8.0 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations presented in this section are based on the data obtained from the Phase III exploration and testing used in conjunction with pertinent data from previous explorations to perform engineering analysis for the configuration of the SNS facilities current at the time of our work. Future changes to location, configuration, or loading conditions should be evaluated to determine if modification of these recommendations is necessary.

8.1 CONCLUSIONS

1. Laboratory test results relative to compaction, hydraulic conductivity, shear strength, compressibility, and dynamic properties are typical of soils with similar index properties (grain size and Atterberg limits).

2. The field measurements such as standard penetration test blow counts, compression and shear wave velocities and pressuremeter moduli are typical of residual silty clays with varying quantities of chert particles.

3. The onsite soils should provide adequate earthfill when compacted to a minimum of 95 percent standard Proctor. When the earthfill is used to support load bearing foundations, the moisture content should be within 2 percent of optimum moisture content. General earthfill may be placed within 3 percent of optimum moisture content.

4. All buildings and structures, with the exception of the Target Building and its associated external experiment tunnels and buildings, can be supported by the residual soils or fill using spread, strip or mat foundations. Allowable bearing capacities should be restricted to 3 KSF or less for static loading conditions to avoid potentially excessive total or differential settlements. Dynamic allowable bearing capacities should be limited to 4 KSF or less.

5. For all tunnels, the dump buildings and the Utilities Building, the foundation contact pressure of the completed structure and any overlying shielding fill will be below the present (pre-construction) overburden pressure at the foundation level. As a result, all the time-dependent settlement for these structures will be recompression which should occur prior to operational testing. The predicted settlement profile for the tunnels is shown on Figures 7.3-2 and 7.3-4. The maximum settlement occurs at the locations of deepest overburden. The values shown on Figures 7.3-2 and 7.3-4 represent the average soil properties. The accuracy of the settlement prediction is estimated to be ± ¼-inch. As such, differential settlement over short distances could be ¼ to ½-inch. We understand that this differential settlement should be within the adjustment range of the beam control facilities. Secondary compression will not occur beneath these structures.

6. Due to very large potential total and differential settlements and the time-required for the primary and secondary settlement to occur, the Target Building should be supported by deep foundations bearing on sound bedrock. Micropiles or drilled shafts are the two best alternative deep foundation support systems.

7. Differential settlement at the interface between the soil-supported RTBT tunnel and the Target Building which will be supported on deep foundations, is recognized as a concern. As a result of obtaining additional boring and laboratory test data, subsequent analyses indicate the differential settlement at the interface is in the order of 1¼ inch ±. Our time-rate of settlement studies suggest that approximately 85 percent of this settlement should occur by the time the Target Building is operational. Theoretically, less than ¼ inch of settlement should remain for the RTBT tunnel. It is our understanding that this amount of settlement should be within the adjustment range of the beam control facilities.

8. Construction excavations in the residual soils are stable at 2H:1V slopes even when exposed for a long period of time.

9. Permanent earthfill slopes composed of soils native to the site are stable at 3H:1V slopes when compacted to at least 95 percent standard Proctor density and within 2 percent of optimum moisture content for the following loading conditions - End of Construction, Long-Term Static Stability, Seismic up to 0.22 g, and Seismic up to 0.22 g with surcharge loading.

10. Apparent voids were encountered in some borings. We have found no indication that these "voids" have any significant lateral extension, but rather they appear to be weathered high-angle joints or joint intersections.

11. Borehole data do not indicate extensive development of bedrock voids (e.g., caves) beneath the SNS site; therefore, in our opinion, the risk of Karst-related collapse on this site is very low.

12. The topography effects of 260 to 300 feet of vertical relief and the approximate 1 on 3 to 1 on 5 slopes of Chestnut Ridge have minimal, if any, real significance on earthquake ground motions and do not warrant serious consideration. Therefore, design basis earthquake spectra commonly used at other Oak Ridge Reservation facilities are appropriate for the SNS site.

