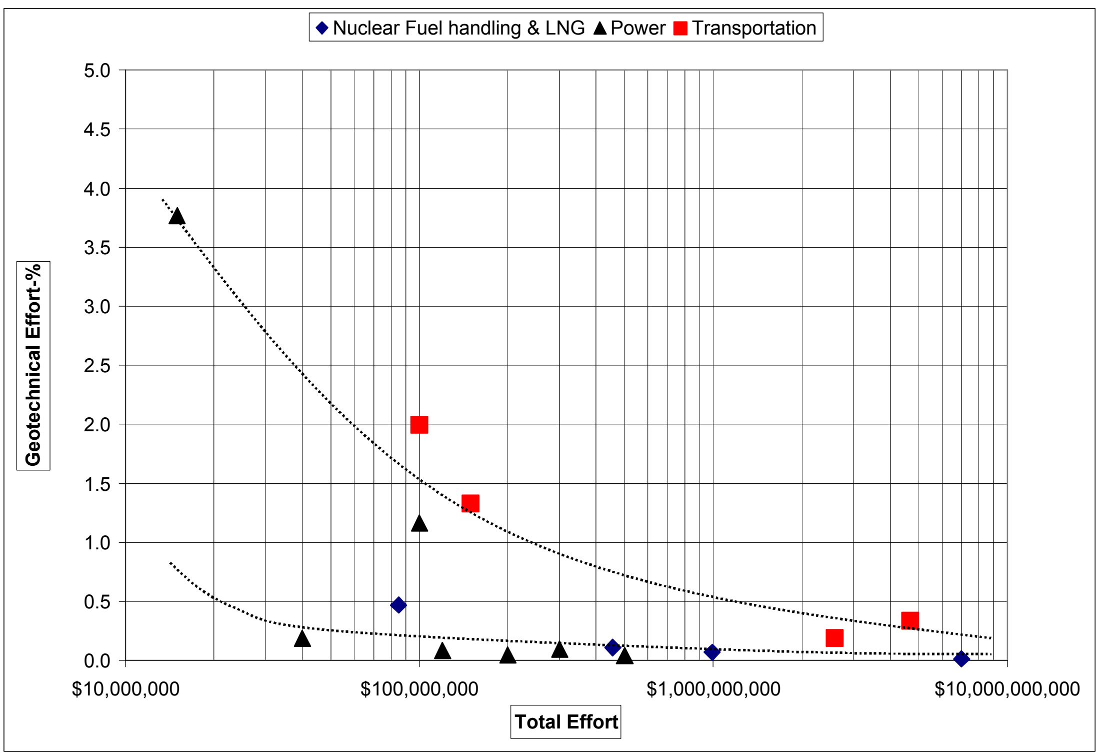

FIG. 3 Geotechnical cost as a function of total project estimated cost

Unfortunately, subsurface investigations are thought of as commodities that can be purchased off the shelf, based on a low cost/high speed mentality. Rather, each program is unique, designed to fit the project and the unique conditions that are inherent to every project with which we become involved. We suggest that information, such as that given in Figures 2 and 3 be developed and used in the planning stages of a project to "calibrate" the decision makers and clients on the level of effort required and to provide a sanity check on the baseline program established. In this way project managers and clients are included in the decision making process and are a part of the risk and uncertainty discussions.

These results are not meant as recommendations, as actual site conditions should dictate what is ultimately carried out. There are no building codes, regulatory documents or other hard and fast criteria that dictate the ultimate level of effort; there are only guidelines. There are however, particular attributes to every well planned and well executed characterization program above and beyond the level of effort discussed above. They are discussed subsequently.

Attributes of a Good Characterization Program

So, what constitutes a good characterization program? First, each site and facility is unique, thus all characterization programs are unique, or should be. Too often characterization programs (including the reporting) are re-cycled with a cut-and-paste mentality. Although this approach may suffice in some instances, it is a slippery slope that really should be avoided.

For a characterization program to be as successful as possible it must be tailored to the specific project under consideration and it must be sufficiently flexible to adapt to changing conditions as they are encountered. From our experience a successful program is done in five basic phases: 1) reconnaissance, 2) proposal or preliminary design, 3) detailed design, 4) construction, and 5) post-construction monitoring. Each phase has a specific purpose and can vary considerably given the specific project conditions.

The reconnaissance phase is generally done for planning purposes and feasibility studies. The effort generally entails researching the site and surrounding area by reviewing historical reports, topographic maps, geologic maps, soil surveys, aerial photographs, field visits, and performance surveys of existing structures.

