REPORT OF CONCEPTUAL PHASE GEOTECHNICAL EXPLORATION

US 701 BRIDGE REPLACEMENTS OVER GREAT PEE DEE RIVER, PEE DEE OVERFLOW AND YAUHANNAH LAKE
SCDOT PROJECT NO. BR-BR88(044)
SCDOT FILE NO. 22.124B
HORRY-GEORGETOWN COUNTIES, SOUTH CAROLINA

Prepared for:

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Prepared by:

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S&ME PROJECT NO. 1611-04-569

MAY 17, 2005

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May 17, 2005

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Attention: Tuhin Basu, PE

Reference: **Report of Conceptual Phase Geotechnical Exploration**
US 701 Bridge Replacements over Great Pee Dee River,
Pee Dee Overflow, and Yauhannah Lake
SCDOT Project No. BR-BR88(044), Pin No. 30688
SCDOT File No. 22.124B
Horry-Georgetown Counties, South Carolina
S&ME Project No. 1611-04-569

Dear Mr. Basu:

We have enclosed our conceptual phase geotechnical report of the proposed project. The exploration was conducted in general accordance with S&ME Proposal No. 1611-3570-04rev3 dated September 15, 2004. The recommendations of the report incorporate all engineering analyses conducted by our firm to the present time.

The purpose of the exploration was to characterize and provide information about the on-site subsurface soils based upon the borings and soundings conducted. Information obtained was then used to provide preliminary recommendations for the proposed construction including their potential utility for foundation support, their relative suitability for use as structural fill, and their lateral earth pressures. The enclosed report includes (1) a description of observed site conditions (2) methods and results of field tests and sampling, (3) laboratory tests of recovered samples, and (4) our assessment of the soil properties as they relate to design issues.

We appreciate working with Tuhin Basu and Associates, Inc. on this project and look forward to continuing our association in subsequent phases of this project. Please do not hesitate to contact us if you have questions or if we may be of any further assistance.

Very Truly Yours,

S&ME, Inc.

John C. Lessley, PE
Chief Engineer

For: Aaron Goldberg, PE
Senior Engineer

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Attachments: Geotechnical Report
Figures and Analysis (APPENDIX I)
Boring Logs (APPENDIX II)
Laboratory Data Sheets (APPENDIX III)

EXECUTIVE SUMMARY

The proposed construction consists of replacement of three bridges on US 701 north of Yauhannah, South Carolina. The bridges consist of two flood plain relief structures (Pee Dee Overflow and Yauhannah Lake) lying to either side of the Great Pee Dee River channel, and the main bridge (Great Pee Dee Bridge) over the channel itself.

The Pee Dee Overflow and Yauhannah Lake bridges are 1320 and 1440 feet in length. There is some indication of settlement of one or more bents of both flood relief structures as well as some evident horizontal offsets of several of the slabs. The Great Pee Dee Bridge is a combination steel and concrete T-beam structure with a total length of 1600 feet, supported by pile bents on the land approaches and piers bearing on timber piles in and immediately adjacent to the river. Typical bent spans on the approaches range from 72 to 90 feet, with a central span in the main channel of 176 feet. The bridge was originally constructed with a steel truss on the central span, but this was replaced in the 1990's by a reinforced concrete structure bearing on two new piers set just inside the original piers at bents, reducing the center span to less than 150 feet.

To briefly summarize the findings of our exploration:

1) Boring data and shear wave velocity profiles conducted to depths up to 65 feet indicate the Yauhannah Lake Bridge to be Site Class D as defined in Section 3.3.2 of the "SCDOT - Seismic Design Specifications for Highway Bridges, October 2001 as amended by the 2002 Interim Revisions". The remaining two bridges are Site Class F due to the presence of deep deposits of liquefiable soils.

2) The bridges are "normal" bridges and Importance Classification III structures as defined by the "SCDOT - Seismic Design Specifications for Highway Bridges, October 2001 as amended by the 2002 Interim Revisions". SEE spectral response accelerations for S_S and S_1 were provided by SCDOT for Geologically Realistic Site Conditions. For the provided spectral response values, the SEE S_{DS} and S_{D1} values were calculated to be 0.64g and 0.42g, respectively. The Site Class D peak ground acceleration taken as $S_{DS}/2.5$ is 0.26g. The Yauhannah Lake Bridge will be designed as a Seismic Performance Category (SPC) B structure based on these criteria.

3) Due to presence of liquefaction in the soil profile, the main PeeDee River Bridge and the PeeDee Overflow Bridge are considered to be Site Class F. S&ME performed a site specific evaluation of the seismic response at these locations using the time history provided by the SCDOT geotechnical group for geologically realistic site conditions. The SEE S_{DS} and S_{D1} values were calculated to be 0.79g and 0.29g, respectively. The peak ground acceleration taken as $S_{DS}/2.5$ is 0.32g.

4) Since liquefaction appears to be limited to a few isolated seams and pockets, it is our opinion that liquefaction-related settlements and lateral movement of the slopes will not be a major factor in design for the Yauhannah Lake Bridge. However, the deep clayey sediments underlying the embankment at the north end of the Yauhannah Lake Bridge are only marginally stable for static conditions, particularly under partial flood, and marginally stable to unstable under the seismic condition. Further definition of the shear strength profile of the

soft clays would be necessary to determine whether ground stabilization would be necessary in this area.

5) A qualitative assessment of the boring data and recovered samples by our staff revealed potential for liquefaction within the recent Quaternary alluvial sand deposits at shallow depths within the bridge alignments at the Pee Dee River and the Pee Dee Overflow. In these areas stabilization of the foundation soils at the abutments, or provision of supplemental berms or bolsters at the base of the fill slopes, may be required to provide the required global stability and to limit settlements due to volumetric compaction.

6) Alluvial deposits are underlain by stiff to very hard Cretaceous silts and clays of the Pee Dee Formation at a typical depth of about 35 to 65 feet below the flood plain surface. In some areas the Pee Dee Formation is overlain by about 5 feet of very dense lithified silty sands with SPT penetration values exceeding 100 bpf. In other areas the PeeDee Formation is overlain by 15 to 20 feet of dense or very dense river laid sands, also exhibiting SPT values of 50 to 100 bpf. The dense river laid sands, the "caprock" and the underlying very hard clays will likely resist more than a few feet of penetration by high displacement concrete piles.

7) Driven piles consisting of H-sections, precast prestressed piles, and open-ended pipe piles were considered for axial support of bridge elements. All piles appear to generate resistance to axial load through end bearing on either the PeeDee caprock, hard indurated seams of limestone within the PeeDee Formation, or in the overlying very dense river laid sands.

8) Due to the presence of liquefiable sands or soft clays at depths of up to 40 feet over much of the flood plain, 18-inch and 24-inch square PSC piles will be essentially unsupported over lengths approaching 50 feet. They will offer relatively low lateral resistance to applied displacements of the bridge superstructure if constructed as pile bents with the piles extending above the ground surface into a cap at the base of the superstructure.

9) We considered use of driven piles consisting of either 18-inch or 24-inch square, prestressed concrete piles or 20-inch diameter open ended pipe piles extended to rigid caps at the ground surface and modeled as fixed-headed. In this case the piles provided significantly higher resistance to lateral deflection, but to achieve effective fixity of the pile tips will require that the 24-inch PSC piles and the 20-inch OE pipe piles penetrate several feet into very dense soils at the top of the Pee Dee Formation.

10) Drilled shafts are likely preferable for support of interior or intermediate piers. Shafts on land will typically be installed using the wet method described in Standard Specification 712, using temporary steel casing to stabilize loose sands of soft clays down to the top of the PeeDee Formation. It may be necessary to have a bid quantity for a short section of permanent casing near the surface to prevent contamination of the upper few feet of the shafts since water levels are so close to the ground surface. Shafts in the river channel will bear in water up to 25 feet deep (from normal pool). These shafts will utilize permanent construction casing through water.

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1.0 INTRODUCTION

The proposed construction consists of replacement of three bridges on US 701 north of Yauhannah, South Carolina. The bridges consist of two flood plain relief structures (Pee Dee Overflow and Yauhannah Lake) lying to either side of the Great Pee Dee River channel, and the main bridge (Great Pee Dee Bridge) over the channel itself. S&ME, Inc. is to provide conceptual design information about subsurface conditions at the three bridge locations, which would include preliminary site preparation and foundation design recommendations for the proposed bridge replacements. Our services were performed in general conformance with recommended SCDOT guidelines for structures.

The geotechnical exploration will be conducted in two phases. This initial reconnaissance phase forms part of conceptual design of the project to develop basic stratigraphy of the flood plain. A design phase investigation will later be conducted to support preliminary and final design.

The purpose of the conceptual phase exploration was to characterize and provide basic stratigraphic information about the on-site subsurface soils based upon the borings conducted. Information obtained was then used to provide general recommendations for conceptual design of the proposed construction, including site preparation and bridge foundation recommendations. Our objectives may be summarized as follows:

1. Site description with photographic documentation.
2. Drilling and testing procedures, along with charts illustrating soil classification terminology and criteria.
3. Microstation compatible boring location plan.
4. Soil test boring and cone penetration sounding logs in gINT (SCDOT template) and in Microstation-compatible format, indicating sampling depths and soil descriptions.
5. Shear wave velocity profiles(s) and Site Class by bridge.
6. Preliminary discussion of foundation support issues, including recommendations for feasible pile or drilled shaft configurations and installation methods, discussion of lateral support under static and earthquake conditions.
7. Preliminary liquefaction evaluation, identification of potentially liquefiable zones, and discussion of likely remedial strategies.
8. Preliminary discussion of embankment stability, including effects of liquefaction on stability.

9. Discussion of site preparation issues, such as undercutting, surface preparation, possible application of geotextiles, or whether staging of embankment construction would be required.

The enclosed report includes (1) a description of observed site conditions (2) methods and results of field tests and sampling, (3) laboratory tests of recovered samples, and (4) our assessment of the soil properties as they relate to specific design issues.

2.0 PROJECT INFORMATION

Tuhin Basu and Associates is responsible for design of roadway and bridge replacements along US 701, northeast of Georgetown between Horry and Georgetown counties. Approximately 10,500 feet of roadway will be widened or improved and three bridges will be replaced. Bridges to be replaced are two flood plain relief structures (Pee Dee Overflow and Yauhannah Lake) lying to either side of the Great Pee Dee River channel, and the main bridge (Great Pee Dee Bridge) over the channel itself. All bridges are currently two lanes with no or very narrow shoulders and were built in 1952-53, replacing earlier bridges situated just southeast of the original bridges built in the 1920's.

The Pee Dee Overflow and Yauhannah Lake bridges are concrete T-beam construction supported by concrete piles with typical bent spans of about 30 feet. Bridge lengths are 1320 and 1440 feet. There is some indication of settlement of one or more bents of both flood relief structures as well as some evident horizontal offsets of several of the slabs.

The Great Pee Dee Bridge is a combination steel and concrete T-beam structure with a total length of 1600 feet, supported by pile bents on the land approaches and piers bearing on timber piles in and immediately adjacent to the river. Typical bent spans on the approaches range from 72 to 90 feet, with a central span in the main channel of 176 feet. The bridge was originally constructed with a steel truss on the central span, but this was replaced in the 1990's by a reinforced concrete structure bearing on two new piers set just inside the original piers at bents, reducing the center span to less than 150 feet.

The roadway embankment within the flood plain is typically about 30 feet wide at the top and 20 to 25 feet high, with side slopes ranging from 1.5H:1V to 2.0H:1V. For the most part the ground surface near the toe of the embankment intermediate of the bridges is wet, undeveloped and in a natural state. Hardwood swamps extend up to the toes of one or both of the abutments of the Pee Dee Overflow and Yauhannah Lake bridges, but there is good road access to both abutments of Great Pee Dee Bridge, as well as most of the approach spans on either side of the Pee Dee River. Side slopes are typically heavily vegetated, preventing close observation. However, several areas of the embankment appear to have undergone some sliding or sloughing, particularly on the south side between the Pee Dee River and the Yauhannah Lake Bridge. It also appears that some erosion or movement has occurred near the west abutment of the westernmost bridge. All of the abutment slopes have been faced with rip rap.

The replacement scheme is not yet known. Replacement structures may either be offset slightly from their current locations to allow phased construction, or located on an entirely new alignment. For the purpose of the conceptual phase exploration, we have assumed that the roadway would be on a new embankment closely paralleling the existing embankment within the current right of way. Although design has not begun for the new bridges, we understand interior bents for the main bridge may possibly be supported with drilled shafts while end bents may likely be supported with driven piles. The new bridges may use one of several construction techniques, but we understand that span lengths under the new arrangement would likely be increased to 60 or 70 feet from the current 30 feet.

From a review of local geologic mapping, we anticipate that the alignment area is underlain by fluvial sediments of the Wando Formation, deposited approximately 90,000 years ago and little altered since that time. The ground surface retains a distinct ridge and swale pattern with point bar ridges (largely sands), natural levees (sands or clayey sands), and filled in channels or swales (mostly silts and clays). There may also be some areas of peat or swamp muck of recent origin, particularly near bridge abutments. Seismic hazards such as liquefaction are potentially present over much of the alignment, as is typical for coastal South Carolina. Foundation support will likely be on consolidated sediments of the Pee Dee Formation, a calcareous clay and limestone sequence typically occurring at depths of 30 to 100 feet below the surface in Horry County.

The bridges are "normal" bridges as specified by SCDOT and classify as Importance Classification III structures as defined by the "SCDOT - Seismic Design Specifications for Highway Bridges, October 2001 with October 2002 Interim Revisions." SEE and FEE spectral accelerations for S_S and S_1 were provided by SCDOT for geologically realistic conditions.

3.0 SUMMARY OF EXPLORATION PROCEDURES

The American Society for Testing and Materials (ASTM) publishes standard methods to explore soil, rock and ground water conditions in Practice D-420-98, "*Standard Guide to Site Characterization for Engineering Design and Construction Purposes.*" These methods are then modified as necessary by the geotechnical engineer to consider the specific geologic or topographic setting, the proposed construction, and the objectives of the client.

- Reconnaissance of the Project Area
- Preparation of Exploration Plan
- Layout and Access to Field Sampling Locations
- Field Sampling and Testing of Earth Materials
- Laboratory Evaluation of Recovered Field Samples
- Evaluation of Subsurface Conditions

The standard methods do not apply to all conditions or to every site. Nor do they replace education and experience, which together make up engineering judgment. Finally, ASTM D 420 does not apply to environmental investigations.

In exploring the site, we generally followed the approach described in S&ME, Inc. Proposal No. 1611-3570-04rev3, with certain exceptions. Right-of-entry to perform borings and other fieldwork on the property was conveyed with acceptance of our proposal.

The exploration plan or drilling assignment sheet consisted of a set of written directions to the drillers or to other field exploration staff. The plan tabulated the minimum depth of borings, method of drilling and stabilizing the boring, sampling methods and depths, procedures for backfilling, and procedures to be followed if certain subsurface conditions were encountered. The location, number and depth of the borings, the method of drilling, and the method and depths of sampling were discussed prior to commencement of the exploration and were outlined in our initial proposal. This scope of work formed the basis of the preliminary exploration plan.

3.1 Configuration and Layout of Borings

Where practical, we reviewed available topographic maps, county soil surveys, reports of nearby investigations and aerial photographs when preparing the boring and sampling plan. Then we walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures. A total of 5 soil test borings, and 3 electronic cone penetrometer (CPT) soundings, were laid out on the site. The three CPT soundings were subsequently re-drilled to depths of 100 feet as soil test borings after CPT tools refused at depths of 30 to 64 feet.

Soil test borings were performed at or near the beginning and ending bridge abutments of each bridge at locations approved by Tuhin Basu and Associates, Inc. and the SCDOT. Borings and soundings were extended to depth of refusal of our drilling or sounding equipment or to at least ten feet below the anticipated depth of driven pile foundations. Soil test borings were also conducted at the crest of the embankment overlooking the PeeDee River channel approximately 350 feet from each abutment. In addition, soil test borings performed by Law Engineering in 1996 in the PeeDee River channel were provided by the SCDOT for inclusion in this phase of exploration.

The Boring Location Plan (included as Figures 2 through 4) and the Subsurface Profiles (included as Figures 5 through 8) indicate approximate locations of borings and soundings and the cross-sectional profiles. Borings and soundings were performed at locations indicated on sketches provided by Tuhin Basu and Associates, Inc. unless offsets were required due to unavoidable circumstances such as slopes, ditches, overhead power or other obstructions.

3.1.1 Checks for Hazardous Conditions

State law requires that we notify the Palmetto Utility Protection Service (PUPS) before we drill or excavate at the site. PUPS is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. PUPS forwarded our location request to the participating utilities. Location crews then marked buried lines with colored flags or paint within 72 hours. They did not mark utility lines beyond junction boxes or meters. We checked proposed sampling points for conflicts with marked utilities, overhead power lines, tree limbs, or man-made structures during the site walkover.

3.1.2 Staking of Borings

S&ME was provided a scaled half-size plan and profile drawing by Tuhin Basu and Associates, Inc. prior to commencement of field work. The site plan indicated the general orientation of the proposed structures in relation to existing features. There is no system of stationing yet available for the project.

S&ME laid out the borings and soundings by measuring distances from existing site features with a measuring wheel and by turning rough right angles from existing features marked on the plan and profile sheets. Interior bent borings were generally located using existing bents on the bridges for reference points. Boring and sounding locations were marked in the field with small colored flags with the boring or sounding numbers and depths inscribed, or marked on the bridge deck or road surface with spray paint. Boring and sounding locations indicated on the attached "Boring Location Plan" and tabulated below must be considered as approximate.

Necessary offsets from staked locations are indicated on the Soil Test Boring Records. Boring locations depicted on the boring location plans are accurate only to the degree of accuracy used in boring layout.

3.1.3 Boring Access

Most of the borings for this exploration were conducted from the ground surface of the flood plain using all-terrain mounted drill rigs. Boring locations were selected so that no heavy clearing equipment was necessary to access boring locations. Interior bent borings for the Pee Dee River Bridge were performed in the flood plain despite wet conditions in and around the boring locations at the south end of the bridge.

3.1.4 Boring Offsets

Borings were performed at the staked locations unless offsets were required due to unavoidable circumstances such as slopes, ditches, overhead power or other obstructions. Where offsets from staked locations were required, the distance of the offset north and east of the staked location was indicated on the field boring records. Offset distances are shown on the final Soil Test Boring Records and on the attached "Boring Location Plan" in the Appendix.

3.1.5 Boring Elevations

Top-of-ground elevations at borings were interpolated from the plan and centerline profile sheets provided to S&ME prior to drilling. Interpolation to actual boring locations offset from the centerline was made using the care and judgment ordinarily exercised in similar work. Boring elevations must be considered accurate only to the degree that the topographic elevations portrayed on the plan accurately reflect site topography. Boring elevations must also be considered accurate only to the degree of accuracy of the boring layout.

3.2 Boring and Sampling Procedures

On December 13, 2004, S&ME, Inc. mobilized to the site to begin the boring and sampling portion of the field exploration. On December 17, 2004, our contract CPT operator mobilized to the site to begin the CPT portion of the field exploration. S&ME re-mobilized to the site on December 27, 2004, to extend boring and sampling at the CPT sounding locations, where refusal had occurred at shallow depths. During drilling and sampling procedures S&ME maintained a minimum of one drill rig on site. The field exploration was completed on December 31, 2004. Presented in Table 1 is a summary of the borings.

3.2.1 Soil Test Boring with Mud Rotary Wash

Soil sampling and penetration testing were performed in general accordance with ASTM D1586, *"Standard Test Method for Penetration Test and Split Barrel Sampling of Soils.* A rotary drilling process was used to advance the hole and a heavy drilling fluid was circulated in the bore holes to stabilize the sides and flush the cuttings. At regular intervals, drilling tools were removed and soil samples were obtained with a standard 1.4 inch I. D., two-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Table 1 - Boring Depths and Remarks

Boring No.	Station and Offset (ft)	Drill Casing Total Footage (From Top of Bridge Deck) (ft)	Soil Test Boring Total Footage (From Ground Surface) (ft)	Remarks
B-1		--	120	Embankment N. of Yauhannah Lake
B-2		--	120	Embankment Boring S. Abutment of Pee Dee River
B-3		--	100	Bent Boring in Flood Plain, S. of Pee Dee River
B-4		--	100	Embankment Boring N. of Pee Dee River
B-5		--	110	Embankment Boring S. of Pee Dee Overflow
B-6		--	120	Extension of CPT-1
B-7		--	120	Extension of CPT-3
B-8		--	120	Extension of CPT-2
C-1		--	(CPT) 35	Embankment S. of Yauhannah Lake
C-2		--	(CPT) 60	Embankment Boring N. of Pee Dee River
C-3		--	(CPT) 65	Embankment Boring N. of Pee Dee Overflow

3.2.2 Electronic Cone Penetrometer (CPT) Soundings

CPT soundings consist of a conical pointed penetrometer, which is hydraulically pushed into the soil at a slow, measured rate. Procedures for measurement of the tip resistance and side friction resistance to push generally follow those described by ASTM D-5778, "*Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils.*"

A penetrometer with a conical tip having a 60 degree apex angle and a cone base area of 10 cm^2 was advanced into the soil at a constant rate of 20 mm/s. The force on the conical point required to penetrate the soil was measured electronically every 50 mm penetration to obtain the *cone resistance* q_c. A friction sleeve is present on the penetrometer immediately behind the cone tip. The force exerted on the sleeve was measured electronically at a minimum of every 50 mm penetration and divided by the surface area of the sleeve to obtain the *friction sleeve resistance value* f_s A pore pressure element mounted immediately behind the cone tip was used to measure the pore pressure induced during advancement of the cone into the soil.

Using this procedure soil samples are not obtained. Soil classification was made on the basis of comparison of the tip resistance, sleeve resistance and pore pressure values to values measured at other locations in known soil types, using experience with similar soils and exercising engineering judgment. Sounding plots are attached in the Appendix. Tabular output of sounding data is archived in the S&ME office.

3.2.3 Refusal to CPT Push

Refusal to the cone penetrometer equipment occurred when the reaction weight of the CPT rig was exceeded by the thrust required to push the conical tip further into the ground. At that point the rods began to buckle or the rig tended to lift off the ground. Refusal may have resulted from encountering hard cemented or indurated soils, heavily overconsolidated clays, soft weathered rock, coarse gravel, or thin rock seams. All soundings met refusal before reaching the planned depth of 100 feet.

3.2.4 Seismic Cone Penetration (SCPT) Measurements

Shear-wave velocity measurements of the near surface soil sequence were made in three soundings using a cone penetrometer instrumented with geophones. The seismic cone penetrometer test (SCPT) measures the travel times of vibrations generated by an impulsive force applied to the ground surface. The data was then analyzed to determine shear wave velocities at approximately three foot increments over the depth of each sounding.

The seismic cone penetrometer measures the travel times of surface generated vibrations to geophones mounted on the penetrometer at various incremental depths in the sounding. At a given depth, the travel time of the first arrival is measured and corrected for the horizontal offset of the source at the surface from the sounding. Interval velocities are calculated by dividing the difference in vertical distance by the travel times between successive measurement depths. Measurements are typically made at 1 meter intervals – the length of commonly available CPT extension rods – unless otherwise noted. SCPT data in the form of interval velocity vs. depth or travel time vs. depth plots are attached in the Appendix.

3.2.5 Water Level Readings

Water level readings were made in the open boreholes immediately after completing drilling and withdrawal of the tools. Where feasible, measurements were repeated after an elapsed period of 24 hours to gauge the stabilized water level. Procedures for measurement of liquid levels in open boreholes are described in ASTM D 4750, "*Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well).*" A weighted measuring tape was slowly lowered into each borehole until the liquid surface was penetrated by the weighted end. The reading on the tape was recorded at a reference point on the surface and compared to the reading at the demarcation of the wetted and unwetted portions of the tape. The difference between the two readings was recorded as the depth of the liquid surface below the reference point. Measurements made by this method were then repeated until approximately consistent values were obtained.

CPT penetration pore pressures include the *in-situ equilibrium pore pressure*, controlled by the local ground water regime, and the *excess pore pressure*, generated by insertion of the probe. In clays and silts, penetration is essentially undrained and recorded pore pressures significantly exceed in-situ equilibrium pore pressures. In sands and gravels, penetration is essentially drained and recorded pore pressures are essentially equal to the in-situ equilibrium pore pressure. The piezometric surface, defined as the point of zero equilibrium pore pressure, was obtained by plotting in-situ equilibrium pore pressure vs. depth using only pore pressure data from sand or gravel soils. Where possible, derived piezometric surface was verified by tape measurement through the sounding opening after removal of the CPT rod and before collapse of the soils.

3.2.6 Borehole Closure

Following collection of relevant geotechnical data, boreholes were filled by slowly pouring auger cuttings into the open hole such that minimal "bridging" of the material occurred in the hole. Backfilling of the upper two feet of each hole was tamped as heavily as possible with a shovel handle or other hand held equipment, and the backfill crowned to direct rainfall away on the surface. Where boreholes exceeded five feet in depth, a plastic hole plug was firmly tamped into place within the backfill at a depth of about two feet.

3.3 Preservation and Handling of Recovered Earth Materials

Procedures for preserving soil samples obtained in the field and transportation of samples to the laboratory generally followed those given in ASTM D 4220, "*Standard Practice for Preserving and Transporting Soil Samples*". Split spoon samples obtained in the borings were handled as Group B samples as defined in Section 4 of ASTM D4220. Representative samples of the cuttings or split spoon samples, or representative bulk samples, were placed in suitably identified, sealed glass jars or plastic containers and transported to the laboratory. Sample identification numbers on the containers corresponded to sample numbers recorded on field boring records.

3.4 Laboratory Examination and Archiving of Samples

The subsurface conditions encountered during drilling were reported on a field test boring record by the staff professional. The record contains information about the drilling method, samples attempted and sample recovery, indications of materials in the borings such as coarse gravel, cobbles, etc, and indications of materials encountered between sample intervals. Representative soil samples were placed in glass jars and transported to the laboratory along with the field boring records. Field boring records are retained at our office. Recovered field samples and field boring records were reviewed in the laboratory by the geotechnical engineer. Finished Soil Test Boring Records and other field data are assembled in Appendix II.

3.4.1 Examination of Split Spoon Samples

Soil and field boring records were reviewed in the laboratory by a geotechnical staff professional. Soils were classified in general accordance with the visual-manual method described in ASTM D 2488, *"Standard Practice for Description and Identification of Soils (Visual-Manual Method)"*. The geotechnical staff professional also prepared the final boring records enclosed with this report.

3.4.2 Sample Retention

Recovered samples not suspected of contamination and not expended in laboratory tests are commonly retained in our laboratory for 90 days following completion of drilling. Samples are then disposed of at our convenience. For this project samples will be held until a permanent storage location is determined.

3.4.3 Preparation of Finished Boring Logs and Cross Sections

The Soil Test Boring Records and Subsurface Profiles enclosed with this report represent our interpretation of the contents of the field records based on the results of engineering examination and tests of field samples. Soil test boring records depict conditions at the specific boring locations at the particular time when drilled. For the purpose of illustration, conditions were interpolated between the borings on the subsurface profiles using reasonable engineering judgment. The nature and extent of variations between the borings will not become evident until construction and are not warranted.

3.5 Laboratory Tests of Soil Physical Properties

After the soil samples were brought to our laboratory selected samples were subjected to index laboratory tests to help classify the site soils and formulate our conclusions and recommendations. Laboratory test data is summarized on the attached Summary of Laboratory Test Data and on the laboratory data sheets included in Appendix III.

3.5.1 Moisture Content Testing of Soil Samples by Oven Drying

Moisture content was determined in general conformance with the methods outlined in ASTM D 2216, "*Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil or Rock by Mass*." This method is limited in scope to Group B, C, or D samples of earth materials which do not contain appreciable amounts of organic material, soluble solids such as salt or reactive solids such as cement. This method is also limited to samples, which do not contain contamination.

A representative portion of the soil was divided from the sample using one of the methods described in Section 9 of ASTM D 2216. The split portion was then placed in a drying oven and heated to approximately 110 degrees C overnight or until a constant mass was achieved after repetitive weighing. The moisture content of the soil was then computed as the mass of water removed from the sample by drying, divided by the mass of the sample dry, times 100 percent.

No attempt was made to exclude any particular particle size from the portion split from the sample.

3.5.2 Mechanical Sieve Analysis of Samples

The distribution of sand size particle sizes was determined in general accordance with the procedures described by ASTM D 421, "*Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants*", and D 422, "*Standard Test Method for Particle Size Analysis of Soils.*" During preparation samples were divided into two portions. The material coarser than the No. 30 U.S. sieve size fraction was dry sieved through a nest of standard sieves as described in Article 6. Material passing the No. 30 sieve was independently passed through a nest of sieves down to the No. 200 size.

3.5.3 Percent Fines Determination of Samples

A selected specimen of soils was washed over a No. 200 sieve after being thoroughly mixed and dried. This test was conducted in general accordance with ASTM D 1140, "*Standard Test Method for Amount of Material Finer Than the No. 200 Sieve.*" Method A, using water to wash the sample through the sieve without soaking the sample for a prescribed period of time, was used and the percentage by weight of material washing through the sieve was deemed the "percent fines" or percent clay and silt fraction.

3.5.4 Liquid and Plastic Limits Testing

Atterberg limits of the soils was determined generally following the methods described by ASTM D 4318, "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.*" Albert Atterberg originally defined "limits of consistency" of fine-grained soils in terms of their relative ease of deformation at various moisture contents. In current engineering usage, the *liquid limit* of a soil is defined as the moisture content, in percent, marking the upper limit of viscous flow and the boundary with a semi-liquid state. The *plastic limit* defines the lower limit of plastic behavior, above which a soil behaves plastically, below which it retains its shape upon drying. The *plasticity index* (PI) is the range of water content over which a soil behaves plastically.

Numerically, the PI is the difference between liquid limit and plastic limit values. Representative portions of fine grained Group A, B, C, or D samples were prepared using the wet method described in Section 10.1 of ASTM D 4318. The liquid limit of each sample was determined using the multi-point method (Method A) described in Section 11. The liquid limit is by definition the moisture content where 25 drops of a hand operated liquid limit device are required to close a standard width groove cut in a soil sample placed in the device. After each test, the moisture content of the sample was adjusted and the sample replaced in the device. The test was repeated to provide a minimum of three widely spaced combinations of N versus moisture content. When plotted on semi-log paper, the liquid limit moisture content was determined by straight-line interpolation between the data points at N equals 25 blows.

The plastic limit was determined using the procedure described in Section 17 of ASTM D 4318. A selected portion of the soil used in the liquid limit test was kneaded and rolled by hand until it could no longer be rolled to a 3.2 mm thread on a glass plate. This procedure was repeated until at least 6 grams of material was accumulated, at which point the moisture content was determined using the methods described in ASTM D 2216.

4.0 SITE CONDITIONS

S&ME's assessment of the geotechnical conditions began with a reconnaissance of the topography and physical features of the site. To the extent feasible, we consulted available topographic and geologic maps, soil maps, or SCDOT and South Carolina Geologic Survey boring data for relevant information. The site is located in South Carolina just north of Yauhannah on US 701 in Georgetown County. Flood relief bridges over Yauhannah Lake and the Pee Dee Overflow are located within the Pee Dee River floodplain north and south of the main Pee Dee River Bridge. All 3 bridges lie entirely within the floodplain of the Pee Dee River. Topographic relief ranges from gently sloping to level, with a maximum elevation change of about 5 feet along the alignment within the floodplain. There are bluffs on either end of the alignment at the edge of the floodplain, about 20 feet in height.

The site is located within the Lower Coastal Plain of South Carolina. The floodplain adjacent to the river has a poorly developed drainage pattern typical of a nearly level plain comprised of largely sandy soils. Major streams have numerous oxbows and meanders typical of a low-energy system with very low stream gradients. Slopes along drainageways are typically very gentle. Roads and other improvements mostly lie on more elevated terraces and bluffs out of the flood plains. Most of the lowlands are wooded. More elevated areas adjacent to the floodplain are cultivated or used for timber plantations. There appeared to be no cultivated land along the alignment.

Bottom lands along the rivers in the vicinity are nearly level and typically about one mile wide. At the proposed bridge locations along the floodplain, maximum floodplain width appears to be approximately 1 ½ miles. Floodplain surface deposits consist mostly of poor to moderately well drained sandy alluvium.

4.1 Site Reconnaissance

Messrs. John Lessley and Tommy Still of S&ME, Inc. viewed proposed boring locations in the major areas of the project on September 13, 2004. At that time conditions were fairly dry following lower than average rainfall over an extended period of time. The portion of the main Pee Dee River Bridge north of the river channel is an asphalt-surfaced parkway lot for a public boat-ramp access. The surface of the floodplain was mostly dry in the areas of the Pee Dee Overflow and the main Pee Dee River Bridge of the surface very wet in the land portion of the Yauhannah Lake Bridge. There were also some very wet areas at the eastern end of the Pee Dee Overflow Bridge, but they were fairly confined and access was not restricted.

During the initial visits to the site, we checked boring locations for accessibility to the drilling equipment scheduled for the project and, where necessary, adjusted locations in the field to facilitate the work. Surface conditions which could influence design and construction were also noted. These included presence of ditches or existing overhead or underground utilities in construction areas, presence of all fills, ponded water, conditions of surface soils, type of ground cover and general topography.

4.2 Local Physiographic Conditions and Geology

The site lies within the Atlantic Flatwoods Region of the Lower Coastal Plain of South Carolina. The Atlantic Flatwoods comprises most of the Lower Coastal Plain, lying between the Citronelle and Surry escarpments, and ranging from 15 to 100 miles inland from the sea. The topography of this region is dominated by up to six archaic beach terraces, exposed by uplifting of the local area over the last one million years. The lower coastal plain terraces are relatively young features, exhibit only minor surface erosion, and can be traced large distances on the basis of surface elevation. Each terrace forms a thin veneer over older, underlying Coastal Plain soils. Materials comprising the terraces typically consist of a strand or beach ridge deposit of clean sands at the seaward margin. Between the strand and the toe of the next inland terrace are mainly finely interlayered clays and sands termed backbarrier deposits. Old swamp deposits, stumps and buried trees have in some areas been covered by the backbarrier deposits and are usually not evident at the surface.