Deep Foundation Considerations		
Consideration	**Discussion**	
	Micropile	**Drilled Shaft**
Penetration of overburden containing obstructions	Micropiles are regularly used in random fill areas containing construction debris. Air rotary penetrated this area easily during the exploration.	Augers can penetrate some cobbles, boulders, etc. A rock auger may be required for larger size boulders.
Penetration of weathered rock	The majority of techniques used to install micropiles will penetrate with no change in drill techniques. Drilling causes less disturbance and removal of in-situ materials than drilled pier installation.	Requires change to rock auger or downhole drilling techniques. Multiple Telescoping may be required.
Contractor prequalification	Contractor must be prequalified.	Contractor must be prequalified.
Vertical load-carrying capacity	Requires multiple units because of lower individual capacity.	Capable of high load capacity on individual unit.
Lateral load carrying capacity	Requires multiple units because of smaller area. Annular space must be grouted.	Can be designed to carry significant lateral loads. Annulus must be grouted or a neat interface with soil.
Local history of use	Use of Micropiles for structural support in East Tennessee has not been common in the past. The technique has recently gained acceptance and become better understood. Several large projects have recently incorporated the system and there are local contractors which perform the work.	Use of drilled shafts is common practice and well understood in East Tennessee.
Drilling time/schedule impacts	Do not anticipate unforeseen slow-downs. Most drilling techniques will be similar to the air rotary exploratory holes which drilled easily. Typically rigs are smaller, more mobile, and generate less waste than drilled shaft rigs.	Anticipate some slowdown because of chert and weathered rock. Telescoping casing will be required.
Verification of founding material	Not as critical for non-end bearing units. Can verify assumptions by extending some piles deeper during installation. Requires on-site inspection	Can usually inspect shaft bottom and probe rock beneath bottom elevation. Requires on-site inspection.
Additional exploration required	Field observation during installation.	Common practice to drill each shaft location.
Load testing required	Recommended for verification and/or adjustment of design assumptions.	Recommend for structure of this importance.
Redundancy-number of units	There will be many micro-pile units as a result of their smaller individual load-carrying capacity. This results in less impact on the performance of the structure if a problem should develop in one of the units.	Since there would be a smaller number of highly loaded drilled shafts, a problem in an individual unit would impact a large part of the structure.
Integration of foundation system into structure	There is not a special requirement to modify the slab because the multiple units spread the load.	The slab must be thickened or grade beam installed to carry the load to the drilled shafts.
Deformation	Piles will deform elastically under load. Some deformation reduced by grouting entire length of pile.	Shaft will deform elastically under load.
Grout takes	Close field control required to modify procedures and reduce overrun.	Procedures will need to be modified if large takes are observed.

For all other structures and considering the nature of the proposed project and the subsurface data obtained, it is our opinion the subsurface conditions are suitable for the use of spread, strip, or mat foundations bearing on residual or fill soils.

Provided the site is prepared in accordance with our recommendations in Section 8.2.3 and 8.2.4, we recommend an allowable bearing pressure of 3,000 psf for static loading conditions. We further recommend an allowable bearing pressure of 4,000 psf for dynamic loading conditions.

The minimum column and continuous wall foundation widths should be 24 and 18 inches, respectively, for ease of construction and to avoid a local punching failure of the foundations into the foundation soils. All exterior foundations should be founded at least 18 inches below finished exterior grade to protect against frost heave and to provide protective embedment. Interior foundations may be founded at nominal depths unless the completed foundation subgrade will be exposed to freezing weather or severe evaporation during construction.

We recommend that the structural design accommodate some potential differential movement. Individual column foundations should be entirely supported by either residual soil or by compacted fill.

Exposure to the environment may weaken soils at the foundation bearing level if foundation excavations remain open for an extended period of time. Therefore, foundation concrete should be placed as soon as possible after excavations are made. Bearing soils softened by surface water intrusion or exposure are unable to provide the required foundation support. If rainfall becomes imminent while bearing soils are exposed, we recommend plastic sheeting or a 2- to 4-inch-thick "mud-mat" of "lean" concrete be placed on the bearing soils for protection.

We recommend that one of our geotechnical engineers observe the foundation excavations before the placement of reinforcing steel or concrete. Based on these observations, selected foundation excavations may be tested with a dynamic cone penetrometer to aid in evaluating the suitability of the bearing soils for the design bearing pressure. The foundation bearing area should be level or suitably benched. It should also be free of loose soil, ponded water, and debris. If soils unsuitable for the design bearing pressure are encountered in the foundation excavations, the condition, along with appropriate recommendations, will be brought to the attention of the owner's representative.

8.2.3 Site Preparation and Excavation Recommendations

All topsoil, vegetation, debris, and surface soil containing organic material should be stripped from the construction area and either wasted from the site or, if suitable, used as topsoil or fill in areas to be landscaped. Topsoil was encountered in the borings to an average depth of 1.5 feet. The thickness of topsoil may be greater or less in unexplored areas.

