United States Department of Energy

Savannah River Site

SRS Soft Zone Initiative

Historical Perspective

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SRS Soft Zone Initiative – Historical Perspective

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1. Introduction

At the Savannah River Site (SRS) a geologic layer exists between approximately 100 to 250 feet below ground surface named the Santee Formation. It is a marine deposit laid down during the Middle Eocene epoch, which occurred 35 to 50 million years ago. Past geologic studies have characterized the upper middle portion of this deposit as having locally high concentrations of calcium carbonate. Often found within these sediments, particularly in the upper third are weak zones interspersed within stronger matrix materials. It has been the focus of many subsurface exploration programs dating back to the original United States Army Corps of Engineers (COE) studies in the early 1950s (COE 1952a and 1952b) and is a major concern for foundation design and performance for facilities at the SRS.

The first attempts to deal with the weaker, or soft zones within the Santee Formation are summarized in the reports by the U. S. Army Corps of Engineers (COE 1952a and 1952b) and Moran, Proctor, Mueser & Rutledge Consulting Engineers (MPMRCE 1963). At the time these reports were issued, the analytical tools available to the geotechnical engineer were much more limited than they are today. Therefore, engineers faced with the problems presented by the weak zones had to make do with the tools available to them and to use conservative assumptions in the absence of more powerful analytical tools. Thus at the time, remediation via cement grouting was selected to mitigate any potential affects of these soft sediments.

Several hypotheses exist regarding the processes responsible for soft zone formation; all share a common assumption that soft zones result from post-depositional and/or early diagenetic changes (in other words, the soft zones were not originally deposited as weak or “low strength” materials). One prevailing idea, which is supported by a substantial body of geologic, geochemical, and mineralogic evidence collected at SRS, invokes the percolation of groundwater and the dissolution of carbonate material and partial replacement by silica resulting in a residuum (soft zone) that is porous but still self-supporting, not unlike a “honeycomb” or sponge-like structure (WSRC, 1999).

The material in the soft zone is more deformable and more compressible than the original (unaltered) sediment. The *in situ* vertical effective stress acting on the soft zone is less than the vertical effective stress at the same depth in a region that does not contain a soft zone. In other words, the dissolution and partial replacement by silica *in situ* and the subsequent redistribution of vertical overburden stress have created a zone in which the vertical effective stresses are less than would be computed by simple summation of overburden effects. Since the existing vertical stresses acting on the soft zones are less than the apparent geostatic stress, these soils may be described as “underconsolidated” with respect to the vertical stress at an equivalent depth in unaltered (non-soft zones) sediments.

Even though the soft zones are underconsolidated, with respect to geostatic stress, they are assumed to be normally consolidated within their own stress regime (i.e., within the area/volume of stress redistribution or arching). The condition of lower-than-geostatic stress is possible because of the relatively strong matrix soils surrounding the soft zones and the relatively dense overlying sands (in this case, sands of the Dry Branch Formation). Thus, when a consolidation test is performed on an intact specimen of soft zone soil, the preconsolidation pressure (P_P) determined is assumed to be the effective vertical stresses acting on the soft zone (i.e., OCR of 1 within soft zone stress regime). However, the P_P determined from the consolidation test for a soft zone will be less than the geostatic stress (P'_{0G}), hence the term underconsolidated.

The general computational methodology that has been utilized at SRS for a number of years relies on consolidation theory to determine soft zone settlement (i.e., settlement that will result if the full overburden pressure, P'_{0G}, acts on the soft soils) at depth and on empirical correlations used in the soft ground tunneling industry to propagate the compressions at depth to the ground surface. Past studies at SRS have also relied on more sophisticated numerical models to propagate compressions at depth to the ground surface, however the use of these more sophisticated models has been bogged down by the inherent assumptions that must be used; for example assumptions regarding an appropriate constitutive soil model.

The purpose of this white paper is to describe and discuss the analytical models used at the SRS to determine what effect, if any, these weaker, "soft zones" within the Santee Formation may have on existing and new facilities, particularly the determination of settlement at depth. This paper summarizes the practical consequences of the existence of the soft zones at the Savannah River Site, the geotechnical procedures that have been used to deal with them and modifications that might be made in the light of recent improvements in the analytical tools available in soil mechanics. It attempts to separate concepts that are unquestioned from extrapolations and proposals. This paper also discusses some of the terminology that has been used to describe these weaker zones and how these weaker zones have been and could be modeled in terms of assessing potential settlement at the ground surface; however, it does not discuss all of the details of these models. This paper is not intended to replace any other document prepared on this subject, but to compliment that effort in terms of describing the analytical models utilized.

Included within this document is a discussion of the more relevant, pertinent documents that contributed to the present understanding and development of the current SRS analytical model. These documents are discussed below, as they relate directly to current and alternate analytical models, however, by no means are they the only documents that discuss this issue. Also included is a summary of relevant soil parameters measured from laboratory tests performed on samples of soft zone soils. Properties vary from location to location, however as will be shown there are distinct trends.

2. Background

At the onset of the early COE investigation programs (~55 years ago), the COE recognized that the weaker zones within the Santee Formation had to be addressed beneath critical facilities (the five reactor sites and the two canyon facilities). Maps of "surface sinks" (NCState, 1951) were prepared (Figure 1, the dark aerial features) and presumably used, in conjunction with subsurface investigations, to site the industrial areas (the five reactors and two separations areas). At the time, the prevailing model (COE Model) assumed karst-based conditions. Thus, possibly due to investigation and analytical tools available at the time, and partly due to schedule concerns, the COE decided to remediate and embarked on an extensive investigation and pressure (albeit low pressure) grouting program of the soft soils beneath the foundations for critical facilities.

Although historical records are somewhat elusive, a summary of the grouting for K-Area is contained in WSRC (1992). That summary shows that the grouting program generally consisted of jetting borings (termed "fishtail" borings in the historical record) across the footprint of the facility at 35 foot centers. For example, in K-Area that amounted to 388 of these borings for the reactor area. Where rod drops or fluid losses occurred, pressure grouting was performed. In K-Area 15% of the fishtail borings were pressure grouted as a result of rod drops, and an additional 11% were pressure grouted as a result of water or grout venting from the borings during grouting of other borings. Thus, about 25% of the borings (approximately 25% of the footprint area) received some amount of grout. However, the volume of grout received was extremely small when compared to the volume of soil potentially influenced by the presence of the facility.

Figure 1; Surface Depressions

Measured settlement (contained in the historic record) of the K reactor structures during the first two years of construction ranged from about 1 to 3 inches (Figure 2). Gross loads were estimated to range from about 4,500 to 7,500 pounds/ft^2 (psf) with net loads (all of the reactors were founded approximately 50 feet below original ground surface) generally 2,000 psf and less. Settlement analysis performed by the COE indicated that approximately 75% of the settlement would occur as the load was applied; thus no or very little long term settlement was anticipated. From the historical records, this appears to be a reasonable assumption. The effectiveness of the grout injection program was presumed to support this approach, however it was never demonstrated. Nowhere in the settlement analysis was there mention of grout and the potential improvement due to grouting. Thus it appears the settlement estimates were based on *in situ* properties without regard to the effects of grouting.

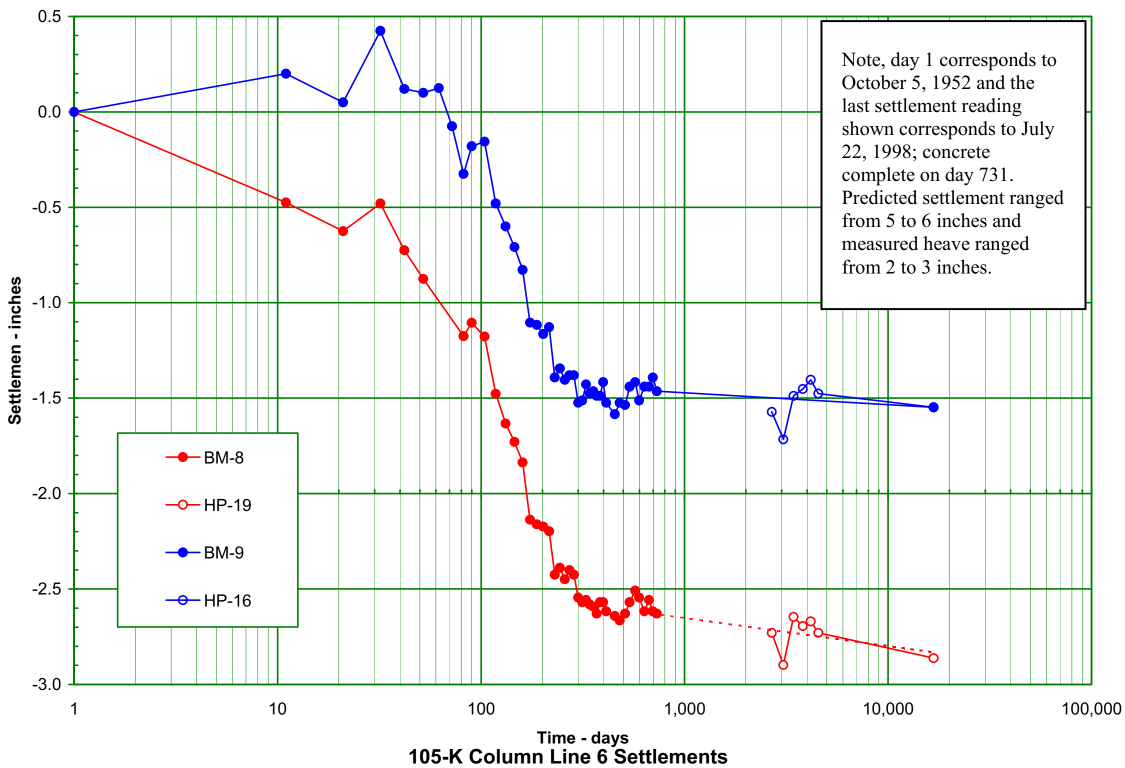

Figure 2; K-Area Settlement

As a follow up about 40 years later, investigations performed at K-Area during the reactor restart program (RRP) of the late 1980s and early 1990s, found that soft zones still existed in areas where grout was injected during the original COE program. Although quantitative measures were not available to compare original 1950s vintage soft zone strengths to strengths inferred during the RRP (e.g., CPT resistances), it appears no improvement was made to the soft soils. As documented in WSRC (1992), there is the possibility that the act of grouting actually destroyed the soil fabric/structure and that if settlement were to occur it would have occurred during the grouting operation. Today in hindsight, the best that could be said was that the grout injected “reinforced” the *in situ* soils, which would tend to increase it’s resistance to deformation. However, this has not been nor can it be quantified. Thus, the effectiveness of the COE grouting program is debatable. In any case, original COE estimates of settlement for the K reactor based solely on *in situ* properties of the geologic layers without specific influence of the injected grout closely match the measured settlements that have actually occurred. Thus, it is concluded that grouting had a negligible affect on the measured surface settlement, and it appears the soft zones had very little affect on the structures as well, based on visual examination of the facilities.