Most of the Pee Dee River floodplain is comprised of one of several Quaternary age river terraces that flank the major streams of the area. These terraces were formed over several periods in which the Atlantic Ocean intruded up the valley during warm periods of the Pleistocene Epoch. The more recent terraces are relatively young features, exhibit only minor erosion, and can be traced large distances on the basis of surface elevation. Older terraces have been more severely eroded and have less surface expression. Materials comprising high-energy terraces may consist almost entirely of clean sands. Other terraces deposited under low-flow conditions more typically consist of medium dense, coarse-grained red-brown clayey sands or stiff reddish sandy silts or clays where groundwater levels are sufficiently deep to allow oxidation and consolidation of the soils to occur. In poorly drained areas, a seasonally high water table limits oxidation of the soils and the soil matrix assumes a distinctly mottled appearance. In many cases there are evident bedload features near the base of the terrace soils, with numerous rounded quartz pebbles, stumps, logs and other debris embedded in the soil binder.

4.3 Interpreted Stratification of Soils On Site

S&ME's interpreted subsurface stratification is indicated in the subsurface profiles included as Figures 3 through 8 in Appendix I. Our borings encountered up to five general strata or horizons based on visual appearance and apparent geologic origin. A general description of the samples recovered from each strata is included below, and summarized in Table 2 at the end of this section.

Strata I – Embankment Fill

Occurrence appears limited to highway embankments constructed as part of the 1952 construction. Typical embankment height ranges from 20 to 25 feet at abutments facing the Great Pee Dee River, 10 to 22 feet at the Pee Dee Overflow, and about 16 feet at Yauhannah

Lake. These soils will form the immediate bearing surface for a portion of any new fills placed to widen the existing roadway, assuming the revised roadway closely parallels or overlaps the existing alignment.

Three of the soil test borings penetrated the full height of the embankments. Recovered samples were narrowly graded fine to medium grained sands with trace to no discernable fines. There were also a few recovered samples which were found to consist of mixtures of fine sand and clayey fines. Predominant classification by the Unified Soil Classification was SP based on visual-manual procedure.

Meaningful thickness of embankment fill was encountered in CPT sounding C-3, at the north end of the Pee Dee Overflow Bridge. Fill thickness at this location appears to be approximately 14 feet. Soil behavior based on CPT point resistance and friction ratio was typical of sands containing little to some silty fines.

Moistened samples are tan in color. Samples were generally moist becoming wet or saturated, containing considerable free moisture, near the base of the embankment. Soil specimens washed over a No. 200 sieve indicated 0 to 5 percent clay or silt by weight. North of the Pee Dee Overflow Bridge, the CPT soil index parameter Ic in sounding C-2 suggests a fines content of 3 to 6 percent using the relationship by Robertson and Fear (1998).

Field N-value counts before correction for overburden stress ranged from 10 to 20 blows per foot, typical of medium dense consistency sands. Cone penetrometer tip resistances obtained in the fill section of sounding C-3 north of the Pee Dee Overflow ranged from 80 to 150 tons per square foot. Cone penetrometer tip resistances in the fill section of sounding C-1 south of Yauhannah Lake are substantially lower, typically 30 to 50 tons per sq. ft. Relative density estimated using the relationship by Kulhawy and Mayne (1990) range from 42 to 70 percent, consistent with loose to medium dense sands.

These soils might be subject to some additional compression if the embankment is raised or widened. However, soils in this layer lie mostly above the water table and consist mostly of sands. Even where fine grained soils are present they will be only partially saturated. Primary consolidation will for that reason be very rapid and impossible to tell apart from immediate settlement. Secondary compression will be very small and can be neglected in settlement estimates.

Stratum II – Holocene Alluvial Clays and Silts

These soils were encountered over a limited area near the south bank of the Pee Dee River and north of the Pee Dee River to the north margin of the flood plain. In this area these soils extend from the existing ground surface to a depth of about 35 feet below the surface. Based on drawings indicating the profile of the existing piers, these soils appear to have been completely penetrated and scoured away within the channel of the Pee Dee River. Local geologic mapping indicate Holocene age soils not to be present at the surface beyond a short distance south of the Pee Dee channel.

These soils will form the principal resisting strata for laterally loaded deep foundations supporting the proposed Pee Dee River and Pee Dee Overflow structures. They will also form the immediate subgrade for new structural fill in widened embankments north of the Pee Dee River.

These alluvial soils appear to have been recently deposited by river erosion and depositional processes. Borings and soundings penetrating these soils exhibit a highly interlayered soil fabric typical of levee type deposits. Boring B-3 penetrated these soils to a depth of 35 feet just south of the Pee Dee River channel. North of the Pee Dee River, the deposits consist of highly interlayered sands, silts and clays to a depth of about 15 feet. These highly variable soils appear to represent a zone of recent scour and redeposition on top of underlying deposits of alluvial sands and gravels, in the north one-half of the flood plain.

Recovered samples were moderate to high plasticity, clayey fines with considerable dry strength to a depth of about 12 feet. South of the river these soils were underlain by about 4 feet of loose clean sands and then about 10 feet of very soft clayey silts. Individual split spoon samples recovered in this layer contain organic debris. Soil behavior based on CPT point resistance and friction ratio was typical of clays or silty clays which exhibit substantially undrained behavior. Limited thicknesses or zones with CPT soil behavior more typical of sensitive silts or clays which exhibit very great strength loss when disturbed were also identified. CPT penetration data imply a highly variable, layered strata consisting of thin (<6 inch) seams of sands and silts or clays.

Moistened samples are dark brown to black in color. Samples were wet or saturated and contained considerable free moisture. Soil specimens washed over a No. 200 sieve indicated as much as 90 percent clay or silt by weight. The CPT soil index parameter Ic suggests soils near the north bank of the river to have a fines content of 45 to 80 percent using the Robertson and Fear criteria. The grain size distribution of the minus No. 200 fraction calculated from the time rate of sedimentation of the various size particles by the hydrometer method was 63 percent silt and 28 percent clay, for a sample of the organic laden silts obtained in Boring B-3. A similar test conducted in boring B-4 within this zone resulted in 58 percent silt and 28 percent clay.

Minus No. 40 sieve sizes exhibited liquid limit values of 34 to 49 percent and plastic limit values ranging from 20 to 25 percent. Plasticity Index values thus ranged from 14 to 24 percent.

Standard penetration test values typically ranged from 3 to 5 blows per foot, typically soft consistency, with some intervals exhibiting a full 18-inch drop under the weight of the hammer. Unconfined compressive strength of disturbed fine grained soil samples in the laboratory ranged from .1 to .5 tsf using a pocket penetrometer. Values are consistent with soft cohesive soils. Cone penetrometer tip resistances ranged from 1 to 2 tons per square foot within this zone in sounding C-2. The relationship by Robertson & Campanella (1988) gives values of undrained shear strength ranging from 100 to 300 lbs per square foot, using a cone factor N_{kt} of 15. This is also consistent with soft, cohesive soils.

Undrained shear strength based on CPT tip stresses in sounding C-2 are approximately 0.2 to 0.3 times the vertical effective stresses within this zone. Using the relationship given by Andresen et. al. (1979) and Ladd (1974) this ratio would be consistent with a fine grained soil which has been preconsolidated by a factor of only about 1 to 1.2 times the present effective overburden stress. This would suggest a maximum prior applied vertical stress approximately equal to the vertical stress now applied by soil self-weight.

These soils form a soft layer on top of a profile consisting mostly of hard or dense sandy soils below. Compression of this stratum will comprise the majority of the settlement experienced under static load. These soils lie mostly below the water table at this site and most of these soils will be entirely saturated upon application of load. But consolidation is still likely to be rapid since the soils tend to be highly stratified horizontally and drainage paths from impervious layers tend to be very short. Primary consolidation of this layer may thus be to some degree hard to tell apart from immediate compression. Secondary compression will be very small and can be neglected in settlement estimates.

Stratum III – Late Quaternary to Holocene Alluvial Sands and Gravels

These soils occur below the alluvial silts and clays over most of the alignment, but appear to thin and become absent near the north and south ends of the project. These soils were encountered in all of the borings conducted within the flood plain, but appeared to be largely absent in borings B-1 and B-2 at the north and south ends of the Yauhannah Lake Bridge. In most locations south of the main channel the alluvial soils consisted of fine grained materials which were penetrated to depths of over 25 feet below the original ground surface, possibly reflecting almost complete erosion or scouring of the sands. But north of the main channel the overlying clays and silts appear to thin to less than 10 feet in thickness or become intermittent and the sands comprise nearly the entire section of the alluvial profile to a depth of about 35 feet below the original ground surface.

These soils will form an intermediate or deep bearing strata below area fills in the northern portion of the site. These soils will also form one of the principal resisting strata for laterally loaded deep foundations supporting the proposed structures north of the main river channel.

These coarse grained soils appear to represent a zone of high energy deposition characteristic of a beach or sand bar. Recovered samples were narrowly graded fine to medium grained sands with trace to no discernable fines. Predominant classification by the Unified Soil Classification was SP based on visual-manual procedure. Soil behavior based on CPT point resistance and friction ratio was typical of sands containing little to some silty fines. Soil specimens washed over a No. 200 sieve indicated 2 to 5 percent clay or silt by weight. The CPT soil index parameter Ic suggests a fines content of 4 to 5 percent using the relationship by Robertson and Fear.

Dry sieving of samples obtained at different levels within these soils indicates a significant coarsening of the soil fabric with depth. Near the top of the layer, representative samples contain no significant fraction of gravel sizes (plus No. 4 sieve size), 0.3 percent coarse sands (plus No. 10), approximately 10 percent medium sands (plus No. 40) and 80 percent fine sands (plus No. 200). Approximately midway through the layer the percentage medium sand increases to over 60 percent and the percentage fine sands decreases to 35 percent. Near the base of the layer samples contained 5 percent fine gravel, 11 percent coarse sands, about 70 percent medium sand, and only 12 percent fine sands.

Since fines content was less than 12 percent, a coefficient of uniformity Cu and a coefficient of curvature Cc was computed from the cumulative particle size distribution for each sample. Near the top of the layer coefficient of uniformity was 2.21, at the middle 2.31, and near the base 4.33. Coefficient of curvature ranged from 1.12 to 1.26. These data are consistant with a sand that is gap or skip graded in the fine sand size range near the top of the layer and in the medium sand size range with depth.

Field N-values when corrected for overburden stress ranged from 6 to 14 blows per foot, typical of loose to medium dense consistency sands. Cone penetrometer tip resistances ranged from 100 to 200 tons per square foot. Relative density estimated using the relationship by Kulhawy and Mayne (1990) range from 55 to 65 percent, consistent with medium dense sands.

The empirical method by Robertson and Campanella (1983) using uncorrected tip stress and vertical effective stress indicates a lower bound value of friction angle of 40 degrees. This method assumes uncemented, unaged, moderately compressible quartz silica sands which exhibit Robertson soil behavior type 4 or higher.

These soils lie mostly below the water table at this site and most of these soils will be entirely saturated upon application of load. But compression is still likely to be rapid since the soils are free draining.

Stratum IV – Late Quaternary Alluvial Clays and Silts (Wando Formation)

These soils form the surface over most of the alignment south of the Pee Dee River, but appeared to be absent in Sounding C-3 adjacent to the main Pee Dee River channel, and in borings north of the Pee Dee River. In borings B-1 and B-2 south of the main channel the fine alluvium was penetrated to depths of 15 feet to as much as 30 feet below a thin surface layer of organic material. Local geologic maps indicate these areas to form part of the fluvial facies of the Wando Formation of late Pleistocene age.

These soils will form the immediate subgrade for new structural fill for any new embankments within the flood plain. These soils will also form the principal resisting strata for laterally loaded deep foundations supporting the proposed Yauhannah Lake structure. These alluvial soils form part of the Late-Quaternary age terrace system of the Carolina Coastal Plain. The base of these deposits represents a zone of scour and redeposition on top of underlying buried channel sands or lithified marine sediments.

Recovered split spoon samples were mostly moderate to high plasticity, clayey fines with considerable dry strength when oven-dried. Individual samples consisting of low plasticity, silty fines with very little perceptible dry strength were also obtained. Predominant classification by the Unified Soil Classification was CH based on visual-manual procedure. There were also a few recovered samples which were found to consist of CL or MH. This material was not penetrated by the CPT soundings.

Moistened samples are gray to dark gray in color. Samples were wet or saturated and contained considerable free moisture. Portions of the upper 10 feet of the soils exhibited essentially nonliquid- nonplastic behavior. Minus No. 40 sieve sizes exhibited liquid limit values of 26 to 36 percent and plastic limit values ranging from 21 to 28 percent. Plasticity Index values thus ranged from 5 to 8 percent. Liquidity Index values determined by comparison of plasticity indices to in-place moisture content varied from 0.6 to 1.0.

Standard penetration test values typically ranged from weight of hammer to about 8 blows per foot. Soil consistency was typically firm to stiff near the ground surface and firm within approximately the middle two-thirds of the layer. SPT N-values obtained in this layer exceed the confining stress (expressed in tsf) by a factor of about 2 to 4. This is a very rough indication of soil stress history but consistent with soils which are normally consolidated to slightly preconsolidated.

These soils form a thick, soft layer within a profile consisting mostly of medium dense sandy soils at depth. Compression of this stratum will comprise the majority of the settlement experienced under static loads. These soils lie mostly below the water table at this site and most of these soils will be entirely saturated upon application of load. Consolidation is likely to be relatively slow since the soils may be stratified horizontally to only a limited degree and drainage paths from impervious layers will tend to be long. Primary consolidation will thus constitute a distinct phase apart from immediate compression and will require a marked length of time to occur after load application. Secondary compression may also be significant and would need to be considered in design.

Stratum V – Deeply Buried Channel or Terrace Sands

These soils were encountered from 30 to as much as 60 feet below the existing ground surface in the portion of the flood plain bounded by the south end of the Pee Dee Overflow Bridge and the north end of the Yauhannah Lake Bridge. These soils appear to thin and become absent at the margins of the flood plain. Maximum thickness reached was about 30 feet at some sample points.

These soils extended to refusal to the CPT push at Sounding C-3 at a depth of 65 feet. We note that refusal of our drilling or sounding tools in this layer may have resulted from the presence of gravel beds, lenses or seams of cemented or hard soils, boulders or ledges of cemented materials or highly calcareous materials occurring at the base of this layer. From a review of old bridge plans, these soils appear to form the principal bearing strata for 18-inch square precast piles supporting the existing trestle structures.

These soils form part of the Quaternary age terrace system of the Carolina Coastal Plain. Since they are not calcareous, they likely represent a terrestrial high energy deposition such as a point bar or sand bar.

Recovered samples were narrowly graded fine to medium grained sands with trace to no discernable fines. Predominant classification by the Unified Soil Classification was SP based visual-manual procedure. Soil behavior based on CPT point resistance and friction ratio was typical of free draining soils with relatively high normalized tip stresses such as gravelly sand to medium dense sand, with some zones more typical of heavily overconsolidated or cemented sands to clayey sands. There were also several areas where CPT soil behavior was more typical of overconsolidated undrained soils, with very high excess pore pressures, over intervals of 3 to 5 feet. Both SPT and CPT penetration data imply a variable, stratified deposit consisting of moderately thick (3 to 5 foot) layers of alternating medium dense to dense sands, with occasional layers of stiff to very stiff clays or silts.

Moistened samples are gray. Samples were wet or saturated and contained considerable free moisture. Soil specimens washed over a No. 200 sieve indicated 5 to 7 percent clay or silt by weight. The CPT soil index parameter Ic suggests a fines content of 1 to 3 percent using the relationship by Robertson and Fear (1998).

Field N-values when corrected for overburden stress ranged from 18 to over 50 blows per foot, typical of medium dense to very dense consistency sands. Cone penetrometer tip resistances ranged from 150 to 400 tons per square foot. Relative density estimated using the relationship by Kulhawy and Mayne (1990) range from 80 to 90 percent, consistent with dense sands. The empirical method by Robertson and Campanella (1983) using uncorrected tip stress and vertical effective stress indicates a lower bound value of friction angle of 42 to 44 degrees.

A rough estimate of sand compressibility was made using uncorrected CPT tip resistance values. For the typical range of sand-sized grain sizes, elastic modulus of the soils is typically represented by a value equivalent to 2 to 4 times the tip resistance. In this case, tip resistances of 150 to 400 tons per square foot likely represent a material with an elastic modulus of 400 to 1200 tons per square foot. These soils would be anticipated to be relatively incompressible under area fills.

Stratum VI – Coastal Terrace Sands and Clays (Socastee Formation)

Occurrence appears limited to the abutment areas at the extreme north and south ends of the alignment, at the margins of the Pee Dee River Flood Plain. These soils reached depths of 30 to 35 feet south of Yauhannah Lake, and about 30 feet below original ground surface north of the Pee Dee Overflow Bridge. These soils become thin or absent in the flood plain portions of the site.

Local geologic maps indicate these soils to form part of the Socastee Formation, a mostly marine Coastal Plain deposit of late-Pleistocene age which has been largely eroded through by the Pee Dee River within the flood plain. These fine grained soils form part of the Quaternary age terrace system of the Carolina Coastal Plain, mostly tidal but some river deposited sediments that in some areas were subsequently covered by beach sands and compressed to some degree by migrating sand dunes. These soils extended to refusal to the CPT push at a depth of 35 feet south of the flood plain, and to the top of underlying overconsolidated clays and silts at 52 feet at the north end of the Pee Dee Overflow Bridge.

Approximately 20 feet of this material was penetrated on the south side of the flood plain in Sounding C-1. Soil behavior based on CPT point resistance and friction ratio was typical of sands containing little to some silty fines in the upper one-half of the layer, with the lower portion containing lenses or seams of clays or silty clays which exhibit substantially undrained behavior. CPT penetration data imply a variable, stratified deposit consisting of moderately thick (6 inch to 12 inch) layers of alternating sands and clays or silts, particularly below a depth of 30 feet.

North of the Pee Dee Flood Plain sounding C-3 penetrated roughly the same thickness of material. But at this location the Coastal Plain soils consisted of about 10 feet of clays or silty clays which exhibit substantially undrained behavior, underlain by sands containing little to some silty fines. Cone penetrometer tip resistances ranged from 14 to 20 tons per square foot in the upper clayey portion of the zone. The relationship by Robertson & Campanella (1988) gives values of undrained shear strength ranging from 1 to 1.5 tons per square foot, using a cone factor N_{kt} of 15. Fine grained soil undrained shear strength based on CPT excess pore pressure ranges from 0.8 to 1.0 tons per square foot using a pore pressure cone factor of 10. These values would be consistent with stiff, cohesive soils.

Undrained shear strength based on CPT tip stresses are 5 to 8 times the vertical effective stresses within this zone. Using the relationship given by Andresen et. al. (1979) and Ladd (1974) this ratio would be consistent with a clay which has been preconsolidated by a factor of at least 2 to 2.5 times the present effective overburden stress. This would suggest a maximum prior applied vertical stress of at least 5 to 6 kips per square foot in this layer.

These soils lie mostly below the water table at this site and most of these soils will be entirely saturated upon application of load. But consolidation is still likely to be rapid since the soils tend to be highly stratified horizontally and drainage paths from impervious layers tend to be very short. Primary consolidation of this layer may thus be to some degree hard to tell apart from immediate compression. Secondary compression will be very small and can be neglected in settlement estimates.

Stratum VII – Calcareous Sands and Fines (Pee Dee Formation)

These soils extend from a depth of 30 feet to termination depth of our exploration at the south end of the project. These soils have likely been scoured deeply within the flood plain at some point in geologic history. The upper contact of these soils decreases greatly in altitude progressively northward from the south end of the project to a low point at about 65 feet below the level of the flood plain surface near the existing Pee Dee River channel. Near the north end of the project the upper contact appears to increase somewhat in elevation to about elevation -40 feet.

These soils will form the principal bearing strata for deep foundations supporting the proposed structures. These soils form part of the weathered profile of the Cretaceous age Pee Dee Formation, well consolidated or semi-lithified Coastal Plain sediments indicated on local geologic maps.

The upper contact of these soils is marked by a layer of very dense calcareous sands ranging from 5 to 10 feet in thickness. These soils cap underlying moderate to high plasticity, clayey fines with considerable dry strength which then extend to termination of the borings. Predominant classification of the upper caprock by the Unified Soil Classification was SC based on visual-manual procedure, or in some areas alternating seams of SP and CH clays. Underlying materials appear to be predominantly CH. Samples reacted strongly to dilute muratic acid.

Standard penetration test values typically ranged from 12 to over 100 blows per foot, typically between 20 and 40 blows per foot. Soil consistency based on SPT N-values ranges from stiff to very hard. Hard layers appear to occur in finite layers or seams 6 to 12 inches in thickness occurring at irregular intervals. Unconfined compressive strength of disturbed soil samples in the laboratory ranged from 2 to 4.5 tsf using a pocket penetrometer.

SPT N-values obtained in this layer exceed the confining stress (expressed in tsf) by a factor of at least 10 in almost every sample interval. This is a very rough indication of soil stress history but consistent with soils which have been preloaded well in excess of stresses now applied by the overlying soils. These soils are consolidated under pressures greatly exceeding those that would be applied as either area or line loads by foundation elements supporting the structures. For this reason they may be considered a hard layer below which soils are essentially incompressible for purpose of practical engineering computations.

Table 2 – Typical Soil Units Penetrated By Borings and Soundings

GEOLOGIC STRATA	UNIT	SOIL DESCRIPTION	USCS CLASS		SPT N	MOIST.	DRY STRENGTH	REMOLDED BEHAVIOR	REMARKS
			Primary	Minor					
Manmade deposits	F_E	EMBANKMENT FILL - Soils placed as fill during previous site construction for which no documentation of compaction or engineering control is available. These soils form the existing roadway embankments and are 20 to 25 feet thick.	SP	SM SC	CPT Qt 30-100 tsf SPT 10-15	Dry to Moist	Low to Moderate	Moderately Cohesive to Cohesionless	Upper several feet may provide limited resistance to abutment movements in longitudinal direction. These soils might be subject to some additional compression if the embankment is raised or widened. Primary consolidation will be very rapid. Secondary compression will be very small.
Holocene Alluvium	A_1	HOLOCENE ALLUVIAL CLAYS AND SILTS - Geologically recent deposits along Pee Dee River and north side of flood plain, consisting mostly of very soft clayey to sandy silts and silty clays.	CL, ML	Lenses of SM	Ranges from 0 to 5	Wet to Saturated	Low	Cohesive, Nonplastic to Moderately Plastic	Most samples were saturated, containing free water and heavily remolded., undrained shear strength ranging from 100 to 300 psf. Organic material in the form of vegetable remnants, stumps, limbs, etc also present.
	A_2	LATE QUATERNARY TO HOLOCENE ALLUVIAL SANDS AND GRAVELS – River deposited sands north of the Pee Dee River and the north one-half of the flood plain . Sampled materials typically well-rounded to subangular, loose, fine to medium sands.	SP SM	SW	Ranges from 2 to 28 $N_{AVG} = 12$	Wet to Saturated	None	Cohesionless	Most samples fail to retain any integrity in the sampler. Zones are liquefiable under earthquake conditions. Average fines < 5% Layer is liquefaction prone
Quaternary Alluvium Wando Formation	Q_1	LATE QUATERNARY ALLUVIAL CLAYS AND SILTS (WANDO FORMATION) – Present at the surface south of the Pee Dee River, absent north of the Pee Dee River. Thickness ranges from 15 feet to 30 feet below a thin surface layer of organics.	CH	CL MH	SPT 0-8	saturated	Moderate to high	Cohesive, plastic	Compression of this stratum will comprise most settlement of embankments south of the Pee Dee. Consolidation relatively slow - soils not well stratified horizontally. Secondary compression may also be significant and would need to be considered in design.
	Q_2	BURIED CHANNEL SANDS – Quaternary age river deposited sands which are deeply buried by near surface alluvium. These sands thin and become absent at the margins of the flood plain. Not generally present south of the Pee Dee River.							Typically non-liquefiable based on shear wave velocity measurements and SPT data. Average fines about 5 percent. Layer thins and becomes absent near Yauhannah Lake.

Table 2 – Typical Soil Units Penetrated By Borings and Soundings *(continues)*

GEOLOGIC STRATA	UNIT	SOIL DESCRIPTION	USCS CLASS		SPT N	MOIST.	DRY STRENGTH	REMOLDED BEHAVIOR	REMARKS
			Primary	Minor					
Quaternary Terrace Socastee Formation	Q_3	COASTAL TERRACE SANDS AND CLAYS Limited to abutment areas at north and south ends of the alignment. Soils reached 30 to 35 feet south of Yauhannah Lake, and about 30 feet north of Pee Dee Overflow Bridge. Become thin or absent in the flood plain portions of the site.	SP	SM	20-50	Saturated	None	Competely cohesionless	Soils not liquefiable. Sand compressibility using CPT tip resistance values, represent a material with an elastic modulus of 400 to 1200 tsf. Soils relatively incompressible under area fills. Tip bearing for most existing piles at main Pee Dee bridge.
Cretaceous Deposits (Marine Shelf) Pee Dee Formation	K_2	CRETACEOUS RESIDUAL SANDS (PEE DEE CAPROCK) - Recovered samples generally firm, white or gray kaolinitic sands with a tacky silt or clay matrix. Samples are typically somewhat micaceous.	SC	CH	Ranges from 21 to 100+	Moist to wet	Low to none	Typically weakly cohesive to noncohesive	Very dense calcareous sands forming upper cap of Pee Dee Formation. Typ. 5 to 10 feet in thickness. Become absent north of Yauhannah Lake. Samples reacted strongly to dilute muratic acid. Most piles on existing Yauhannah Lake bridge appear to bear on this layer.
	K_3	VERY HARD CRETACEOUS CLAYS AND SILTS (PEE DEE FORMATION) - Soils occur as a thick seam or ledge of very hard, highly desiccated fines containing minor amounts of gritty sand. Predominant soil color is dark gray or black samples are slightly micaceous.	CL CH ML MH	SM SC	Ranges from 22 to >100	Dry to moist	High	Cohesive, shaly, structured, plastic	S_U typically 8000 – 10,000 psf based on CPT data. Layer is not free draining and often resists insertion of high displacement piles. Very high set up of driven piles.

4.4 Groundwater

Groundwater measurements were obtained in each boring upon completion of drilling and at least 24 hours after completion of drilling. The groundwater levels observed at each boring location are indicated on the individual Soil Test Boring Records attached and in Table 3 below.

Table 3 - Summary of Groundwater Measurements

Boring Number	Groundwater Depth (Feet)	Time of Measurement (Days)	Elevation (ft)	Remarks
B-1	20	24 hr.	+2.5	
B-2	21	24 hr.	+2.0	
B-3	2.5	24 hr.	+2.5	
B-4	2.5	24 hr.	+1.5	
B-5	20	24 hr.	+2.5	
B-6	11	24 hr.	+11.5	
B-7	13.5	24 hr.	+9.0	
B-8	3.5	24 hr.	+1.5	
C-1	15	TOB	+10.0	Derived from pore pressures
C-2	5	TOB	+0.0	Derived from pore pressures
C-3	17	TOB	+5.5	Derived from pore pressures

*T.O.B = Time of Boring
24-hr. = Reading made 24-hours after completing boring
Datum obtained from 1952 SCDOT Bridge Cross Section

We note that groundwater levels are influenced by precipitation, long term climatic variations, and nearby construction. Groundwater measurements made at different times than our exploration may indicate groundwater levels substantially different than indicated on the boring records in Appendix II.

Groundwater elevations in soil test borings are obtained by direct measurement. Sometimes a direct measurement can be obtained from a CPT push if the hole remains open for a short period of time after the tools are withdrawn. Otherwise, the piezometric surface was indirectly derived from a line fitted to the trend of the pore pressure gage measurements from the CPT push data.

5.0 EARTHQUAKE DESIGN ISSUES

Seismic induced ground shaking at the foundation is the effect taken into account by "SCDOT – Seismic Design Specifications for Highway Bridges, October 2001 as revised". Other effects, including landslides or soil liquefaction, are not addressed in the specifications but must also be considered for certain performance category structures.

Bridge structures on the state highway system have been classified as "normal bridges", "essential bridges", or "critical bridges" in the SCDOT - Seismic Design Specifications. The bridges in this project are "normal" bridges as specified by the SCDOT. The bridges classify as Importance Classification III structures as defined by Section 3.5 of the SCDOT – Seismic Design Specifications. Normal bridges require an evaluation for the Safety Evaluation Earthquake (SEE) only.

5.1 Ground Motion

The "SCDOT – Seismic Design Specifications for Highway bridges, October 2001 as subsequently revised" use two different earthquake motions. The Functional Evaluation Earthquake (FEE) is defined as an earthquake with a 10 percent probability of exceedance in 50 years (474 year return period). The Safety Evaluation Earthquake (SEE) is an earthquake with a 2 percent probability of exceedance in 50 years (2500 year return period).

The South Carolina Department of Transportation has obtained site specific seismic spectral response accelerations from research conducted around the State of South Carolina for a seismicity assessment made under a separate contract. S&ME was provided spectral acceleration values for this site by Ms. Lucera Mesa of the SCDOT to be incorporated into design. Below are the values provided by the SCDOT specifically for the US 701 Bridges.

Safety Evaluation Earthquake Short Period Spectral Acceleration ($S_{S\text{-}SEE}$)	0.45g
Safety Evaluation Earthquake 1 Second Spectral Acceleration ($S_{1\text{-}SEE}$)	0.21g

Performances required subsequent to each earthquake are tabulated in terms of service levels and damage levels in Section 3.2.3 of the SCDOT – Seismic Design Specifications. For Normal bridges, a service level of *impaired* and a damage level of *significant* is acceptable after the Safety Evaluation Earthquake. Impaired service signifies extended closure to the public but open to emergency vehicles and replacement may be needed. Significant damage implies that there is minimum risk for collapse, though permanent offsets may occur in structural elements other than foundations. The damage usually consists of concrete cracking, reinforcement yielding, major spalling of concrete, and deformations in minor bridge components. This damage may require closure to repair and partial or complete demolition and replacement may be required in some cases. For Normal bridges, the functional evaluation is not required.

5.2 Site Class Effects

We classified the bridge site as one of the Site Classes defined in Section 3.3.2 (Table 3.3.2A) of the SCDOT – Seismic Design Specifications using the procedures described in Steps 1 through 3. The Site Class is used in conjunction with mapped spectral accelerations S_S and S_1 to determine spectral response Site Coefficients F_A and F_V in Section 3.3.3, tables 3.3.3A and 3.3.3B.

Table 4 – Site Class Definitions Defined in SCDOT – Seismic Design Specifications

SOIL CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 FT (30 M)		
		SOIL SHEAR WAVE VELOCITY $\bar{V}_s$	STANDARD PENERTRATION RESISTANCE $\bar{N}$ or $\bar{N}_{ch}$	UNDRAINED SHEAR STRENGTH $\bar{s}_u$
A	Hard Rock	>5,000 ft/sec (>1500 m/s)	Not applicable	Not applicable
B	Rock	2,500 to 5,000 ft/sec (760 to 1500 m/s)	Not applicable	Not applicable
C	Very dense soil and soft rock	1,200 to 2,500 ft/sec (360 to 760 m/s)	>50	≥ 2,000 psf (≥ 100kPa)
D	Stiff soil	600 to 1,200 ft/sec (180 to 360 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100kPa)
E	Soft soil	< 600 fps (< 180 m/s)	<15	< 1,000 psf (<50 kPa)
E		Any profile with more than 10 ft (3m) of soft clay defined with: $PI^{(1)}>20$; $w^{(2)} \geq 40$ percent; and $\bar{s}_u < 500$ psf (25 kPa)		
F		Any soil profile containing one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H>10 ft [3 m] of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H>25 ft [8 m] with $PI^{(1)} > 75$) 4. Very thick soft/medium stiff clays (H> 120 ft [36 m])		

(1) The plasticity index, PI, is determined according to ASTM D4318-93.

(2) Moisture content, w, is determined according to ASTM D2216-92.

Site conditions were initially compared to the three conditions described for Site Class E. These are soft soils vulnerable to large strains under seismic motion. For classification on Class E, borings must include at least 10 feet having 1) plasticity index greater than 20, 2) moisture content greater than 40 percent, and 3) undrained shear strength less than 500 psf. Borings B-1 and B-2 lying between Yauhannah Lake and Pee Dee River penetrated approximately 30 feet of alluvial silts and clays with plasticity index greater than 20 and with low apparent strength. However, moisture content of this soil ranged from 24 to 36 percent in five representative locations within this interval. Since all three of the criteria stated for Site Class E are not present, Site Class E does not appear justified in this area.

The next step in site class definition is a check for the four conditions described for Site Class F, which would require a site specific evaluation to determine site coefficients F_A and F_V. Soils vulnerable to potential failure under item 1) including quick and highly sensitive clays or collapsible weakly cemented soils, were not observed in the borings. Three other conditions, 2) peats and highly organic clays; 3) very high plasticity clays; and 4) very thick soft/medium stiff clays, were also not evident in the borings performed.

The remaining vulnerability - liquefaction of saturated, cohesionless soils - appears possible at shallow depths within the Holocene to Quaternary alluvial sands in portions of the bridge alignment at the Pee Dee River and the Pee Dee Overflow Bridge under the SEE earthquake. In these areas, river-deposited sandy soils encountered at shallow depth north of the Pee Dee River appear to contain little fines, lie below the water table, and often exhibit relative density values of less than 70 percent based on a comparison of SPT N-values with effective overburden stress. Potentially liquefiable soils also appear to be to some extent horizontally continuous between adjacent borings north of the Pee Dee River and also vertically between adjacent sample intervals. Finally, the geologic setting of the north over-half of the floodplain, Quaternary or Holocene terrace, is known historically to have experienced liquefaction in previous earthquakes.

Potentially liquefiable areas are indicated on the profiles in figures 6, 7 and 8. Very limited liquefaction potential appears evident near the Yauhannah Lake Bridge. Where liquefaction is evident, thickness of the liquefaction zone appears to be typically greater than about 10 feet, occurring near the base of the Quaternary river alluvium. In some borings liquefaction appears to be present to as deep as 40 feet. Soils at or below the tips of the existing piles of the PeeDee Overflow Bridge and the north approach of the main Pee Dee River bridge are typically subject to liquefaction in this area.

For the above reasons it would be reasonable to expect structural response would be significantly affected by liquefaction occurrence under the SEE earthquake. Site Classification F, requiring a specific determination of the site response, appears justified by conditions encountered in the borings at the Pee Dee River and Pee Dee Overflow bridges.

We believe the intent of the SCDOT Seismic Design Specifications can be met at the Yauhannah Lake Bridge by using the Site Class D response spectrum for design, while explicitly considering the consequences of any minor areas of liquefaction in analysis of foundations and embankments.

5.3 Spectral Response Analysis

5.3.1 Yauhannah Lake

The bridge site was categorized using the method described in Section 3.3.2, Method 1 (V_S Method), Method 2 (N method) and Method 3 (N_{ch} method) of the SCDOT – Seismic Design Specifications for a 100 foot depth below finished grade. Undrained shear strength of the upper clays are assigned for each sampling interval based on inspection of the recovered soils, simple penetration tests in the laboratory and our experience in similar soils. Site Class based on the S_U

method is D for borings B-1 and B-6. We have selected Site Coefficients F_A and F_V from tables 3.3.3A and 3.3.3B using Site Class D. Site Class D will need to be confirmed during the final exploration by further evaluation of the soil profile at Yauhannah Lake.