The proposal or preliminary design phase may only include the reconnaissance phase, but it could also include a limited field exploration with widely spaced borings, CPTs, and geophysical tests. It can include some laboratory testing and simplified analyses for conceptual design, and/or cost estimating purposes.

The detailed design phase is where the bulk of the characterization program is performed. It includes detailed field exploration, such as sample borings (SPT and undisturbed sample borings), CPT soundings, dilatometer and pressuremeter soundings, field vane tests, and geophysics. Representative samples of the subsurface materials are taken and sent to a laboratory for testing. Testing generally includes index tests, and tests for static and dynamic strength and compressibility. Depending on the size of the project and the complexity of the subsurface, this phase may be subdivided into additional phases. For example, the initial phase might include CPT soundings to determine site stratigraphy. A second phase would then target specific horizons for undisturbed samples for laboratory testing, in addition to the more routine standard penetration test borings. In the experience of the authors, for critical projects (critical can be defined in terms of safety or monetary expenditure) a phased approach for the detailed design phase is highly recommended. It allows "pinpoint" sampling of specific horizons rather than sampling at pre-selected depths. This tends to focus the effort on those strata that have the greatest potential effect on the facility. It also adds needed flexibility to the program, which is required if the exploration program is to be successful. Without the flexibility to adjust locations, depths, sample types, and the type of exploration to meet the conditions encountered, the characterization program is doomed. Unfortunately, in many cases once the program has been agreed to and initiated, we tend to manage the cost and schedule at the expense of gathering needed data. Communication with the project team, and in particular the project manager, is critical to success. This also requires full time oversight and direction of the program by qualified geotechnical engineers and geologists dedicated to the effort and who will continue and follow through on the project as it moves from the investigation phase into the design and later construction phases.

Phases 1, 2, and 3 should be carried out on every project. Phases 4 and 5 will depend on the success of the initial program and on any scope changes or any unknown subsurface conditions encountered during construction. However, the level of effort required for each phase may vary considerably based on the type and size of the project and complexity of the subsurface. The key point is that whatever the program entails, it needs to be flexible and the engineer/geologist must be able and allowed to adapt the program to the conditions encountered. In the authors' experience, this doesn't necessarily mean the program will grow, however communication with the project team and/or owner is crucial. On any project, large or small, simple or complex, we are still working with a schedule and budget. And any deviation from either causes concern even though the deviations are valid given the subsurface conditions that dictated the change.

Use of the CPT

Following the trend observed in the industry in general, use of CPT technology at the SRS, and within Bechtel, has progressively increased since the late 1980s in an effort to meet the aforementioned objectives on a project by project basis. This evolution has resulted in a basic exploration philosophy for critical facilities. "Utilize the CPT early and often in a project following with borings and "pinpoint sampling" of targeted strata for further evaluation and laboratory testing."

Several particularly important advantages of CPT technology have been recognized and thus used to further enhance the quantity and quality of geotechnical exploration performed by the authors:

- More exploratory penetrations due to lower cost and less field time as compared to traditional drilled borings (CPTs at the SRS are about ½ to ⅓ the cost and take about $^1/_5$ the time of a SPT boring of equal depth),
- Higher vertical resolution due to nearly continuous measurements, allowing for superior stratigraphic interpretation, detection of layers of special interest, including very thin, loose, or compressible layers, which can be used to determine target intervals for further adjacent sampling and subsequent laboratory testing,

- Highly repeatable measurements within similar material types or layers because of standard and automatic testing and data acquisition methods, and
- Multiple measured parameters including tip stress, sleeve stress, friction ratio, pore pressure, and shear wave velocity for resolving material characteristics, including initial stiffness.

At the SRS, and within Bechtel, the CPT is used primarily to establish stratigraphy, identify any anomalous strata (soft or compressible soil), and acquire a preliminary estimation of specific engineering soil properties for design. However a word of caution, verification and calibration of site-specific correlations of engineering parameters determined with CPT parameters is highly recommended, as correlations shown in the literature do not fit all conditions.