3. Foundation Investigations and Treatment at Savannah River Plant

In the early 1960s a report was prepared by Moran, Proctor, Mueser & Rutledge Consulting Engineers (MPMRCE, 1963) summarizing the thoughts at the time regarding soft zones and how to deal with them. The report was not geared to any one facility; rather, it gave general considerations for facilities at SRS. Some of the main points and findings of the report were:

- Surface sinks or depressions (sometimes referred to as Carolina Bays) that were mapped (Figure 1) in the early 1950s were attributed to solution of calcareous materials at depths of 100 to 200 feet below ground surface. The extent of solution is variable, but it is more pronounced towards the east and south of the site. The solution-related defects are restricted to the McBean (Santee) Formation or the material directly overlying it. The mapped surface sinks are large and do not involve sharp variations in surface elevation within small horizontal distances. Thus, it was believed at the time that if any movement were to occur, it would be gradual and would occur over large distances (angular distortions would be small).
- A conservative approach (pressure grouting) was adopted even though it was thought that solutioning occurred very slowly and there was no danger to facilities. Thus, grouting was performed beneath critical facilities for safety and, more importantly, to permit immediate construction.
- Areas where these soft deposits occur should remain stable unless subjected to: 1) large stress increase due to construction, 2) major changes to surface drainage, 3) subsurface water movement due to pumping, and 4) shocks such as earthquakes, blasting, or explosions. All of these were taken into account, except that item 4 has taken on much more significance since original design. As discussed earlier, this is most probably due to the analytical procedures and tools available at the time.
- Structures were placed in one of three classifications; I – non-sensitive or minor structures; II – medium sensitivity or large cost; and III – maximum sensitivity. Structures in Category III required extensive exploration and foundation treatment via grouting. Guidelines for exploration and foundation treatment were developed for each of the three structure classifications.
- Settlement monitoring is essential, and it was implied that, over time, the results of settlement monitoring could be used to demonstrate the low risk the Santee Formation poses to surface facilities.
- An acceptable degree of risk was recognized considering structural, operational, and economic factors associated with a foundation treatment program.

Our current thinking and understanding of the surface sinks and/or Carolina Bays have evolved through time. Today, the most accepted origin of the Carolina Bays is Aeolian (transported by wind). However, as noted in early studies of the SRS by the COE, surface depressions may represent a mix of both Carolina Bays as well as sinks resulting from karst conditions. As carbonate content thickens to the southeast, conditions become more favorable for karstic-like conditions although, there are no substantiated accounts (recent or otherwise) of sinkholes forming. Thus, in relative terms, we would expect karstic-like conditions to be unlikely in the northern portion of SRS, less likely in the General Separations Area (GSA), and somewhat more likely in the southernmost portions of the site. However, as will be mentioned later, there is evidence of "draped stratigraphy" and "soft sediment" deformation at the SRS, which may be attributed to settlement at depth. Borings made 40 years after grouting had been performed in K-Area, failed to reveal thick layers of grout, as expected (WSRC, 1992). In addition, where grout was found soft soils still existed. This indicated that probably the most severe, common subsoil condition existing throughout the solutioned zone is a porous, spongy, relatively thin open stratum that accepts some fluid grout, but has structural competence (honeycombed structure).

Settlement monitoring of critical facilities over time shows that the majority of the settlement occurs as the load is applied, confirming earlier COE predictions, and that the magnitude of that settlement is generally in the range of 1 to 3 inches for the types of structures (aerial extent and load) at SRS. This indicates that the soil column generally behaves elastically (for the loads imposed), indicating some overconsolidation. There is a long-term creep or secondary component that, for the Defense Waste Processing Facility (DWPF) for example, ranges from about 0.3 to 0.8 inches per log cycle of time. Furthermore, there have been no known operational issues with respect to settlement at depth for any facility.

Note that in the above report by MPMRCE (1963) there is no discussion related to analysis of these soft layers and their impact on a foundation or a structure. Although geotechnical engineering was flourishing at the time (1960s), the analytical capability available compared to today was rudimentary. Thus, a prudent and safe course of action at the time was foundation treatment via grouting, even though there was no demonstration that such grouting produced thick layers of grout, nor reduced structure settlement. It appears to us today that the grouting program designed and implemented by the COE was simply meant to be a "cavity filling" exercise rather than a "soil improvement" exercise.

3.1 K Reactor Area Geotechnical Investigation for Seismic Issues – Technical Evaluation and Remediation of Soft Zones in the K-Reactor Area

In the late 1980s and early 1990s, GEI (WSRC 1991a and WSRC 1991b) performed extensive geologic, geotechnical, and analytical studies in K-Area as part of the Reactor Restart Program (RRP). Much of the effort was focused on the characterization and behavior of the Santee Formation under seismic conditions. Of significance were the changes that had occurred over the 25-year period between the aforementioned MPMRCE (1963) effort and the GEI effort, in particular, the exploration tools, laboratory testing, and the analytical capabilities. Some of the main points and findings of the GEI report were:

- The soft zones consist of soils that have been altered over geologic time and are now subjected to relatively low effective stresses as a result of arching or some other form of stress re-distribution. The alteration is due to removal of calcium carbonate by groundwater dissolution followed by silica replacement. Calcium carbonate still exists; however, the rate of solutioning is so slow that it is not expected to affect facilities at the site.
- This was the first program to utilize the cone penetration test (CPT) to characterize soft zones. The soft zones were quantifiably defined in terms of engineering parameters that could be measured. The criterion established was a CPT tip resistance (q_t) of 200 pounds/in^2 (psi) or 14.4 tons/ft^2 (tsf) and less for a minimum of a continuous 1-foot thick layer. The tip resistance criterion was based on what could be expected from a normally consolidated, medium plastic clay at the depths in question utilizing;

  $S_u/P \approx 0.2$, and $q_t \approx (N_k \times S_u) \times P_0$, with a low-bound value for N_k of 9
- CPT dissipation tests performed within the soft zones soils indicated no excess hydrostatic pressures. In addition, the CPT results (tip and sleeve resistance, along with dissipation test results) indicated the soft zones were not water-filled cavities, but consisted of soil.
- The soft zones occur in the Santee Formation at a depth of about 115 to 145 feet (critical layer) below the ground surface (K-Area). They are quite erratic, channel-like features that

comprise about 10 to 15% of the “critical layer” by volume (thus the much stronger matrix material comprises 85 to 90% of the “critical layer”). The maximum thickness found was 15 feet, while the average was about 6 feet.

- The soft zones were quite erratic with SPT N-values ranging from weight of rods to over 100 blows/foot and shear wave velocities (V_s) ranging from about 350 to 2,500 feet/second (fps). Shear wave velocities measured via crosshole and downhole methods were consistent. Further, V_s measurements in previously grouted areas were no different from V_s measurements in non-grouted areas.
- The matrix material was not found to be cemented and must resist long term (drained) loading through frictional forces. This is consistent with the finding that the least lateral dimension of the soft zones is similar to their height, since uncemented matrix material would not arch over a soft zone significantly wider than its height. Thus it is not surprising that soft zones comprise a small percentage of the “critical layer”.
- The average of five consolidation tests on soft zone soil samples indicated the average preconsolidation pressure was 32% of the effective overburden pressure. Thus, the consolidation test results demonstrate that vertical stresses have been transferred around these zones to more competent soils, i.e., some form of arching or stress re-distribution has occurred.
- The K-Reactor area soils are adequate to support the existing facilities under the design basis earthquake (DBE). Liquefaction factors of safety were high, while dynamic settlement resulting from the DBE was small, less than 2 inches. Maximum pore pressure increases in the matrix material and in the soil above the soft zone due to the DBE are expected to be about 14% of the existing effective stress. Thus, arching and stress redistribution provided by the matrix materials is not expected to change as a result of the DBE. In addition, paleoliquefaction studies indicate a return period of about 600 years for a repeat of the 1886 Charleston earthquake. The fact that soft zones still exist today indicates that similar events in the past have not caused “collapse”. This is an indication that future events of a similar nature will not cause “collapse”. Note that the site-specific DBE used for design during the RRP (RG 1.60 spectrum anchored at 0.2g) envelopes the current SRS DBE for DOE Performance Category (PC) 3 facilities (Figure 3).
- The effectiveness of the earlier COE grouting program could not be determined. However, through sampling and field testing it was demonstrated that soft zones that existed prior to COE grouting existed after (40 years) grouting as well. Thus it appears the COE grouting program had little affect reducing building settlement.
- As discussed above, settlement was initially determined based on the fact that the matrix soils were stable (they have been for millions of years and have experienced many seismic events, particularly a repeat of the 1886 Charleston event, which has an estimated return period of about 600 years) and, through analysis, were stable after the DBE. Thus, settlements were only due to dissipation of excess pore pressures after the seismic event. Through discussions with the DNFSB, it was suggested (by the DNFSB) that the settlement should be based on full consolidation (i.e., the settlement be based on consolidation from the existing state of stress to full overburden pressure) of the soft zone soils (WSRC 1991b). The subsequent settlement ranged from about 6-½ inches to about 42 inches, depending on assumptions used. The expression of the computed settlement at depth to the surface (or

foundation of the service water piping) was determined using a soft ground tunneling analogy (Peck, 1969).

- In terms of surface settlement in the K-Area study (WSRC, 1991b), the tunnel approach resulted in greater settlement than the numerical method (FLAC) and a mining approach (Bals) developed in the coal industry (Adamek and Jeran, date unknown).

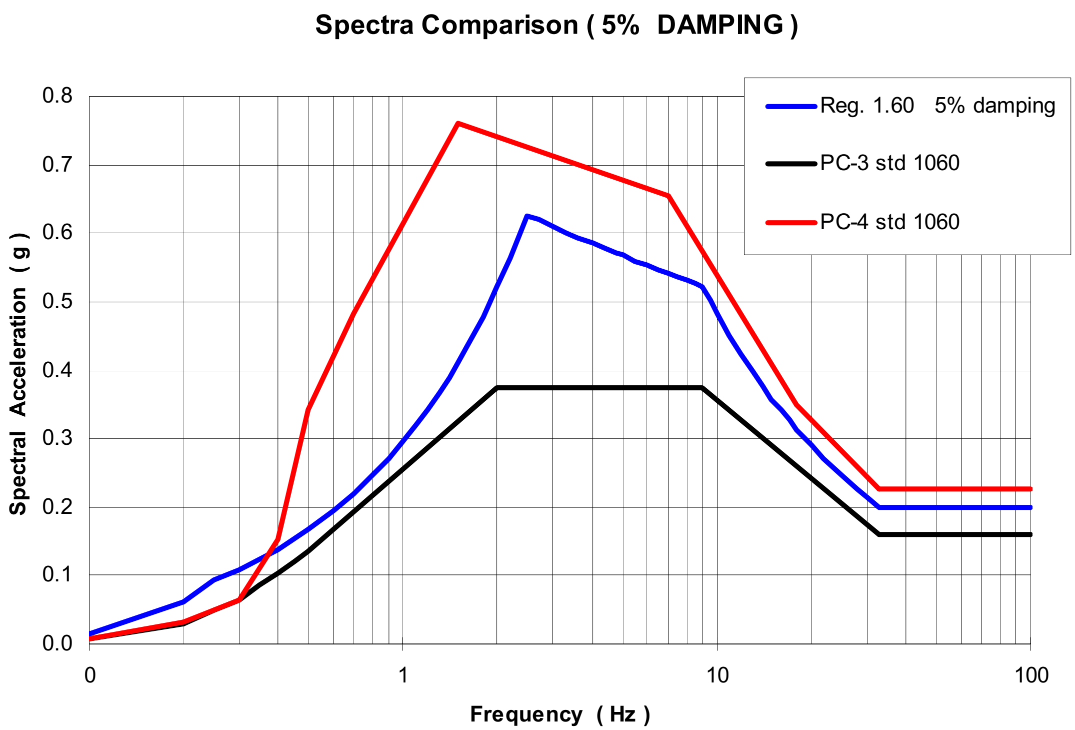

Figure 3; Response Spectra

As noted above, the concept of "arching" was used in the context of stress re-distribution around these weaker soft zones. "Arching" has become a common way of describing the situation at SRS, but in some instances the use of this term may have resulted in an oversimplification regarding the potential future behavior of the geologic layers above and around the soft zone; *can the arch be broken?* Such usage starts to deviate from the real issue, which is one of stress re-distribution. Thus, more properly then, *can the current effective vertical stresses within a soft zone be altered?* The studies by GEI would suggest the answer to the latter question is no. However, a fundamental result of the two GEI studies and the interaction with the DNFSB at that time (1991) was to assume the arch would break and the full overburden pressures would eventually bear on the underconsolidated soft soils. As will be seen later in this document, the SRS analytical model has not deviated from this concept, and still uses the model suggested by the DNFSB of full consolidation. However, the fundamental question still remains; *can the current effective vertical stresses within a soft zone be altered?*

The GEI study (WSRC, 1991a) was the first to quantify the definition of a soft zone utilizing the CPT. SRS has embraced the use of the CPT over the years because of its superior capability for soil profiling. However, SRS has increased the defining threshold for soft zone identification from 14.4 tsf to 15 tsf over a continuous 2-foot layer.