Design spectral values S_{DS} and S_{D1} were obtained by multiplying Site Coefficients F_A and F_V by the values obtained for the Safety Evaluation Earthquake for short and 1-second periods, respectively, obtained in Section 5.1. The Seismic Performance Category (SPC) of the structure was determined by comparing the values of S_{D1}-SEE above with Table 3.6 in the SCDOT – Seismic Design Specifications, considering the structure to have an Importance Classification of III for a Normal Bridge. Values of S_{DS}, S_{D1}, F_A, F_V, and SPC are presented below in Table 5.

Table 5 – SCDOT Design Spectral Response Acceleration Parameters (per SCDOT) Yauhannah Lake

$S_{S\text{-}SEE} = 0.45g$ $S_{1\text{-}SEE} = 0.21g$

Sounding/Boring	Site Class	F_A	F_V	$S_{DS\text{-}SEE}$	$S_{D1\text{-}SEE}$	Peak Ground Acceleration	Remarks
C-1	D	1.44	2.0	0.64g	0.42g	0.26g	
B-1	D	1.44	2.0	0.64g	0.42g	0.26g	

5.3.2 Site Specific Response Analysis

This site specific seismic response analysis is prepared to meet the SCDOT Seismic Design requirements. Soil stratigraphy for the site response analysis was based on CPT and shear wave velocity data at soundings CPT-C1, CPT-C2 and CPT-C3 and comparison to profile drawings. The Pee Dee Formation was modeled using the peninsular range dynamic soil properties. Input motion used in the analyses was provided by the SCDOT Geotechnical Design Section (referred to as *"scaledshake"*). The Safety Evaluation Earthquake (SEE) for 2 percent probability of exceedance in 50 years was used in the analysis. Consultant Geotechnical Seismic Response and Consultant Seismic Information Request sheets referenced in the analysis are attached. Site location (longitude and latitude values), a description of site conditions, and seismic information for the SEE seismic event (and FEE seismic event) are included on these sheets.

The commercially available and widely used computer program, SHAKE2000 was used to compute the response of the soil deposit at the site to the input seismic motion. The computer program SHAKE 2000 is used to edit input files and present results of the analyses performed. A sensitivity analysis consisting of variations in depth to the B-C boundary, soil strata types, shear wave velocity soil profiles (based on the three separate CPT soundings) and shear wave velocity values was performed to develop the appropriate response spectrum for design. Results of the individual analyses performed as part of the sensitivity analysis, and the parameters varied are presented below.

Variations were inherently created by the fact that three SCPT soundings were performed at the site, subsequently creating three different shear wave velocity profiles. The three shear wave velocity profiles included clay layers of various thickness and depths below grade. The "soil type" within the clay layers was varied by varying the PI for the clay layer at CPT-C1 only. It was noted that this did not noticeably affect the response. As such, the PI of the clay layer was left at 30 for remaining iterations. Additional iterations were created by varying the location of the B-C boundary from 150 ft to 175 ft for each iteration set and each shear wave velocity profile.

The measured shear wave velocities were also varied for each profile by decreasing and increasing their values by 20 percent in all soil layers located above the Pee Dee Formation, and for some of the upper layers of the Pee Dee formation as needed to maintain a linear increase in the shear wave velocity values with depth (up to the 2,500 fps boundary). It should be noted that the shear wave velocity values were increased linearly from the last value recorded in each sounding to the boundary value of 2,500 fps. This was conducted for each profile.

The recommended response spectrum is S_{DS} = 0.79g and S_{D1} = 0.29g, giving consideration to Section 3.4.5.1 of the SCDOT Seismic Code. For comparison purposes, the SCDOT general procedure response spectrum for a site class D is presented along with the site specific response spectra in Figure 56. This analysis implies that soil column at the site has a slightly higher spectral amplitude than the general procedure at relatively high frequencies <0.5 sec. period, but in the displacement dominated zone of the spectrum the response displays considerably lower spectral amplitudes than the general procedure. In effect, S_{DS} is somewhat higher than the class D S_{DS} value obtained using the general procedure, but S_{D1} is a lot lower.

The response values are in the 1.0 - 1.5 second region are sufficiently lower than the general procedure solution that to use the site specific response spectrum directly would not be allowed without a third party opinion, as stated in section 3.4.5.1 of the SCDOT seismic manual. Alternately, the 70 percent of the general procedure values (shown as a red dashed line in Figure 56) could be used without a third party opinion.

5.4 Liquefaction

Liquefaction of saturated, cohesionless soils occurs when they are subject to earthquake loading which causes the pore pressures to increase, and effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results.

We considered potential for liquefaction of bearing soils using the empirical procedure described in the 1996 NCEER and 1998 NCEER/NSF workshop summary report, "Liquefaction Resistance of Soils," Journal of Soil Mechanics and Foundation Division, American Society of Civil Engineers (2001) vol. 127, SM 10, pp. 817-833. This approach characterizes the stress state of the soil by a *cyclic stress ratio (CSR)*, the ratio of the average earthquake-induced shear stress to the effective confining pressure. Cyclic stress ratio plotted against Standard penetration test (SPT) or cone penetration test (CPT) point stress values indicates in general terms potential for liquefaction to occur at given elevations in each boring, based on comparison of plotted values for this site to plotted values for locations where occurrence of liquefaction following earthquakes of known magnitude is documented in the literature.

CRR values were then plotted against the Cyclic Stress Ratio computed as above. Corrections to the CRR were made for earthquake magnitude; however, no correction was made to the point stress values to account for the presence of thin clay or silt layers in the soil column.

Earthquake-induced ground surface acceleration at the site was assumed from the SEE design peak ground acceleration of 0.26g determined assuming Site Class D response factors obtained using the general procedure, and 0.31g using site specific response analyses. This procedure indicates potential for liquefaction of portions of the clean river laid sands (Layer A_2) encountered in the borings and soundings between depths of 10 and 15 feet intermittently across the northern portion of the site. SPT and CPT values were corrected for overburden stress and fines content using the procedures described by the summary report and plotted against CSR values at each depth. A magnitude correction factor for a $M = 7.3$ earthquake was used for the SEE earthquake. A factor of safety against liquefaction of 1.1 was considered as the point of liquefaction initiation.

Several borings performed using an automatic trip hammer likely resulted in low penetration resistances which needed to be corrected for lower efficiency or striking rate. The automatic SPT hammer has a transfer efficiency of 90 percent. A hammer correction of 1.5 was made for these borings to represent a standard 60 percent efficiency. No correction was made for borings performed using the standard rope and cat head arrangement. Areas subject to liquefaction under the SEE earthquake are shaded in figures 6, 7 and 8.

5.5 Post-Liquefaction Surface Rupture

Potential impacts of liquefaction often include loss of bearing capacity, loss of lateral support to piles, lateral spreading of the surface, and post-liquefaction settlement. Our exploration has identified relatively thick zones of liquefaction prone sands in the upper 10 to 30 feet of the natural soil profile across the alignment north of the Pee Dee River. Potentially liquefiable layers are overlain by liquefaction resistant soils to a depth of about 5 feet over much of the alignment. In other areas muck overlies liquefiable sands and the soil profile would be essentially liquefied from the ground surface to the base of layer A_2.

Assuming full liquefaction of these layers, we estimated for the potential for surface damage using the relationship by Ishihara (1985), who also correlated observations of damage to structures related to liquefaction to the thickness of the surficial layer of non-liquefiable soils and to the thickness of the liquefiable layer. For a peak ground acceleration of 0.26g at the surface, a surficial layer 5 feet thick overlying a 20 foot thick liquefiable zone would be likely to experience surface damage.

5.6 Surface Settlements Due to Volumetric Compaction

Settlement of sands due to volumetric compression of the liquefied soils depends on the induced cyclic stresses from the earthquake, the vertical effective stress at the depth of the layer being examined, and the equivalent clean sand corrected SPT value. Settlements were in general terms evaluated by comparing corrected SPT and CPT values within the liquefied zone to empirical data developed by K. Tokimatsu and H. B. Seed, "*Evaluation of Settlements in Sands for Earthquake Shaking*", Journal of Soil Mechanics and Foundation Division, American Society of Civil Engineers (1987) vol. 113, SM 8, pp. 861-879. Tokimatsu and Seed plot volumetric strain vs. CSR at sites where liquefaction was observed and deformations monitored.

Data from the site suggest average volumetric strains within the liquefied zones on the order of 1.0 to 3.0 percent of the thickness of the deposit for the SEE earthquake where full liquefaction occurs. Limited settlements also occur in sands, which have initially liquefied but remain slightly above liquefaction safety factor of 1.0. Average volumetric strains under these conditions range from 0.1 to 0.5. Abutment settlements at each bridge are totaled below for the SEE earthquake.

	SEE pga = 0.31g
Yauhannah Lake, North Abutment	nil.
Yauhannah Lake, South Abutment	nil.
Pee Dee Main, North Abutment	7 in.
Pee Dee Main, South Abutment	2 in.
Pee Dee Overflow, North Abutment	2 in.
Pee Dee Overflow, South Abutment	7 in.

Our experience is that the above settlements are acceptable for roadway embankments under the SEE earthquake. But they are only marginally acceptable to the SCDOT for normal bridges under the SEE earthquake at the north end of the Pee Dee River Bridge and the south end of the PeeDee Overflow Bridge. Settlements occurring due to volumetric strains within the liquefied soils are likely to be variable across any given structure within the site. Differential settlement of slabs or structures bearing on the surface, though relatively small, are likely to approach the total cumulative settlement value over short horizontal distances.

5.7 Lateral Spreading

The potential for free-field liquefaction-induced lateral displacements were estimated using the empirical approach of Bartlett and Youd (1995) developed from regression analyses of United States and Japanese case histories. Using this approach, potential for lateral displacement towards a free-face at the river channel would be substantial for a Magnitude 7.3 earthquake, since most SPT N-values after correction for overburden stress and hammer efficiency are less than 15 blows per foot, and the free-face at the river channel approaches 6m.

Potential for free-field liquefaction-induced lateral displacements within the floodplain itself were estimated using the empirical approach of Bartlett and Youd (1995), reproduced in "Geotechnical Earthquake Engineering", FHWA Publication No. HI-99-012. The most recent update to Youd's body of work on lateral spread displacement prediction is given in Youd, T. L.;Hansen, C. M.; and Bartlett, S. F.; "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement', Journal of Geotechnical Engineering, vol. 128, no. 12, pp. 1007-1017 (2002).

$$\mathrm{Log}(L) = -16.213+1.532M-1.406\log R^{*}-0.012R+0.338\log(S)+.54\log(T_{15})+3.413\log(100-F) - 0.795(\log(D_{50}+0.1\mathrm{mm})$$

Where:

M = Earthquake magnitude for controlling earthquake, taken as 7.3.
R = Horizontal distance to earthquake source, in km – deaggregation data not provided by SCDOT along with spectral values but assumed as 64 km based on deaggregation plot for Georgetown posted on SCDOT webpage.
$R^{*} = R + R_0$, where $R_0 = 10^{(0.89M-5.65)}$
S = Ground slope in percent, variable but 2 to 5 percent assumed.
T_{15} = Thickness of granular soils with corrected blow count less than 15 blows per foot, taken as 8 meters (26 ft). This would include non-liquefied clay sediments of soft consistency.
F = fines content of granular soils in percent, taken as 5 percent.
D_{50} = mean grain size of granular soils in liquefied zone, taken as 0.5 mm.
L = lateral spreading displacement in meters

The thickness of sediments subject to lateral spreading is equal to the worst case observed in any boring. This set of variables yields a lateral displacement of 0.13 meters or 5 inches for a ground slope of 2 percent and a lateral displacement of 7 inches for a ground slope of 5 percent. Either amount appears satisfactory for roadway or bridge approach sections considering SCDOT allowable horizontal movement under the SEE earthquake for normal bridges.

A preliminary estimate of the potential for lateral spreading at the banks of the PeeDee River was made using the Bartlett and Youd equation for a free face, given as:

$$\mathrm{Log}(L) = -16.366+1.178M-0.927R-0.13R+0.657\log(W)+.348\log(H)+0.457\log(100-F) - 0.922D_{50}$$

With variables as previously defined. W is the ratio of the height of the free face to the distance of the free face (L) in percent (100H/L). For a cut exposing a free face of 8 meters in saturated granular soils, movements by lateral spreading towards the river would approach 80 inches within a few feet of the bank, and would be 3 to 4 inches to a distance up to 300 feet from the bank on the north side of the river. Lateral spreading is indicated graphically in the soil profiles on either side of the river shown in Figures 6 and 7. Lateral spreading vs distance to the river are indicated graphically in figures 9 and 10.

In practice either formula is more highly dependent on the horizontal distance to the earthquake than any of the other variables. The 64 km distance to the earthquake for Georgetown may or may not reflect the actual distance, but seems reasonable given the proximity of this site and the other assumptions being made in the estimate.

Conventional limit equilibrium analysis was also used to evaluate potential for flow sliding at the abutments, using residual strength values derived from penetration resistance values of the soils as described below. The steps of the limit equilibrium analysis are described as part of Report Section 6.1.

6.0 EMBANKMENT SLOPES

The following paragraphs include our conceptual conclusions and recommendations for site preparation, fill placement and compaction, design and construction of new or widened approach embankments. Once the construction scope is further defined, the proposed embankment locations are set, and conditions are further explored by additional soil test borings, S&ME, Inc. will review these data based upon the new information and make any necessary changes.

6.1 Stability Assessment

The source of embankment soils for the project is not presently known. We consider it likely that the fills will be constructed of local materials, so soil parameters assumed for stability analyses are taken from those assumed representative for fill material comprising the slopes. The predominant materials indicated by the borings are clean to clayey sands. While shear strength tests were not performed during the preliminary exploration, correlations in the literature suggest a negligible cohesion and an effective stress angle of internal friction of 32 degrees under undrained loading when compacted to at least 95 percent of maximum dry density. Stability of fill slopes were assessed assuming compacted embankment material with no internal water table. In addition, fills were assumed to have no significant areas of poor soils or shoulder material within the 2H:1V envelope.

Abutment slope stability was modeled using the PCSTABL6H computer code developed by Purdue University. PCSTABL is a computer program written in FORTRAN IV source language for the general solution of slope stability problems by a two-dimensional limiting equilibrium method. It is written for the Microsoft Fortran compiler package. The calculation of the factor of safety against instability of a slope is performed by the method of slices. The method of slices is an interactive method that requires computation of the factor of safety against sliding over many trial surfaces before determining the surface with the minimum safety factor.

Parameters used in preliminary assessment of the stability of the embankments are as presented in Table 6:

Table 6 – Strength Properties Assumed for Preliminary Slope Stability and Foundation Evaluation

GEOLOGIC STRATA	UNIT	SOIL DESCRIPTION	USCS CLASS		UNDRAINED LOADING		DRAINED LOADING		REMOLDED BEHAVIOR	REMARKS
			Primary	Minor	Cohesion	Friction Angle	Cohesion	Friction Angle		
Manmade deposits	F_E	EMBANKMENT FILL - These soils form the existing roadway embankements and are 20 to 25 feet thick.	SP	SM SC	0	32 deg	0	32 deg	No change	
Holocene Alluvium	A_1	HOLOCENE ALLUVIAL CLAYS AND SILTS	CL, ML	Lenses of SM	300 - 500 psf	0 deg	0 psf	22 deg	240 psf zero friction	
	A_2	LATE QUATERNARY TO HOLOCENE ALLUVIAL SANDS AND GRAVELS	SP SM	SW	0	30 -34 deg	0	32 deg	Liquefaction prone Su_{res} = 200 - 750 psf	Average fines < 5%
Quaternary Alluvium Wando Formation	Q_1	LATE QUATERNARY ALLUVIAL CLAYS AND SILTS (WANDO FORMATION)	CH	CL MH	300-600 psf	0 deg	0	22 deg	300 psf cohesion 0 deg friction angle	
	Q_2	BURIED CHANNEL SANDS	SP	SW	0 psf	38 deg	0 psf	38 deg	No change	Non-liquefiable.
Quaternary Terrace Socastee Formation	Q_3	COASTAL TERRACE SANDS AND CLAYS	SP	SM	0 psf	34 deg	0 psf	34 deg	No change	Non-liquefiable
Cretaceous Deposits (Marine Shelf) Pee Dee Formation	K_2	CRETACEOUS RESIDUAL SANDS (PEE DEE CAPROCK)	SC	CH	0 psf	38 deg	0 psf	38 deg	No change	Non-liquefiable
	K_3	VERY HARD CRETACEOUS CLAYS AND SILTS (PEE DEE FORMATION)	CL CH ML MH	SM SC	2000 psf	16 deg	800 psf	32 deg	No change	Non-liquefiable

6.2 Rotational Stability Under Static Conditions

Analyses were performed assuming a circular arc failure surface within the foundation and/or embankments. Factors of safety obtained using this approach where then checked against a sliding block analysis for the same geometry using the Janbu Sliding Block method.

Stability under static conditions was computed at representative locations in the flood plain for 2H:1V side slope heights of 20 feet. Graphical output indicate failure geometry for each case is included in the appendix as Figures 11 - 25. A minimum factor of safety of about 1.3 was obtained in all locations. This assumes minimal undercutting and replacement of the upper soils below the abutment and approach fills as required.

Location	Fill Height	Slope	FS (Static)
Yauhannah Lake, North Abutment	20 ft	2H:1V	1.31
Pee Dee River, North Abutment	20 ft	2H:1V	1.54
Pee Dee River, South Abutment	20 ft	2H:1V	1.37
Pee Dee Overflow, South Abutment	20 ft	2H:1V	1.40

Static stability exceeds a factor of safety considered adequate for long term static conditions by only a small amount in several locations, particularly within the flood plain at Yauhannah Lake. The strength parameters assumed for the thick clays underlying this area will need to be more definitively assessed during the final exploration.

6.3 Short Term Stability

Pore pressures within the embankment generated by consolidation of the fill material are anticipated to be low and are anticipated to be dissipated as the fill is brought to height. Where embankments are constructed on compressible fine grained soils, which do not drain freely, consolidation pore pressures in the foundation must be considered in the stability analysis.

As fill is placed, excess pore pressures are built up at the center of the impervious compressible stratum. Initially they are equal to the total pressure of the applied load. Dissipation then occurs as the soils consolidate under the applied stress. Stability was computed for various fill heights assuming that instantaneous placement of each increment of height.

Computations utilized the Modified Bishop Method circular failure methods using effective stress parameters, representing the pore pressures generated by a peizometric surface equal to the original groundwater elevation plus an additional height equal to the applied load divided by the unit weight of water.

Excess pore pressure at the end of construction assumes instantaneous placement of fill to the full height plus 10 ft. surcharge. Factor of safety against rotational failure was computed for a point immediately after placement of the surcharge layer. For purpose of these computations we assumed pore pressure increase to equal nearly the full load near the toe of the slope, and excess pore pressures exceeding 20 ft. of head well beyond the toe. The minimum factor of safety of 1.4 obtained indicates that there probably will not need to be any restriction to rate of fill placement on the embankments or need for "wait" periods during fill placement.

6.4 Seismic Stability

Dynamic stability of the slopes were estimated using pseudo-static analyses using the unified approach outlined in FHWA Pub. HI-99-012. The peak average horizontal acceleration applied to each slice in the method of slices was taken as about ½ of the peak ground acceleration at the surface for preliminary evaluation of stability, or $k_S = 0.14g$.

We assumed that the soils were fully consolidated under the applied fill loads. Where a factor of safety equal to or greater than 1.1(SEE) was indicated using the assumed soil strength values and k*max,* the slope is assumed to be stable under earthquake loads with some minor settlement or displacement.

Deformation analysis using a peak ground acceleration averaged for the centroid of the sliding mass depends on the vertical component of the allowable movement, since the vector of movement at the uphill end of a rotational failure surface is almost entirely vertical. Allowable vertical movement is given as 6 inches for bridge structure (including liquefaction settlements) and 12 inches for roadways under the SEE earthquake. Allowable deflections were used to estimate an average allowable acceleration within the slope for the seismic case, considering allowable deformation. The average allowable acceleration was then applied to each slice in the Bishop Method of Slices, to estimate a factor of safety against failure under earthquake loads.

6.4.1 Average Seismic Coefficient for Deformation Analyses

We estimated the maximum transverse or longitudinal crest acceleration to be equal to approximately 0.60g considering a peak ground acceleration value of 0.30g and using the graphical relationship given by Harder (1991), represented as Figure 6-4 of FHWA Pub. HI-99-012. The 0.60g value represents the likely upper bound value of the top of the embankment based on motions presented in the 2001 specifications for the SEE earthquake. The resulting peak acceleration value was then averaged for the centroid of the sliding mass to determine the average acceleration k*max* within the slope using the graphical relationship given in Figure 6-5 of the FHWA publication. Our computations indicated the minimum stability for sliding masses to extend well below the existing ground surface to the bottom of the liquefaction zone. Accordingly, use of a peak horizontal average acceleration equal to 0.35 times the peak transverse acceleration (pga) value for the sliding mass was deemed appropriate for the driving earthquake acceleration. The value k_{max} was computed to be 0.60g x 0.35 = 0.21g for the SEE earthquake.

The Permanent Seismic Deformation Chart prepared by Hynes and Franklin (1984) and reproduced in FHWA Publication HI-99-012 depicts average, average plus one standard deviation and upper bound plots of deformation vs. the ratio of the average applied acceleration to the maximum applied acceleration, computed from 348 strong motion records using the Newmark Sliding Block Analysis. The Hynes and Franklin chart does not differentiate between earthquake magnitude and source to distance. The upper bound curve would be appropriate for Magnitude > 7.5 earthquakes located at least 25 km away. The lower bound "average" curve would be used for small earthquakes (Magnitude 5.5 or less) or intermediate magnitude ($5.5<M<6.5$) earthquakes within 10 km. The mean plus one standard deviation curve would be used for all other earthquakes, including the SEE earthquake at this site. Assuming no liquefaction and a maximum permanent vertical displacement component of sliding equal to 6 inches (150 mm), the slope configuration must provide a yield acceleration of at least 0.35 times k_{max} or 0.07g to 0.08g.

A similar chart of normalized acceleration vs. permanent displacement is provided by Makdisi and Seed. The Makdisi and Seed chart brackets the range of possible displacements for $M = 6.5$, 7.5 and 8.5 earthquakes in terms of the acceleration as a function of the peak acceleration. In this case the SEE earthquake at $M=7.3$ was taken at the midpoint of the overlap area for the $M=6.5$ and $M=7.5$ earthquakes. Again, assuming no liquefaction and a maximum permanent vertical displacement component of sliding equal to 6 inches (150 mm), the slope configuration using the Makdisi and Seed chart must provide a yield acceleration of at least 0.45 times k_{max} or 0.09g to 0.10g.

6.4.2 Computation of Seismic Factor of Safety.

Provided that the slope crest can tolerate some limited lateral movement, global stability computed using the Modified Bishop Circular Arc method may utilize an average acceleration lower than the peak average acceleration of 0.21g given by Figure 6-5 of FHWA publication HI-99-012 considering a peak surface acceleration of 0.30g.

Where a factor of safety less than unity was indicated for an average acceleration of 0.14g, the analysis was repeated for different acceleration values until a value of A_{yield}, representing the acceleration giving a factor of safety equal to 1.1 was determined. This acceleration is the maximum that could be withstood without substantial permanent displacement of the slope. Comparing A_{yield} to k_{max} in the mean plus one standard deviation curve on the Hynes and Franklin chart, or to the midpoint of the overlap between $M=6.5$ and $M=7.5$ data on the Makdisi and Seed chart, gives a range of estimated permanent displacement of the slope in millimeters for the design earthquake.

Considering the allowed displacements under the SEE earthquake, deformation or surface cracking could be anticipated in the roadway surface but are considered acceptable. Horizontal displacement would be progressively smaller as the factor of safety is increased.

Circular arc failure analyses for SEE earthquake are tabulated below are yield accelerations and estimated slope movements based on k_{max} values computed for SEE earthquake events. Estimated permanent displacements based on correlations presented by Makdisi and Seed are shown in parentheses.

Location	Fill Height	Slope	Pseudostatic ks=0.14g Min Fs=1.1 (SEE)	Newmark deformation kmax=0.21g Min Fs=1.1 Ayield (SEE)	Remarks
Yauhannah Lake, North Abutment	20 ft	2H:1V	Fs= 0.86	0.08g (4in)	OK but marginal
Pee Dee River, North Abutment	20 ft	2H:1V	Fs = 1.09	0.18g (0.5 in)	Liquefaction settlement adds 2 in.
Pee Dee River, South Abutment	20 ft	2H:1V	Fs = 0.88	0.10g (3-4 in)	Liquefaction governs
Pee Dee Overflow, South Abutment	20 ft	2H:1V	Fs = 1.15	0.20g (nil)	Liquefaction governs

*Note: computed K*max* for SEE earthquake based on deep seated failure below embankment base.

Embankments in the flood plain do not appear to provide the minimum 1.1 safety factor against rotational failure for the pseudostatic analysis without supplemental stabilization in some locations. However, they do appear to provide sufficient resistance to sliding to limit permanent deformation to values less than allowable by the SCDOT for the SEE earthquake. In two of the locations evaluated liquefaction settlements exceed allowable settlements stated for normal bridges by the SCDOT, so the liquefaction case would be said to govern design. Computer plots of PCSTAB6 runs are attached in the appendix for each location above.

6.4.3 Computation of Post-Earthquake Flow Sliding.

The potential for liquefaction-induced flow sliding was analyzed using the method of slices and employing residual shear strengths to the liquefied layers. In this type of post-earthquake stability assessment, the seismic coefficient was set equal to zero as recommended in FHWA Publication HI-99-012.

Location	Fill Height	Slope	FS	Remarks
Yauhannah Lake, North Abutment	20 ft	2H:1V	-	No Liquefaction
Pee Dee River, North Abutment	20 ft	2H:1V	1.4	
Pee Dee River, South Abutment	20 ft	2H:1V	1.13	
Pee Dee Overflow, North Abutment	20 ft	2H:1V	1.4	
Pee Dee Overflow, South Abutment	20 ft	2H:1V	1.62	

FHWA Publication HI-99-012 indicates a factor of safety of 1.1 to be acceptable when liquefied soil residual strengths are estimated on the basis of CPT or SPT N-values. Factors of safety of 1.2 or greater obtained using this approach indicate flow sliding not to occur following cessation of earthquake motion.

6.5 Embankment Settlements

Widening of existing embankments will result in placement of up to 20 feet of new fill along the existing slope faces. Embankment consolidation could require the contractor to place additional fill to attain the planned grades. In addition, the contractor or resident engineer may need to place fill to a level higher than final finished grade to accommodate time dependent settlements occurring between completion of rough grading and prior to initiating trim grading immediately before paving. Settlements may also occur subsequent to paving which could affect roadway performance. Borings indicated foundation soils to be generally free-draining sands, with some layers or pockets of silts or clays, north of the Pee Dee River. South of the Pee Dee River foundation soils consist of relatively thick, river laid deposits of silts and clays.

6.5.1 Additional Fill Requirement

To estimate the thickness of additional fill required on embankments to compensate for consolidation in cut/fill balancing of grading operations, we first estimated the consolidation of the bearing materials below the embankment, considering instantaneous placement of an incompressible fill to full height. Settlements for each subsurface layer were estimated by multiplying the increase in overburden stress by the layer thickness and then dividing by the compression modulus for the material as estimated from CPT and SPT tests as well as our general experience with similar soils in the coastal plain region.

Distribution of stresses below the stabilized soil mass was estimated by assuming the bearing soils to constitute a semi-infinite elastic continuum using the Westergaard stress distribution. The fill mass was assumed to represent a perfectly flexible foundation of infinite length. Settlements were computed at the new centerline of the proposed embankment, interpreted to be the current shoulder (column 3), and at the new shoulder (column 2), interpreted to be the point of maximum new fill height (column 1), using the soil profiles identified in representative nearby borings. We did not attempt to modify the distribution of stresses to reflect the presence of a multi-layer bearing strata. Totaled settlements estimated by station are presented in Table 7 below.

Table 7 – Estimated Additional Fill Height to Compensate for Settlements of Embankment

Fill Location	(1) Max. New Fill Height (Feet)	(2) Settlement at New Shoulder (Inches)	(3) Settlement at New Center Line (Inches)
Yauhannah Lake, North Abutment	16	9-½ in	6-½ in
Yauhannah Lake, South Abutment	13	2-½ in	1-½ in
Pee Dee River, North Abutment	19	6-½ in	3-½ in
Pee Dee River, South Abutment	21	9 in	5 in
Pee Dee Overflow, North Abutment	17	1- ½ in	1-½ in
Pee Dee Overflow, South Abutment	22	4 -½ in	2-½ in

6.5.2 Time Dependent Settlements

Since embankment fills will be constructed incrementally, most of the settlements occurring within embankments due to fill placement will be built out during construction. The soils just below the embankments will be mostly below the water table at all three bridge sites. They will be for the most part entirely saturated. However, foundation soils are almost entirely comprised of sands or thinly interlayered sands and silts or clays deposited in a flood plain environment. Drainage distances from the clays to sand seams within the alluvial deposits are likely to be very short. Primary consolidation will for that reason be rapid. Assuming a reasonable construction rate on most embankments of approximately 5 feet per week, settlements occurring subsequent to topping out of most slopes will be minor, on the order of 1 inch or less for a 20-foot high embankment. It is unlikely that significant impact to the construction schedule would occur due to the need for wait periods to allow primary settlement to occur.

Secondary consolidation will be very small compared to primary consolidation. Assuming a secondary consolidation coefficient of 0.005, a reasonable preliminary estimate of secondary consolidation would be ½ inch or less the first year after construction, with approximately ½ inch after that over the next 5 years. Total secondary consolidation was estimated to range from ½ to 1 inch. Secondary compression is unlikely to result in long term settlements exceeding SCDOT criteria for either roadway or bridge approach sections.

6.5.3 Settlements of Existing Roadway and Approach Structures

Where new embankments closely parallel or partially overlap existing embankments, additional settlements may be sustained by the existing roadway and structures. This is because of superposition of the imposed stresses of the widened area load in the compressible strata underlying the embankments. In some locations this additional settlement may be substantial, on the order of 6 inches or more, and may require reworking of grades or laying of temporary asphalt to limit offsets at abutments to maintain traffic.

6.5.4 Settlements Due to Earthquake Forces

Compression of the fill embankments due to pore pressure increase of the foundation materials under earthquake motion was estimated using the free field volumetric compaction values computed at each abutment. Settlements due to seismic loads in embankment fills are expected to be on the order of 1 inch for an embankment 15 to 20 feet high, for the SEE earthquake. This would be in addition to any volumetric compaction of liquefied soils.

7.0 RECOMMENDATIONS FOR CONCEPTUAL FOUNDATION DESIGN

Structural design for the bridges is as yet in the conceptual stages. In preparation of foundation recommendations we have assumed that typical foundation loads follow those provided for similar bridges by the SCDOT on similar projects. We also made the following assumptions:

- No significant scour will be realized at the Pee Dee Overflow and Yauhannah Lake bridges. Scour depth at the Pee Dee River will reach to elevation -35 feet in the channel and -5 feet outside of the channel.
- Drilled shafts are able to rotate at the ground surface (free-head condition).
- Steel pipe piles were assumed to be fixed in pile caps at the surface and were modeled as fixed headed in either the transverse or longitudinal directions.
- Steel H-piles were modeled as embedded at the abutment wall base (fixed-head condition) for loading in the either the transverse or longitudinal directions.
- Precast concrete piles would either be extended to bent caps at the base of the bridge superstructure (free-headed) or fixed in pile caps at the surface (fixed headed).

7.1 Foundation Alternatives

For our analysis we initially considered prestressed concrete piles as well as driven open-ended steel pipe piles, driven steel "H" piles and drilled shafts. The relative advantages and disadvantages of each type are discussed briefly below. In past discussions with bridge consultants on SCDOT projects, steel H piles were considered more appropriate at abutments with integral loadings and drilled shafts were considered the preferred foundation alternative for more heavily loaded interior bents where lateral loads would be applied under seismic conditions and where lateral support is very low near the surface. Advantages and disadvantages of each type are presented in the following sections.

7.1.1 Pile Axial Capacity

We estimated static capacity using the Nordlund method outlined in FHWA Publication FHWA-SA-98-074, "*DRIVEN 2.0: A Microsoft Windows Based Program for Determining Ultimate Vertical Static Pile Capacity*" and similar static capacity methods presented in NAVFACS *Design Manual DM 7.2* (1984). In this case H-piles and open-ended pipe piles were modeled as fully plugged for long-term conditions.

We analyzed 20-inch diameter open-ended steel pipe piles, HP 12 x 53 and HP 14 x 73 steel piles, and 18-inch and 24-inch square PSC piles to estimate working axial load vs. depth using a resistance factor of 0.45 and a λ_v factor of 1.0. A λ_v factor of 1.0 represents capacity verification based on pile driving analyzer measurements and CAPWAP Analyses during production pile installation and on a representative selection of production piles (typically 5 percent) periodically during driving.

Nearly all piles develop significant resistance on the Pee Dee caprock south of the Pee Dee River and on the dense to very dense river laid sands north of the Pee Dee River. Since a large proportion of the loading resistance is developed through end bearing, undulations or discontinuities in the very dense or lithified soil layers will likely result relatively inconsistent driving depths to achieve design capacities at proposed pile supported foundations in some portions of the project.

Negative skin friction maybe assumed to be negligible under static conditions at the Pee Dee Overflow Bridge and at the north abutment of the Pee Dee Bridge, but will be substantial elsewhere due to the presence of compressible clayey soils to depth of cap to 40 feet below proposed new embankment fills. In addition, liquefaction settlements at the Pee Dee River Bridge and the Pee Dee Overflow would result in substantial magnitude of regative skin friction at the three locations under the seismic case.

Abutments will receive approximately 15 to 20 feet of new fill to achieve proposed grades at the bridge crossing. Negative skin friction is the effect of downdrag occurring on piles extending through a soil deposit undergoing consolidation, in which relative soil-pile movements are downward. Observations indicate that a relative downward soil movement of 0.6 inches is sufficient to mobilize the full negative skin friction. The "neutral point" – that point of no relative movement between the soil and pile – is estimated to be at a depth very close to the bottom of the soft clays for the soil column indicated in the abutment borings at Yauhannah Lake and north of the Pee Dee River. Even assuming that piles are wrapped or cased as described below, negative skin friction would be substantial and would need to be included as a load.