We recommend that the subgrade in cut areas upon which buildings or pavements are to be constructed be scarified to a depth of at least 9 inches and compacted as discussed in our subsequent "Compacted Fill Recommendations." The intent of this procedure is to lessen the number of potential pathways for the downward migration of surface water and reduce the potential for sinkhole development resulting from such water migration.

After stripping and before placing fill, we recommend the exposed subgrade in the building and pavement areas be proofrolled to detect unsuitable soil conditions. Proofrolling should be done after a suitable period of dry weather to avoid degrading an otherwise acceptable subgrade. Proofrolling should be performed with a heavily-loaded dump truck or with similar approved construction equipment. The proofrolling equipment should make at least four passes over each section, with the last two passes perpendicular to the first two.

We recommend the exposed subgrade and proofrolling operation be observed and documented by our personnel. If unsuitable conditions are encountered at the subgrade level, our geotechnical engineer will make appropriate recommendations to the owner's representative for dealing with the conditions. Soft, organic, highly plastic, wet soils, or soils that pump, rut, or wave, during site grading or proofrolling operations should be excavated and replaced with compacted fill or evaluated for stabilization alternatives.

Based on our understanding of the project requirements and the subsurface conditions encountered, we expect subgrade stabilization may be required occasionally. Such stabilization, if required, can typically be achieved using one of the following stabilization alternatives:

- Undercutting poor subgrade soils to expose competent soils, and then backfilling with compacted soil fill to planned subgrade levels.

- Undercutting poor subgrade soils to a depth sufficient to allow the placement of a "bridging layer" of soil or stone backfill upon which an interval of compacted soil fill can be constructed for subgrade support.

- Undercutting poor subgrade soils a minimum of 2 to 3 feet below the subgrade elevation and then placing a high-modulus geotextile and/or a layer of aggregate for stabilization.

We recommend subgrade stabilization requirements be determined at the time of site preparation, based on the stability of the subgrades as determined by proofrolling. The construction budget should include contingency moneys for this purpose.

During the undercutting and rough grading, positive surface drainage should be maintained to prevent the accumulation of water. If the exposed subgrade becomes excessively wet or frozen, or if conditions different from those described previously in this report are encountered, our geotechnical engineer should be contacted.

Construction excavations that will be exposed for a long period of time should be excavated no steeper than 2H:1V. Steeper slopes may be acceptable in localized areas that are not very deep and that will be open for only a short period of time. However, all excavated slopes must comply with OSHA requirements.

8.2.4 Compacted Fill Recommendations

We recommend all compacted fill be constructed by spreading acceptable soil in loose layers not more than 8 inches thick. The soils used within the proposed construction areas should be compacted in lifts to at least 95 percent of the standard Proctor maximum dry density (ASTM D 698). The upper 24 inches of fill beneath pavements and upper 12 inches beneath grade slabs should be compacted to at least 100 percent of standard Proctor maximum dry density.

The moisture content of the soils compacted to 95 percent standard Proctor that support load bearing foundations should be maintained within +2 to -2 percentage points of the optimum moisture content as determined from the standard Proctor compaction test. General earthfill compacted to 95 percent standard Proctor may be placed within 3 percent of optimum moisture content. This provision may require the contractor to dry soils during periods of wet weather or to wet soils during the hot summer months. The fill soils should have a maximum dry density of no less than 90 pounds per cubic foot (pcf).

Samples of potential borrow material were collected from numerous borings throughout the site and tested to determine the maximum dry density, optimum moisture content, natural moisture content, and PI. These tests are used to determine if the soil is usable for fill and for quality control during compaction. The results of these laboratory tests along with a description of each test are provided in Appendix E of Volume III. A family of compaction curves for use in the field compaction control of earthfill are shown in Figures 8.2.4-1 and 8.2.4-2.

Our laboratory test data indicate potential on-site borrow soils are typically wetter than optimum moisture content. The contractor should anticipate occasional drying of soils.

The fill surface must be adequately maintained during construction in order to achieve an acceptable compacted fill. We recommend the fill surface be sloped to achieve sufficient drainage and to prevent ponding of water on the fill. If precipitation is expected while fill construction is temporarily halted, the surface should be rolled with rubber-tired or steel-drummed equipment to improve surface run-off. If the surface soils become excessively wet or frozen, fill operations should be halted and we should be consulted for guidance.