It should be noted that an alternate finite element analytical model (Prototype Engineering, 2005) described below (developed for PDCF in F-Area) give very nearly the same result for the ratio of actual (measured in consolidation tests) vertical effective stress to apparent overburden, measured on K-Area samples, after the soft zones have softened (created by geologic processes) and before the building load has been applied.

3.2 K-Area Soil Stabilization (KASS), and In-Tank Precipitation (ITP) and H-Tank Farm (HTF) Geotechnical Reports

In the early to mid 1990s two reports were prepared, the first on the results of the K-Area Soil Stabilization (KASS) effort (WSRC, 1992) and the second a geotechnical evaluation for the In-Tank Precipitation (ITP) facility (WSRC, 1995). It was during these two efforts that a numerical model was introduced utilizing the software Fast Lagrangian Analysis of Continua (FLAC™), to determine the influence of soft zone settlement at depth on surface facilities. It was also when the concept of the "moveable" soft zone was introduced, to evaluate several soft zone geometries that may be plausible beneath structures. Some of the main points and findings of the reports related to the current SRS analytical model are as follows;

- Use of the dilatometer sounding (DMT) in the KASS study confirmed that the matrix material surrounding the soft zones and the Dry Branch Formation above the soft zones were transferring (re-distributing) stresses around the soft zone soils.
- Numerical modeling utilizing FLAC™ in the ITP study corroborated the findings in the field that around and above soft zones, horizontal pressures were higher than expected, possibly due to arch action (stress re-distribution).
- Numerical modeling utilizing FLAC™ in the ITP study confirmed the measured static settlement at the time (1 ¾ to 2 inches) for the ITP tanks versus 2 ½ inches predicted from the FLAC model; and the *in situ* vertical stress within the soft zones soils (about 65 to 75% of the full effective overburden pressure) versus an overconsolidation ratio (OCR) of about 0.7 determined from laboratory testing. Thus, in the eyes of the analyst, the FLAC™ numerical model and the parameters used were validated.
- According to analysis in the K-Area study, soft zones much wider than 50 feet could not physically exist, they would have already compressed due to full overburden pressure.

3.3 Tritium Extraction Facility Geotechnical Report (TEF) and the TEF Excavation Mapping Program

In the late 1990s two reports were prepared related to the Tritium Extraction Facility (TEF). The first was the geotechnical report, which described the investigations and analyses performed and the second presented the results from the foundation mapping done for the TEF Remote Handling Building (RHB). Some of the main findings are as follows;

- The concept of a "moveable" soft zone was employed, which essentially assumes a soft zone of known thickness (the thickest found beneath a facility) and extent (determined during the exploration phase) could be anywhere under the facility. The facility is then designed for this assumption.
- Both heave of the foundation subgrade (measured during excavation) and subsequent settlement of the TEF RHB were much less than calculated or predicted.

- Use of the CPT to determine stratigraphy was demonstrated very well by direct comparison of pre-excavation CPTs with post excavation mapping.
- The mapping report uncovered flexures and small-scale faults, similar to features found across the SRS, which may be associated with secondary structural tectonic features or carbonate dissolution. However, the most likely causal mechanism appears to be soft sediment deformation in the Tobacco Road Formation (Eocene) and Altamaha Formation (Miocene), indicating movement below this level, as materials are deposited.
- In subsequent reviews with the DNFSB it was concluded that the settlement estimates were reasonable, however it was suggested that the settlements be doubled. Subsequent analysis performed by the structural group demonstrated the adequacy of the structures by spanning the postulated soft zone. It was through this process that the use of grade beams beneath the TEF Process Building was born, with the concept of the foundation system spanning any potential surface subsidence.

3.4 Summary of Early Studies

As noted, the early studies discussed herein (and others not explicitly discussed in this document) were performed over a period of years by various organizations. All of them presented valuable insights, many that have remained valid (in our view) to this date, including:

- Soft zones exist despite the many seismic events that have been postulated over the millions of years since they were altered. They tend to occur along horizontal, or nearly horizontal, planes. They vary in thickness and width. In some cases they are nearly equi-dimensional in plan, and in others they seem to be long stringers. In general, the widths of the soft zones are less than about 15 feet, but are postulated to be larger, however no more than about 50 feet. They appear to be in a honeycombed structure surrounded by more competent matrix soils. Velocity measurements (V_s) in K-Area show no real differences between grouted and non-grouted areas.
- The zones are difficult to sample effectively because they occur at such depths. It is difficult to establish precisely the extent of the zones at any site, and sampling the soft materials themselves presents major challenges. It is virtually impossible to obtain undisturbed samples of soft soils in general; it requires special care and the use of piston samplers under carefully controlled conditions. Various *in situ* methods have been employed to obtain data on the properties, but direct laboratory measurement on undisturbed samples has proven to be somewhat elusive. With this in mind, representative compressibility properties (particularly for the GSA) were established based on known properties measured at the time (i.e., CR of 0.24 and OCR of 0.7). However, it has always been recognized that facility- or site-specific properties, if measured with representative sampling, can and should be used (e.g., K-Area).
- The materials in the soft zones are mostly clayey sands and sandy clays. However, there are some regions of silty sands, clays, and silts as well.
- All evidence is that the materials in the soft zones exist at effective vertical stresses that are lower than the apparent overburden stress at those same depths outside of the soft zone (in the matrix material). This indicates that the overburden stress has been redistributed around the soft zones as the chemical processes formed the weaker and more compressible materials in the soft zones. At SRS they have been characterized as “underconsolidated” relative to soils at the same depth in unaltered (non-soft zones) sediments.

- There is little to no evidence of recent surface expression due to soft zones, either as a result of measured long-term settlement in the GSA or deformation after postulated historic earthquakes across the entire SRS. There is however, evidence of soft sediment deformation, which may be due to compression at depth. However, the deformation has occurred over a period of time and appears to have been very gradual, as originally postulated by the COE in their early studies. Today we believe the most likely causal mechanism is soft sediment deformation that occurred during deposition.
- Use of consolidation theory and the concept of full consolidation (an outcome of the DNFSB/GEI interactions), although conservative is a valid approach and has been embraced by the site since its inception. The process evolved to include the concept of the “moveable” soft zone utilizing the thickest soft zone found beneath a facility.
- Use of the tunnel approach to determine surface expression of settlement at depth is a valid and conservative approach. Because it is based on empirical data that is relatively simple to verify, it is a simple procedure to implement and does not require sophisticated sampling, testing, and modeling.

It is the results and conclusions of these studies (and many others not explicitly discussed herein) evolving over time that have led to the current SRS soft zone analytical model. That basic model (with some modifications) has been utilized for every major facility, with the exception of the Mixed Oxide Fuel Fabrication Facility (MFFF), since the 1991 GEI studies (WSRC, 1991a; and 1991b).

4. Current SRS Analytical Model

The current SRS analytical settlement model is based on two large and broad sources of information; 1) geology and subsurface conditions of the site (much of which was given in the preceding sections), and 2) observed facility performance data in the form of settlement records. Each of these two sources is discussed below in the context of the SRS analytical settlement model.

4.1 Geology and Subsurface Conditions

An understanding of the geologic environment developed over the years since initial explorations carried out in the early 1950s, and continuing with exploration and geologic programs since the late 1980s up to the present day. The essence of that understanding is summarized in “Significance of Soft Zone Sediments at the Savannah River Site (U): Historical Review of Significant Investigations and Current Understanding of Soft Zone Origin, Extent, and Stability,” (WSRC, 1999). The main points are:

- The soft zone soils occur in the Santee Formation, a marine deposit laid down 35 to 50 million years ago. Soft zones originally contained quartz sand, silt, and shell fragments. With geologic time, percolating ground water removed the shelly material by dissolution, leaving a porous but still self-supporting soil, similar to a honey-combed structure. Shell fragments were locally replaced by precipitated silica as they dissolved. Thus, the soils at the SRS have inclusions of soft materials. The soft zones occur (in the GSA) at or near the top of the Santee Formation (ST) in the soil layer identified as DB4/5 and near the bottom of the Santee Formation in the soil layer identified as the ST2 layer. The elevations where these soft materials occur are somewhat higher than 180 ft, msl, which is equivalent to depths of about 100 to 120 feet below the natural surface of the soil. The inclusions are well below the water table.

- There is substantial geological evidence that the soft zones have not had significant effects on surface settlement at the SRS, including when one considers the hypothesized number of large magnitude (M > 6.5 to 7+) earthquakes that have occurred along the South Carolina coast with no paleoliquefaction evidence at the SRS (WSRC, 1996), no recent surface features that would suggest slumping at the SRS, and the fact that soft zones are still present at the SRS.
- There are anecdotal stories regarding surface features that have appeared, but nothing has been substantiated. It is clear that no unusual surface settlement has occurred at the SRS that can be attributed to soft zone settlement since the inception of the SRS. There is however, evidence of soft sediment deformation (WSRC, 2000b).
- The geological, chemical, and mineralogical evidence indicates that these zones formed *in situ*. That is, the materials were not deposited as weak materials. They formed as the result of chemical weathering that altered the mineralogy of the original materials over the several tens of million years since the materials were originally deposited. The materials overlying the soft zones were in place when the chemical processes took place. Thus, there was (and still is) some load (overburden stress), acting on these materials prior to and during alteration.
- The new materials resulting from the chemical processes are more deformable and more compressible than the original materials and the materials above, below, and adjacent to the soft zones.
- The soft zones are small, isolated pockets and channel-like features with an average thickness of only a few feet (in F-Area the average thickness of all soft zones encountered is less than two feet) and postulated lateral dimensions less than 30 feet, but more than likely on the order of 10 to 20 feet or less.
- The *in situ* vertical effective stress is less than the vertical stress at the same depth in a region not containing soft zones. That is, the effect of the softening (in our case dissolutioning) *in situ* and the subsequent redistribution of vertical overburden stress create a zone in which the vertical effective stresses are less than would be computed by simple summation of overburden effects. Since the existing vertical stresses are less than the apparent geostatic vertical effective stresses, these soils have been said to be "underconsolidated", with respect to the vertical stress at the same depth in a region without soft zones (unaltered). The soft zones are underconsolidated compared to the geostatic stress, however they are believed to be normally consolidated under their stress regime (i.e., within the area of stress re-distribution or arching). This concept has not changed since the GEI studies of 1991 (WSRC 1991a and 1991b) and their interaction with the DNFSB.
- The low *in situ* stress within and above the soft zone is possible because of stress re-distribution (arching), within and directly above the Santee Formation. Consolidation tests carried out on recovered samples of soft zone soils confirm the stress is less than the geostatic stress. Other confirmatory data include low penetration resistance (SPT and CPT) and low V_s measurements. It needs to be emphasized here that the criteria to identify soft zones in the field are either a CPT tip stress < 15 tsf or a SPT N-value < 5. However, if a laboratory consolidation test carried out on a sample from the soft zone interval results in an

OCR < 1, it too would be considered a soft zone regardless of the field results. There is a certain amount of judgment that must come into play when analyzing the results, whether they are field or laboratory results.