Methods have been developed for reducing negative skin friction on deep foundations, including predrilling of oversized holes through the compressible material prior to insertion of the pile, with the annular space filled with bentonite slurry or vermiculite. It may also be feasible to install casing or sleeving of the pile to prevent direct contact with the soil, or to use coatings to reduce friction within the fill and allow slippage between the pile and soil. Pile capacity computations above assume that one of the above approaches will be used, and no allowance for negative skin friction was made in determining tip elevations.

7.1.2 Pile Lateral Capacity

Soil-pile interaction was analyzed for conceptual design purposes to determine minimum drawing depths and to assess the relative resistance available for each foundation type. This preliminary assessment was carried out using the computer code LPILE developed by Professor Lyman Reese. Pile loading at the ground surface for cyclical conditions was modeled using assumed working vertical loads and a range of impose lateral deflections ranging from 0.1 inches to 4 inches at the pile heads. Coefficients of horizontal subgrade reaction and other soil parameters for the soils penetrated by the piles are summarized in Table 8 below:

Table 8 – Soil Parameters for Lateral Analysis of Foundations

GEOLOGIC STRATA	UNIT	SOIL DESCRIPTION	USCS CLASS		UNDRAINED LOADING		SUBGRADE MODULUS K	STRAIN COEFF ε	REMARKS
			Primary	Minor	Cohesion	Friction Angle			
Manmade deposits	$\boxed{F_E}$	EMBANKMENT FILL - These soils form the existing roadway embankments and are 20 to 25 feet thick.	SP	SM SC	0	32 deg	90 pci	NA	0-20 ft at boring B-1 0-20 ft at boring B-5
Holocene Alluvium	$\boxed{A_1}$	HOLOCENE ALLUVIAL CLAYS AND SILTS	CL, ML	Lenses of SM	300 - 500 psf	0 deg	(soft clay)	0.02	Not present boring B-1 20-25 ft at boring B-5
	$\boxed{A_2}$	LATE QUATERNARY TO HOLOCENE ALLUVIAL SANDS AND GRAVELS	SP SM	SW	Liquefaction prone Su_{res} = 200 - 750 psf	30 -34 deg	40 pci	NA	Not present boring B-1 25-55 ft at boring B-5
Quaternary Alluvium Wando Formation	$\boxed{Q_1}$	LATE QUATERNARY ALLUVIAL CLAYS AND SILTS (WANDO FORMATION)	CH	CL MH	300-600 psf	0 deg	(soft clay)	0.02	20-55 ft at boring B-1 Not present boring B-5
	$\boxed{Q_2}$	BURIED CHANNEL SANDS	SP	SW	0 psf	38 deg	125 pci	NA	Not present boring B-1 55-80 feet boring B-5
Quaternary Terrace Socastee Formation	$\boxed{Q_3}$	COASTAL TERRACE SANDS AND CLAYS	SP	SM	0 psf	34 deg	90 pci	NA	Not present in either boring B-1 or B-5
Cretaceous Deposits (Marine Shelf) Pee Dee Formation	$\boxed{K_2}$	CRETACEOUS RESIDUAL SANDS (PEE DEE CAPROCK)	SC	CH	0 psf	38 deg	125 pci	NA	45-55 ft boring B-1 Not present boring B-5
	$\boxed{K_3}$	VERY HARD CRETACEOUS CLAYS AND SILTS (PEE DEE FORMATION)	CL CH ML MH	SM SC	2000 psf.	16 deg	800 pci	0.005	55-100 ft boring B-1 80-120 ft boring B-5

An estimate of pile lateral capacity has been made for each pile type in terms of lateral shear. Pile lateral deflection was computed using an assumed 200 kip dead load component of the axial load applied vertically to the pile head in combination with the lateral load.

Pile top deflection as a function of depth for typical piles is shown in Figures 29 to 41. Deflection vs. applied shear is shown in Figures 45 to 48. The critical depth of the piles, beyond which pile length no longer influences pile lateral behavior, is estimated to be 26 feet based on plastic load parameters provided for the HP 14x73 pile types, respectively. The equivalent pile length, the length of freestanding pile, fixed against either translation or rotation at the base, having lateral deflection and bending similar to the embedded pile, is approximately as follows:

Table 9 – Estimated Lateral Resistance by Pile Type for Conceptual Design (Boring B-1)

Pile Type	Defl = ¼ in Lateral Resistance (kips) (ft)	Defl = 1 in Lateral Resistance (kips) (Equiv freestanding length) (ft)	Remarks
HP 12 x 53 at abutment	24 kips	62 kips (17.5 ft)	Strong Orientation Fixed Head
HP 12 x 53 at abutment	12 kips	37 kips (19.0 ft)	Weak Orientation Fixed Head
HP 14 x 73 at abutment	30 kips	84 kips (18.5 ft)	Strong Orientation Fixed Head
HP 14 x 73 at abutment	19 kips	50 kips (20.5 ft)	Weak Orientation Fixed Head
20-in OE Pipe	61 kips	145 kips (34 ft)	Fixed in cap at ground surface (interior bent)
18-in PSC	2 kips	5 kips (22 ft)	Freeheaded in pile bent cap 12 ft above ground surface (interior bent)
18-in PSC	20 kips	41 kips (18 ft)	Fixed in cap at ground surface (interior bent)
24-in PSC	3 kips	10 kips (24 ft)	Freeheaded in pile bent cap 12 ft above ground surface (interior bent)
24-in PSC	25 kips	60 kips (21 ft)	Fixed in cap at ground surface (interior bent)

Table 10 – Estimated Lateral Resistance by Pile Type for Conceptual Design (Boring B-5)

Pile Type	Defl = ¼ in Lateral Resistance (kips) (ft)	Defl = 1 in Lateral Resistance (kips) (Equiv freestanding length) (ft)	Remarks
HP 12 x 53 at abutment	24 kips	62 kips (10 ft)	Strong Orientation Fixed Head
HP 12 x 53 at abutment	12 kips	37 kips (9.5 ft)	Weak Orientation Fixed Head
HP 14 x 73 at abutment	30 kips	85 kips (14 ft)	Strong Orientation Fixed Head
HP 14 x 73 at abutment	20 kips	52 kips (10 ft)	Weak Orientation Fixed Head
20-in OE Pipe	100 kips	7200 kips (25 ft)	Fixed in cap at ground surface (interior bent)
18-in PSC	1 kip	3 kips (27 ft)	Freeheaded in pile bent cap 12 ft above ground surface (interior bent)
18-in PSC	20 kips	58 kips (15 ft)	Fixed in cap at ground surface (interior bent)
24-in PSC	2 kips	8 kips (28 ft)	Freeheaded in pile bent cap 12 ft above ground surface (interior bent)
24-in PSC	38 kips	105 kips (17 ft)	Fixed in cap at ground surface (interior bent)

7.1.3 Imposed Soil Loads due to Lateral Spreading

The soil profile at sounding C-2 was modeled to estimate lateral loads imposed on proposed pile foundations by lateral spreading of the upper soils towards a free face along the bank of the Pee Dee River. Lateral spreading computations in report section 5.7 imply movements ranging from 67 inches to 87 inches in the longitudinal direction at the banks.

Foundations were considered to extend to a height of 15 feet above the ground surface, embedded into the bridge superstructure and restrained against lateral deflection or rotation at the top under the movement imposed by the soils. The lateral spreading was represented by soils having a residual strength value of 400 psf for soft clays and/or liquefied sands comprising the bottom half of the spreading soil mass. We also assumed an upper, non-liquefied crust of material displaced by the lateral spreading, consisting of stiff clay with an undrained shear strength of 1000 psf. Results of analysis are discussed in section 7.3.4.

7.2 Driven Piles

Steel H piles will likely be preferred for abutment piers based on cost, constructability and overall performance issues. Other pile types, open ended pipe piles or prestressed concrete piles, may also be considered for interior bents. The following paragraphs provide preliminary analysis and recommendations for the bridges.

7.2.1 Prestressed Concrete Piles

Prestressed concrete piles have the advantage of being durable in almost any environment and are relatively easy to combine with a concrete superstructure. Generally they are the preferred type on trestle structures. In this case precast concrete piles were considered to have several disadvantages. First, since piles will be mainly end bearing on either the Pee Dee caprock or on the dense sands which immediately overlie the Pee Dee Formation, length of driving is anticipated to be variable and short driving will occur frequently owing to the presence of lenses or seams of dense sands in portions of the alignment underlain by the buried river channel sands.

18-inch square PSC piles fixed at the surface in a pier cap will achieve fixity just above the PeeDee caprock at the Yauhannah Lake bridge, but 24-inch square PSC piles will likely require several feet of penetration into the 100 bpf material to reach fixity. In this layer soil void ratios likely do not exceed 0.5, so high displacement piles are likely not able to penetrate these soils more than a few feet without predrilling or use of an H-pile stinger of sufficient length.

Where liquefaction prone sands are present, particularly north of the Pee Dee River, only limited lateral support will be offered to these piles by the upper 40 feet of strata once liquefaction occurs. During this period these piles may also be subject to large lateral loads where lateral spreading occurs adjacent to the Pee Dee River.

7.2.2 Steel Pipe Piles

Steel pipe piles may be driven open- or closed-ended and may be unfilled or filled with concrete. Steel pipe piles are advantageous due primarily to their relatively high vertical capacity and high section modulus for lateral loading. Pipe piles have relatively greater cost and are less convenient to splice than H-piles. Like H-piles, pipe piles are susceptible to corrosion unless protected and are not suitable in open water foundations unless cofferdams are used to extend piers below the scour line.

Installation of closed-ended pipe piles is typically highly difficult in dense or very stiff soils because of the high displacement required. Open-ended piles tend to form a plug in the bottom when penetrating hard lenses or very stiff cohesive soils and thus may act as closed-ended piles. In these cases penetration may be limited to shallow depth unless piles are first predrilled or reamed out during driving.

7.2.3 Steel H-Piles

Steel "H" piles are advantageous due primarily to their relative cost and ease for pile splicing and cutting. Piles are commonly paid for on an "in-place" basis and no charge is made for the length of steel cut off. This is a particular advantage where variable bearing depths may be encountered. The principal disadvantage associated with steel "H" piles are their relative small tip areas which results in a relatively small end bearing resistance in sandy soils since formation of a soil plug cannot be counted on in all cases to help with end bearing capacity development. Lateral load resistance is relatively low particularly where length to effective fixity of the pile is relatively long. Steel "H" piles are also more susceptible to corrosion unless sufficiently embedded below the surface. H-piles are typically more suited to abutments in coastal areas unless they can be incorporated into piers or caps. We recommend H-piles be equipped with driving points or shoes to facilitate penetration of the Pee Dee caprock materials.

7.3 Drilled Shafts (Interior Bent)

Drilled shafts socketed into the overconsolidated Pee Dee Formation will be the preferred option for heavily loaded bridge foundations, particularly where foundations are constructed over water. Drilled shafts have the advantage of being corrosion resistant and can be readily combined with the concrete superstructure. In addition, drilled shafts are easily adaptable to length changes and provide substantial lateral capacity under earthquake loads.

- Typically, relatively high compression capacities per shaft can be achieved in stiff to hard coastal plain strata.
- The installation of drilled shafts will eliminate the requirement for foundation caps and may require lighter construction traffic than steel H-piles.
- Drilled shafts typically require the use of a specialty contractor for installation. Also, being cast-in-place, they require increased inspection efforts during installation.

Drilled shafts bearing in coastal marl or limestone profiles offer the general advantages of generating relatively high axial load capacity (compressive and uplift) and lower cost per ton supported than driven piles, since less foundation elements are needed to support a given load and often foundation caps for bridges are reduced in size from those of pile caps. Drilled shafts also have high individual lateral load capacities, eliminating the need for battered foundation elements. Although it does not appear to be a significant factor at this site, drilled shafts can be installed with significantly lower vibration and noise levels than driven piles.

7.3.1 Axial Capacity and Bearing Depth

Structural columns at central bridge piers will generally be supported by individual shafts. Shafts will be sized for economy of concrete volume to support the relatively large loads over these bridge spans and to provide a relatively large cross section to resist lateral loads.

Drilled shafts were analyzed utilizing the soil strength properties described in Table 6. Constant diameter drilled shafts would derive their axial capacity primarily through shear transfer (side friction) or interlock between the concrete of the shaft and the soil interface, and from end bearing or point resistance at the base of the shaft in the deeper materials.

The amount of relative movement between the shaft perimeter and the bearing formation required to fully mobilize the ultimate strength of side shear transfer is usually on the order of 1 percent of the drilled shaft diameter, or ¾ to 1 inch. The end bearing or point resistance, however, reaches its ultimate load capacity upon tip movements on the order of 5 to 10 percent of the shaft tip diameter when bearing is in very dense or very hard soils. Settlements of up to 6 to 8 inches would be associated with full development of the end bearing. Consequently, we have utilized only partial end bearing (1/3 of ultimate value) in calculation of drilled shaft working capacities.

Ultimate shaft capacities were determined using the computer code SHAFT 4.0 for Windows. The ultimate capacities computed were then factored using a resistance factor of 0.7. No allowance was made for negative skin friction in the interior bents. For effective stress conditions after completion of the project and full dissipation of pore pressures induced by shaft construction, soils within the Pee Dee Formation were considered as fully drained. End bearing and side friction values were estimated using the β-relationship for cohesionless soils by Reese and O'Neill given in Section 4.6.5.1.4 of the Standard Specification for Highway Bridges (15th Edition).

60, 72 and 84-inch diameter shafts were analyzed for conceptual design. The upper and lower portion of the shaft corresponding to one shaft diameter was assumed as non-contributing to shaft skin friction capacity. All shafts were assumed straight-sided. Internally, the SHAFT program uses the method specified in FHWA and AASHTO guidelines. The method uses a formula that assumes a drilled-shaft construction with a concrete slump of 6 in. or higher. The method also considers that drilling slurry, if employed, should be such that it would not cause a weak layer of bentonite to develop at the wall of the excavation. The following formula is used internally to compute β values when using S.I. units:

$\beta = 1.5 - 0.135\ z.^{0.5}$

where

β = empirical value, limited to $1.2 > \beta > 0.25$
z = depth below ground surface, in units of ft.

Ultimate capacity vs. depth is depicted for representative bents in figures 26 through 28. Computed shaft capacities suggest generally similar contributions to capacity by end bearing and side friction within the Pee Dee Formation. Contribution by skin friction is typically on the order of one-half to two-thirds of total capacity at the ultimate capacity. Depending on capacity, shafts must extend through the Pee Dee caprock to bear in stiff to hard clayey materials occurring below a depth of 60 to 70 feet below the ground surface over most of the alignment. At that depth end bearing values at ultimate capacity computed for drained (effective stress) loading are typically 70 kips/square foot.

7.3.2 Uplift Capacity

Uplift applied to drilled shafts is expected to be minimal. The result of our compression capacity analysis was utilized to estimate the shaft uplift capacities. Allowable static shaft uplift capacities will be equal to some fraction of the available skin frictional component of axial compressive loads. We have assumed 70 percent of the ultimate shaft side resistance will be available to resist uplift. Based on our analysis side friction available to resist uplift forces will be approximately 200 tons for a 72-inch diameter; 90 foot long shaft socketed a minimum of 10 to 20 feet into stiff to hard Pee Dee Formation soils at the Pee Dee River Bridge.

In addition, the estimated self-weight of the shafts may be added to the uplift capacities obtained from side friction.

7.3.3 Drilled Shaft Foundation Settlement Potential

Settlement of individual drilled shaft foundation elements will be essentially given by elastic shortening of the concrete shaft under the working static axial loads. Tip movement of the drilled shaft will be negligible at working loads. For static loads, shaft displacement occurring as tip movement required to mobilize the shaft side resistance capacity was neglected in load deflection curves for a 72-inch diameter.

7.3.4 Lateral Loads Applied by Bridge Displacement

Soil-pier interaction was analyzed using the LPILE program developed by Reese and previously described. The analysis was performed for the 60, 72 and 84-inch diameter shafts considered in design. Lateral subgrade modulus values presented in the table below were used for lateral load analyses under static and cyclic loading. All seismic lateral analyses incorporated a stiffness value (EI) representing an uncracked shaft section.

Shaft loading at the ground surface was modeled using coefficients of horizontal subgrade reaction for the soils penetrated by the piles as summarized in Table 8.

Shafts were modeled as free headed, applying shear and moment loads at the butt though a range of butt displacement from 0 to 6 inches, and calculating deflection versus applied shear and slope angle versus applied shear individually.

Table 11 – Estimated Lateral Resistance by Drilled Shafts for Conceptual Design

Pile Type	Defl = ¼ in Lateral Resistance (kips) (ft)	Defl = 1 in Lateral Resistance (kips) (Equiv freestanding length) (ft)	Remarks
Yauhannah Lake	60 dia 47 kips	60 dia 110 kips (43 ft)	Free head at surface
	72 dia 65 kips	72 dia 160 kips (49 ft)	
	84 dia 90 kips	84 dia 240 kips (50 ft)	
Pee Dee Main River	60 dia 5 kips	60 dia 24 kips (53 ft)	Free head with stickup above el. -35 feet
	72 dia 10 kips	72 dia 55 kips (67 ft)	
	84 dia 20 kips	84 dia 70 kips (70 ft)	
Pee Dee Overflow	60 dia 60 kips	60 dia 205 kips (25 ft)	Free head at surface
	72 dia 90 kips	72 dia 300 kips (29 ft)	
	84 dia 110 kips	84 dia 403 kips (35 ft)	

Depth to fixity of the shafts was estimated by inspection of load-displacement diagrams. Point-of-fixity is defined as the second point of zero deflection of equal load increment curve. Maximum depth to effective fixity of the tip is estimated to be 60 feet relative to the cut-off elevation. Assuming the embedded shaft to be represented as an unsupported cantilevered shaft fixed against translation or rotation at the base, the equivalent length of the cantilevered shaft giving roughly the same load vs deflection at the top would be 53 to 70 feet below the water surface depending on diameter.

Imposed soil loads due to lateral spreading were computed for typical drilled shaft using the soil profile at sounding C-2 as before. Imposed loads ranged from 2200, 2400 to 2700 lb/in for 60-inch, 72-inch and 84-inch diameters, respectively. This load would be applied per unit length in inches, at the base of a triangular load in the non-liquefied soils. Within the liquefied zone, imposed loads by the residual strength of the liquefied sands range from 1200, 1400 to 2250 lb/in, applied as a uniform load per unit length in inches on 60, 72, and 84-inch diameters, respectively. Load vs depth is shown for each shaft diameter in Figure 55.

7.3.5 Installation Recommendations

The wet construction method utilizing either temporary casing or the slurry displacement method will be applicable at shaft locations on land with high groundwater levels, cohesionless overburden soils, and potentially permeable bearing zones. The wet method utilizes a relatively thick bentonite or attapulgite clay mineral drilling fluid to stabilize the drilled hole below the groundwater level. Use of this method should be anticipated by the contractor in all shafts, particularly in sands encountered to a depth of approximately 40 feet.

In addition, the contractor may need to leave casing permanently in place in all shafts, particularly if reinforcing steel stick-up restricts ability be withdraw the casing. In this case provision should be made for permanent casing in each shaft. We recommend a minimum of 10 feet of permanent casing for estimation of quantities for bid documents.

8.0 LIMITATIONS OF REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations between the borings will not become evident until construction. If variations appear evident, then we will need to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

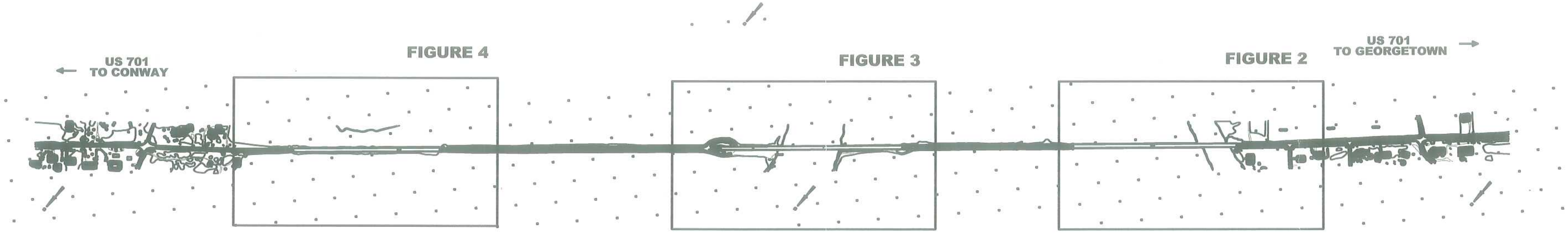

FIGURE NO: 1

BORING LOCATION PLAN

US 701 YAUHANNAH

GEORGETOWN/HORRY COUNTY
SOUTH CAROLINA

JOB NO: 1611-04-459

S&ME
ENVIRONMENTAL SERVICES
ENGINEERING TESTING

134 SUBER ROAD
COLUMBIA, SC 29210
(803) 561-9024

SCALE: NA
CHECKED BY: TSR
DRAWN BY: JCL
DATE: 4/5/05

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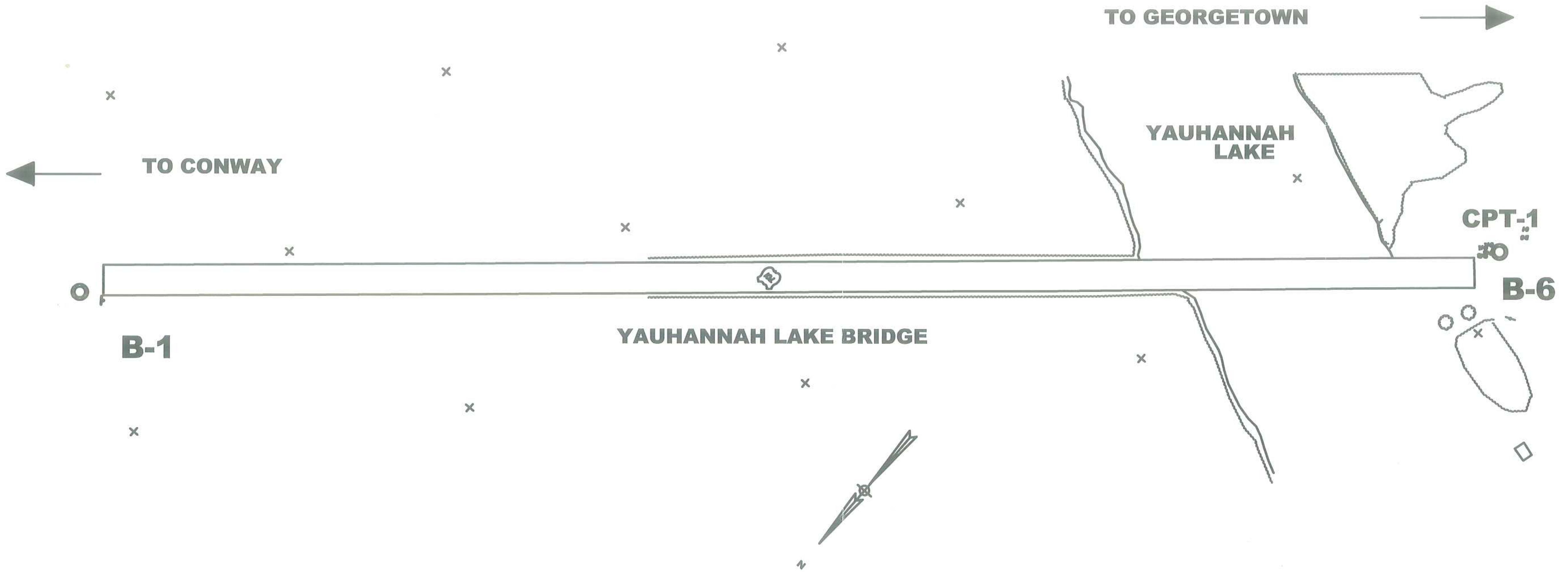

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CHECKED BY: JCL
DRAWN BY: JCL
DATE: 3/21/05

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134 SUBER ROAD
COLUMBIA, SC 29210
(803) 561-9024

BORING LOCATION PLAN
US 701 OVER YAUHANNAH
GEORGETOWN/HORRY COUNTY
SOUTH CAROLINA

JOB NO: 1611-04-459

FIGURE NO: 2

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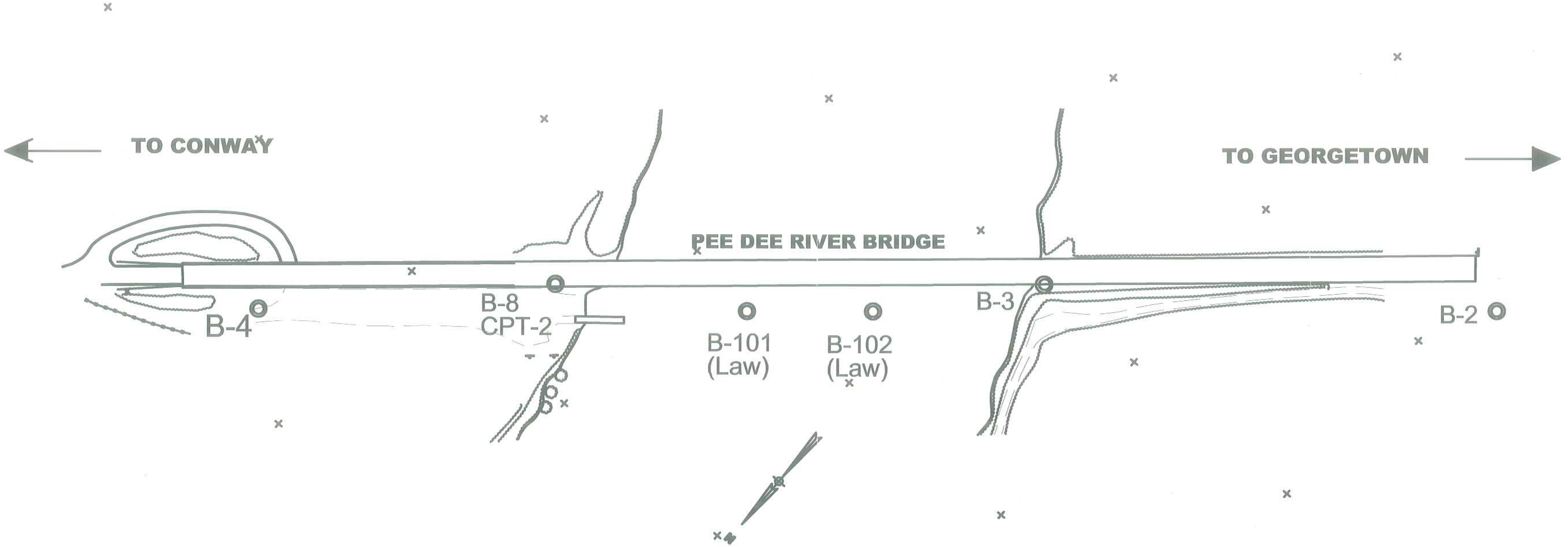

FIGURE NO: 3

BORING LOCATION PLAN

US 701 OVER YAUHANNAH
GEORGETOWN/HORRY COUNTY
SOUTH CAROLINA

JOB NO: 1611-04-459

S&ME
ENVIRONMENTAL SERVICES
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134 SUBER ROAD
COLUMBIA, SC 29210
(803) 561-9024

SCALE: 1"=100'
CHECKED BY: JCL
DRAWN BY: JCL
DATE: 3/21/05

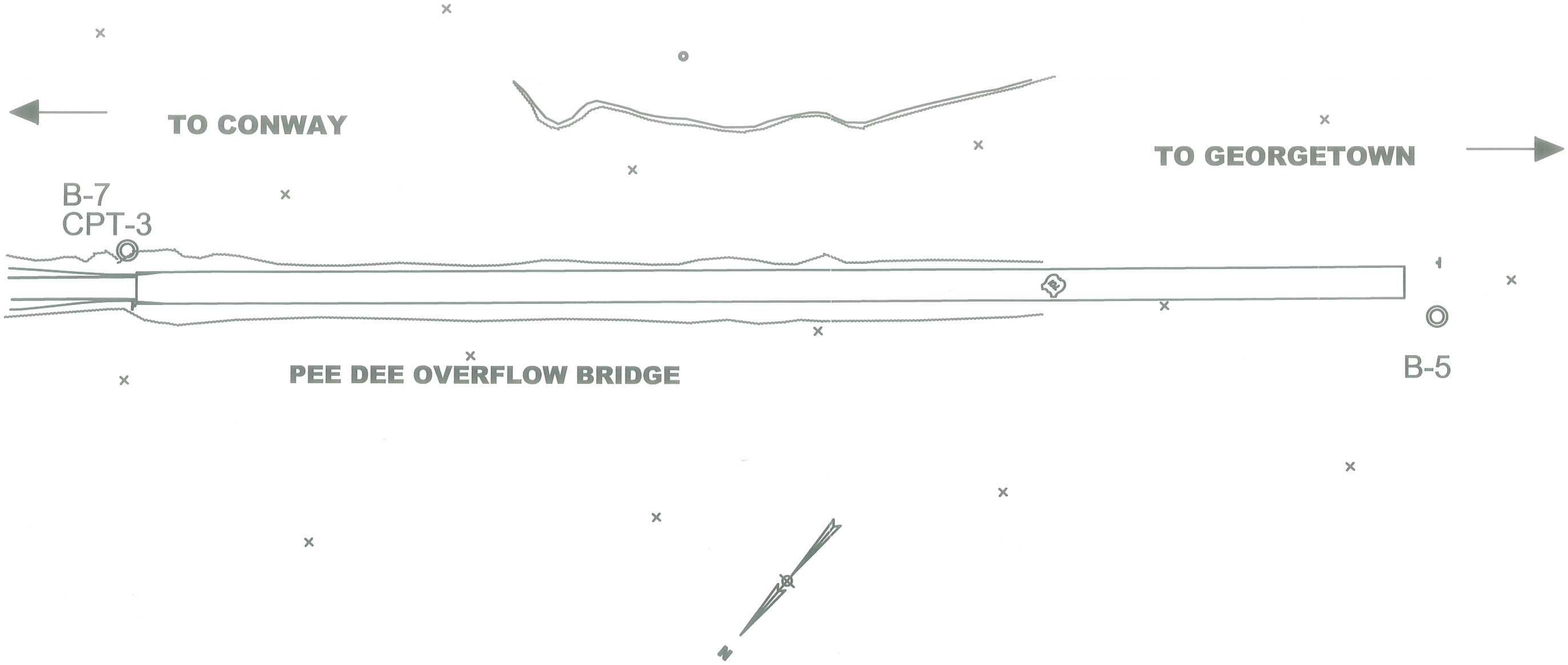

FIGURE NO: 4

BORING LOCATION PLAN
US 701 OVER YAUHANNAH
GEORGETOWN/HORRY COUNTY
SOUTH CAROLINA

JOB NO: 1611-04-459

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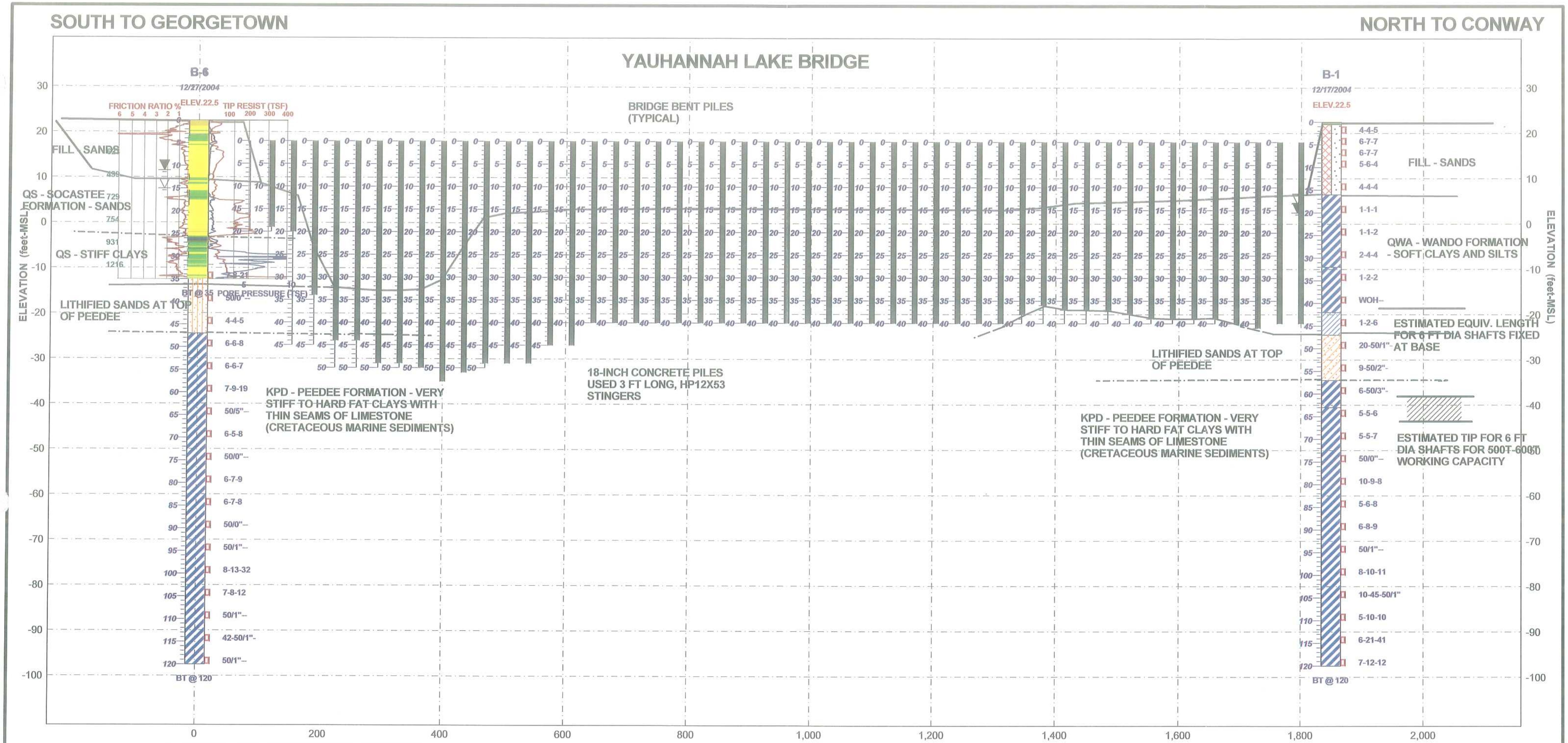
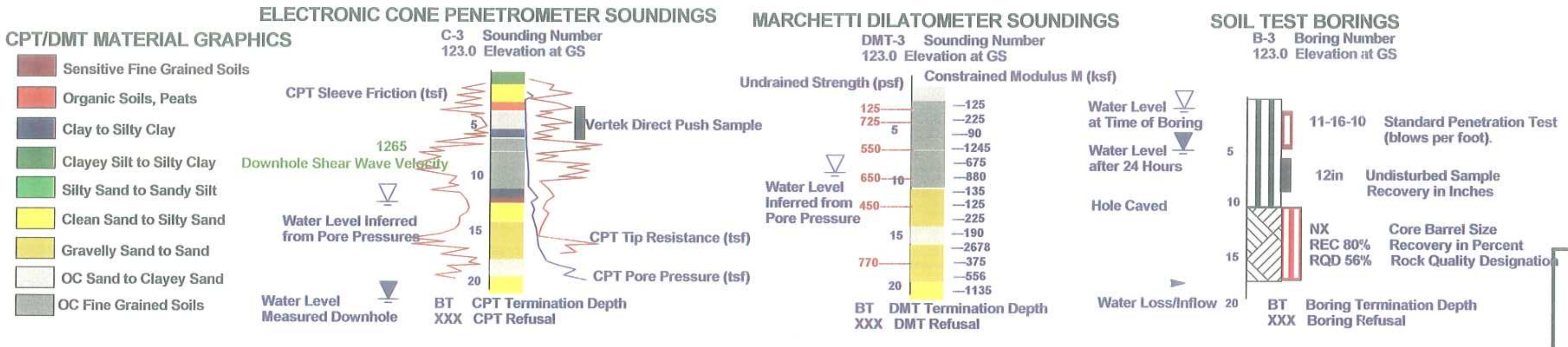
134 SUBER ROAD
COLUMBIA, SC 29210
(803) 561-9024