AGED SOILS AT THE SRS

In the shallow subsurface beneath the SRS, Eocene and Miocene age (35 to 50 million years old) sediments of the Altamaha, Tobacco Road and Upper Dry Branch Formations are composed primarily of laminated, gap-graded, clayey sands deposited under alternating marginal marine and fluvial conditions. The clay, chiefly kaolinite and illite, binds the sand grains and appears to have been formed by *in situ* weathering. The CPT tip resistances of this material range from less than 1 MPa to about 15 MPa, and CPT friction ratios (sleeve resistance divided by tip resistance) range from less than 1% to over 10%. Corresponding Standard Penetration Test (SPT) N-values range from less than 10 to over 20 blows per 0.3 m. Because of the relatively low penetration values and the relatively high seismic exposure (proximity to Charleston, SC), studies of the liquefaction vulnerability utilizing the empirical liquefaction chart suggested by Seed et al. (1985) indicated that the site was potentially vulnerable to seismic liquefaction. It is well known that the empirical chart is based on observations of the performance of Holocene deposits. Since the soils at the SRS are geologically much older than Holocene-age, the question logically arose regarding whether the empirical chart was appropriate for the liquefaction evaluation at the SRS. To resolve the concern, two tasks were completed: an extensive field and laboratory geotechnical investigation at the site, and a review of available opinions and data in the technical literature on the liquefaction vulnerability of geologically old sand deposits.

Geotechnical Investigations at the SRS

For the formations of interest (Tobacco Road and Dry Branch Formations) there were no paleoliquefaction events (case histories) to draw upon at the SRS or in the vicinity. For this reason, the decision was made to perform a detailed geotechnical exploration program including field testing, undisturbed sampling, and dynamic testing of carefully sampled soil specimens in the laboratory. The program was developed and implemented by Bechtel Savannah River Incorporated (BSRI), and the dynamic laboratory testing was carried out at the University of California at Berkeley (UCB) laboratory (BSRI, 1993 and 1995).

Samples were obtained by a fixed-piston sampler using controlled techniques and sampling procedures well established at SRS. Measurements, including X-ray photography, were performed on each sample tube prior to packing and transporting and after being received at the UCB laboratory. The laboratory testing included index testing and the determination of dynamic strength, volumetric strain after liquefaction and an evaluation of the influence of confining pressure, leading to site-specific recommendations for K_σ, a factor that normalizes the cyclic resistance of a soil (CRR) to an overburden pressure equal to one atmosphere. All of the test results were correlated back to a sample specific $(N_l)_{60}$, or $(q_t)_l$, where $(N_l)_{60}$ is the SPT penetration resistance normalized to one atmosphere and 60% energy level, and $(q_t)_1$ is the CPT tip resistance normalized to one atmosphere.

Site-specific sampling and laboratory testing were completed for two facilities at the SRS. In the first, a series of 17 stress-controlled, isotropically consolidated undrained cyclic triaxial tests were performed for site A. The samples were obtained adjacent to (within 1.5 to 3 m) SPT boreholes at locations exhibiting low N-values. To aid in the evaluation, SPT energy measurements were obtained and used later to correct the field N-values to N_{60}. Though the fines content of the samples varied, a single "CRR design curve" for these soils was established. The overall assessment resulted in three data points relating $(N_1)_{60}$ to CRR. The samples tested to develop the SRS curve were classified as SC soils (unified classification system) and had plastic fines with contents ranging from about 9 to 29%, and an average of about 17%.

Secondly, to correlate CPT $(q_t)_1$ with the cyclic resistance values obtained in the laboratory, CPT soundings were pushed adjacent to the Site A borings described above. In the same way, 18 additional high quality fixed-piston samples were obtained at site B from boreholes adjacent to (1.5 to 3 m) 18 CPT soundings. These samples were also sent to the UCB laboratory for dynamic testing (stress-controlled, istropically consolidated, undrained cyclic triaxial tests). The laboratory results were evaluated in the same manner described above except that the CPT $(q_t)_1$ was utilized instead of $(N_1)_{60}$. The samples tested were SC soils and had plastic fines contents ranging from about 16 to 34%, with an average of about 24%. Table 1 summarizes the relevant data for the combined data set. Note that although 35 cyclic triaxial tests were performed, only 12 data points are shown. This is due to grouping of like material in terms of fines content and $(q_t)_1$.

Prior to the re-evaluation (discussed subsequently), a suite of curves based on plastic fines content was established. The lines representing various fines contents were constructed based on the laboratory test results described above and the trends of Seed et al.'s (1985) empirical chart, (i.e., the clean sand passing through the origin of coordinates), and engineering judgment, assuming the suite of curves was more or less parallel. That relationship (developed in 1994-1995) has recently undergone a re-evaluation to take into account newer information (since 1995), including results from Youd et al. (2001) and Idriss and Boulanger (2004).