4.2 Observed Facility Performance

It was recognized early that settlement monitoring would play a key role in determining existing as well as future facility performance (see Background, MPMRCE, 1963). All of the major facilities onsite have had settlement monitoring for various periods throughout their existence. The best summary of how settlement monitoring can play a role is found in the In Tank Precipitation Facility (ITP) and H-Tank Farm (HTF) Geotechnical Report (WSRC, 1995). The overriding conclusion from this report and the settlement monitoring programs is that analysts and designers have tended to over predict settlement for structures onsite. This is not only true for the soft zone soils, but for the overlying sediments as well. The over prediction is due, in part, on the failure to utilize all available information and to rely too heavily on laboratory-derived parameters, at the expense of field monitoring results and other field tests. For example, original settlement estimates for DWPF did not fully account for the overconsolidated state of the subsurface soils and predicted settlements were made assuming more normally consolidated conditions. Predicted settlements for DWPF were eventually revised, based on monitoring results from the nearby H Tank Farm facilities, resulting in predicted settlements of 3 to 5 inches. This compares quite well with the actual measured settlement to date of about 1½ to 3 inches.

So how does static settlement relate to soft zones? It is well established that soft zones exist in the subsurface throughout the SRS. The frequency is open for debate, but in the GSA it is estimated that one has a 25 to 50% chance of encountering a soft zone each time a penetration is made, although this estimate seems large in light of more recent information on the aerial extent of the zones. It is also well established that these zones are weak: SPT N-values of near zero, CPT q_t values less than 15 tsf, and V_s measurements less than about 500 fps. Laboratory consolidation test results have demonstrated that the soft zones have effective vertical stresses lower than the apparent *in situ* geostatic stress. This raises the question, if soft zones are present and they are as prevalent as predicted and they are as large as some believe, why has the corresponding static settlement not been observed, particularly under large structures and aerial fills? There are two possible explanations; either the soft zones are not as compressible as thought; or they are small, isolated pockets, in a honeycombed structure that transfers (or re-distributes) the vertical loads to the more competent matrix soils. Thus, as long as the matrix soils remain intact and continue to re-distribute stresses; settlement of the soft zones will not occur, however if settlement were to occur, it would be gradual over a long period of time as suggested by the early COE studies. We believe the latter to be more indicative of the actual condition. Thus, could settlement monitoring be used as means for early detection of subsurface movement? Although this has not been explicitly done, it is something the COE had in mind when they discussed reviewing settlement records to demonstrate predicted settlements were within acceptable limits. It would require real-time monitoring with multiple instruments at various depths.

Since there is strong evidence that the soft zones are indeed soft, the analytical model developed is predicated on the presence of a zone of stress re-distribution (arch) causing low effective vertical stress (via a honeycombed structure) within the soft zones. Vertical overburden pressures are transferred around the soft zones through the more competent matrix materials within and above the soft zones to more competent soils beneath the Santee Formation. Under present static conditions, the more competent matrix soils are more than adequate to carry the load. This is clearly demonstrated by the fact that several large, heavy, critical facilities have existed onsite for up to 50 years with no adverse or unusual settlement. The question then becomes *will the stronger matrix soils be as competent after a seismic event?* Analyses carried out previously (GEI and BSRI) demonstrated that, even after a seismic event, the matrix soils would still be more than adequate to carry the overburden load (WSRC 1991 and WSRC 1995). However, as has been mentioned and will be discussed subsequently, the current SRS model assumes the full overburden pressure is transferred to the soft zone soils after the DBE.

4.3 General Analytical Procedure

As with the term "arching", the use of the term "underconsolidated" to describe the soft zone may result in some confusion. More classically the term "underconsolidated soil" refers to, for example, the unusual situation where high pore water pressures keep geostatic stresses off of a soil layer, and if not accounted for can result in high settlement situations when constructing a building. In our case the term "underconsolidated" has been used in the past to refer to a situation in which the actual vertical effective stress in the soft zone interval is less than the vertical effective stress that would exist in an unaltered material at the same depth. In any case, similar to the use of "arching", the situation at SRS relates to a low effective vertical stress situation, and whether these existing vertical stresses can be increased to the full geostatic load case. As discussed earlier herein, analysis indicates no. However, based on the discussions between GEI and the DNFSB in 1991, the current SRS model includes this fundamental assumption, i.e., the *in situ* vertical effective stress acting on the soft zone soils is assumed to reach the full vertical effective overburden pressure after the DBE. This current SRS model is believed to be conservative, since soft zones exist in the subsurface today and the site has experienced many seismic events similar to the 1886 Charleston event, which has an estimated return period of about 600 years. However, as discussed throughout, and to be conservative, the current SRS analytical model assumes the matrix material loses <u>all</u> strength after a seismic event and full overburden pressures are transferred to the soft zone soils. Once the overburden pressures are transferred, virgin compression within the soft zone soils occurs, and that compression is propagated to the ground surface. The analytical model used follows Terzaghi's one dimensional consolidation theory. The equation (found in many textbooks) is as follows;

$$S = \frac{C_c}{1+e_0} H \times Log \frac{P'_0 + \Delta P}{P'_0} \qquad (1)$$

Where; C_c is the compression index, e_0 is the void ratio (note CR, the compression ratio, is equal to C_c / {1 + e_0}), P'_0 is the existing vertical stress acting on the soft zone, H is the soft zone thickness, and Δp is the stress required to increase P'_0 to the full geostatic stress (within the matrix soils, σ'_{vG}). Note because the soft zones are assumed to be normally consolidated under their stress regime, the preconsolidation pressure (P_p) determined from a consolidation test from a soft zone would equal P'_0 within the soft zone (for a normally consolidated soil the overconsolidation ratio, which is defined as P_c / P'_0, equals 1), and ΔP in equation 1 is the pressure required to increase the existing overburden pressure (P_p) within the soft zone to the full geostatic stress in the matrix soils (σ'_{vG}). Thus, for a soft zone (relative to the matrix soils) the second term in equation 1 becomes;

$$Log_{10} \frac{P_p + \left(\sigma'_{vG} - P_p\right)}{P_p} = Log_{10} \frac{\sigma'_{vG}}{P_p} = Log_{10} \frac{1}{OCR} \tag{1a}$$

Thus, equation 1 becomes;

$$S = CR \times H \times Log_{10}\left(OCR\right)^{-1} \tag{1b}$$

In addition, since all of the compression is assumed to be virgin compression (another conservative assumption), there is no need to be concerned with the recompression index (C_r). This analytical model is a direct result from the GEI studies of 1991 based on full consolidation, as suggested by the DNFSB, and has been used at the SRS ever since. It was peer-reviewed (Mitchell, Marcuson, Sowers, Schmertmann, and Gould) and accepted. The generalized procedural steps are as follows:

1) Establish a nominal width and thickness for the soft zone, through exploration. The width is usually 10 to 20 ft, but our analysis often assumes greater widths (in the form of parametric analysis) to be conservative. Current analysis generally uses the maximum thickness encountered (again conservative) in any one exploration hole for a particular facility.

2) Assume that after the earthquake, the material around and above the soft zone will have lost strength to the point that all the overburden stress will be placed on the material in the soft zone, which will then compress to accommodate the added stress. Estimate the compressibility of the materials within the soft zone. Design values that have been used at SRS since 1999 are compression ratio (CR) of 0.24 and overconsolidation ratio (OCR) of 0.7 (WSRC, 1999). These values were developed (particularly for the GSA) based on data available and are thought to be conservative "representative" values, particularly when used with the maximum soft zone thickness. However, facility-specific data can be generated. The problem in the past is in obtaining a representative sample that is "undisturbed".

3) Compute the compression at depth within the soft zone utilizing equation 1b, the properties from step 2, and the maximum soft zone thickness.

4) Assume the settlements computed in step 3 express themselves at the surface of the site and use the resulting values to design the foundations of the structures. The finite difference program FLAC™ has been used to determine the surface expression of settlement at depth. One modification is to treat the compression of the soft zone as though it were the deformation resulting from soft ground tunneling and use the well established and accepted empirical methods from the soft ground tunneling industry to propagate the deformation at depth to the surface, usually in the form of an inverted Gaussian distribution. Both analyses (FLAC™ and the empirical tunnel analogy) have given similar results in the past, in terms of total settlement and distribution of surface settlement. Typically, however, the soft ground tunnel analogy has been more consistent and conservative (resulting in greater total settlement and larger angular distortion) and it is the preferred approach. A more detailed discussion of the current methodology to propagate settlement at depth utilizing the soft ground tunnel analogy is discussed in Section 4.5.

5) Apply the resulting settlement anywhere beneath the facility and design accordingly. This has been the preferred practice at SRS since the TEF project (1999), to employ the "moveable" or random soft zone concept. However, other approaches, such as fully defining the soft zone lateral extent (used for the Actinide Packaging and Storage Facility [APSF] project), have been successfully employed.

As already discussed, this approach utilizes the maximum soft zone thickness encountered and assumes the strata above the soft zones have no strength after the earthquake. Thus, full overburden pressures are transferred to the soft zones and, in this context, is thought to be very conservative. It also requires a very difficult estimation of the compressibility properties of the materials in the soft zones, including the maximum past pressure. This is because the soft zones are difficult to locate, sample, and test, and once they are tested there is debate regarding the results. However, in the view of the site, because the analysis has many "built-in conservatisms, there can be little doubt that, if a structure can be designed to meet the requirements of the method, it will have adequate capacity to withstand both long-term static settlements and settlements from the effects of earthquakes.

Section 4.4 discusses the application of the finite element method (FEM) to propagate settlement at depth to the ground surface in an alternate (Prototype Engineering) analytical model, developed for the PDCF project, while Section 4.5 discusses the use of the tunnel analogy to propagate settlement at depth to the ground surface.