SCALE: 1"=100'
CHECKED BY: JCL
DRAWN BY: JCL
DATE: 3/21/05





LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

▨	Fill	▨	CL, Low Plasticity Clay	▨	SM, Silty Sand
🟩	Topsoil	▨	SC, Clayey Sand		
▨	CH, High Plasticity Clay	⁘	SP, Poorly-graded Sand		

The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

FIG. 5

SUBSURFACE PROFILE A-A'

PROJECT: US 701 Bridges at Yauhannah

LOCATION:

JOB NO:
1611-04-569

DATE:
3/11/05

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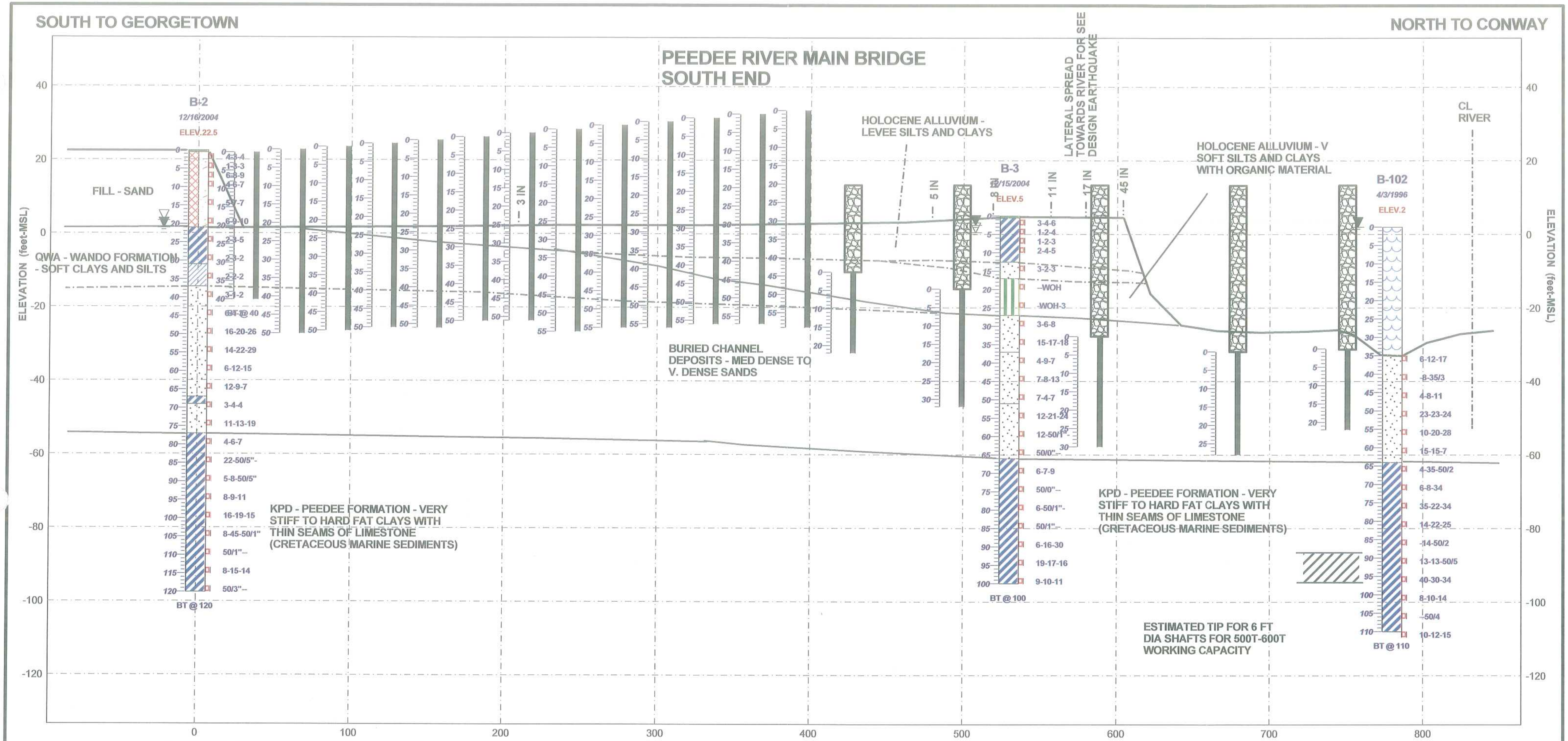

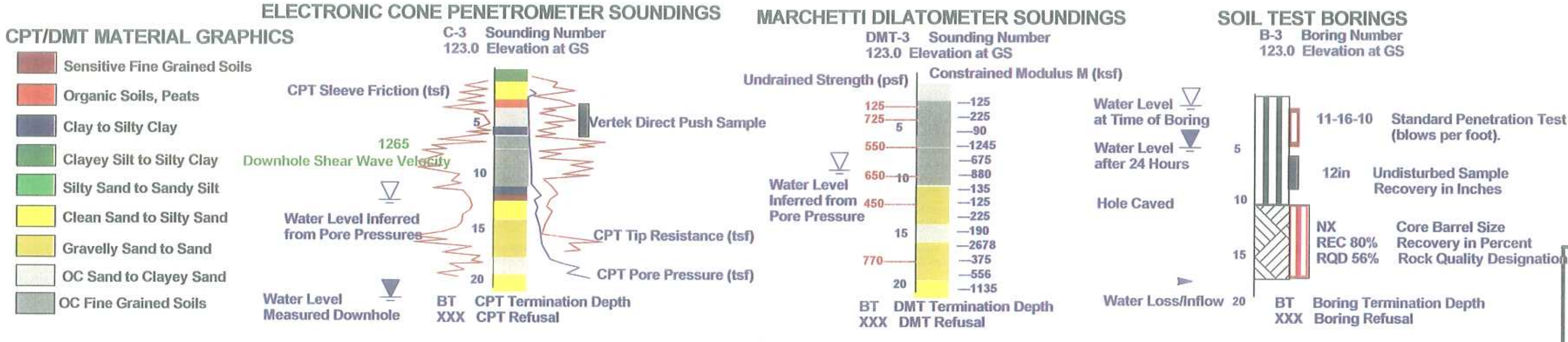

LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

▨	Fill	▨	CH, High Plasticity Clay	⁛	SP/SC, Poorly-graded Sand with Clay
≈	Water	🟩	Topsoil	▥	MH, High Plasticity Silt
⁛	SP, Poorly-graded Sand	▨	CL, Low Plasticity Clay		

FIG. 6

The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

SUBSURFACE PROFILE **B-B'**

PROJECT: US 701 Bridges at Yauhannah

LOCATION:

JOB NO:
1611-04-569

DATE:
3/11/05

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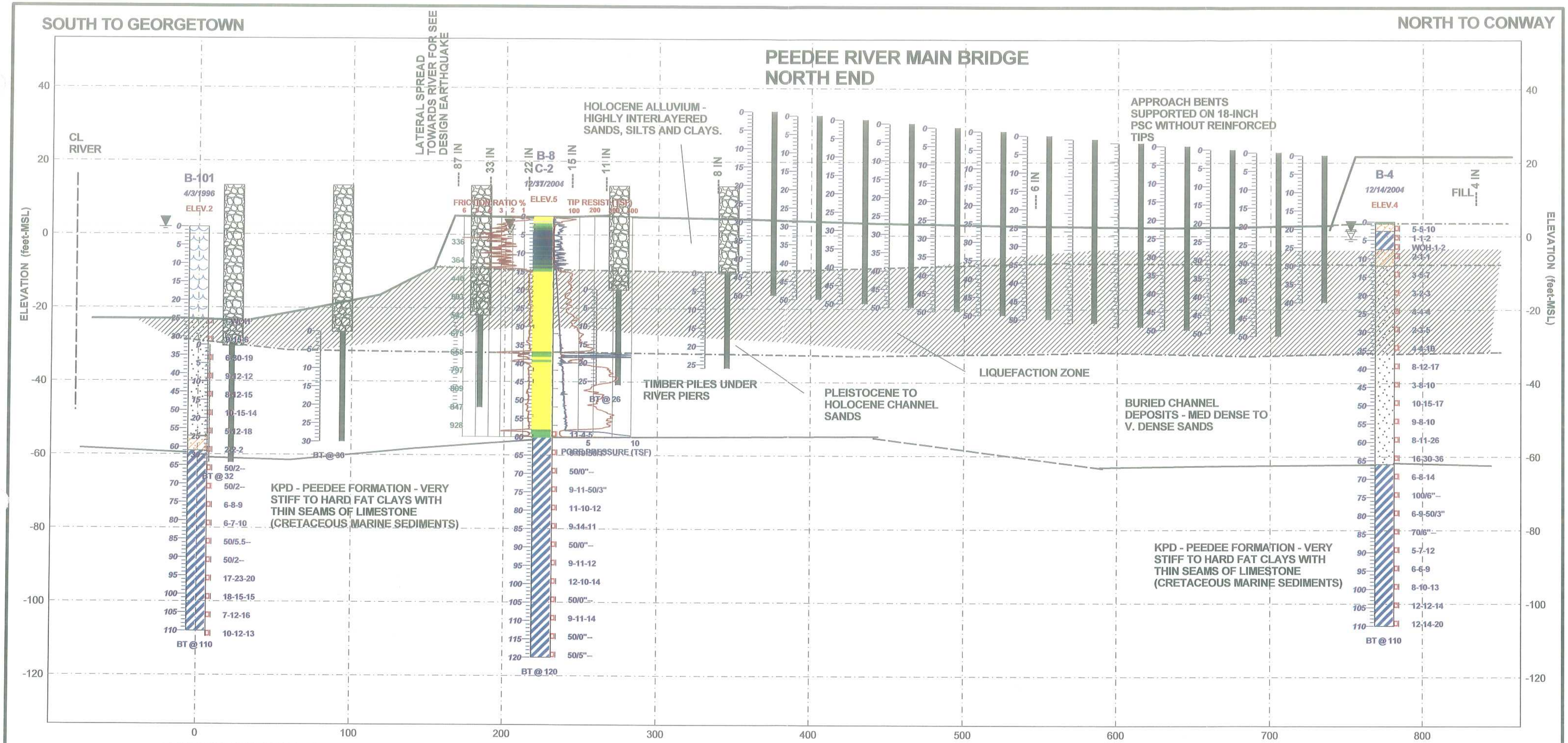
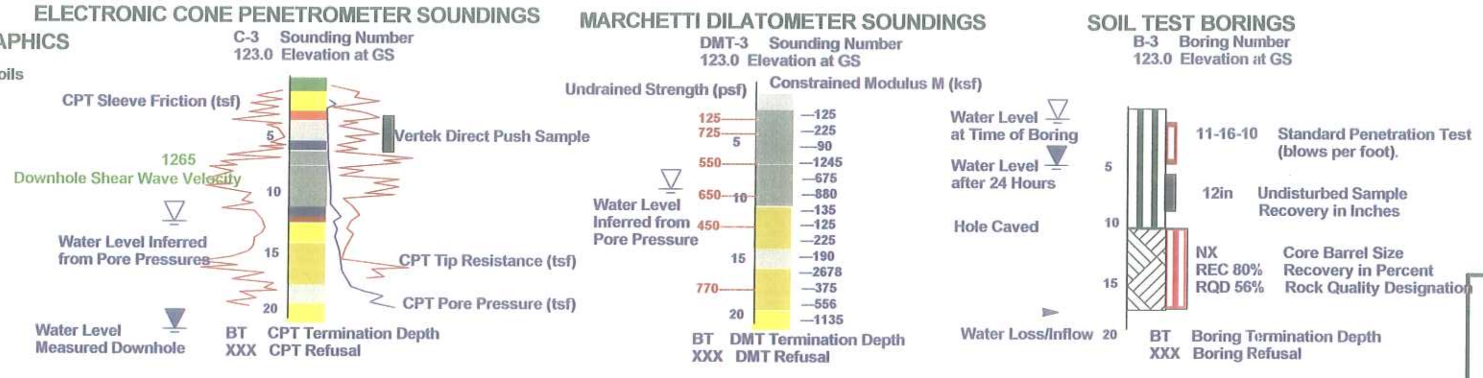


CPT/DMT MATERIAL GRAPHICS

- 🟥 Sensitive Fine Grained Soils
- 🟥 Organic Soils, Peats
- 🟦 Clay to Silty Clay
- 🟩 Clayey Silt to Silty Clay
- 🟩 Silty Sand to Sandy Silt
- 🟨 Clean Sand to Silty Sand
- 🟨 Gravelly Sand to Sand
- ☐ OC Sand to Clayey Sand
- ▨ OC Fine Grained Soils



LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

☐ Water	▨ CH, High Plasticity Clay
☐ SP, Poorly-graded Sand	🟩 Topsoil
▨ SC, Clayey Sand	

FIG 7

The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

SUBSURFACE PROFILE C-C'

PROJECT: US 701 Bridges at Yauhannah

LOCATION:

JOB NO:
1611-04-569

DATE:
3/11/05

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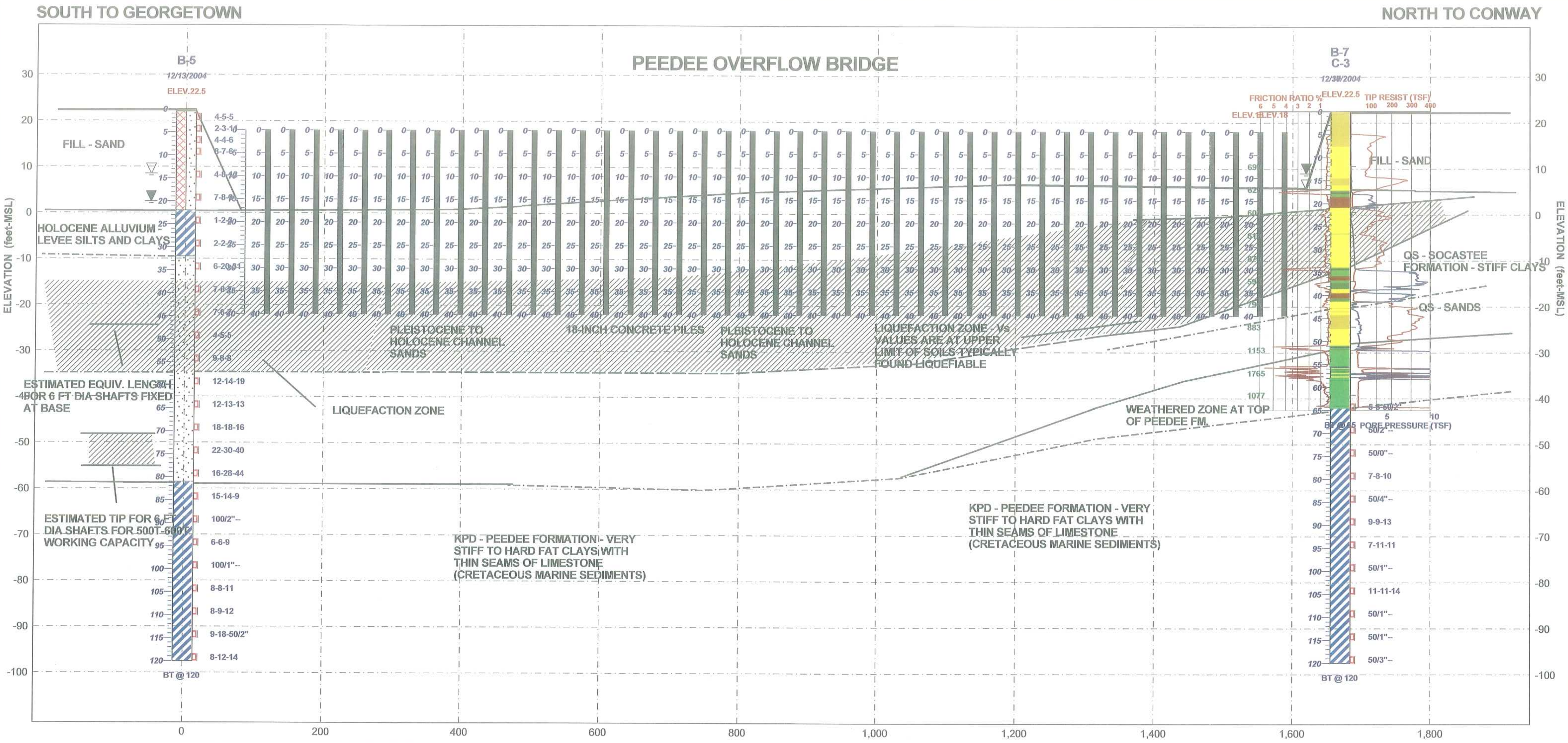
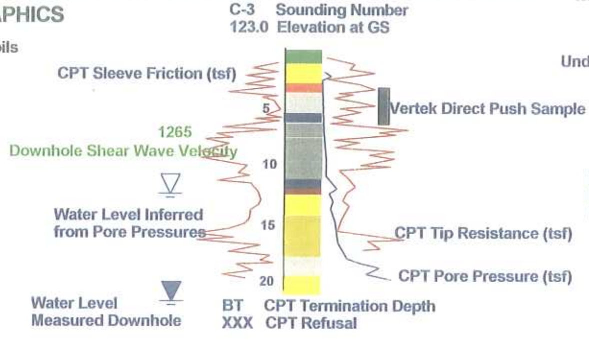


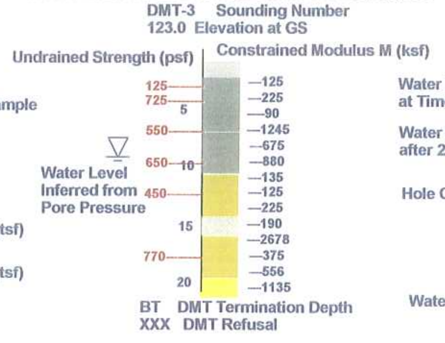
CPT/DMT MATERIAL GRAPHICS

- 🟥 Sensitive Fine Grained Soils
- 🟥 Organic Soils, Peats
- 🟦 Clay to Silty Clay
- 🟩 Clayey Silt to Silty Clay
- 🟩 Silty Sand to Sandy Silt
- 🟨 Clean Sand to Silty Sand
- 🟨 Gravelly Sand to Sand
- ⬜ OC Sand to Clayey Sand
- ⬜ OC Fine Grained Soils

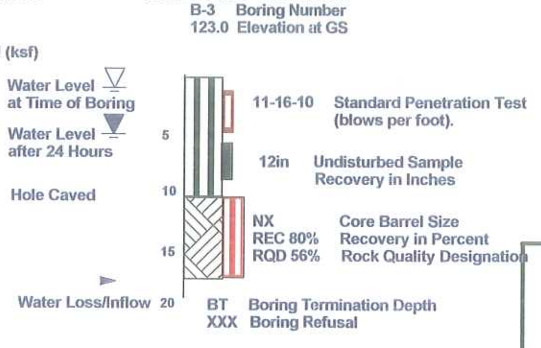
ELECTRONIC CONE PENETROMETER SOUNDINGS



MARCHETTI DILATOMETER SOUNDINGS



SOIL TEST BORINGS



LEGEND OF MATERIAL GRAPHICS for SOIL TEST BORINGS

- ▨ Fill
- 🟩 Topsoil
- ▨ CH, High Plasticity Clay
- ⁘ SP, Poorly-graded Sand

The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

FIG 8

SUBSURFACE PROFILE D-D'

PROJECT: US 701 Bridges at Yauhannah

LOCATION:

JOB NO: 1611-04-569

DATE: 3/11/05

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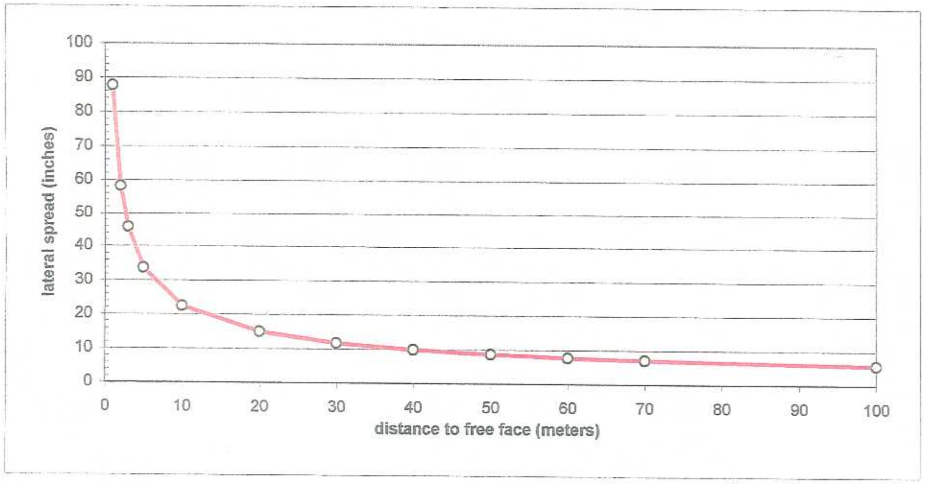
Log(L) = -16.713+1.532M-1.406logR-0.012R+0.592log(W)+.54log(t15)+3.413log(100-F) – 0.795(log(d15+.1))

L	0	meters in distance to free face from point of reference
H	8	Height of free face in N<15 bpf material (meters)
T15	**10**	cumulative thickness of granular soils with N < 15 bpf
D50	**0.5**	mean grain size in mm
F	**5**	average fines content in percent
M	**7.3**	earthquake magnitude
R	**64**	horizontal distance from seismic energy source, in km
S	**0.1**	ground slope in percent

800.00	W
1.718629	0.592log W
11.1836	1.532M
2.539489	1.406logR
0.768	0.013R
-0.338	0.338logS
0.54	0.54logH
8.953155	3.413log(100-F)
-0.17637	0.795log(D50+.1)

log(delta L)	
delta L	meters
	inches

L	meters											
1	2	3	5	10	20	30	40	50	60	70	100	200
800.00	400.00	266.67	160.00	80.00	40.00	26.67	20.00	16.00	13.33	11.43	8.00	4.00
1.718629	1.54042	1.436173	1.304839	1.126629	0.94842	0.844173	0.77021	0.712839	0.665964	0.626331	0.534629	0.35642
11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836
2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489
0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768
0	0	0	0	0	0	0	0	0	0	0	1	2
0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971
-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637
0.348081	0.169871	0.065625	-0.06571	-0.243919	-0.422129	-0.526375	-0.600339	-0.65771	-0.704585	-0.744217	-0.835919	-1.014129
2.228849	1.478669	1.163121	0.859588	0.57027	0.37833	0.297594	0.250993	0.219933	0.197431	0.180212	0.145909	0.096799
87.7275	58.2004	45.78044	33.83339	22.44583	14.89107	11.71332	9.879074	8.656563	7.77088	7.093126	5.742959	3.810009



NORTH ABUTMENT, PEEDEE RIVER BRIDGE
COMPUTATION OF LATERAL SPREAD
USING BARTLETT AND YOUD EQUATION
FOR HORIZONTAL MOVEMENT TO FREE-FACE

COMPUTATION OF LATERAL SPREAD
USING BARTLETT AND YOUD EQUATION
FOR HORIZONTAL MOVEMENT TO FREE-FACE
S&ME PROJ. 1611-04-569

S&ME

FIG 9

Log(L) = -16.713+1.532M-1.406logR-0.012R+0.592log(W)+.54log(t15)+3.413log(100-F) – 0.795(log(d15+.1))

L	0	meters in distance to free face from point of reference
H	8	Height of free face in N<15 bpf material (meters)
T15	3	cumulative thickness of granular soils with N < 15 bpf
D50	**0.5**	mean grain size in mm
F	5	average fines content in percent
M	**7.3**	earthquake magnitude
R	**64**	horizontal distance from seismic energy source, in km
S	**0.1**	ground slope in percent

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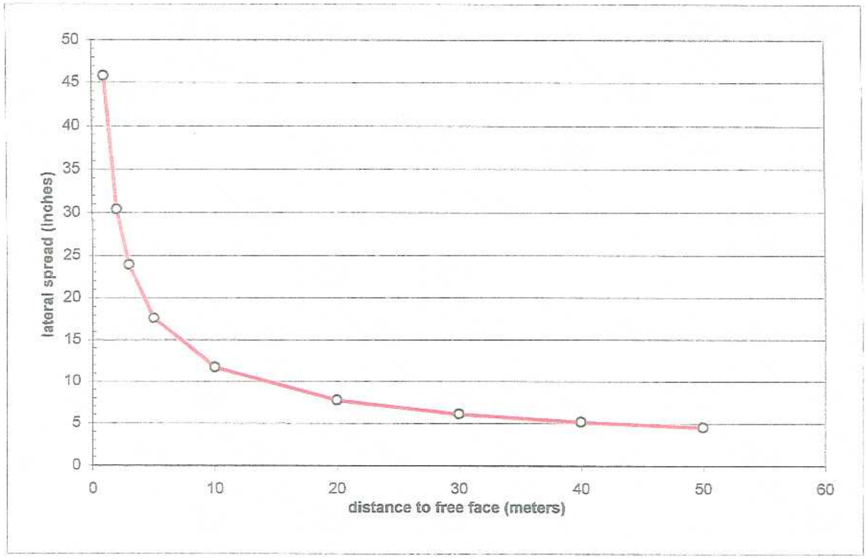
11.1836	1.532M
2.539489	1.406logR
0.768	0.013R
-0.338	0.338logS
0.257645	0.54logH
8.953155	3.413log(100-F)
-0.17637	0.795log(D50+.1)

log(delta L)		
delta L	meters	
	inches	

L	meters										
1	2	3	5	10	20	30	40	50	60	70	100
800.00	400.00	266.67	160.00	80.00	40.00	26.67	20.00	16.00	13.33	11.43	8.00
1.718629	1.54042	1.436173	1.304839	1.126629	0.94842	0.844173	0.77021	0.712839	0.665964	0.626331	0.534629
11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836	11.1836
2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489	2.539489
0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768	0.768
0	0	0	0	0	0	0	0	0	0	0	1
0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645	0.257645
6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971	6.749971
-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637	-0.17637
0.065726	-0.11248	-0.21673	-0.34806	-0.52627	-0.70448	-0.80873	-0.88269	-0.94006	-0.98694	-1.02657	-1.11827
1.163392	0.771821	0.607114	0.448679	0.297664	0.197477	0.155335	0.131011	0.114798	0.103053	0.094065	0.07616
45.79111	30.37886	23.89601	17.66001	11.71605	7.772692	6.114	5.156579	4.518466	4.056166	3.702399	2.997652



SOUTH ABUTMENT, PEEDEE RIVER BRIDGE
COMPUTATION OF LATERAL SPREAD
USING BARTLETT AND YOUD EQUATION
FOR HORIZONTAL MOVEMENT TO FREE-FACE

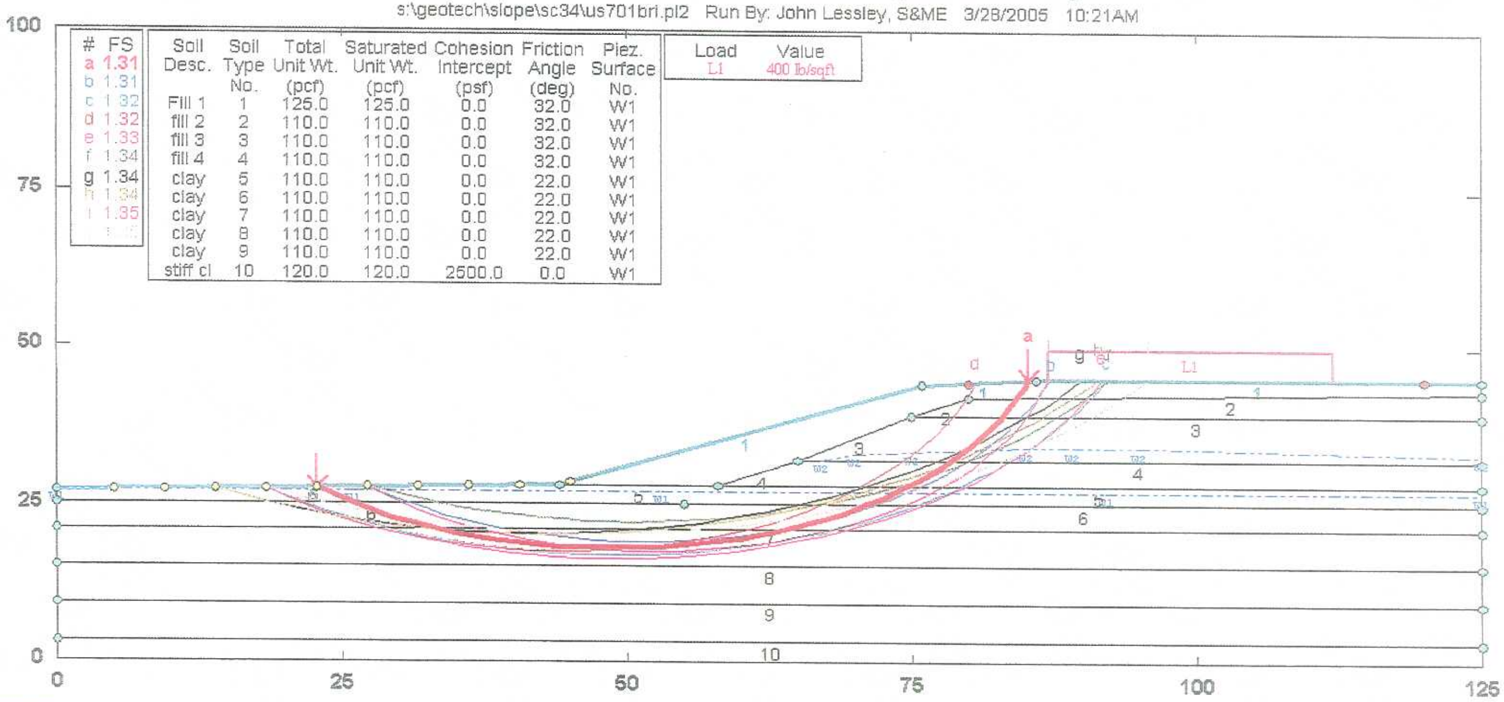
US 701 CROSSING OF THE GREAT PEEDEE RIVER,
PEEDEE OVERFLOW AND YAUHANNAH LAKE
S&ME PROJ. 1611-04-569

S&ME

FIG 10

us 701 bridge 1 north end Static global stability

s:\geotech\slope\sc34\us701bri.pl2 Run By: John Lessley, S&ME 3/28/2005 10:21AM



#	FS
a	1.31
b	1.31
c	1.32
d	1.32
e	1.33
f	1.34
g	1.34
h	1.34
i	1.35
[illegible]	[illegible]

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	0.0	22.0	W1
clay	6	110.0	110.0	0.0	22.0	W1
clay	7	110.0	110.0	0.0	22.0	W1
clay	8	110.0	110.0	0.0	22.0	W1
clay	9	110.0	110.0	0.0	22.0	W1
stiff cl	10	120.0	120.0	2500.0	0.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.31
Safety Factors Are Calculated By The Modified Bishop Method

YAUHANNAH LAKE GLOBAL STABILITY STATIC CONDITION
NORTH ABUTMENT

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 3/28/2005

S&ME
ENVIRONMENTAL SERVICES • ENGINEERING • TESTING

ABUTMENT SLOPE GLOBAL STABILITY

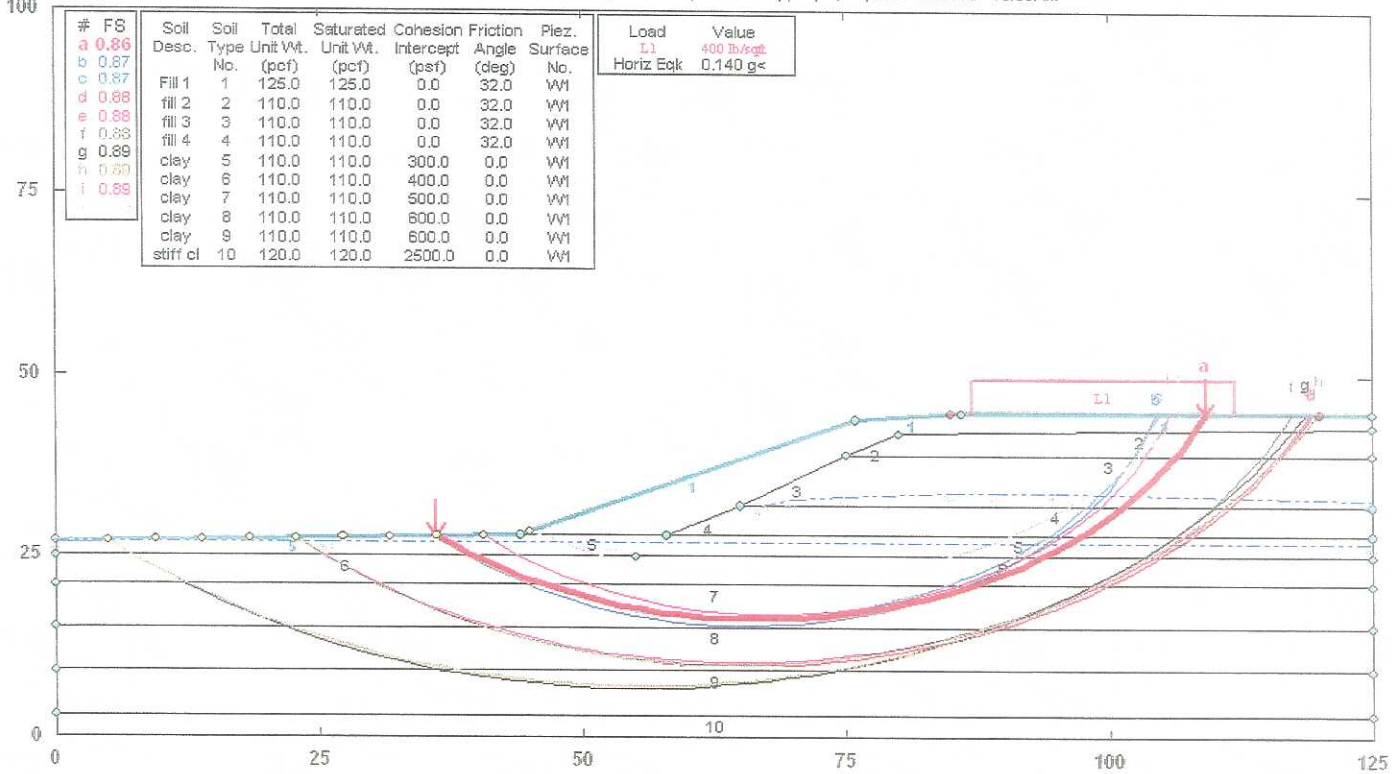
US 701 OVER GREAT PEEDEE RIVER, PEEDEE
OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 11

us 701 bridge 1 north end Static global stability - earthquake

s:\geotech\slope\sc34\us701b1e.pl2 Run By: Tim S. Ray, PE, S$ME, Inc. 4/1/2005 10:08AM