The re-evaluation included a review of all the SRS data, but in particular, it centered on the shape of the clean (<5% fines) curve at low penetration resistances. For example, both Youd et al. (2001) and Idriss and Boulanger (2004) show the clean sand (≤5% fines) curve becoming flatter at low penetration resistances and intersecting the ordinate at a CRR value of 0.05.

However, for the re-evaluation of the SRS data we relied more on the Idriss and Boulanger relationship because; 1) the Idriss and Boulanger clean curve “fits” our site-specific data more closely than the Youd et al. curves, and 2) as expected, at higher penetration resistances the Idriss and Boulanger curve for Holocene soils results in a more conservative estimate of CRR. Thus, for the revised SRS clean curve we have adopted the Idriss and Boulanger (2004) relationship for the CPT to “construct” the revised SRS-specific relationships.

Table 1. SRS Data Summary

No.	Fac.	Geo. Form.	USCS	$(q_t)_1$ (MPa)	D_r (%)	$(V_s)_1$ (mps)	Fines (%)	LL (%)	PI (%)	γ_d (kN/m^3)	CRR_f
1	A	UTR	SC	0.9	<5	293	28.7	51	30	16.6	0.167
2	A	UTR	SC	2.0	15	268	18.5	49	26	15.5	0.138
3	A	LTR	SP-SM	2.3	20	255	9.6	NP	NP	15.7	0.095
4	A	LTR	SC, SM, SP-SC	3.3	32	257	15.7	41	17	15.2	0.135
5	A	LTR	SP-SM, SC	5.0	45	253	10.8	NP*	NP*	15.5	0.115
6	B	TR3	SC	1.7	10	247	34.0	47	28	16.5	0.152
7	B	TR3	SC	0.5	<5	223	33.6	48	32	17.1	0.173
8	B	TR3	SM-SC, SC	1.1	<5	267	26.3	50	29	16.4	0.165
9	B	TR3	SC	0.6	<5	201	20.5	48	29	16.6	0.149
10	B	TR3/TR1	SC	1.8	11	162	19.7	60	37	16.7	0.134
11	B	DB1/3	SC	1.9	14	269	16.1	39	18	16.3	0.117
12	B	DB1/3	SP-SC, SC	1.1	<5	206	18.6	80	62	15.2	0.139

UTR is Upper Tobacco Road; LTR is Lower Tobacco Road; TR1 is Tobacco Road 1; TR3 is Tobacco Road 3; DB1 is Dry Branch 1; DB3 is Dry Branch 3; USCS is the unified soil classification system; $(q_t)_1$ is the normalized cone tip resistance; $(V_s)_1$ is the normalized shear wave velocity (normalization per Andrus and Stokoe); LL is the liquid limit; PI is the plasticity index; γ_d is the dry density and CRR_f is the field-corrected cyclic resistance ratio. The estimate of relative density (D_r) is based on Tatsuoka et al., 1990 (given in Ishihara and Yoshimine, 1992). * Two of the three samples for point #5 were NP.

To develop the SRS “aged” clean curve, we utilized the low end of the strength gain factor range (1.3) proposed by Lewis et al. (1999) for clean sands (discussed below). Thus, for the revised SRS “aged” clean sand relationship, the y-intercept was 1.3 times 0.05, (the revised ordinate for the clean curve, Youd et al., 2001) or 0.065. In addition, the shape of the revised clean sand curve was assumed to be similar to that of the Idriss and Boulanger (2004) clean sand curve, which is in turn similar to the shape given in Youd et al. (2001) at low penetration resistances. Thus, for the clean “aged” curve, a factor of 1.3 was applied over all penetration resistances to the Idriss and Boulanger clean curve to derive the CRR corresponding to the “aged” clean SRS curve. Utilizing a constant factor to increase the curve across penetration resistances is consistent with the work of Polito (1999) and Polito and Martin (2001), for fines contents below about 40%. Using a constant factor for a given fines content independent of the penetration resistance and adopting the shape of the Idriss and Boulanger (2004) Holocene clean sand curve, the remainder of the SRS CRR curves were developed for various fines contents simply by the ratio of the site-specific data (CRR_f) to the adopted Holocene clean sand curve ($CRR_{I/B}$) of Idriss and Boulanger (2004), applied over all penetration resistances. For example, the 10% fines content (FC) curve utilized data points 3 and 5 from Table 1. Point 3 has a normalized tip stress ($[q_t]_1$) of 2.3 MPa and point 5 has a $(q_t)_1$ of 5.0 MPa. The ratios of the CRR at corresponding CRR from the Idriss and Boulanger clean curve is; 0.095/0.058 = 1.64 for point 3; and 0.115/0.079 = 1.45 for point 5; considering both points the ratio would be about 1.6. The resulting SRS 10% CRR curve would be 1.6 times higher than the Idriss and Boulanger clean curve. In the same way curves can be constructed for fines contents of 15, 20, 25 and 30%. Table 2 and Figure 4 summarize the evaluation results for each fines content curve (Figure 4 also shows the previous, 1995, relationship).