4.4 PDCF Prototype Engineering Analytical Model

The analytical model proposed by Prototype Engineering (PE), 2005 is predicated on modeling the entire sequence of events from deposition through softening of the soft zones to placement of the building loads and subsequent earthquakes. This was accomplished through a series of steps analogous to deposition of the soils and applying gravity, through excavation and eventual construction and loading of the foundation. Foundation and building stiffness was also accounted for in the model. The result is that the soft zones have compressed during deposition or during alteration with geologic time. Thus, as long as the overlying materials do not liquefy during the earthquake, there will be no additional settlement after a seismic event due to increasing the vertical effective stress (geostatic stress) on the soft zones because there is no "arch" to break and thus no load transfer or load re-distribution. However there will be settlement due to seismic shaking (increased pore pressure and subsequent dissipation) and as additional static load is applied (e.g., a structure). The consequence of this is that initial predicted static settlement may be higher than previously thought, but the settlement due to a seismic event will be substantially less than previously thought.

The Prototype Engineering (PE) model is similar to the SRS model. Both models contain soft zones with similar dimensions, soil properties, and compressibility. The main difference is the postulated stress conditions. The current SRS model contains arches or a honeycomb structure that re-distributes vertical load from the soft zone, and assumes the full overburden pressure is eventually transferred to the soft zone soils after a seismic event, whereas the PE model computes directly the redistribution of stresses during the softening process (over geologic time), and during the addition of building loads and after a subsequent seismic event. With the PE model the soft zone consolidation slowly keeps pace with changing stiffness of the soft zones (during deposition and subsequent softening), and settlements slowly propagate to the ground surface (over geologic time). Given a change in vertical stresses (e.g. added building weight) the soft zones and neighboring more competent soils would consolidate or settle (albeit to varying degrees) and the differential settlements would propagate to the surface. These settlements are due to static loads and would occur as new loads are added. For narrow soft zones, the PE model suggests that the shape of the effective vertical stresses takes on an arch-like look, but this is not critical. It is the vertical effective stresses that are critical. For wider soft zones, the PE model suggests that the shape of the effective vertical stresses takes on a vertical shaft-like look. In all cases however, as the zone softens the effective vertical stresses are lowered.

Using the PE model, dynamic settlement due to earthquake loading is based on seismic shaking (increased pore pressure and subsequent dissipation). Once the soft zones have softened, they have virtually no stiffness and contribute little to resisting the deformations that result from dynamic shaking. Thus, liquefaction of the soft zones is irrelevant; it is the soils above the soft zones that are critical from a liquefaction standpoint. Furthermore, the PE model allows the Congaree Formation (CG) layer below the Santee Formation (location of the soft zones) to deform or to be a rigid boundary.

Since advanced numerical techniques such as the finite element and finite difference methods are now available, it is possible to perform an analysis that more closely conforms to the conditions and geologic history at the site. It also involves two-dimensional analyses instead of the earlier one-dimensional approach. The steps are as follows;

1) Generate a finite element mesh consistent with the soil profile.
2) Solve for the initial geostatic stresses. The analysis is a “gravity-turn-on” analysis, but it must be done in several incremental steps because of the non-linear material properties. No soft zones exist at this point. This is the initial condition for the analysis of the effects of soft zones.
3) Soften the soft zones in several steps, as discussed above. This involves reducing the stiffness and strength of the soft zones in the upper Santee Formation (DB4/DB5) layer. Stresses at the end of this step represent initial conditions before the PDCF structure is built. There is significant redistribution of vertical stress over the soft zones when the zones soften, and these conditions are reflected in the rest of the analyses. Note it is understood that the actual softening process occurred over several million years, and it is not possible to model that process exactly. However, the analytical process used in the PE model is believed to be a very good approximation given the available information.
4) Impose a surface load to represent the building. Stresses and settlements at the end of this step represent conditions after construction of the building and during its productive use but before any additional event such as an earthquake. In some later analyses the stiffness of the building was included as a stiff layer at the surface of the profile.
5) Impose settlements at the top of the soft zone to represent the additional compression of the overlying materials caused by an earthquake (this is reasonable because the compressing overlying materials are not far above the soft zones). The settlements are estimated from the results of level ground liquefaction analysis (with and without the influence of the structure) of the soils above the soft zones. The results of these analyses are the volumetric deformations that would be caused by the earthquake.

This procedure improves on the past methodology in several ways. First, it conforms to the actual history of the site and recognizes that the effects of developing the soft zones have taken place long ago and are already reflected in the surface profile. Thus, the deformations at the surface that are relevant to design are only those due to placing the building load and any subsequent effects of an earthquake. Second, it does not require that the maximum past pressure and one-dimensional compression properties of the materials in the soft zones be estimated. It does require that the analyst estimate the shear strength and stress-strain characteristics of the materials. It allows the engineer to assume reasonable, conservative values of the compressibility of the materials and to proceed from there. Third, it allows the size and location of the soft zones to be moved in order to estimate parametric effects.

4.4.1 Summary Regarding the PE Model

The assessment of soil settlement at SRS depends on the behavioral model associated with soft zone formation, state of stress, and impact of potential stress changes. The current SRS analytical model is thought to be very conservative and structures designed to withstand deformations determined from that model will have more than adequate margin. Recent finite element modeling (Prototype) has provided additional insight regarding some concepts, and has also clarified others. The Prototype model is believed to be more realistic, attempting to incorporate the sequence of events from deposition throughout the life of the facility. It is critically important to recognize that measured building settlements over the past 50 years strongly suggest that soft zones are not of sufficient size or character to impact static settlement conditions, thus corroborating the evidence that these features are small isolated pockets, possibly in a honey-combed structure, surrounded by much more competent matrix soils. In addition, measured settlements have corroborated the use of average or best estimate soil compressibility properties. The extension of this is that because they are small isolated pockets they would also not be affected by a seismic event. Given this situation the following critical factors are important in terms of understanding the current behavioral model for soft zones:

- Through the very slow process (geologic time; likely millions of years) of chemical alteration soft zones are created. As this alteration takes place the effective vertical stresses above and around these soft zones are lowered.
- Soft zone consolidation slowly keeps pace with changing vertical stresses and settlements slowly propagate to the surface. As this slow process takes place over hundreds of thousand to millions of years, this settlement would not be detectable. This settlement has already occurred.
- Studies performed at the SRS have shown no paleoliquefaction features.
- The majority of the depressions mapped on the SRS are aeolian in nature. Karstic-like features (if they exist) are more likely in the extreme southern portion of the SRS, few have been located within the GSA of the SRS.
- Depending on the width and thickness for the soft zones, the shape of the lower effective vertical stresses may take on an “arch” like look; however there is no arch to break. The key question today is under what conditions can the effective vertical stresses be returned to the full geostatic condition?
- Settlements as a result of added building weight should be based on average or best estimate properties for the various geologic layers (including soft zones) below the building. The soft zones (by volume and by their physical honeycombed like character) have little overall impact. Fifty years of building settlement at SRS support this model.

- Returning the effective vertical stresses to the geostatic condition would require large pore water pressure increases in the layers above the soft zones and below the water table. This would require liquefaction of these layers. This is a key difference between what is now supported versus previous thinking that in part was based on assuming the "arch" was near incipient failure. To reiterate, there is no "arch" to break, rather the evolution of soft zones creates a low effective vertical stress zone that may have the shape of an "arch".
- The annual frequency of liquefaction is the critical issue in terms of assessing the additional consolidation potential for the soft zones. Previous studies for F- and H-Areas indicate that the probability of liquefaction is well below 0.0001 per year. This conclusion would not change given the current (PE) model.

The overall seismic safety for nuclear facilities at SRS depends on assessing the liquefaction potential for the geologic layers below the structure of interest. Numerous liquefaction assessments have concluded that there is adequate margin of safety against liquefaction at SRS for the soils directly above the soft zones. In this context, the soft zones are not expected to represent a unique seismic issue in terms of assessing SRS nuclear facilities.

4.5 SWPF Soft Zone Settlement

As discussed earlier, the SRS analytical model is based on soft zones at depth that have been altered over geologic time to the degree that they are not subjected to the full geostatic overburden pressure. Overburden stresses are redistributed (arched) to the more competent matrix sediments that surround these soft zones or pockets. The concerning issue regarding soft zones is the determination of surface settlement, if any, that would result if soft zones were subjected to full overburden pressures.

During the last 15 years, analytical solutions have been used to quantify the surface effect of soft zone compression at depth. Analytical solutions have included two approaches: (1) Computation of stresses, strains, and displacements by solving a system of equations containing equilibrium, compatibility, and constitutive equations (numerical models); and (2) Computation of surface displacements using kinematic relations of displacement propagation below ground surface (empirical models). The system of equations in the first approach can be solved using finite element or finite difference methods, while the kinematic relations in the second approach can be obtained utilizing empirical data from soft ground tunneling construction. Due in part to the difficulty in concurring on specific input parameters, constitutive soil models, and software applications to use, the kinematic (soft ground tunneling) approach, which is based on real measured data from coastal plain sediments, has been used for many previous SRS projects. Recently for the Salt Waste processing Facility (SWPF), the methodology was enhanced in an attempt to refine the discretization of the soil elements at depth and to include 3D effects. That methodology is briefly summarized below.

4.5.1 Methodology

Settlement due to the compression of soft zones can be estimated using kinematic relations based on empirical data from soft ground tunneling. Settlement data from soft ground tunneling construction (Cording et al., 1976) indicate that:

- The vertical displacement occurring in an area at depth will propagate to a larger area at the ground surface.

- The surface settlement will be in the shape of an inverted normal distribution curve (for an individual "tunnel" or, in our case, a soft zone).
- The width of the settlement trough depends on the angle of propagation, which is dependent on the soil type.
- The volume of the surface settlement trough depends on the volume of ground lost (settlement of soft zones) at depth.

The resulting surface settlement manifests itself at the ground surface as shown on Figure 4, where the surface settlement *z(x)* at any point *x* from the center of the normal distribution curve is (Cording et al., 1976; and several others):

$$z(x) = z_0 e^{\left(-x^2 / 2i^2\right)} \qquad (2)$$

Where; z_o is the maximum settlement at ground surface, located at the center of the normal distribution curve, $i = W / (2\pi)^{1/2}$ is the distance from the center of the normal distribution curve to the point of inflection, and W is the half-width of the settlement trough curve and may be estimated as:

$$W = Z \tan \beta + a. \qquad (3)$$

Where; Z is the depth to top of soft zone, a is the half-width of the area at depth where the displacement originated, and β is the angle of propagation