#	FS
a	0.86
b	0.87
c	0.87
d	0.88
e	0.88
f	0.88
g	0.89
h	0.89
i	0.89

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	300.0	0.0	W1
clay	6	110.0	110.0	400.0	0.0	W1
clay	7	110.0	110.0	500.0	0.0	W1
clay	8	110.0	110.0	600.0	0.0	W1
clay	9	110.0	110.0	600.0	0.0	W1
stiff cl	10	120.0	120.0	2500.0	0.0	W1

Load	Value
L1	400 lb/sqft
Horiz Eqk	0.140 g<

STABL6H FSmin=0.86

Safety Factors Are Calculated By The Modified Bishop Method

STED

YAUHANNAH LAKE GLOBAL STABILITY UNDER SEISMIC
NORTH ABUTMENT

SCALE: NTS

CHECKED BY: JCL

DRAWN BY:

DATE: 4/1/2005

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ABUTMENT SLOPE GLOBAL STABILITY

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

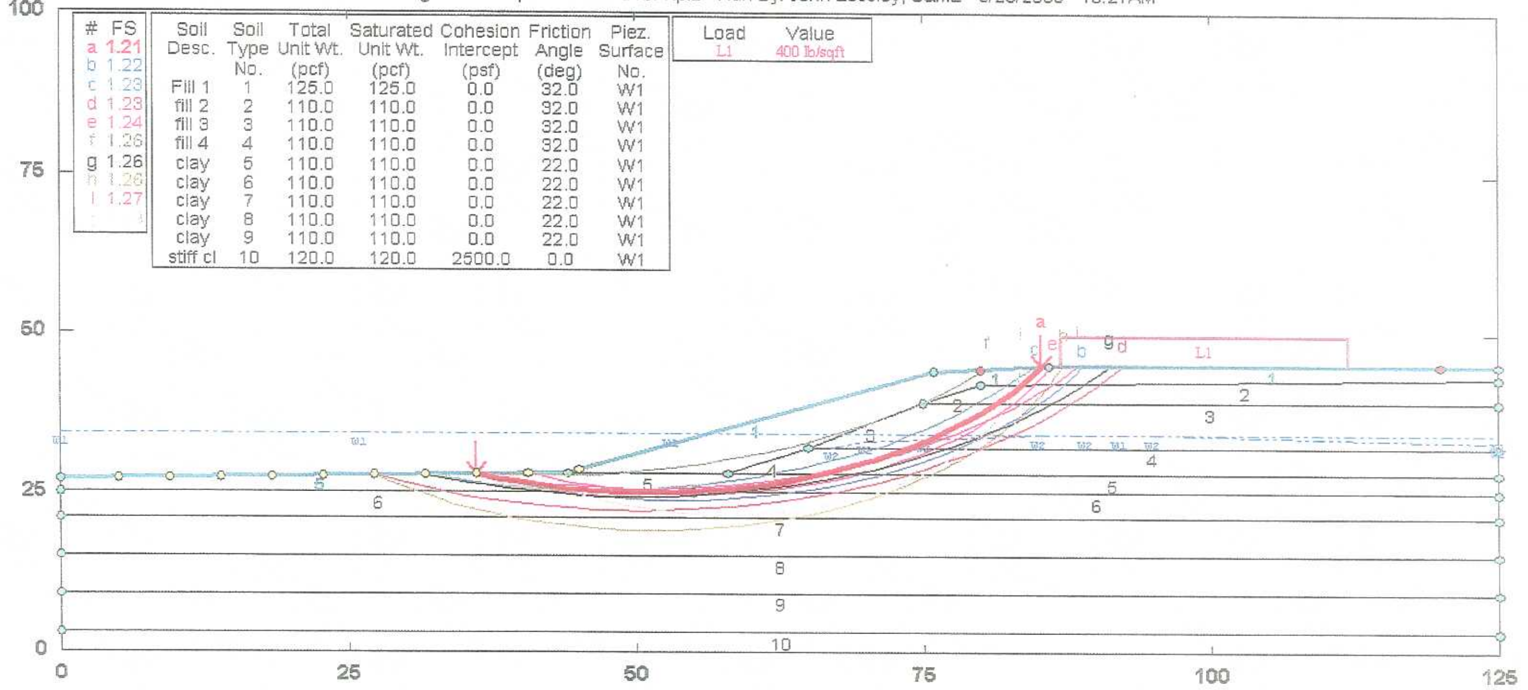
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 12

us 701 bridge 1 north end Static global stability - partial flood

s:\geotech\slope\sc34\us701br1.pl2 Run By: John Lessley, S&ME 3/28/2005 10:27AM



#	FS
a	1.21
b	1.22
c	1.23
d	1.23
e	1.24
f	1.26
g	1.26
h	1.26
i	1.27

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	0.0	22.0	W1
clay	6	110.0	110.0	0.0	22.0	W1
clay	7	110.0	110.0	0.0	22.0	W1
clay	8	110.0	110.0	0.0	22.0	W1
clay	9	110.0	110.0	0.0	22.0	W1
stiff cl	10	120.0	120.0	2500.0	0.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.21

Safety Factors Are Calculated By The Modified Bishop Method

YAUHANNAH LAKE GLOBAL STABILITY PARTIAL FLOOD CONDITION
NORTH ABUTMENT

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 3/28/2005

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US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

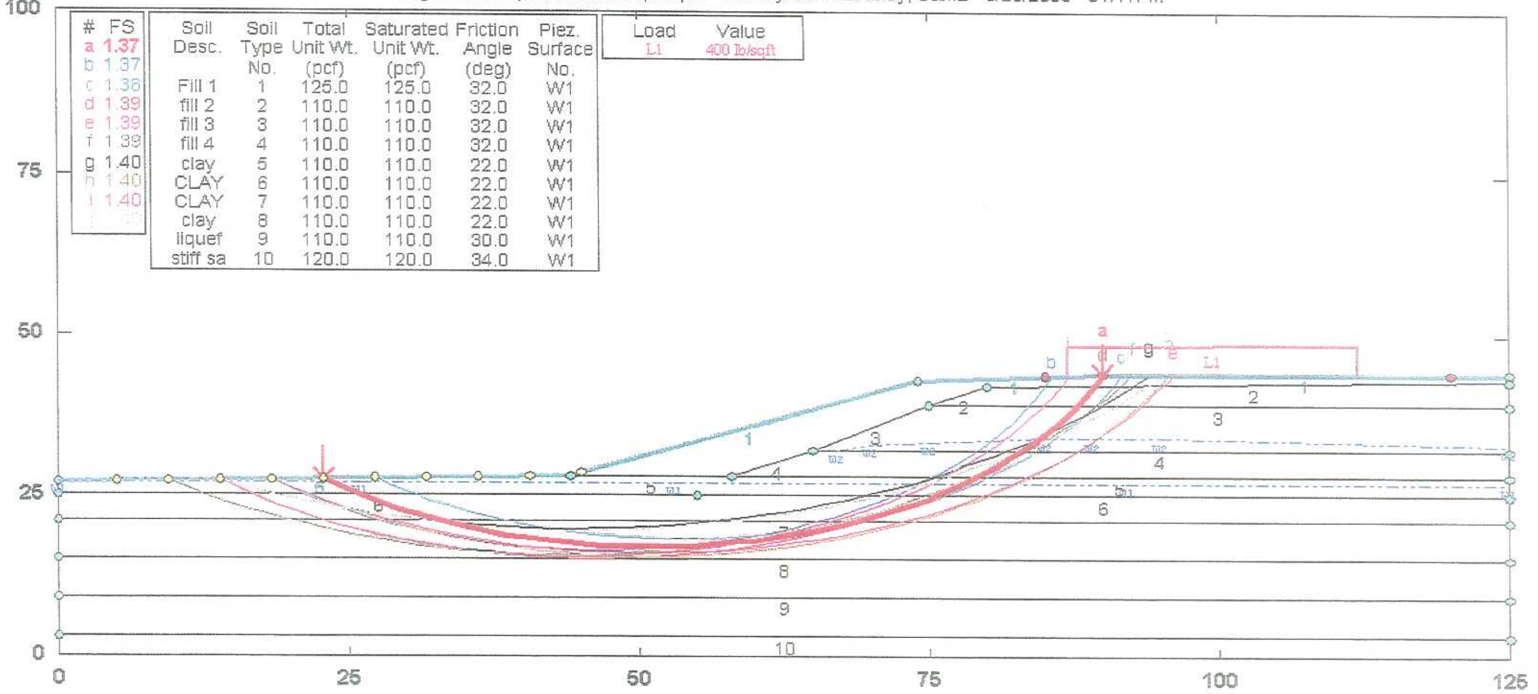
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 13

us 701 PeeDee River South End Global Stability - static

s:\geotech\slope\sc34\us701p2s.pl2 Run By: John Lessley, S&ME 3/28/2005 01:17PM



#	FS
a	1.37
b	1.37
c	1.38
d	1.39
e	1.39
f	1.39
g	1.40
h	1.40
i	1.40

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	22.0	W1
CLAY	6	110.0	110.0	22.0	W1
CLAY	7	110.0	110.0	22.0	W1
clay	8	110.0	110.0	22.0	W1
liquef	9	110.0	110.0	30.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.37
Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE RIVER GLOBAL STABILITY STATIC CONDITION
SOUTH ABUTMENT

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 3/28/2005

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US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

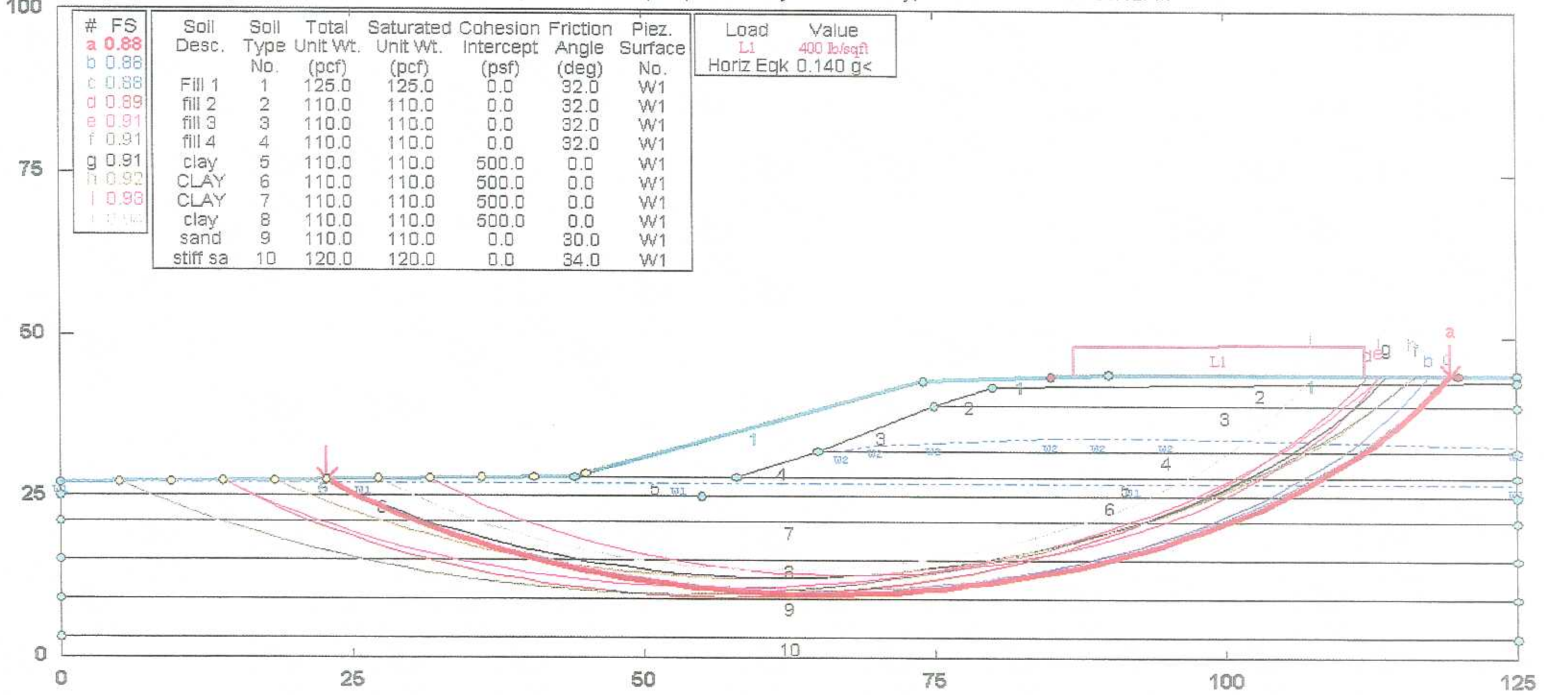
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 14

us 701 PeeDee River South End Global Stability - earthquake

s:\geotech\slope\sc34\us701p2e.pl2   Run By: John Lessley, S&ME   3/28/2005   01:12PM



#	FS
a	0.88
b	0.88
c	0.88
d	0.89
e	0.91
f	0.91
g	0.91
h	0.92
i	0.93
[illegible]	[illegible]

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	500.0	0.0	W1
CLAY	6	110.0	110.0	500.0	0.0	W1
CLAY	7	110.0	110.0	500.0	0.0	W1
clay	8	110.0	110.0	500.0	0.0	W1
sand	9	110.0	110.0	0.0	30.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft
Horiz Eqk	0.140 g<

STABL6H   FSmin=0.88

Safety Factors Are Calculated By The Modified Bishop Method

STED

PEEDEE RIVER GLOBAL STABILITY EARTHQUAKE CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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ABUTMENT SLOPE GLOBAL STABILITY

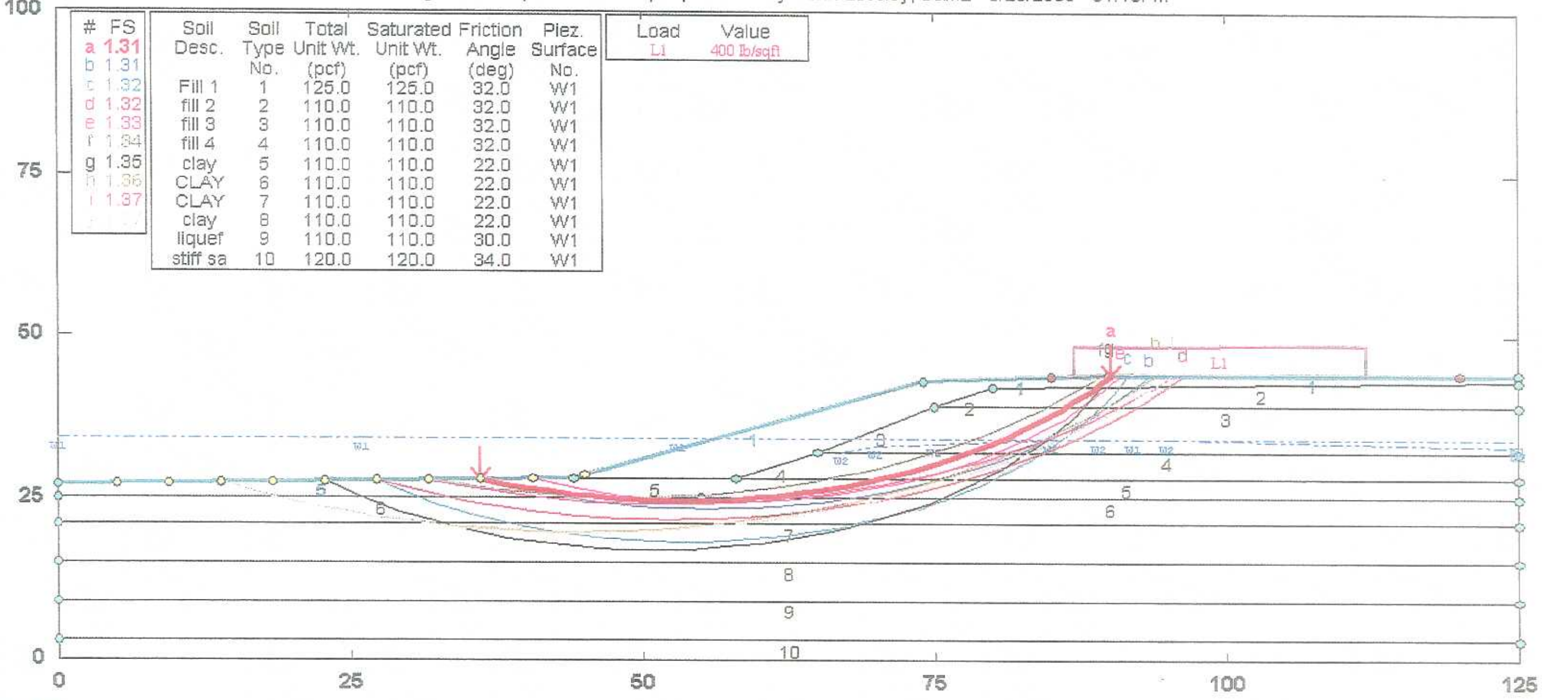
US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 15

us 701 PeeDee River South End Global Stability - flood

s:\geotech\slope\sc34\us701p2f.pl2 Run By: John Lessley, S&ME 3/28/2005 01:18PM



#	FS
a	1.31
b	1.31
c	1.32
d	1.32
e	1.33
f	1.34
g	1.35
h	1.36
i	1.37

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	22.0	W1
CLAY	6	110.0	110.0	22.0	W1
CLAY	7	110.0	110.0	22.0	W1
clay	8	110.0	110.0	22.0	W1
liquef	9	110.0	110.0	30.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.31

Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE RIVER GLOBAL STABILITY PARTIAL FLOOD CONDITION
SOUTH ABUTMENT

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 3/28/2005

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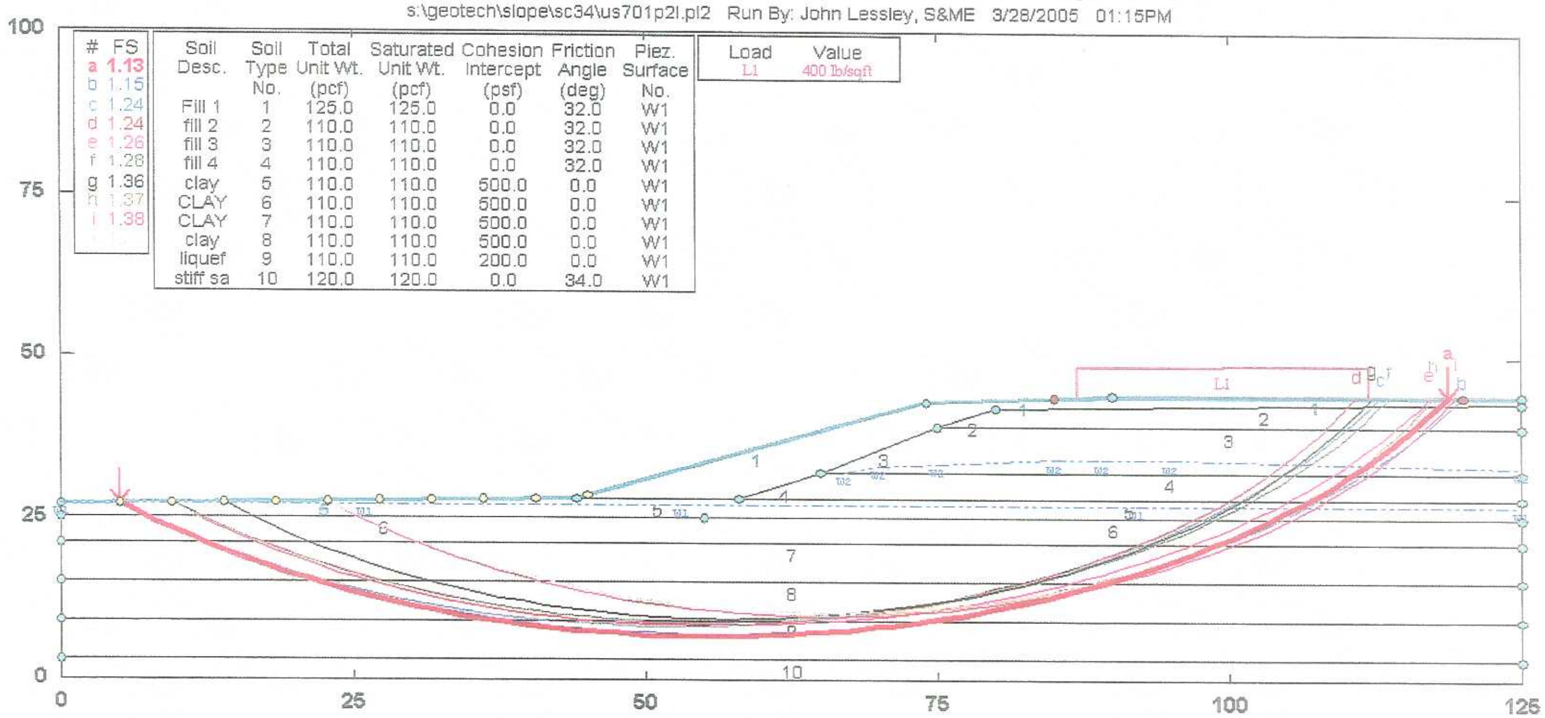
US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 16

us 701 PeeDee River South End Global Stability - liquefaction

s:\geotech\slope\sc34\us701p2l.pl2 Run By: John Lessley, S&ME 3/28/2005 01:15PM



#	FS
a	1.13
b	1.15
c	1.24
d	1.24
e	1.26
f	1.28
g	1.36
h	1.37
i	1.38

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	500.0	0.0	W1
CLAY	6	110.0	110.0	500.0	0.0	W1
CLAY	7	110.0	110.0	500.0	0.0	W1
clay	8	110.0	110.0	500.0	0.0	W1
liquef	9	110.0	110.0	200.0	0.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.13

Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE RIVER GLOBAL STABILITY LIQUEFIED CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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ABUTMENT SLOPE GLOBAL STABILITY

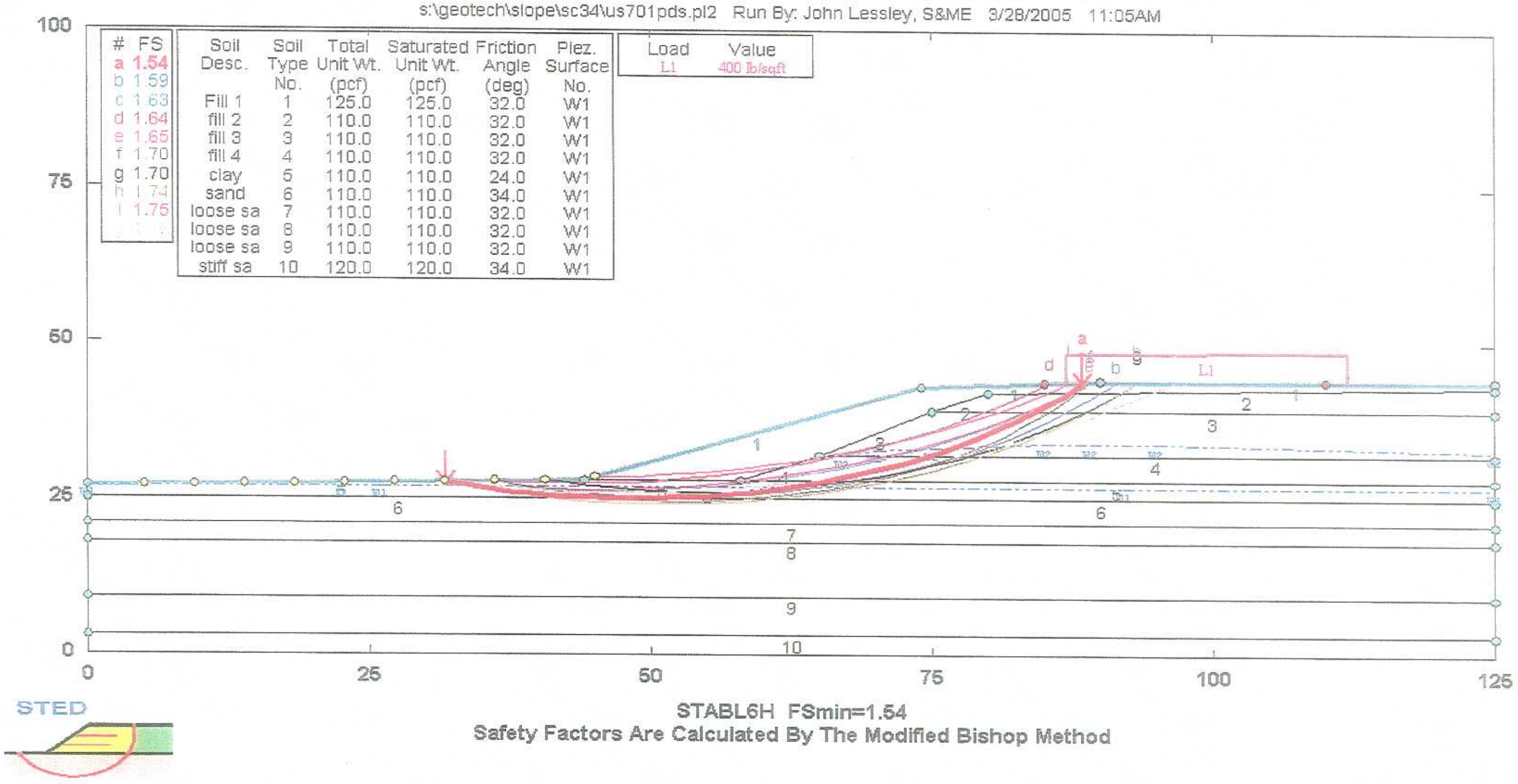
US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO.
17

us 701 main bridge north end Global Stability -LONG TERM STATIC

s:\geotech\slope\sc34\us701pds.pl2 Run By: John Lessley, S&ME 3/28/2005 11:05AM



#	FS
a	1.54
b	1.59
c	1.63
d	1.64
e	1.65
f	1.70
g	1.70
h	1.74
i	1.75

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	24.0	W1
sand	6	110.0	110.0	34.0	W1
loose sa	7	110.0	110.0	32.0	W1
loose sa	8	110.0	110.0	32.0	W1
loose sa	9	110.0	110.0	32.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STABL6H FSmin=1.54
Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE RIVER GLOBAL STABILITY STATIC CONDITION
NORTH ABUTMENT

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 3/28/2005

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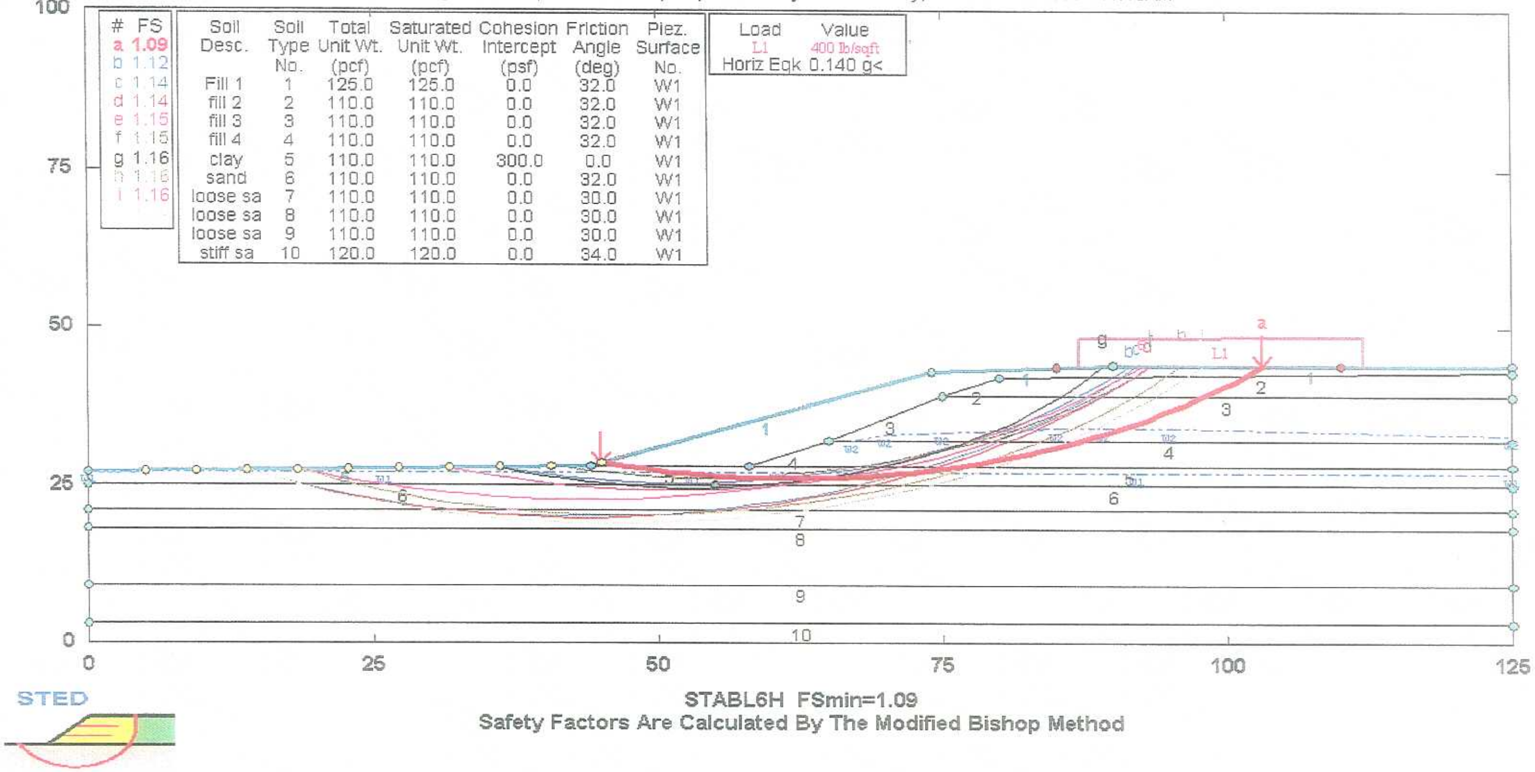
US 701 OVER GREAT PEEDEE RIVER, PEEDEE
OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO.
18

us 701 main bridge north end Global Stability - Seismic

s:\geotech\slope\sc34\us701pde.pl2 Run By: John Lessley, S&ME 3/28/2005 11:13AM



#	FS
a	1.09
b	1.12
c	1.14
d	1.14
e	1.15
f	1.15
g	1.16
h	1.16
i	1.16

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	300.0	0.0	W1
sand	6	110.0	110.0	0.0	32.0	W1
loose sa	7	110.0	110.0	0.0	30.0	W1
loose sa	8	110.0	110.0	0.0	30.0	W1
loose sa	9	110.0	110.0	0.0	30.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft
Horiz Eqk	0.140 g<