Table 2. Summary of CRR Design Curve Factors

FC Curve (%)	Data Point (Table 1)	Actual FC (%)	$(q_t)_1$ (MPa)	CRR_f	$CRR_{I/B}$	Ratio $CRR_f/CRR_{I/B}$	Ratio selected for FC group
10	3	9.6	2.3	0.095	0.058	1.64	1.6
10	5	10.8	5.0	0.115	0.079	1.45	1.6
15	4	15.7	3.3	0.135	0.064	2.10	2.1
15	11	16.1	1.9	0.117	0.056	2.11	2.1
20	2	18.5	2.0	0.138	0.056	2.46	2.6
20	9	20.5	0.6	0.149	0.051	2.93	2.6
20	10	19.7	1.8	0.134	0.055	2.44	2.6
20	12	18.6	1.1	0.139	0.052	2.67	2.6
25	1	28.7	0.9	0.167	0.052	3.24	3.1
25	8	26.3	1.1	0.165	0.052	3.17	3.1
25	9	20.5	0.6	0.149	0.051	2.93	3.1
30	1	28.7	0.9	0.167	0.052	3.24	3.4
30	7	33.6	0.5	0.173	0.050	3.43	3.4

Note: Point 6 from Table 1 is not in Table 2. This data point has not been incorporated in the evaluation because it is not consistent with the results of the entire data set. We believe this data point to be somewhat anomalous; $CRR_{I/B}$ refers to the CRR utilizing the Idriss & Boulanger (2004) clean curve for CPT.

The data show that compared to the Idriss and Boulager (2004) Holocene clean curve for CPTs, the increase in dynamic strength ranges from 1.6 to 3.4 for fines contents from 10 to 30%.