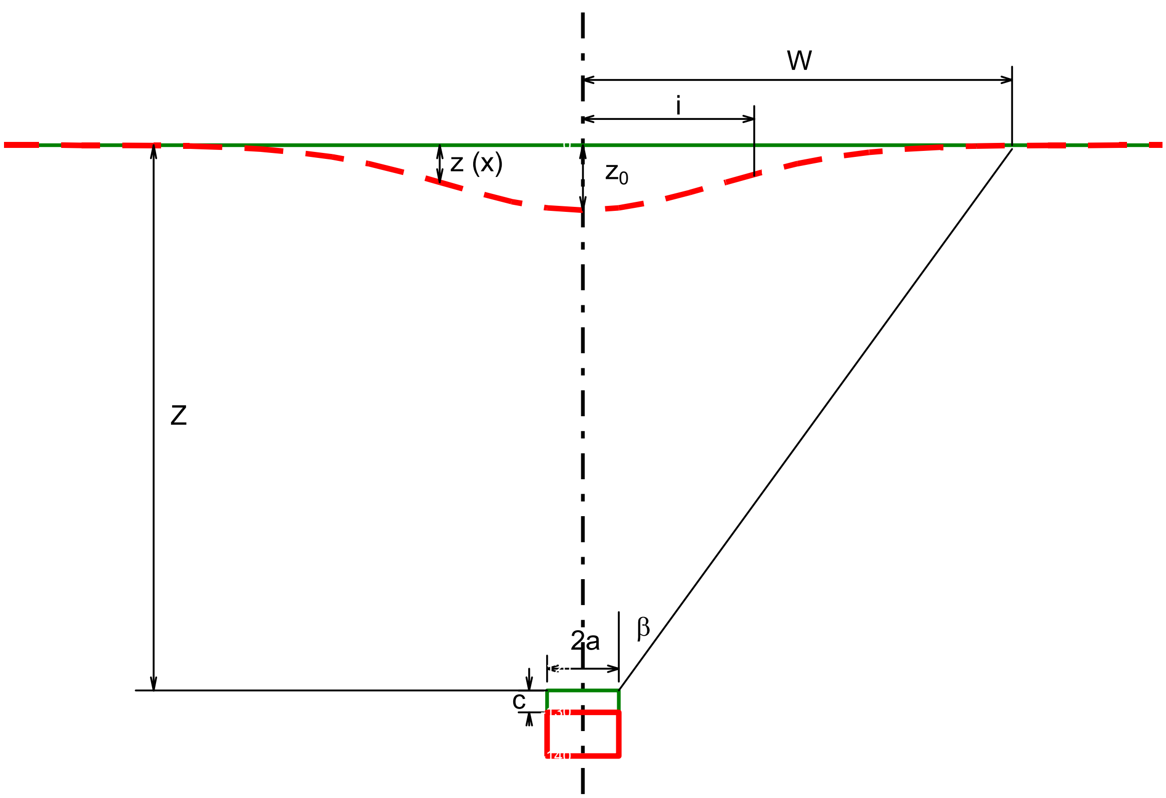

Figure 4; Soft Zone Compression and Resulting Surface Settlement

The settlement data in tunneling construction have related the angle of propagation to the soil type. The settlement data have also been related to the volume of the measured surface settlement trough (Vol_S) and the volume of ground lost (Vol_L) as the tunnel is constructed (Cording et al., 1976). Lost ground refers to the amount of actual excavation in excess of the tunnel volume. In the context of SRS soft zone settlement analyses, volume of ground lost refers to the volume of compression within the soft zone. For our analysis we assume that Vol_S is equal to Vol_L. We believe this assumption is conservative, since the soils above the soft zones would be expected to dilate as settlement propagates to the ground surface thereby reducing the extent of the settlement trough and the magnitude of maximum settlement.

Either 2-D or 3-D analysis can be performed. The choice between 2-D and 3-D analysis is based on subsurface conditions and the results of the soft zone stratigraphy assessment. If the stratigraphy indicates finite, isolated pockets of soft sediments, then the 3-D analysis will be used. On the other hand, if the stratigraphic interpretations support the existence of long, lenticular features, then the 2-D analysis can be performed. Note: For soft zones with long, lenticular features, the 3-D analysis will provide essentially the same results as the 2-D analysis near the center of the feature and therefore either approach can be used. Thus the use of 2-D vs. 3-D methods is based on the amount of data available and whether soft zone geometries can be quantified. It is clear, however, that the subsurface compression and subsequent propagation to the ground surface is a 3-D problem as interaction of the surrounding soils will be engaged in all directions.

4.5.2 Two-Dimensional (2-D) Method

The 2-D method is performed by considering a vertical slice of subsurface with unit length perpendicular to the width of the soft zone. At SRS, a 2-D method utilizing empirical data from tunneling construction was developed in the early 1990s by GEI for the K Reactor Restart efforts (WSRC, 1991) and has since been progressively refined on many projects. The current 2-D method is performed by discretizing the soft zones into a series of sub-areas, computing the results of each sub-area, then superpositioning the results for all sub-areas (WSRC, 2007b). Sub-areas are rectangular in shape and adjacent to each other in the horizontal direction. Multiple clusters of sub-areas may be required to model soft zones at various locations. The width of each sub-area is chosen to be identical, while the elevation and the height of each sub-area replicate the elevation and thickness of the soft zones at the location corresponding to the location of the sub-area, as determined through subsurface investigation and stratigraphic interpretation. When the soft zone is compressed, the resulting displacement at the top of each sub-area will propagate to the ground surface and cause the surface to settle. A FORTRAN code (WSRC, 2007c) as well as spreadsheets (WSRC, 2007d) were developed to compute the resulting surface settlement.

4.5.3 Three-Dimensional (3-D) Method

The 3-D method is performed by discretizing the soft zones into a series of 3-D sub-spaces, computing the results of each sub-space, then superpositioning the results for all sub-spaces (WSRC, 2007b). Sub-spaces are in the shape of square columns and adjacent to each other on the horizontal plane. Multiple clusters of sub-spaces may be required to model soft zones at various locations. The width of each sub-space is chosen to be identical, while the elevation and height of each sub-space replicate the elevation and thickness of the soft zones at the location corresponding to the location of the sub-space in the subsurface, as determined through subsurface investigation and stratigraphic interpretation.

Surface settlement due to the compression of a sub-space is computed by considering the sub-space as a square column. When the horizontal cross-section is sufficiently small, the resulting surface settlement trough will be the same as the settlement corresponding to a circular column with the same horizontal cross-sectional area. The profile of the settlement trough at the surface would be an axisymmetric space formed by rotating the normal distribution curve (see Fig. 4) about the z axis with the volume of (WSRC, 2007b):

$$Vol_S = z_0 W^2. \qquad (4)$$

When the soft zone is compressed, the resulting displacement at the top of each sub-area will propagate to the ground surface and cause the surface to settle. A FORTRAN code was developed to compute the resulting surface settlement (WSRC, 2007c).

4.5.4 Comparison of Two-Dimensional vs. Three–Dimensional Methods

Based on the previous discussion and derivation (WSRC, 2007b; WSRC, 2007c), the maximum surface settlement associated with a sub-area using the 2-D method is

$$z_0 = 2c \times {}^{a}\!/_{W} \qquad (5)$$

and the maximum surface settlement associated with a sub-space using the 3-D method is

$$z_0 = 4c \times {}^{a^2}\!/_{W^2} \qquad (6)$$

where c is the vertical displacement (compression) of the soft zone at depth.

Therefore, the surface settlement is linearly proportional to the soft zone compression c. Furthermore, if a is small compared to W, using the 2-D method, the maximum settlement is approximately inversely proportional to W or $\tan\beta$. While using the 3-D method, the maximum settlement is approximately inversely proportional to W^2 or $(\tan\beta)^2$.

For a soft zone with small horizontal extent, the ratio between the settlements using the 3-D and the 2-D methods can be seen based on the ratio of z_0 in Equations (5) and (6):

$$\frac{4c \times {}^{a^2}\!/_{W^2}}{2c \times {}^{a}\!/_{W}} = 2 \times {}^{a}\!/_{W} \qquad (7)$$

Therefore, the difference between settlements derived using the 3-D method and the 2-D method can be very significant. The relation (Equation 7) between the settlements using 3-D and 2-D methods is applicable for soft zones with small horizontal dimensions. For soft zones with larger horizontal dimensions, this ratio will increase. Figure 5 shows the ratio of settlements using 3-D and 2-D methods (as applied utilizing the soft ground tunnel analogy) for typical SRS soft zone dimensions. (Note that the abscissa is soft zone half-width.) A similar trend was also observed in the K Reactor numerical analysis using FLAC™ (WSRC, 1992), which showed that the ratio of 3-D (axisymmetric) to 2-D (plane strain) ranged from 8% to 14% for a 50-ft wide soft zone, even more significant than shown here with the kinematic approach.

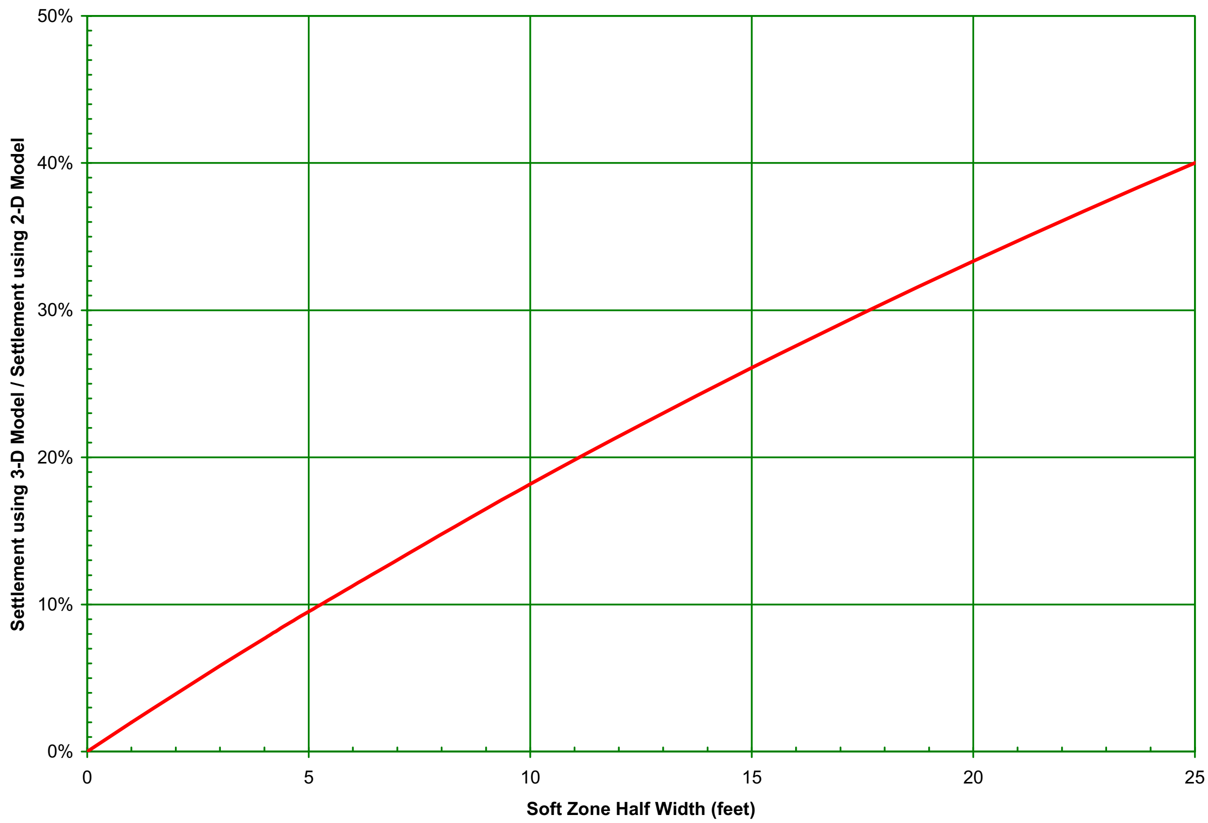

Figure 5; Comparison of Surface Settlement using 3-D and 2-D Analyses

Thus both the SWPF empirical (tunnel analogy) analyses and the K Reactor numerical modeling suggest that the 3-D effects are very significant and cannot be ignored. In our opinion, we expect that any numerical model and subsequent analysis would result in much less settlement than the empirical approach used herein, notwithstanding the differences between 2-D and 3-D results. At SRS this trend has been demonstrated for soft zone settlement analyses by the aforementioned K Reactor effort (WSRC, 1992) and recent analysis performed for the Pit Disassembly and Conversion Facility (Prototype Engineering, 2005).