STABL6H FSmin=1.09
Safety Factors Are Calculated By The Modified Bishop Method

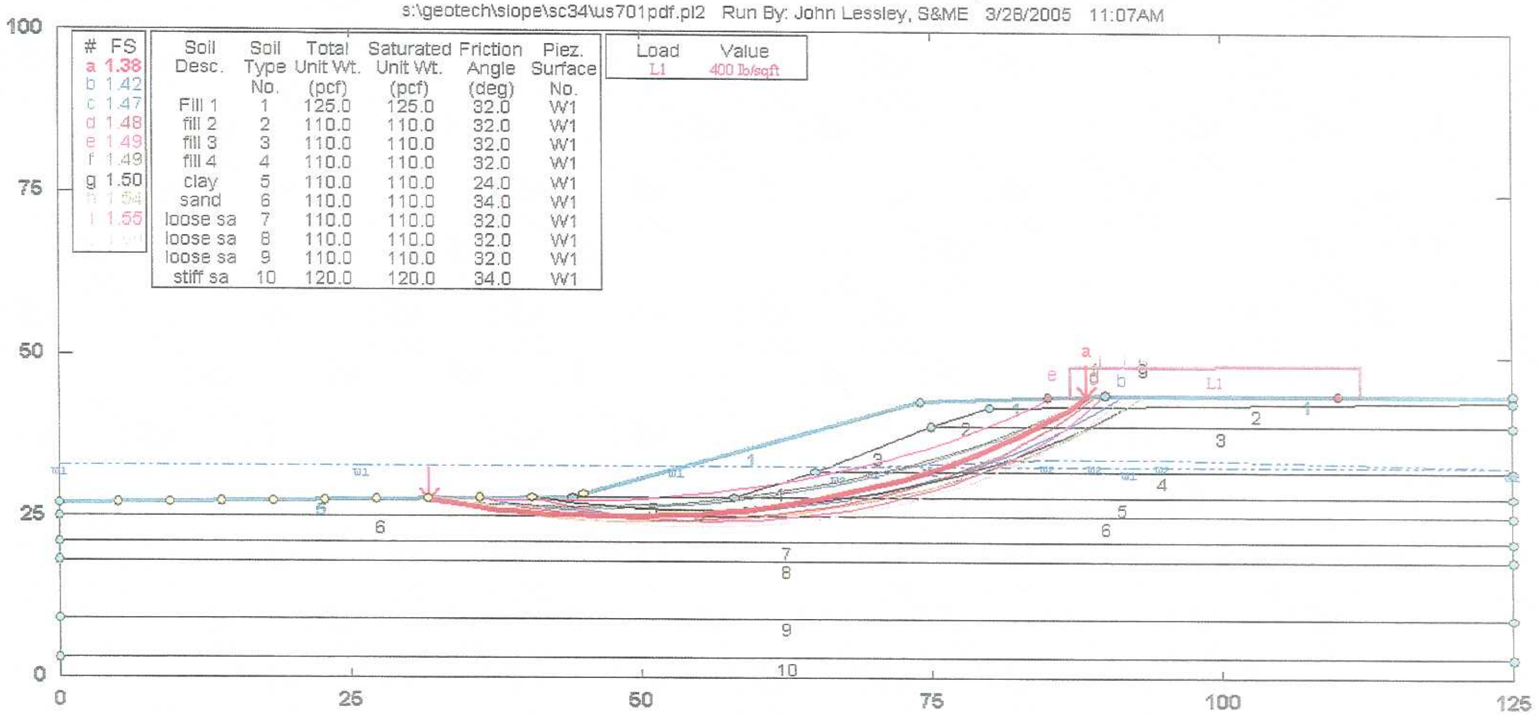
PEEDEE RIVER GLOBAL STABILITY SEISMIC CONDITION

NORTH ABUTMENT

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	ABUTMENT SLOPE GLOBAL STABILITY	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	19
DRAWN BY:			
DATE: 3/28/2005		JOB NO. 1611-04-569	

us 701 main bridge north end Global Stability - Partial Flood

s:\geotech\slope\sc34\us701pdf.pl2 Run By: John Lessley, S&ME 3/28/2005 11:07AM



#	FS
a	1.38
b	1.42
c	1.47
d	1.48
e	1.49
f	1.49
g	1.50
h	1.54
i	1.55

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	24.0	W1
sand	6	110.0	110.0	34.0	W1
loose sa	7	110.0	110.0	32.0	W1
loose sa	8	110.0	110.0	32.0	W1
loose sa	9	110.0	110.0	32.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.38

Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE RIVER GLOBAL STABILITY PARTIAL FLOOD
NORTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

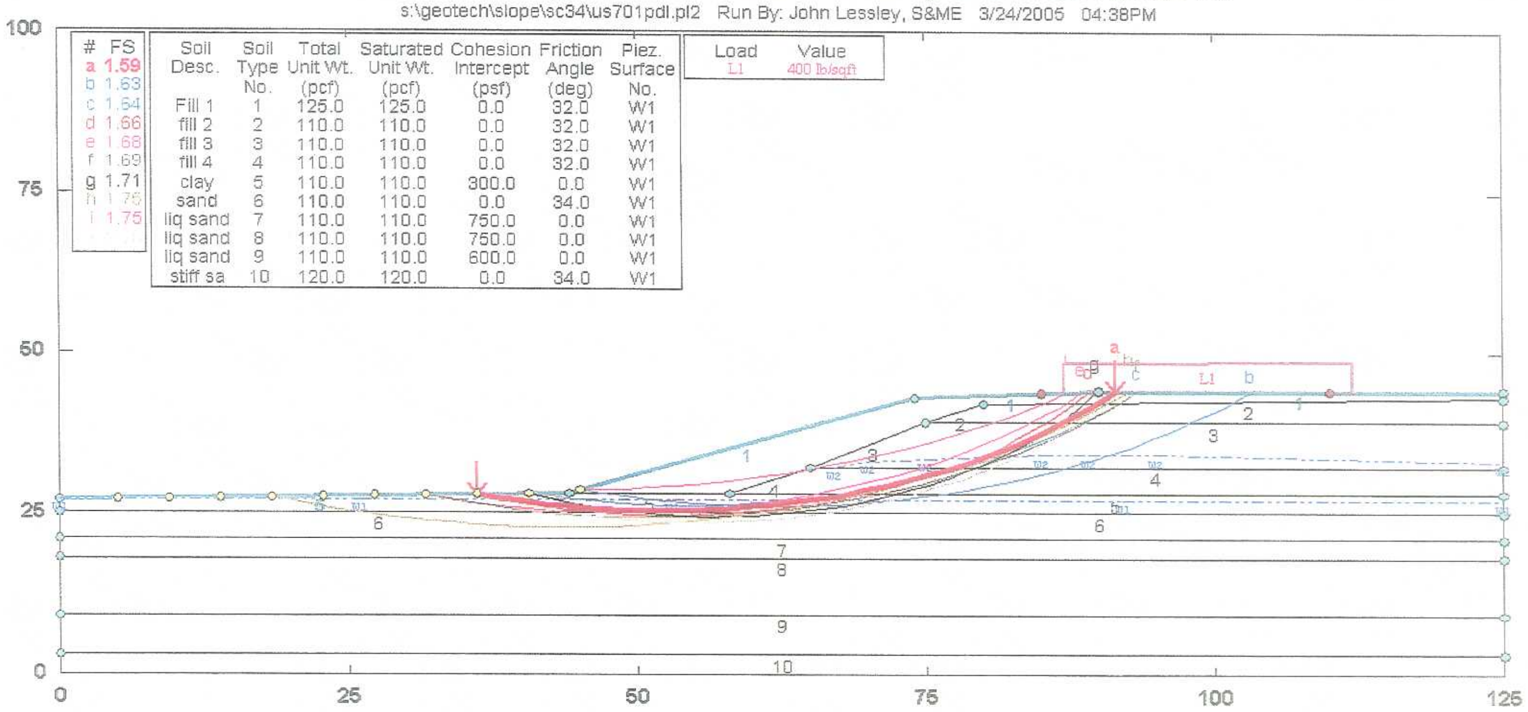
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 20

us 701 main bridge north end Global Stability under earthquake liq

s:\geotech\slope\sc34\us701pdl.pl2 Run By: John Lessley, S&ME 3/24/2005 04:38PM



#	FS
a	1.59
b	1.63
c	1.64
d	1.66
e	1.68
f	1.69
g	1.71
h	1.75
i	1.75

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	300.0	0.0	W1
sand	6	110.0	110.0	0.0	34.0	W1
liq sand	7	110.0	110.0	750.0	0.0	W1
liq sand	8	110.0	110.0	750.0	0.0	W1
liq sand	9	110.0	110.0	600.0	0.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft

STABL6H FSmin=1.59
Safety Factors Are Calculated By The Modified Bishop Method

STED

PEEDEE RIVER GLOBAL STABILITY LIQUEFIED CONDITION
NORTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	4/1/2005

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ABUTMENT SLOPE GLOBAL STABILITY

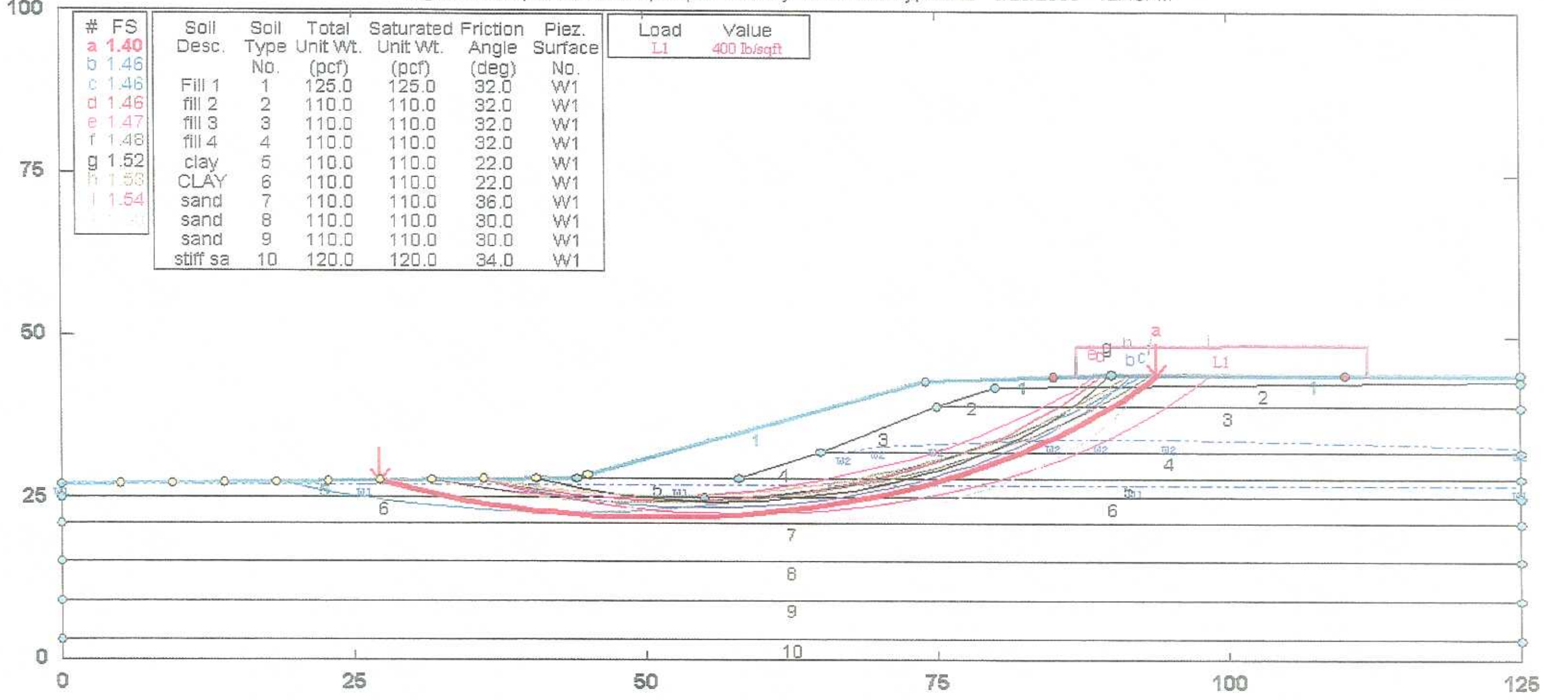
US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 21

us 701 PeeDee Overflow South End Global Stability - static

s:\geotech\slope\sc34\us701pos.pl2 Run By: John Lessley, S&ME 3/28/2005 12:48PM



#	FS
a	1.40
b	1.46
c	1.46
d	1.46
e	1.47
f	1.48
g	1.52
h	1.53
i	1.54

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	22.0	W1
CLAY	6	110.0	110.0	22.0	W1
sand	7	110.0	110.0	36.0	W1
sand	8	110.0	110.0	30.0	W1
sand	9	110.0	110.0	30.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.40

Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE OVERFLOW GLOBAL STABILITY STATIC CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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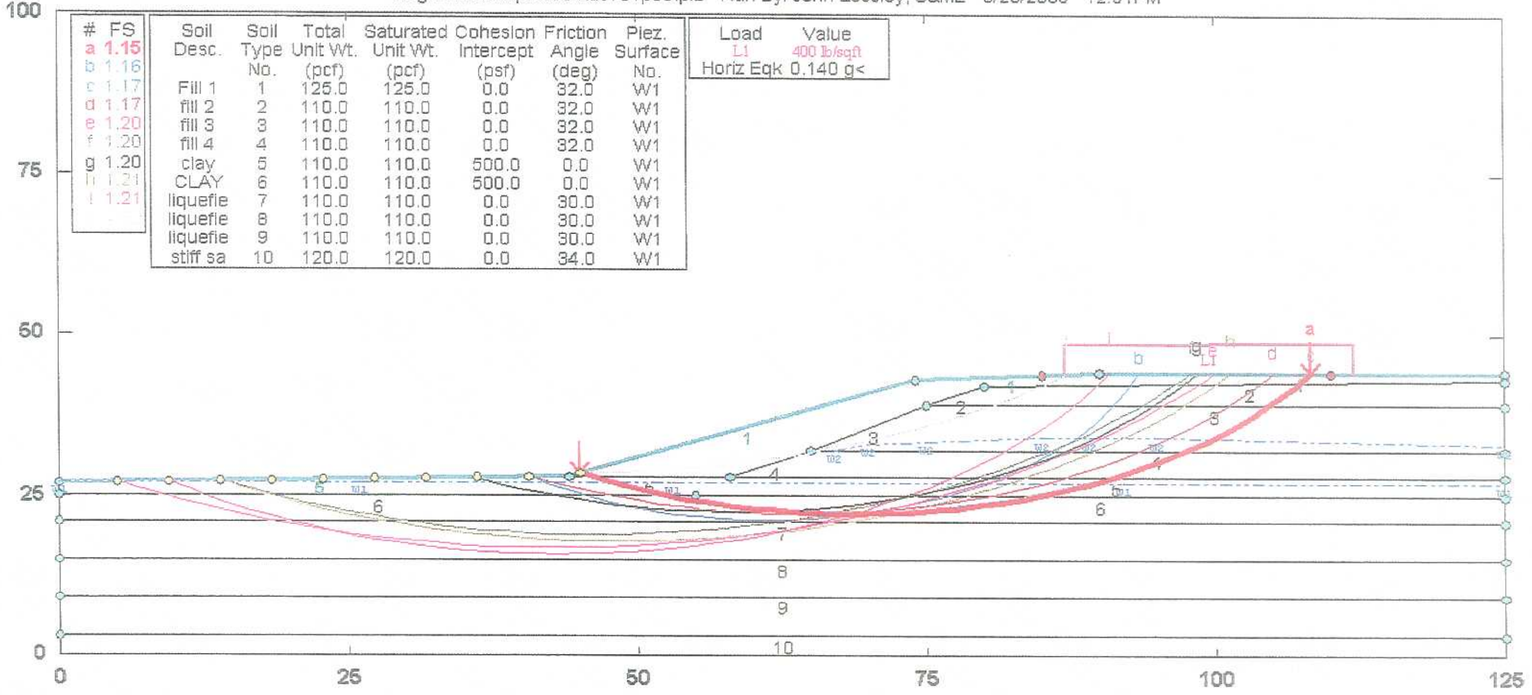
US 701 OVER GREAT PEEDEE RIVER, PEEDEE
OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 22

us 701 PeeDee Overflow South End Global Stability - earthquake

s:\geotech\slope\sc34\us701poe.pl2 Run By: John Lessley, S&ME 3/28/2005 12:51PM



#	FS
a	1.15
b	1.16
c	1.17
d	1.17
e	1.20
f	1.20
g	1.20
h	1.21
i	1.21

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	500.0	0.0	W1
CLAY	6	110.0	110.0	500.0	0.0	W1
liquefie	7	110.0	110.0	0.0	30.0	W1
liquefie	8	110.0	110.0	0.0	30.0	W1
liquefie	9	110.0	110.0	0.0	30.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft
Horiz Eqk	0.140 g<

STABL6H FSmin=1.15
Safety Factors Are Calculated By The Modified Bishop Method

STED

PEEDEE OVERFLOW GLOBAL STABILITY EARTHQUAKE CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	4/1/2005

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ABUTMENT SLOPE GLOBAL STABILITY

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

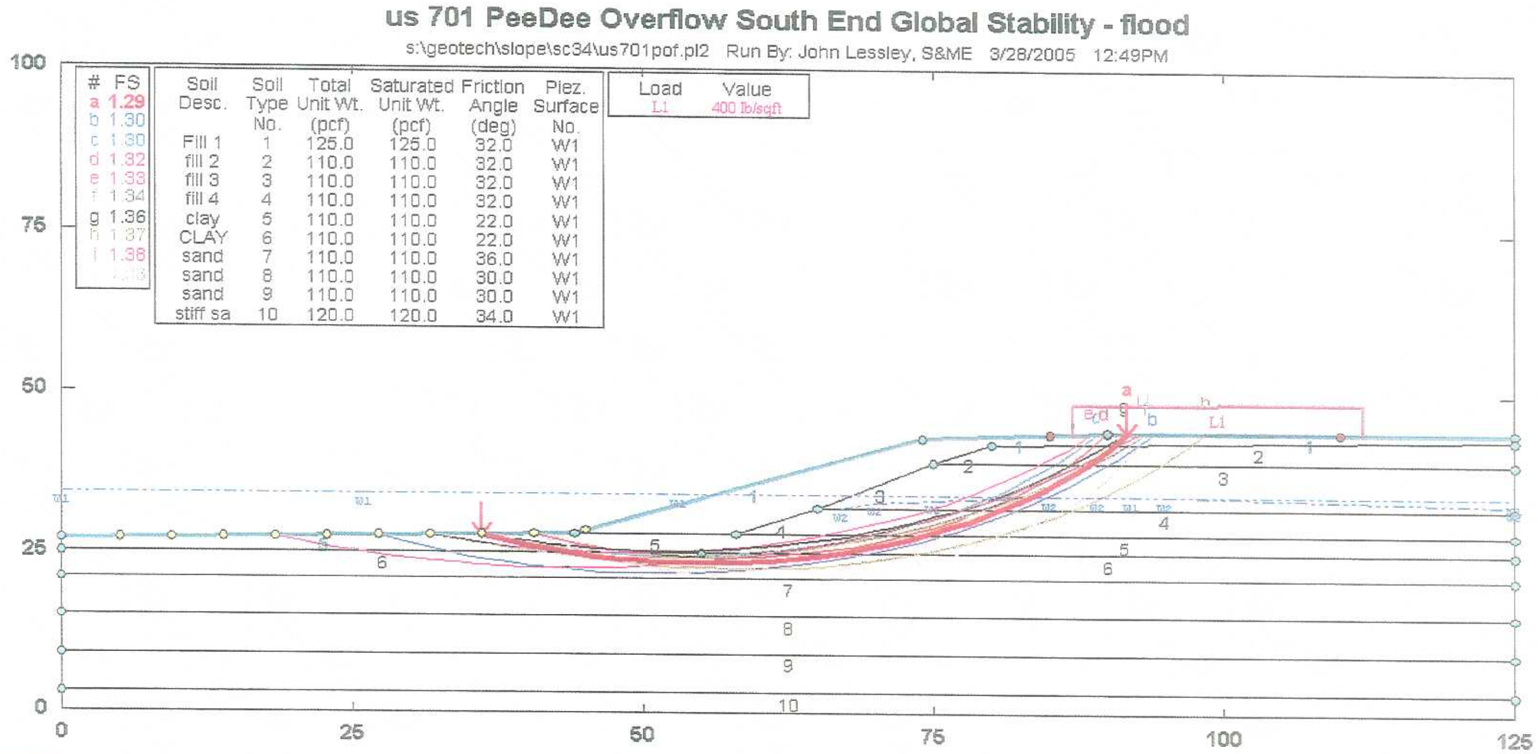
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 23

us 701 PeeDee Overflow South End Global Stability - flood

s:\geotech\slope\sc34\us701pof.pl2 Run By: John Lessley, S&ME 3/28/2005 12:49PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	32.0	W1
fill 2	2	110.0	110.0	32.0	W1
fill 3	3	110.0	110.0	32.0	W1
fill 4	4	110.0	110.0	32.0	W1
clay	5	110.0	110.0	22.0	W1
CLAY	6	110.0	110.0	22.0	W1
sand	7	110.0	110.0	36.0	W1
sand	8	110.0	110.0	30.0	W1
sand	9	110.0	110.0	30.0	W1
stiff sa	10	120.0	120.0	34.0	W1

Load	Value
L1	400 lb/sqft

STED

STABL6H FSmin=1.29

Safety Factors Are Calculated By The Modified Bishop Method

PEEDEE OVERFLOW GLOBAL STABILITY FLOOD CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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ABUTMENT SLOPE GLOBAL STABILITY

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

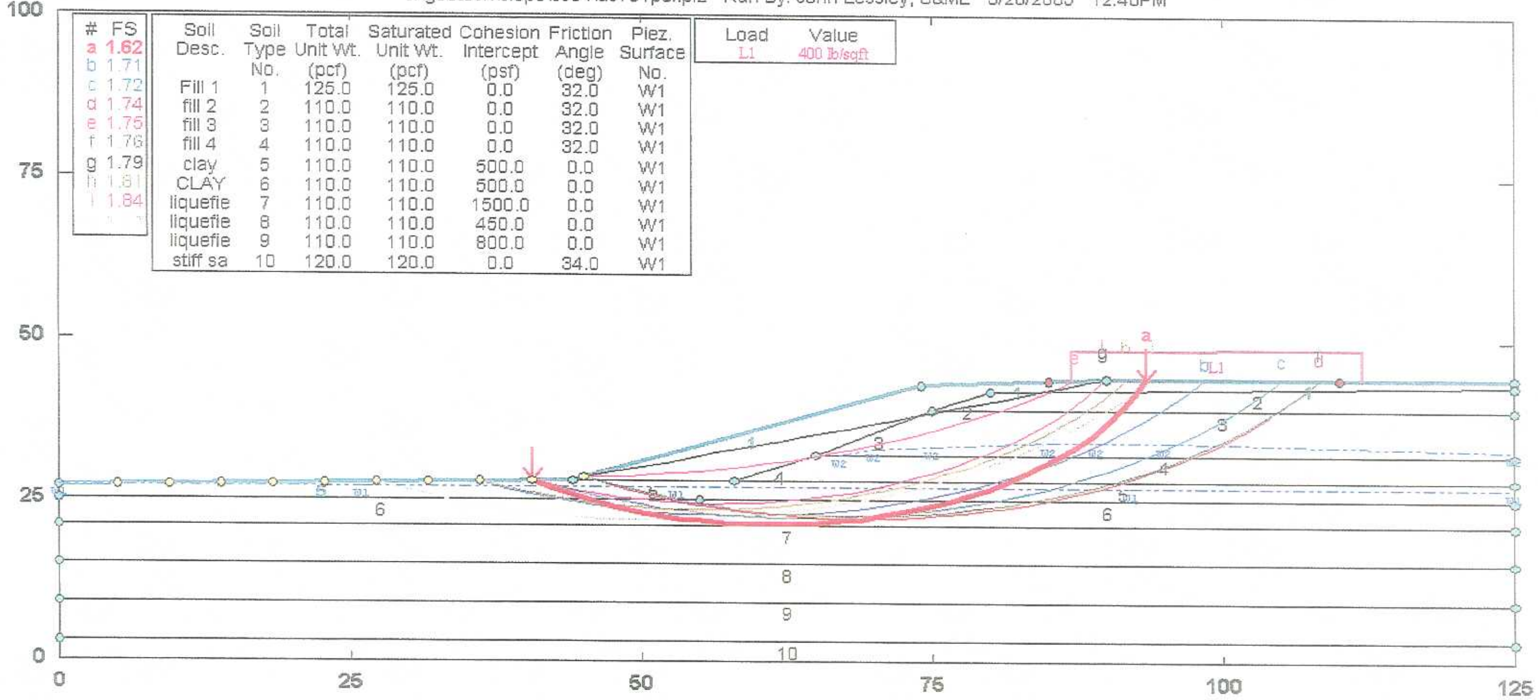
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 24

us 701 PeeDee Overflow South End Global Stability - liquefaction

s:\geotech\slope\sc34\us701pol.pl2 Run By: John Lessley, S&ME 3/28/2005 12:40PM



#	FS
a	1.62
b	1.71
c	1.72
d	1.74
e	1.75
f	1.76
g	1.79
h	1.81
i	1.84

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill 1	1	125.0	125.0	0.0	32.0	W1
fill 2	2	110.0	110.0	0.0	32.0	W1
fill 3	3	110.0	110.0	0.0	32.0	W1
fill 4	4	110.0	110.0	0.0	32.0	W1
clay	5	110.0	110.0	500.0	0.0	W1
CLAY	6	110.0	110.0	500.0	0.0	W1
liquefie	7	110.0	110.0	1500.0	0.0	W1
liquefie	8	110.0	110.0	450.0	0.0	W1
liquefie	9	110.0	110.0	800.0	0.0	W1
stiff sa	10	120.0	120.0	0.0	34.0	W1

Load	Value
L1	400 lb/sqft

STABL6H FSmin=1.62

Safety Factors Are Calculated By The Modified Bishop Method

STED

PEEDEE OVERFLOW GLOBAL STABILITY LIQUEFIED CONDITION
SOUTH ABUTMENT

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	3/28/2005

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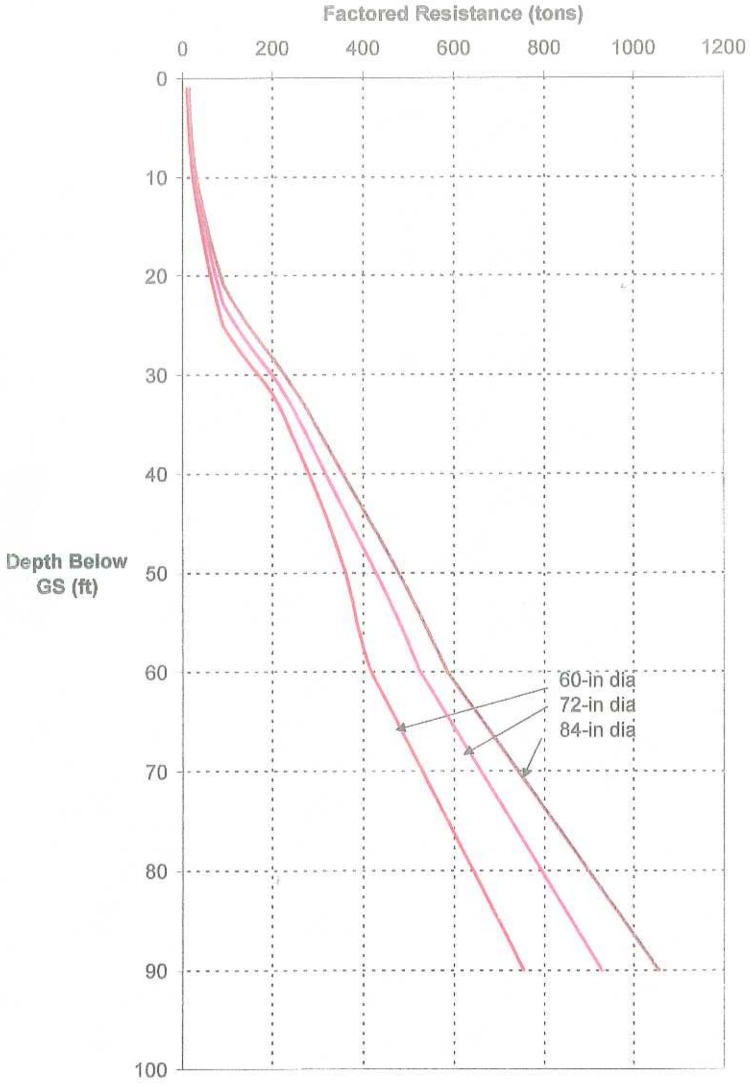
ABUTMENT SLOPE GLOBAL STABILITY

US 701 OVER GREAT PEEDEE RIVER, PEEDEE
OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 25



RESISTANCE FACTOR = 0.7

YAUHANNAH LAKE BRIDGE

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	4/4/2005

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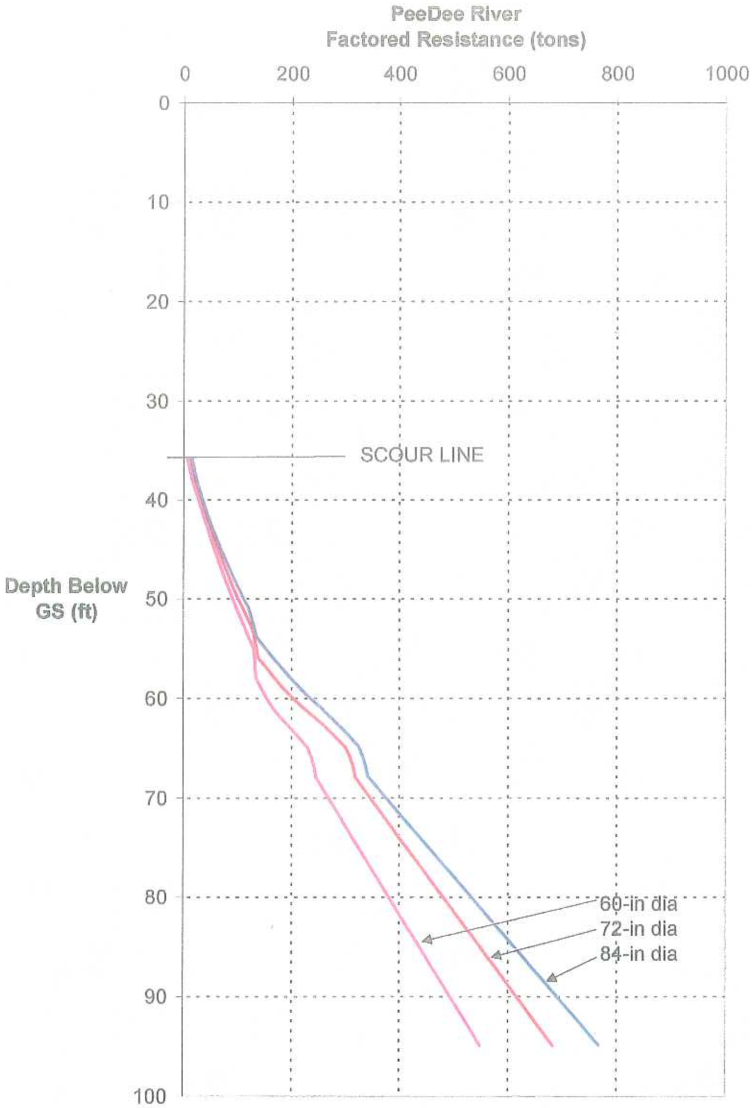
DRILLED SHAFT FACTORED RESISTANCE

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

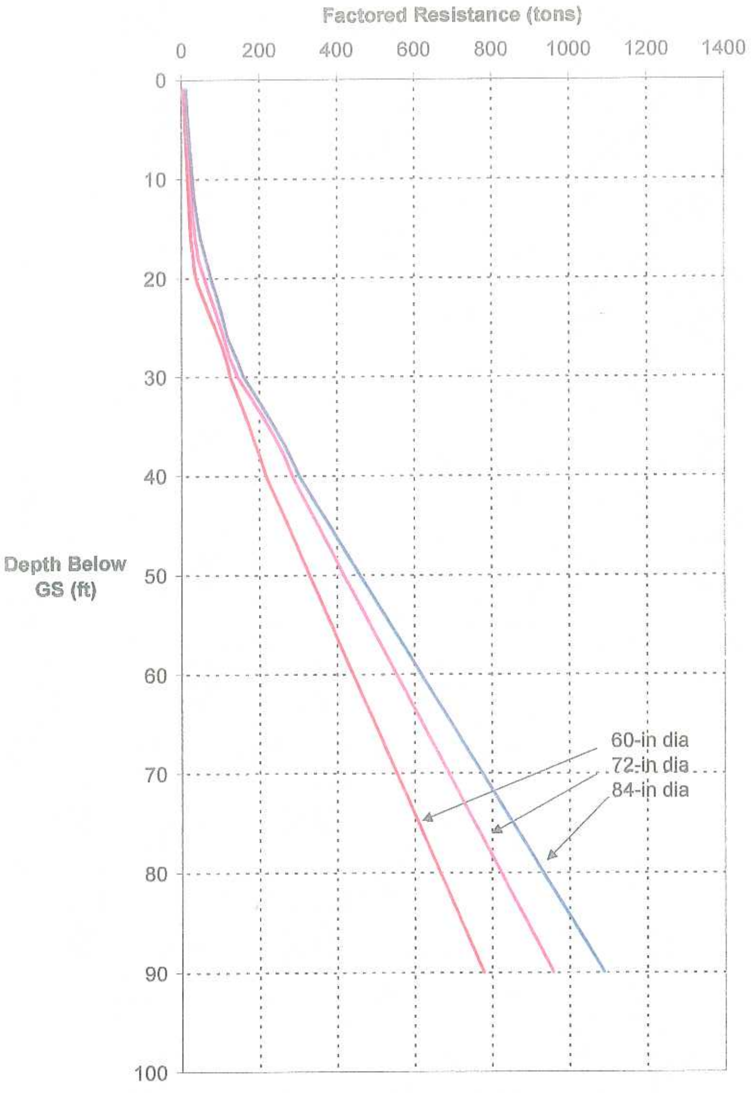
FIGURE NO. 26



RESISTANCE FACTOR = 0.7

PEE DEE RIVER BRIDGE

SCALE: NTS CHECKED BY: JCL DRAWN BY: DATE: 4/4/2005	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	DRILLED SHAFT FACTORED RESISTANCE US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina JOB NO. 1611-04-569	FIGURE NO. 27



RESISTANCE FACTOR = 0.7

PEE DEE OVERFLOW BRIDGE

SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	4/4/2005

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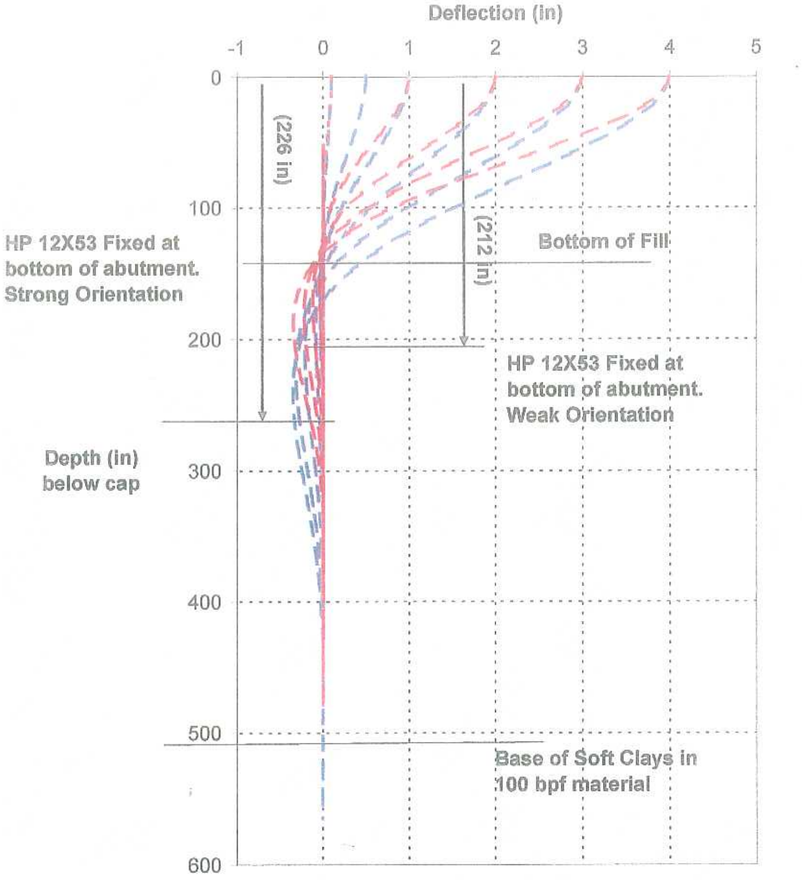
DRILLED SHAFT FACTORED RESISTANCE