Before sloping, the edge of the compacted fill should extend at least 10 feet horizontally beyond the outside edge of any building foundation, beyond areas of proposed future building expansion, and beyond paved areas. Fill slopes should be grassed to protect from erosion. Based on our slope stability analysis, we recommend compacted fill slopes be constructed at 3H:1V.

In areas where "thin slivers" of fill are to be placed upon steep slopes of residual soils, we recommend that a series of localized "benching" excavations be made into the natural slopes prior to placing the overlying compacted earthfill.

Before filling operations begin, representative samples of the proposed fill material should be collected and tested to determine the maximum dry density, optimum moisture content and natural moisture content. These tests are needed to determine if the fill material is acceptable and for quality control during compaction.

We recommend the fill placement and compaction be observed and documented by our engineering technician. To verify compaction level obtained, we recommend frequent field density tests of fill soils as they are placed. Significant deviations, either from the project specifications or from good practice, will be brought to the attention of the owner's representative along with appropriate recommendations.

8.2.5 Soil Plasticity Considerations

Shrinking and swelling problems are generally not as severe in the East Tennessee area as in other areas, since extended periods of excessively wet or excessively dry weather do not normally occur. Therefore, changes in the moisture content of foundation soils are usually minimal. However, it is not uncommon for significant drying of soils to occur if grading is performed during dry weather. If these soils resaturate after completion of construction of lightly loaded foundations, there is the potential for structural distress. Therefore, the volume change potential of the soils at this site should be considered, and the following construction precautions are recommended:

- Foundation construction should be completed as rapidly as possible to prevent deterioration of the load carrying capacity of foundation soils by exposure to the elements. If possible, it is most desirable to complete concreting of individual foundation excavations the same day they are made.

- Subgrades of floor slabs should be protected from excessive drying or wetting by covering the subgrade prior to floor slab construction. This can be done by leaving the floor subgrade several inches high and then making the final excavation to subgrade shortly before floor construction or by use of a working mat of aggregate material.

- Soils native to the SNS site are suitable for backfill beneath floor slabs and other structural elements. This is because soils with a high susceptibility to volume change are uncommon (5 percent ±) and typically do not exist in large zones. Therefore, we anticipate they will be intermixed with lower plasticity soils during the excavation and placement process. Soils with suspected high shrink or swell potential should be discarded or tested in the laboratory for acceptability. Soils with a plasticity index (PI) greater than 50 should be well blended with lower plasticity soils and not placed in large, continuous areas beneath slabs or foundation elements.

- The site should be graded to promote rapid drainage of surface water during construction.

In addition to these construction precautions, we recommend that the following considerations be incorporated into design:

The structural design should accommodate the potential for some differential movement.

Roof drains should discharge well away from the building area to prevent ponding of water near foundations.

Heat sources should be isolated from foundation soils to minimize drying and the resultant shrinkage of foundation soils.

8.2.6 Compacted Aggregate Recommendation

Two off-site commercially available aggregate materials have been proposed for use as backfill behind walls or as working mats beneath structural foundations. These aggregates are:

- ASTM D 448, Size No. 57 or 67, and
- Tennessee Department of Transportation (TDOT) Class A, Grade D, Base Stone.

Screen Size	% Passing
1-1/2 inches	100
1 inch	85-100
3/4 inches	60-95
3/8 inches	50-80
No. 4	40-65
No. 16	20-40
No. 100	9-18

When used as backfill behind walls, the TDOT Base Stone does not require an overlying geotextile fabric to prevent the inward migration of compacted earthfill or in-situ residual soils.

When required, compaction of large fills should be performed with smooth drum vibratory rollers. In areas where heavy construction loads on an adjacent wall or structure are a concern, small vibrating rollers or power tampers should be used for compaction. The construction specification must contain a provision that restricts the moist unit weights of compacted aggregate behind walls to a maximum of 125 pounds per cubic foot. In all cases, testfills or testpads should be constructed to select appropriate compaction equipment, number of equipment passes, and lift thickness best suited to meet the compaction requirement.

8.2.7 Slab Recommendations

Densified aggregate, such as ASTM D 448, Number 57 or Number 67 stone, should be placed beneath slabs to provide a suitable working base and reduce degradation of the prepared subgrade during construction. We recommend the aggregate layer be at least 4 inches thick. The slab should be jointed around columns and along footing-supported walls so the slab and foundations can settle differentially without damage.