5. Soft Zone Properties

Numerous soft zone samples have been collected over the years and tested in the laboratory. Sampling has been difficult due to the difficulty in identifying and finding soft zone intervals in the subsurface and then ensuring that the sample collected was in fact from a soft zone (identified from other exploration methods; e.g., CPT $q_t < 15$ tsf). The other difficulty is actually obtaining a representative sample with a piston sampler. It has been recognized that the samples collected were more than likely "the best of the worst". Thus, we may have only sampled soft zone intervals that reflect the most favorable conditions. In any case, Figures 6, 7, 8, and 10 show the results from laboratory testing on known soft zone intervals from various site locations, while Figure 9 shows results of overconsolidation ratio for soft zones and matrix soils from the Santee Formation.

Figure 6 summarizes the results of Atterberg limit tests indicating that for the most part the fine grained portion of the soft zone soils plot somewhat parallel to and slightly above the "A Line", indicating a clayey material. The data show that about ¼ of the samples tested have liquid limits (LL) > 100%, about 1/3 have LL between 50 and 100%, and the remainder (~ 40%) fall on the low plastic side. In terms of plasticity index (PI), about 2/3 of the data have a PI between 0 and 30%, about 1/10 have a PI between 30 and 60%, and the remainder (~ 1/5) have a PI > 60%.

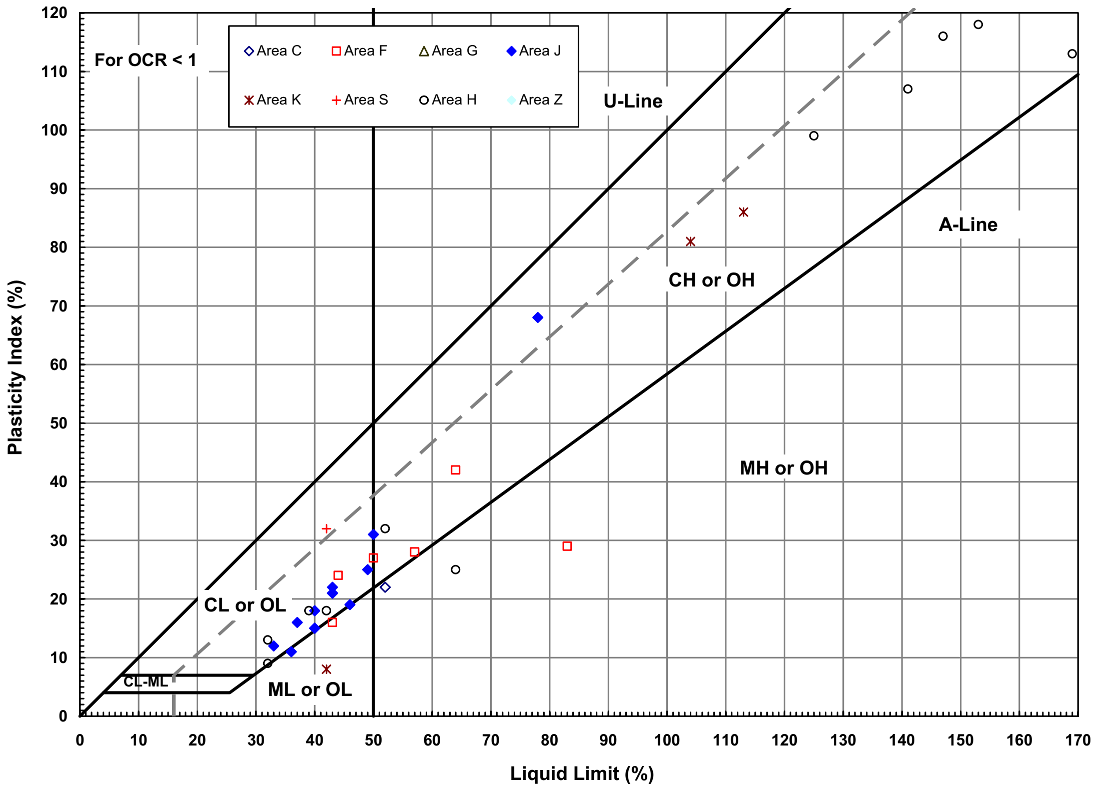

Figure 6; Comparison of Atterberg Limits

Figures 7 and 8 illustrate the compressibility of the material in terms of compression index (C_c) and compression ratio (CR), respectively, versus *in situ* moisture content. Although there is some scatter, the trend is apparent. This would indicate that if an accurate profile of moisture content could be established then compressibility (with some variation) could be determined.

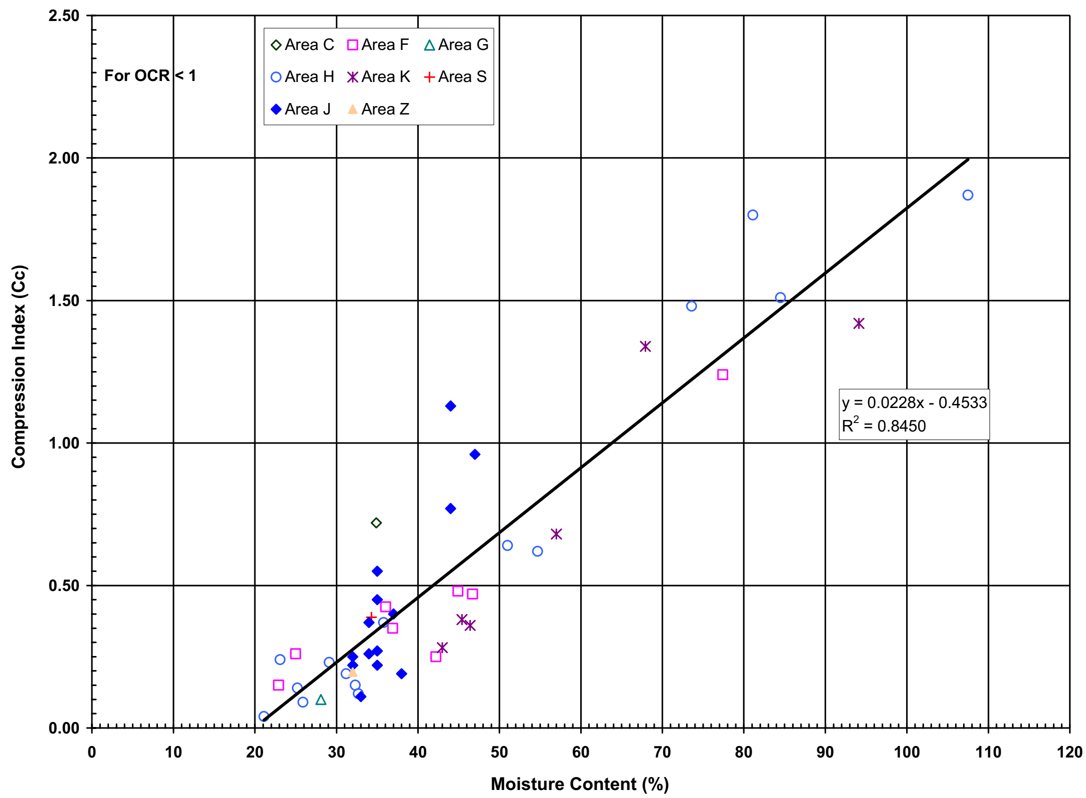

Figure 7; Plot of Compression Index vs. Moisture Content

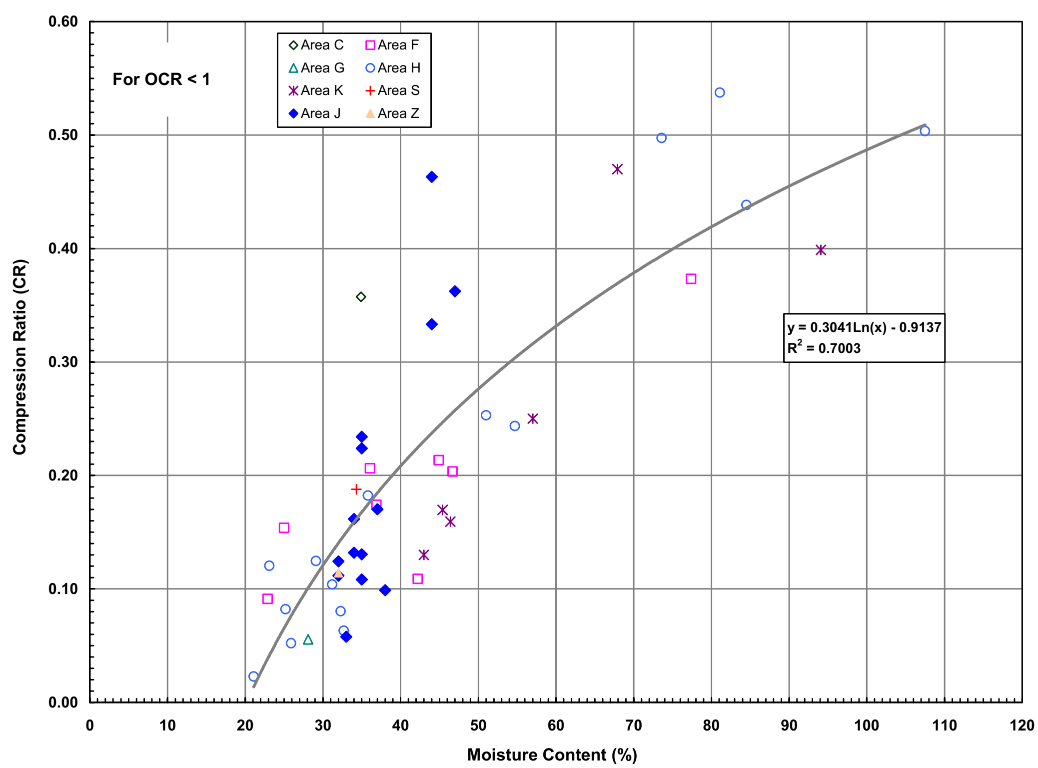

Figure 8; Plot of Compression Ratio vs. Moisture Content

Figure 9 shows the relationship between depth below the ground surface and OCR for soft zones and the surrounding stronger matrix soils. Depth was chosen over elevation in this case because the ground surface elevation varies considerably from area to area. Figure 9 clearly illustrates the issue of "underconsolidation" (relative to the matrix soils within the Santee Formation) of the soft zone soils sampled and tested. The results are about evenly split between OCR < 1 and OCR > 1.