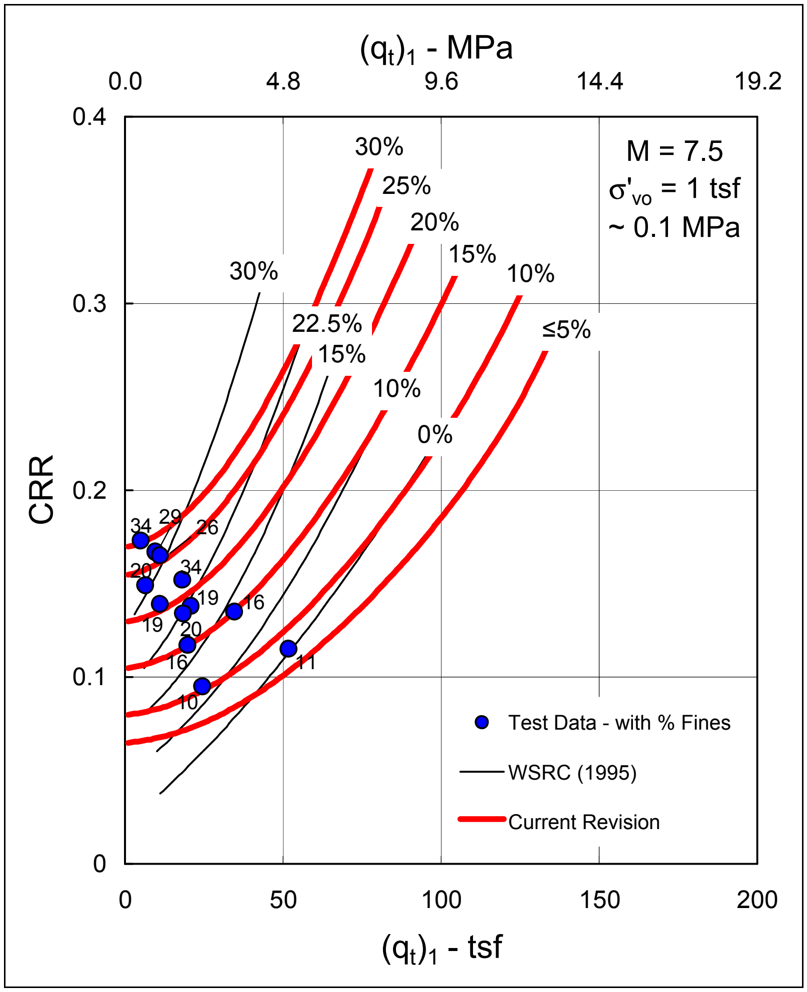

FIG. 4 CRR vs CPT $(q_t)_1$

Review of Data on the Performance of Aged Soil Deposits

Several investigators have addressed the issue of soil aging; among them are Youd and Hoose (1977), Seed (1979), Skempton (1986), Kulhawy and Mayne (1990), Martin and Clough (1990, 1994), Schmertmann (1991, 1993), BSRI (1993, 1995), Arango and Migues (1996), Lewis et al. (1999), and Leon et al. (2006). The results of some of the more significant findings are summarized below.

Seed (1979) considered the cyclic resistance of laboratory-prepared samples and of hydraulic fills of different ages (up to about 3000 years) and concluded that the data indicated "the possibility of increases in cyclic mobility resistance of the order of 75% over the stress ratios causing cyclic pore pressure ratios of 100% in freshly deposited laboratory samples, due to long periods of sustained pressure in older deposits."

Skempton (1986) discussed the evidence for increase in the deformation resistance of sand with the increased duration of sustained loading. He considered the increase in penetration resistance blow count (N-value) as a reflection of the increase in resistance to deformation. He found that the ratio between the normalized SPT,

$(N_1)_{60}$ blow count, and the square of the relative density, (D_r^2), varied with the period of sustained loading. Skempton reported strength gains (increases in $(N_1)_{60}/D_r^2$ relative to that predicted for samples in the laboratory) for normally consolidated sands of about 14% and 57% at 10 years and >100 years, respectively after deposition.

Kulhawy and Mayne (1990) compiled the values of the same parameter as Skempton, $(N_1)_{60}/D_r^2$, for several fine and fine to medium sand deposits of known geologic age, including some of the same data evaluated by Skempton (1986). They concluded that the parameter $(N_1)_{60}/Dr^2$, is influenced by particle size, overconsolidation, and aging. Although they acknowledge some of the data may be "imprecise", they developed a conservative relationship ($C_A = 1.2 + 0.05\log(t/100)$) to account for aging through the parameter $(N_1)_{60}/D_r^2$. We consider this relationship to be a lower bound of potential strength gain with time.

Lewis et al. (1999) reviewed published data compiled from the 1886 Charleston, S.C. earthquake. There are no quantitative data available regarding the magnitude of the event or of associated peak ground accelerations; however, the earthquake's moment magnitude has since been estimated to be between about 7 and 7.5. Independent studies carried out by several investigators estimated the epicentral acceleration somewhere between 0.3g and 1.0g. Lewis et al. (1999) concluded a reasonable range of acceleration was between 0.3-0.5g.

Relic liquefaction features have been investigated by many along the eastern seaboard (e.g., Talwani and Cox, 1985; Obermeier et al., 1986; Dickenson et al., 1988; Amick et al., 1990; and Martin and Clough, 1990 and 1994). Features were found primarily in the sands and silty sands of two ancient ridges dating back 130,000 to 230,000 years and located 10 miles inland. The beach processes led to sands and silty soils being concentrated in the highest portions of the beach ridges. For their study, Lewis et al. (1999) reviewed data collected by Martin and Clough (1990, 1994) and Dickenson, et al. (1988). These included borings, velocity profiles, piezocone probes and trenches. Grain size tests showed that the sands at the sites are non-plastic, clean, with fines content less than 5% (14 locations), and 10% (5 locations). The sands in the remaining sites were described as SP, SM material. For the evaluation of these data, Lewis et al. (1999) calculated the induced cyclic shear stresses at the depths of interest for each deposit utilizing peak ground surface acceleration of 0.5g. A lower boundary was established showing the minimum stress ratios required to cause liquefaction at those sites that experienced marginal liquefaction and liquefaction. The strength gain of the boundary, relative to the clean sands curve in the empirical chart by Seed et al. (1985), was found to be about 2.2. Similarly, an upper boundary was established that separates the maximum cyclic stress ratios tolerated by the soil with no liquefaction, from those sites that experienced limited to widespread liquefaction. The strength gain in this case was calculated to be about 3.0. In the same way, using a lower bound acceleration of 0.3g resulted in computed strength gains of 1.3 to 1.8. (Note: The 1.3 factor was applied above for the re-evaluated SRS "aged" clean curve).