Construction activities and exposure to the environment can cause deterioration of prepared subgrades. Therefore, we recommend proofrolling be performed on the final subgrade soils shortly before slab construction to detect soft areas, filled-in ruts, or disturbed zones that may have been created by normal construction activities.

8.2.8 Pavement Recommendations

Pavement design requires a knowledge of the proposed soil subgrade strength and anticipated traffic conditions. Soil strength is typically expressed in terms of a California Bearing Ratio (CBR) for flexible pavement design, and a modulus of subgrade reaction (K) for rigid pavement design.

If you elect to follow the recommendations provided in this report, a design CBR value of 4 for flexible pavements should be available for subgrade support. Based on published correlations between CBR and modulus of subgrade reaction, a K value of about 100 psi/inch should be available for rigid pavement support.

Our experience has shown that most pavement failures are the result of poor soil subgrade preparation and improper soil subgrade drainage. We emphasize that the upper 24 inches of the subgrade in fill areas must be compacted to at least 100 percent of the soil's standard Proctor maximum dry density. To achieve a higher CBR value, it is critical that the soil be compacted at or below the soil's optimum moisture content based on the standard Proctor compaction data. In addition, just before placement of the base course, the subgrade should be proofrolled to detect soft areas, filled-in ruts, or poorly compacted material that may have been created during construction.

Good surface drainage must also be incorporated into pavement design. If the prepared base course is left in place for an extended period after construction or is rained on prior to asphaltic concrete placement, we recommend additional proofrolling to detect potentially weakened areas.

8.2.9 Retaining and Subsurface Wall Recommendations

We understand the proposed construction will include retaining walls, basement walls and walls of tunnels. The walls should be designed for active, at-rest or passive earth pressures, as appropriate, based upon the amount of movement specified in Table 14B. Table 14B also provides additional criteria for the design of walls.

We recommend TN DOT Class A, Grade D, Base Stone be used as backfill to lessen lateral earth pressures exerted on retaining walls (see maximum permissible unit weight for compacted aggregate backfill in Section 8.2.6). The granular backfill should not be placed on a slope steeper than 62 degrees and no further than 4 feet from the intersection of the wall and footing.

Wall design should include a drainage interval and perforated piping behind the wall to intercept ground-water seepage and thereby reduce hydrostatic pressures. The pipe should be designed to prevent clogging by backfill particles and sloped to drain water from behind the wall. For maintenance purposes, cleaning ports for the piping system should be considered.

Surface-water seepage into the backfill will increase lateral pressures on the wall. To reduce the possibility of excessive surface-water seepage, we recommend capping granular backfill with a 3-foot-thick layer of compacted clay soil, sloping away from the structure.

The use of on-site clay soils as compacted backfill can cause high lateral pressure on the walls. Heavy compactors and grading equipment should not be allowed to operate within 10 feet of the walls during the backfilling to lessen temporary and long-term lateral soil pressures. Backfill adjacent to the walls should be densified by light compaction equipment.

8.2.10 Difficult Excavation

Although none of the test borings at this site encountered refusal near or above plan grades, it has been our experience that rock pinnacles can project upward from the rock mass between the boring locations and not be discovered until actual site grading is being performed. In such cases, rock excavation will be required to achieve plan grades. An estimate of the amount cannot be provided at this time due to the irregular bedrock surface.

Excavation of rock in confined areas will likely require blasting and pneumatic splitters and/or hammers. The speed and ease of excavation will depend on the type of grading equipment, the skill of the equipment operators, and the geologic structure of the material itself, such as the dip of bedding, planes of weakness, and the spacing between discontinuities.

If blasting is required, the need for a pre-blast survey of nearby existing structures should be evaluated. Before performing any blasting, we recommend the contractor consult the Tennessee Blasting Regulations and the appropriate site contacts for guidance.

8.2.11 Solutioning and Sinkholes

The proposed structure locations at the SNS site exhibit a low to moderate potential for sinkhole development. The likelihood of small (less than 10-foot diameter) construction-induced dropouts is thought to be greater in deep cut areas. The potential for occurrence of such dropouts can be reduced by maintaining positive surface drainage and not allowing water to pond. Should they occur, routine repair methods as described in Section 5.1.5 are available. A discussion of sinkhole potential and associated risk levels is provided in Sections 5.1.4 and 5.1.5.