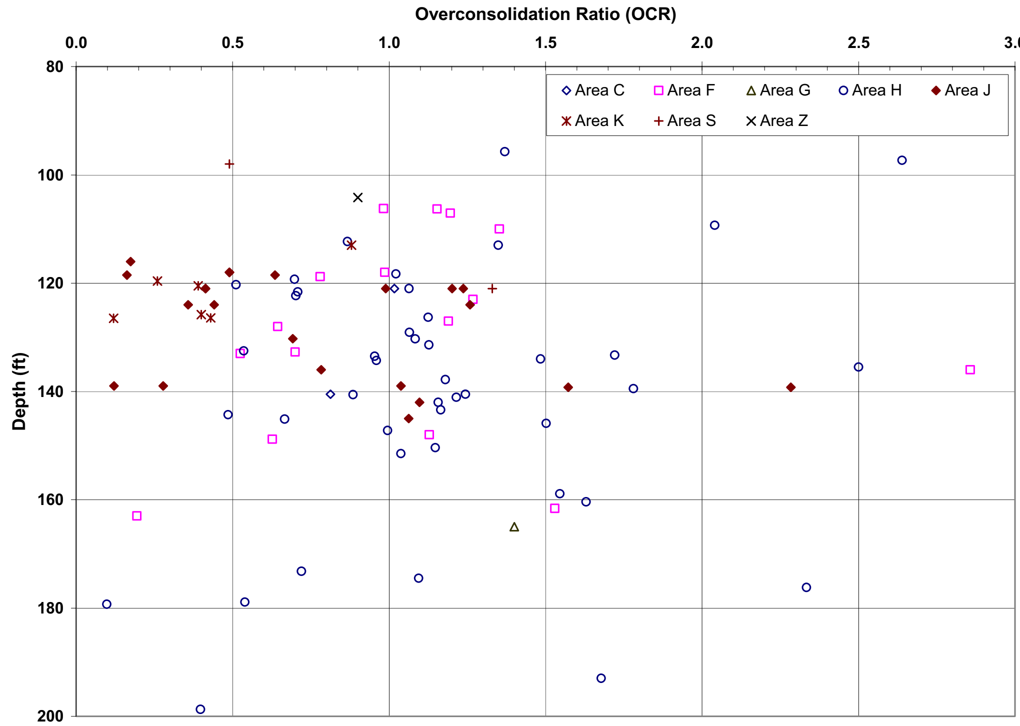

Figure 9; Plot of OCR within the Santee Formation

Figure 10 attempts to combine the results from Figures 7, 8, and 9 illustrating the relationship of strain versus OCR for the soft zone soils tested. Strain in this case is the computed compression, *S*, divided by *H* (the soft zone thickness) in equation 1b (*S/H*). The trend line shown on Figure 10 is based on a representative CR of 0.24, which has been used for many evaluations onsite. In past studies in the GSA, we have utilized what was thought to have been a representative (not necessarily conservative) value for OCR of 0.7.

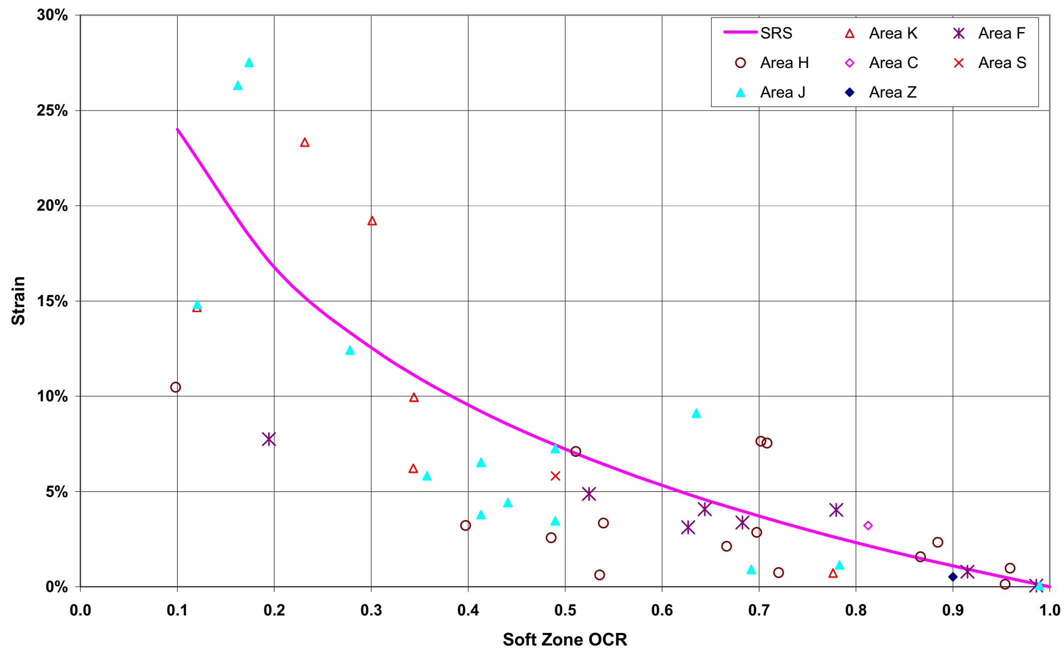

Figure 10; Comparison of Strain

6. Conclusions

Soft zones are present within the subsurface at the SRS. Early treatment of these zones by the COE was performed by grouting. To quote the COE, "…the relatively frequent occurrence of water loss and dropping of drill rods suggested that a zone of cavities or extremely loose deposits underlies the structure sites [referring to the reactor sites]. The presence of such cavities or loose deposits was considered of such importance that grouting was required to the extent necessary for reasonable assurance that all cavities beneath the 190 and 105 buildings were filled." As mentioned, the grouting program was designed to fill cavities. Subsequent analysis by the COE did not account for the presence of grout or demonstrate the benefits of the grouting program. Measured settlements of the reactor facilities and other structures constructed over the years (with and without grouting) demonstrate that the subsurface conditions across the site are capable of supporting large, moderately loaded structures without any adverse settlement. This is an indication of either the lack of large thick zones of soft underconsolidated soils beneath the facilities, or that there is an arching mechanism that is re-distributing load from the structure to more competent materials. Thus, from a static point of view, the presence of soft zones (as presently understood) do not pose a settlement problem.

From a seismic viewpoint, returning or re-establishing the vertical effective stresses within the soft zones to the full geostatic condition would require large pore water pressure increases, and possibly liquefaction, in the layers above the soft zones. Previous probabilistic analyses in H- and F-areas indicate the annual probability of exceedance for liquefaction is on the order of 0.0001 per year. In this context, soft zones are not expected to impact SRS facilities. However, it has been assumed by the SRS that the DBE can alter the state of stress (an assumption that bears additional scrutiny) such that the full vertical overburden pressure bears on the soft zone soils, resulting in compression of the soft zone soils. In this context, soft zones do pose a potential settlement issue. The resulting settlements at depth are computed utilizing the one-dimensional consolidation equation with OCRs and CRs determined from laboratory tests on recovered soft zone soil. Soft zone compressibility properties appear to follow established trends relative to moisture content. Propagation of the computed settlement at depth to the foundation level has been performed both numerically (FLAC™ and FEM methods) and empirically (tunnel analogy). The empirical tunnel analogy model was developed in conjunction with the DOE (including WSRC, BNL, and GEI) during the Reactor Restart Program and has been enhanced over the years; we believe it to be conservative. Thus, as long as facilities can be designed to withstand surface settlement computed in this way, there should not be any compromise in terms of facility safety.

7. Acknowledgments

This report has summarized five decades of exploration, testing, and analysis of the soft zone soils at the SRS. Thanks to the entire SRS Geotechnical Engineering organization, particularly Ms. Laura Bagwell, Dr. William Li, Mr. Michael McHood, and Mr. Rucker Williams. The work summarized was greatly improved by the recommendations, suggestions, and discussions with others including Dr. Brent Gutierrez (DOE-SR), Dr. Carl Constantino, Dr. Frank Syms, Dr. Randolph Cumbest, Mr. Larry Salomone, and Mr. Jeffrey Kimball.

8. References

Adamek, V. and Jeran, P. (no date), "Precalculation of subsidence over long wall panels in the northern Appalachian coal region."

Cording, E.J., Hansmire, W.H., MacPherson, H.H., Lenzini, P.A., and Vonderohe, A.P., (1976), "Displacements around tunnels in soil," U.S. Department of Transportation, Report No. DOT-TST 76T-22, August, Washington, DC.

Moran, Proctor, Mueser & Rutledge Consulting Engineers (MPMRCE 1963) "Foundation Investigations and Treatment at Savannah River Plant," April 19, 1963.

Department of Engineering Research, Civil Engineering Department, North Carolina State College (NCState 1951), "An Engineering Soils Map Prepared From Aerial Photographs of the Savannah River Area," prepared for the Charleston District Office of the U. S. Army Corps of Engineers, March 1951.

Peck, R.B., (1969), "Deep excavations and tunneling in soft ground," State-of-the-art report, *7th Int. Conf. on Soil Mechanics and Foundation Engrg.*, Mexico City, pp. 225-290.

Prototype Engineering, Inc. (2005), "Report on finite element and level ground liquefaction analyses of soft zones for the PDCF at the SRS," Winchester, MA, October.

U. S. Army Corps of Engineers, Charleston District (COE 1952a) "Savannah River Plant Foundation Investigation – P Area," Miscellaneous Paper No. 3-7, Waterways Experiment Station, Vicksburg, MS, May 1952.

U. S. Army Corps of Engineers, Charleston District (COE 1952b) "Geologic Engineering Investigations, Savannah River Plant," Volumes 1 & 2, Waterways Experiment Station, Vicksburg, MS, 1952.

Westinghouse Savannah River Company (WSRC 1991a), "K Reactor Area Geotechnical Investigation for Seismic Issues – Technical Evaluation," GEI Consultants, Inc., WSRC-TR-91-47, Rev. 0, March 1991.

Westinghouse Savannah River Company (WSRC 1991b), "Remediation of Soft Zones in the K-Reactor Area," GEI Consultants, Inc., WSRC-TR-91-415, Rev. 0, June 1991.

Westinghouse Savannah River Company (WSRC 1992), "K-Area Soil Stabilization Program (KASS)", WSRC-TR-92-299, Rev. 0, July 1992.

Westinghouse Savannah River Company (WSRC 1995), "In-Tank Precipitation (ITP) and H-Tank Farm (HTF) Geotechnical Report," WSRC-TR-95-0057, Rev. 0, September 1995.

Westinghouse Savannah River Company (WSRC 1996), "Paleoliquefaction Assessment of SRS (U)", Fiscal Year Status Report, Subcontract C001015P, September 1996.

Westinghouse Savannah River Company (WSRC 1999), "Significance of Soft Zone Sediments at the Savannah River Site (U): Historical Review of Significant Investigations and Current Understanding of Soft Zone Origin, Extent and Stability," WSRC-TR-99-4083, September, 1999.

Westinghouse Savannah River Company (WSRC 2000a), "Commercial Light Water Reactor Tritium Extraction Facility, Geotechnical Summary Report (U)," K-ESR-H-00010, Rev. 2, September 2000.

Westinghouse Savannah River Company (WSRC 2000b), "Tritium Extraction Facility (U) Excavation Mapping Program," K-TRT-00001, Rev. 0, December 2000.

Washington Savannah River Company (WSRC 2007a), "Salt Waste Processing Facility (SWPF) Dynamic Settlement Evaluation (U)," K-ESR-J-00002, Rev. 1, August 2007.

Washington Savannah River Company (WSRC 2007b), "Surface settlement due to the compression of soft zones," K-CLC-G-00087, Rev. 1.

Washington Savannah River Company (WSRC 2007c), "Computation of surface settlement due to the compression of soft zones," K-CLC-G-00088, Rev. 0.

Washington Savannah River Company (WSRC 2007d), "Soft zone settlement analysis for building 235-F," K-CLC-F-00061, attachment C, Rev. 1.