Arango and Migues (1996) performed investigations after the occurrence of the January 17, 1994, Northridge earthquake. The area selected for the study was within the Gillibrand Quarry site in the Tapo Canyon, north of Los Angeles. Area acceleration levels exceeded 0.5g, resulting in the failure of a small water-retaining dam in the quarry. However, in nearby Simi Valley, an old deposit of sand showed no signs of liquefaction. This sand has been estimated to be approximately one million years old. Although this deposit is now exposed in outcrops, it was previously buried by as much as 460 m of overlying soil. It is relatively uniform, fine quartz sand (SP) with less than 5% non-plastic fines. In its current state, the sand is lightly cemented, such that it can support vertical faces when dry, but is weak enough to crush with the slightest pressure between one's fingers. Microscopic examination reveals a high degree of quartz grain overgrowth, evidence of age and burial.

The field exploration program utilized drilled and augered boreholes, CPT soundings, test pits, and undisturbed block sampling techniques. A total of 18 stress-controlled cyclic triaxial tests to classify and determine the static and dynamic strengths of the sand were carried out at the Geotechnical Laboratory at the University of California, Berkeley. The range of field cyclic resistance ratio (CRR), based on results of laboratory testing, was estimated to vary between 0.80 and 1.37. Based on these results, and adopting a predicted induced cyclic stress ratio equal to 0.50 for Holocene-age sands from Seed, et al., (1985), the increase in dynamic strength ranges from 1.6 to 2.7.

Leon, et al. (2006) investigated the effect of age at four sites in the South Carolina coastal plain. The parameters reviewed were SPT N-values, CPT $(q_c)_1$, and normalized shear wave velocity, $(V_s)_1$. The four sites ranged in age from 546 years to 450,000 years old. The fines content from samples at all of the sites ranged from 0-9%, averaging 4%. The results of their evaluation indicated that these coastal plain soils had increased resistance to liquefaction by a factor ranging from 1.3 to 2, with an average of 1.6 (compared to the Youd et al., 2001 relationships for Holocene soils), induced by a magnitude 7.5 earthquake. The specific factors and ages reported for each of the four sites were; Ten Mile Hill site A: 1.3, 3548 yrs., Ten Mile Hill site B: 2, 200,000 yrs., Sampit: 1.5, 546-450,000 yrs., and Gapway: 1.7, 3548-450,000 yrs.

Figure 5 compares the predicted strength gain from the SRS studies reported above, and the historical data reviewed. Note that for the SRS re-evaluated data, the strength gain is relative to the clean CPT curve from Idriss and Boulanger (2004). Utilizing the Seed et al. (1985) relationship would result in strength gains of approximately 10 to 20% less. In either case we note the consistency between the results of the SRS investigation and the field data from South Carolina, and Southern California. Furthermore, the results are also compatible with the extrapolated trends suggested by Seed, Skempton, and Kulhawy and Mayne.

It is interesting to note that trends shown on Figure 5 relating strength gain with age utilizing the work based on SPT N-values (Skempton, and Kulhawy and Mayne) are at the low end of the data shown, particularly older than about 1,000,000 years. This may be an indication that the SPT N-value is a poor indicator of strength gain with time for very old deposits (Note: The trend of Kulhawy and Mayne is an acknowledged conservative trend, as they show other data with higher $(N_1)_{60}/D_r^2$ for ages up to 10^8 years). However, it appears the Kulhawy and Mayne relationship can be used as a lower bound for the data shown, and a similar relationship (utilizing the same functional form suggested by Kulhawy and Mayne) can be used for an upper bound trend ($C_A = 1.92 + 0.23\log(t/100)$).

CONCLUSIONS

The CPT has enhanced our capability to perform subsurface exploration within Bechtel and at the SRS. Utilizing the CPT early in a project adds flexibility to the program and allows greater site coverage with a given budget in a shorter period of time. Initial program development following the suggestions of Sowers provides a reasonable starting point. Properly verified site-specific correlations between CPT parameters and laboratory testing add a dimension that can be very powerful when assessing site conditions and performing design related activities.

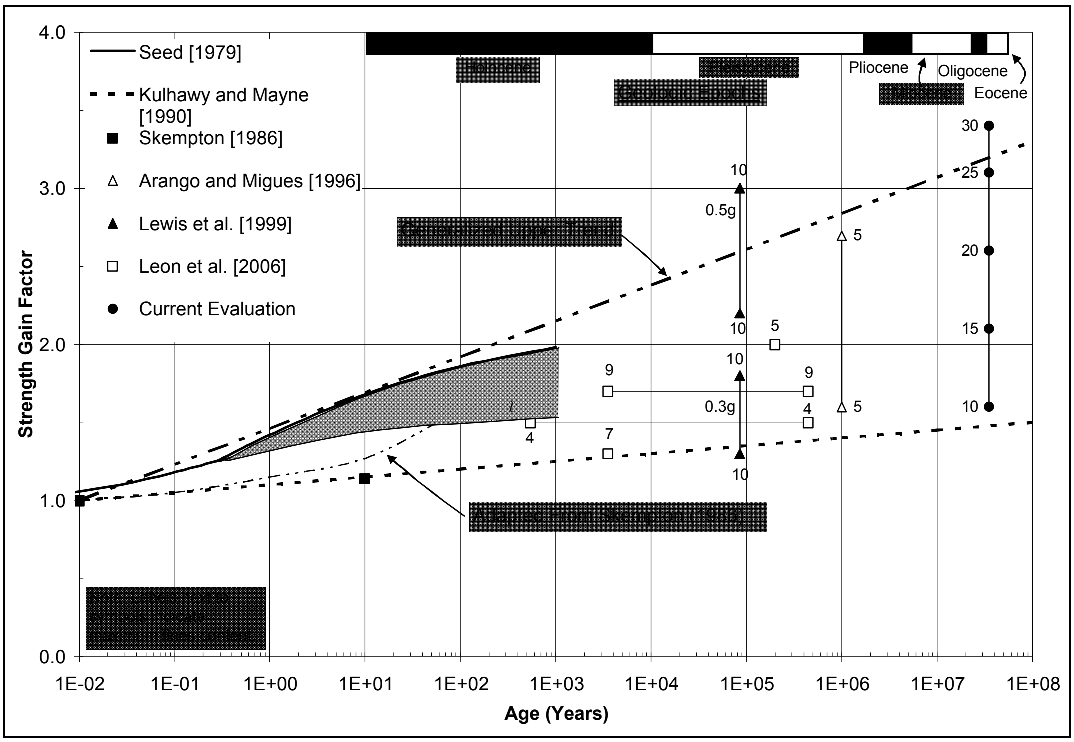

FIG. 5 Strength Gain with Age

The key component however in any exploration program is still effective communication with decision makers at all levels of the program. Full time geotechnical oversight will enhance and facilitate the needed communication and will allow quick and early decisions to be made during the program. With these attributes, utilizing the CPT will result in a program that allows maximum flexibility and affords superior stratigraphic definition through continuous or near continuous data, excellent repeatability and data reliability, and time and cost savings. The result is more high quality data, including "pinpoint" sampling and testing of targeted strata. While there will always be a need for soil borings and laboratory testing, the amount should decrease with increased use of the CPT. However, knowledge about the subsurface conditions will increase. This has been commonly recognized as far back as 1978 (Schmertmann, 1978): "Although engineers with much CPT experience in a local area sometimes conduct site investigations without actual sampling, in general one must obtain appropriate samples for the proper interpretation of CPT data. But, prior CPT data can greatly reduce sampling requirements."

In terms of aging, it is widely recognized by the technical community that the geotechnical properties of sand deposits are influenced by their age. Cyclic resistance data about the behavior of soils under dynamic loading summarized in the widely accepted "empirical chart" by Seed et al. (1985) are limited to the relatively geological young soil of Holocene age. For use at the SRS the need arose to define the cyclic resistance of older sands. Lacking information about the performance of the sands at the site, it was necessary to carry out the field and laboratory test programs, and the literature review. The results, summarized in Figure 5, provide confidence in the validity of the investigations. The case histories reviewed in this paper confirm the observations of Professor Schmertmann, namely that age does play a major role in the strength of soil deposits and cannot be ignored and that strength does increase as time passes.

ACKNOWLEDGEMENTS

Through the years Professor Schmertmann has been involved with Bechtel projects including several projects at the SRS. It has been through this direct interaction that we have come to the methodologies and conclusions described herein. We owe a great deal to Professor Schmertmann for his wisdom, insight, and his fundamental knowledge of soil mechanics. We also acknowledge the contribution of Laura Bagwell and Rucker Williams for their patience and much needed assistance in preparing this manuscript.

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