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

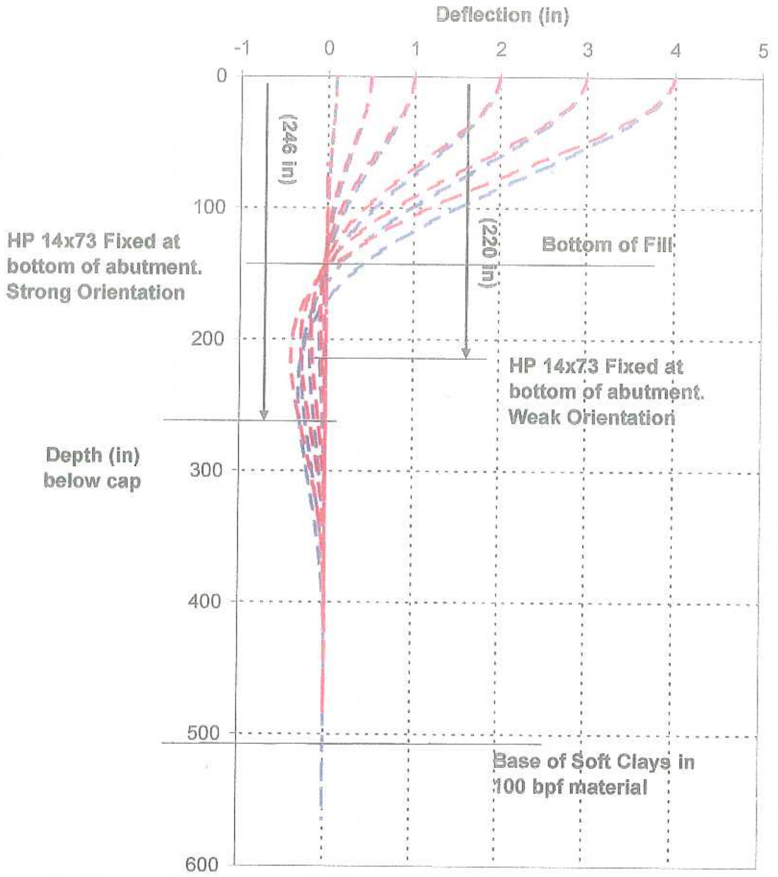
FIGURE NO. 28



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
HP 12x53

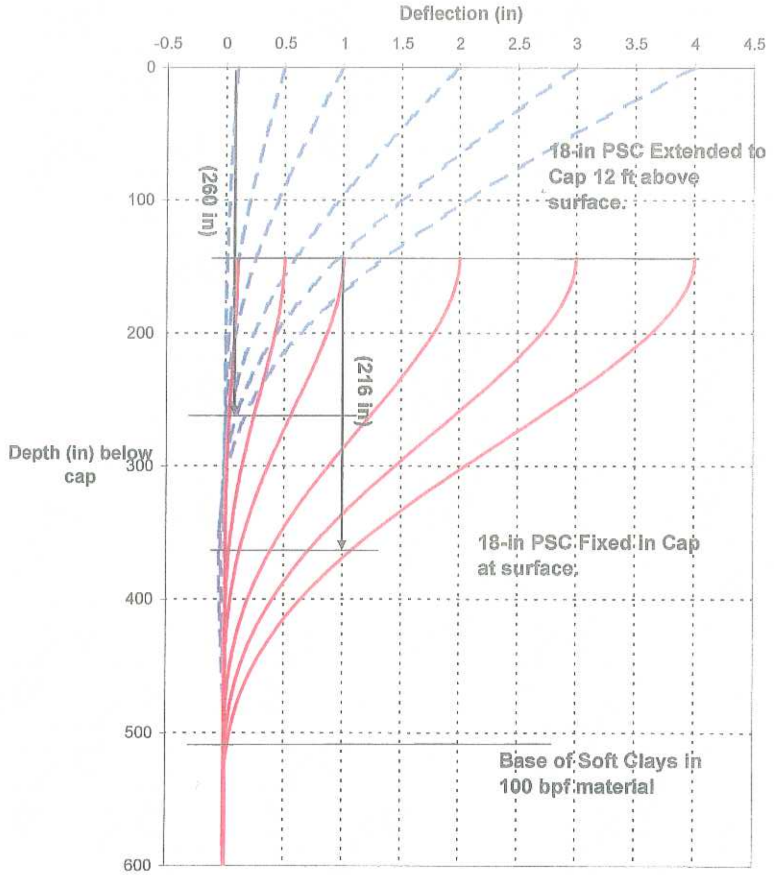
SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	ABUTMENT PILE LATERAL DEFLECTION VS. DEPTH	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	29
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
HP 14x73

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	**ABUTMENT PILE LATERAL DEFLECTION VS. DEPTH**	FIGURE NO.	**30**
CHECKED BY: JCL		**US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina		
DRAWN BY:				
DATE: 4/4/2005		JOB NO. 1611-04-569		



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE

18-IN. SQUARE PSC PILE

1) **FREE-HEADED IN BENT CAP 12 FT. ABOVE G.S.**
2) **FIXED HEADED IN COLUMN CAP AT G.S.**

SCALE: NTS

CHECKED BY: JCL

DRAWN BY:

DATE: 4/4/2005

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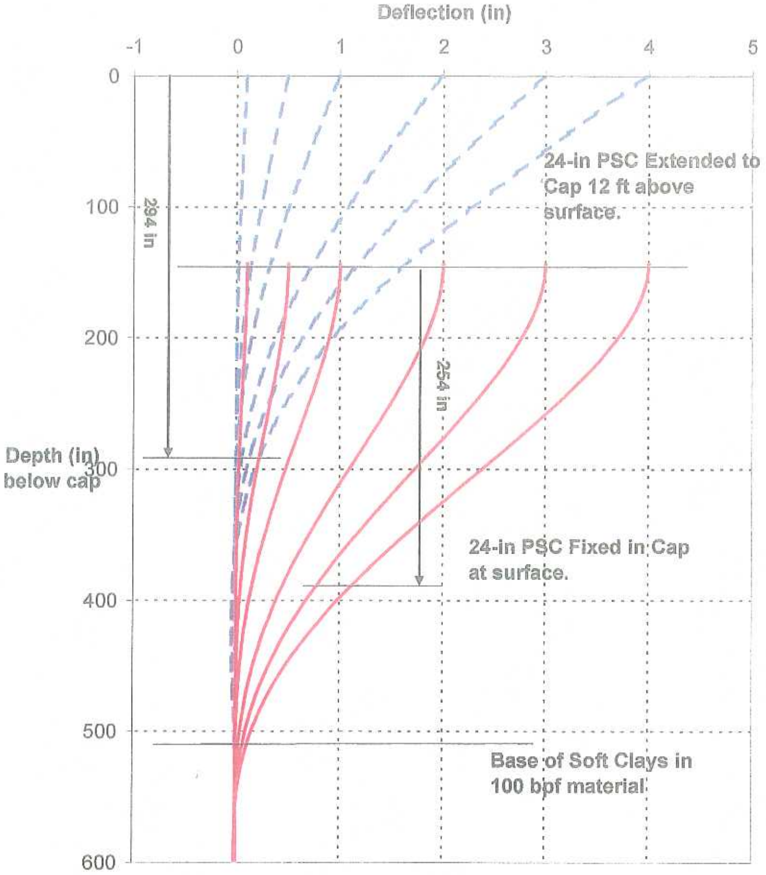
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. **31**



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE

24-IN. SQUARE PSC PILE

1) FREE-HEADED IN BENT CAP 12 FT. ABOVE G.S.

2) FIXED HEADED IN COLUMN CAP AT G.S.

SCALE: NTS

CHECKED BY: JCL

DRAWN BY:

DATE: 4/4/2005

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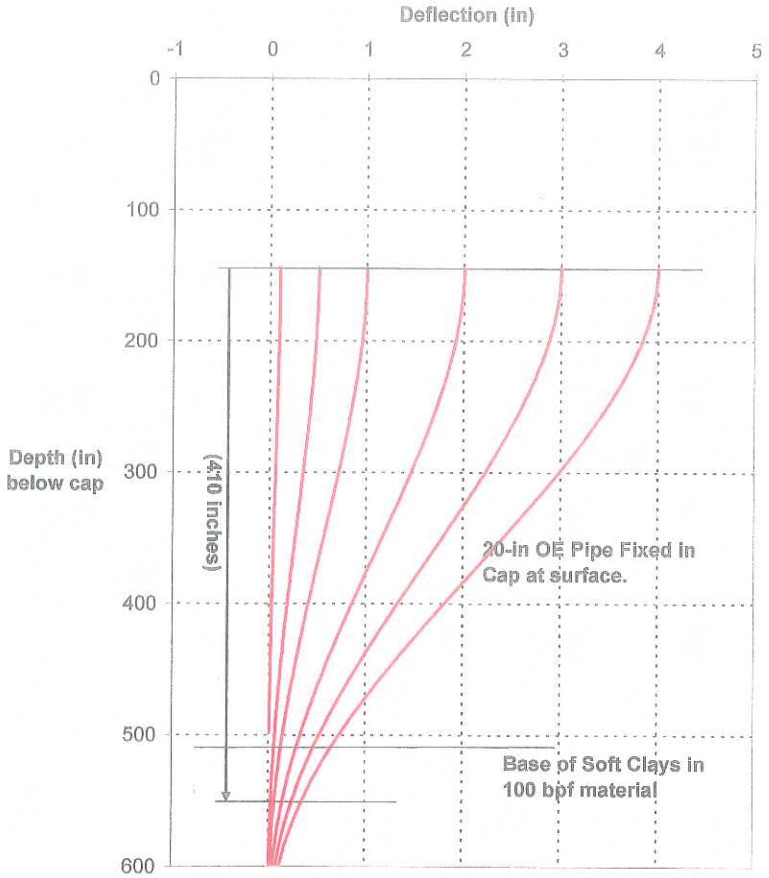
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. **32**



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
20-IN. DIAMETER PIPE PILE
FIXED HEADED IN COLUMN CAP AT G.S.

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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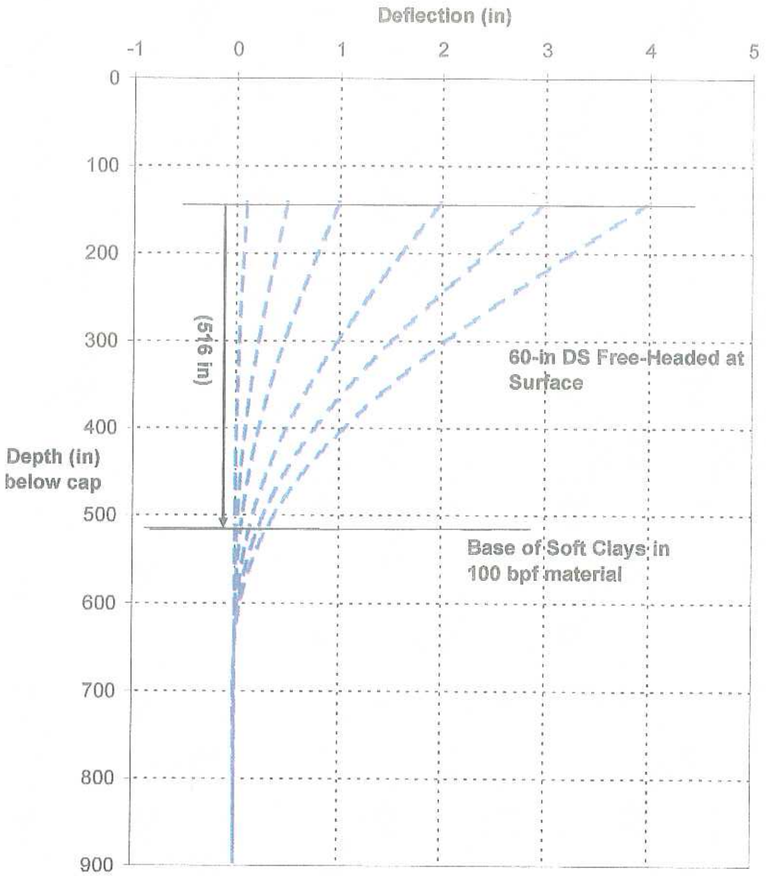
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

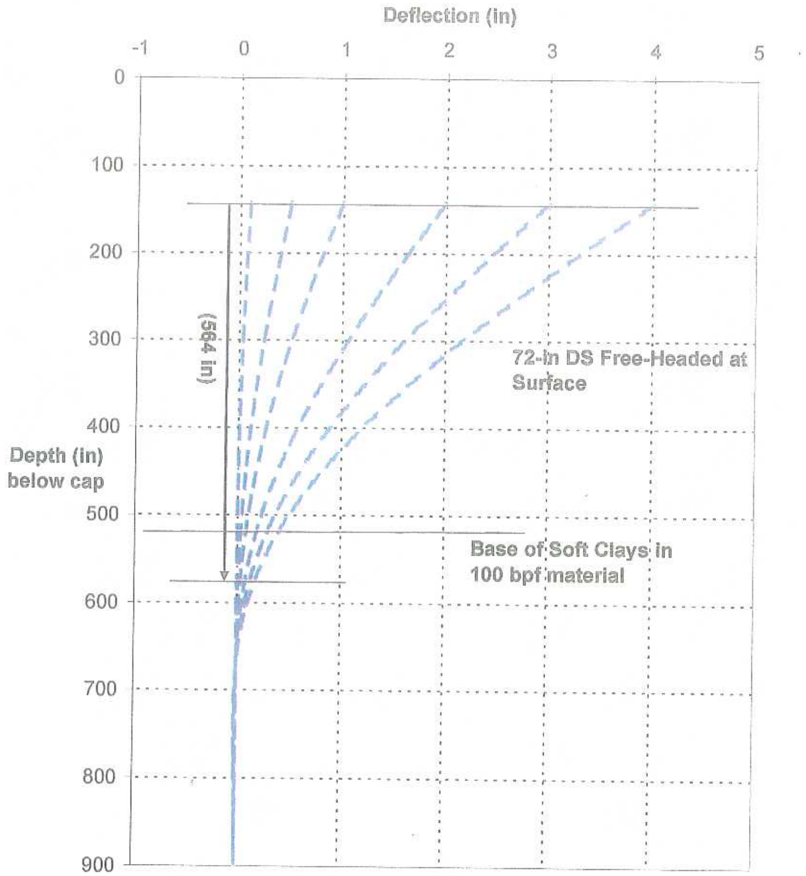
FIGURE NO. **33**



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
60-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

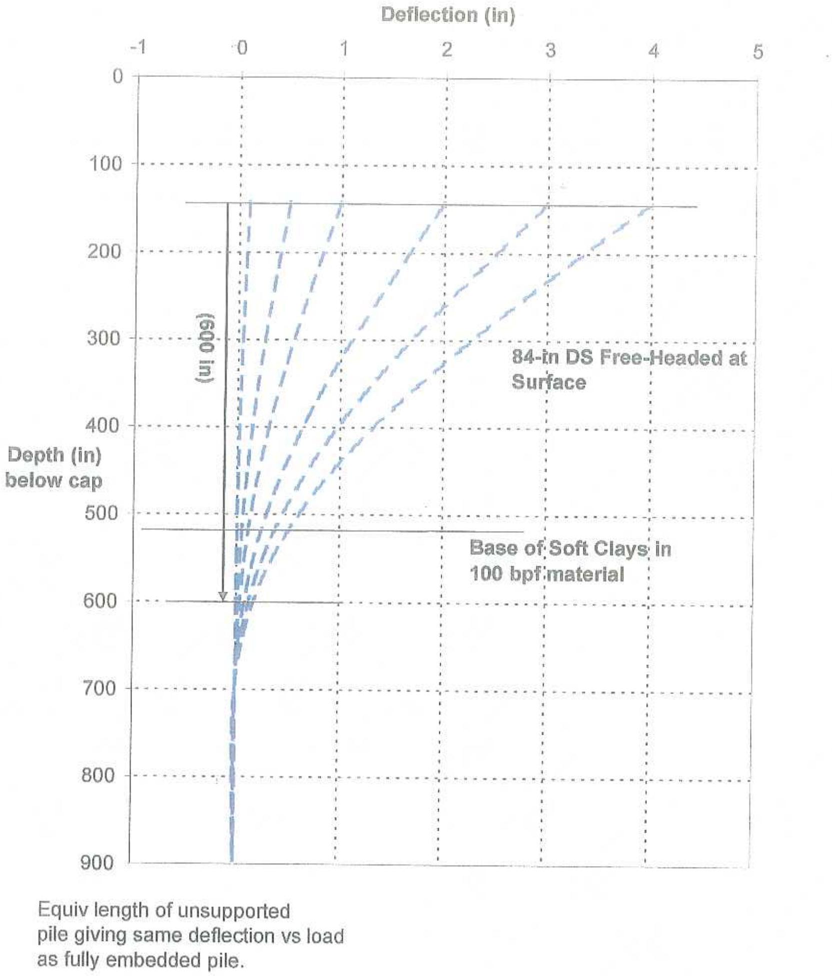
SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	INTERIOR PILE LATERAL DEFLECTION VS. DEPTH	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	34
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
72-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

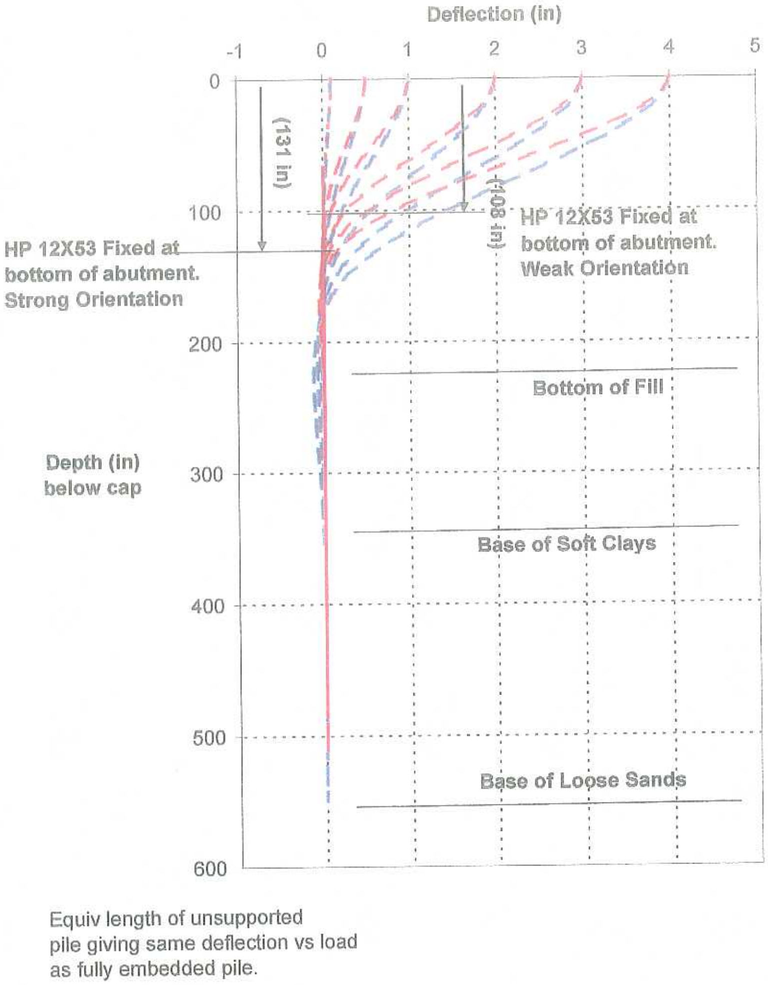
SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	INTERIOR PILE LATERAL DEFLECTION VS. DEPTH	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	35
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-1 YAUHANNAH LAKE
84-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

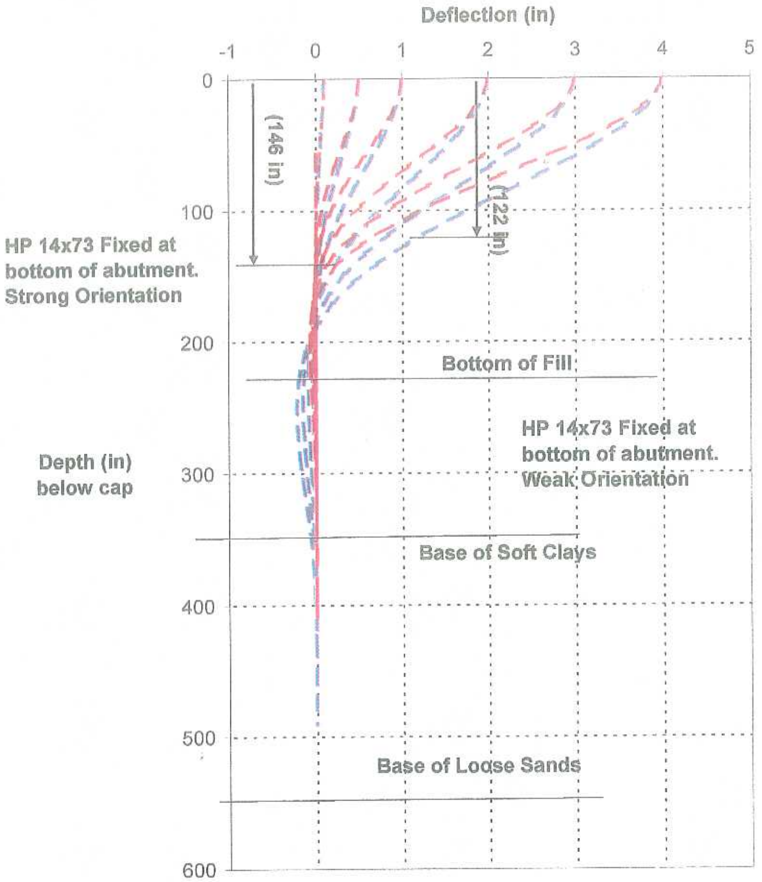
SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	**INTERIOR PILE LATERAL DEFLECTION VS. DEPTH**	FIGURE NO.	
CHECKED BY: JCL		**US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina	**36**	
DRAWN BY:				
DATE: 4/4/2005		JOB NO. 1611-04-569		



BORING B-5 PEE DEE OVERFLOW

HP 12x53

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**ABUTMENT PILE LATERAL DEFLECTION VS. DEPTH**	FIGURE NO.
CHECKED BY: JCL		**US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina	**37**
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-5 PEE DEE OVERFLOW
HP 14x73

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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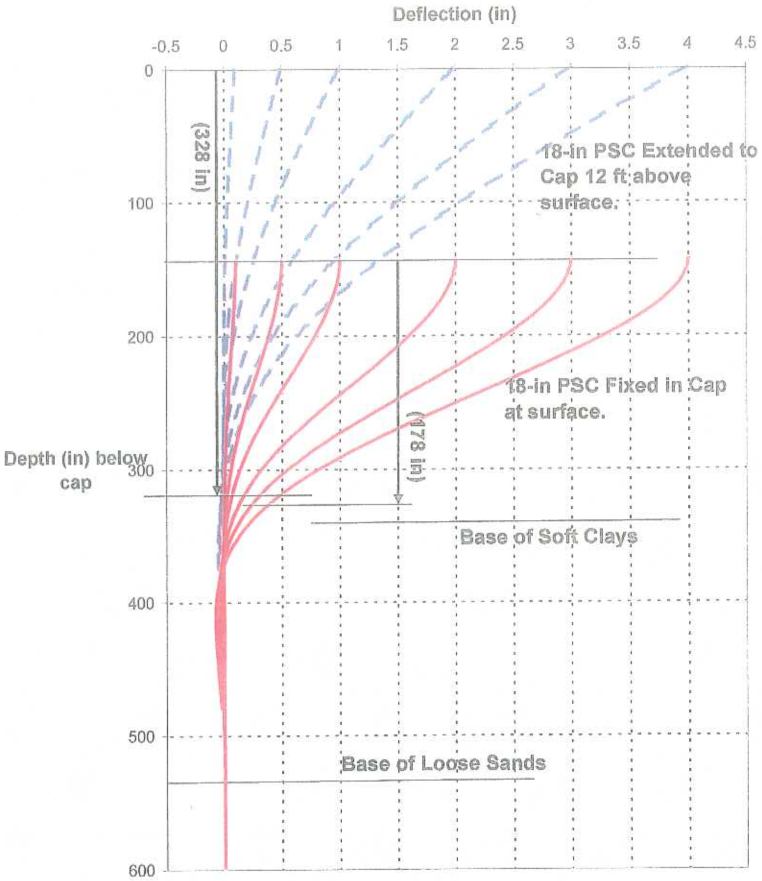
ABUTMENT PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. **38**



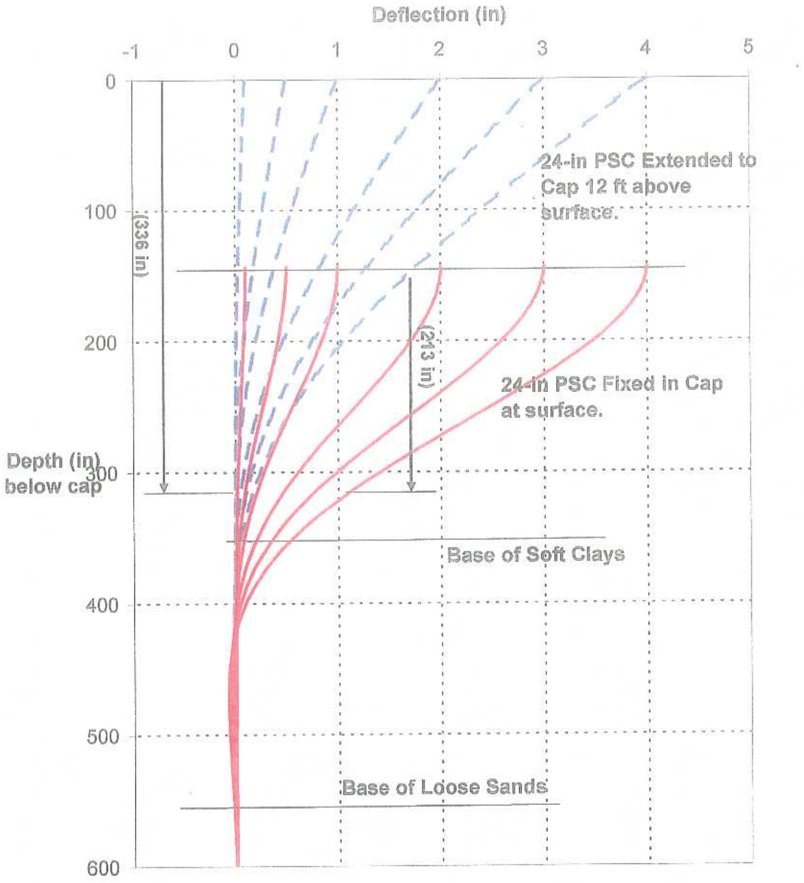
Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-5 PEE DEE OVERFLOW

18-IN. SQUARE PSC PILE

1) **FREE-HEADED IN BENT CAP 12 FT. ABOVE G.S.**
2) **FIXED HEADED IN COLUMN CAP AT G.S.**

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	INTERIOR PILE LATERAL DEFLECTION VS. DEPTH	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	39
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	

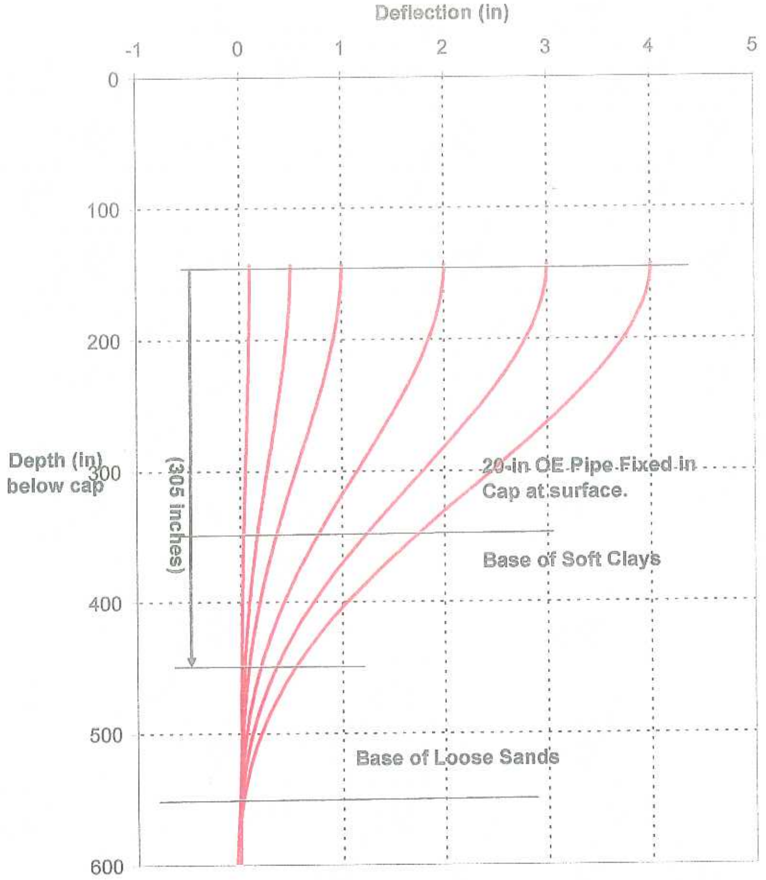


BORING B-5 PEE DEE OVERFLOW

24-IN. SQUARE PSC PILE

1) **FREE-HEADED IN BENT CAP 12 FT. ABOVE G.S.**
2) **FIXED HEADED IN COLUMN CAP AT G.S.**

SCALE: NTS CHECKED BY: JCL DRAWN BY: DATE: 4/4/2005	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**INTERIOR PILE LATERAL DEFLECTION VS. DEPTH** **US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina JOB NO. 1611-04-569	FIGURE NO. **40**



uiv length of unsupported
e giving same deflection vs load
fully embedded pile.

BORING B-5 PEE DEE OVERFLOW
20-IN. DIAMETER PIPE PILE
FIXED HEADED IN COLUMN CAP AT G.S.

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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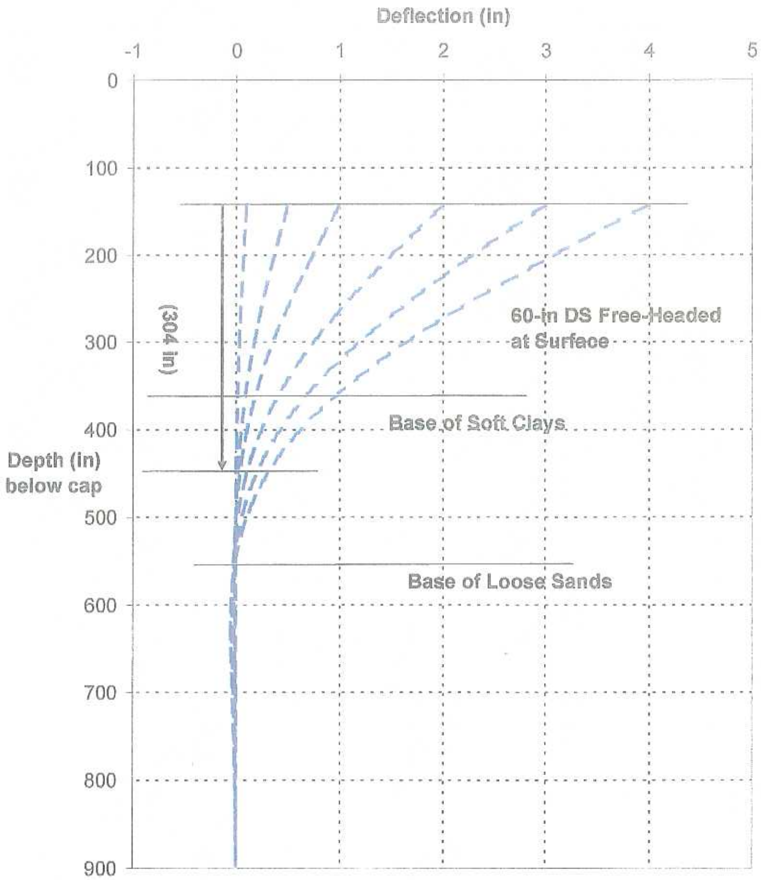
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. **41**



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-5 PEE DEE OVERFLOW
60-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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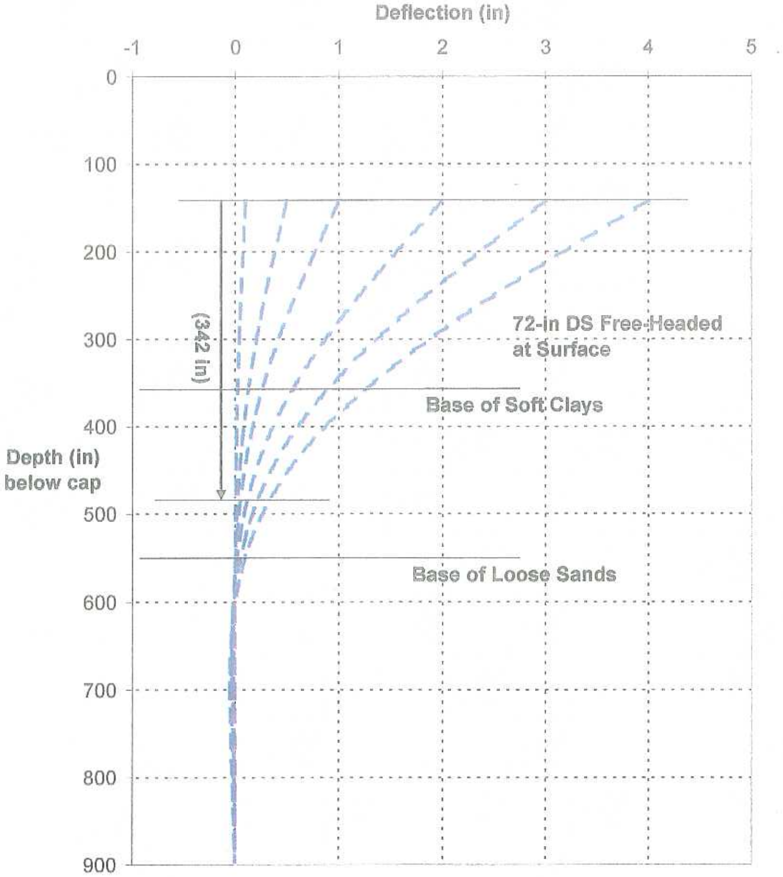
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

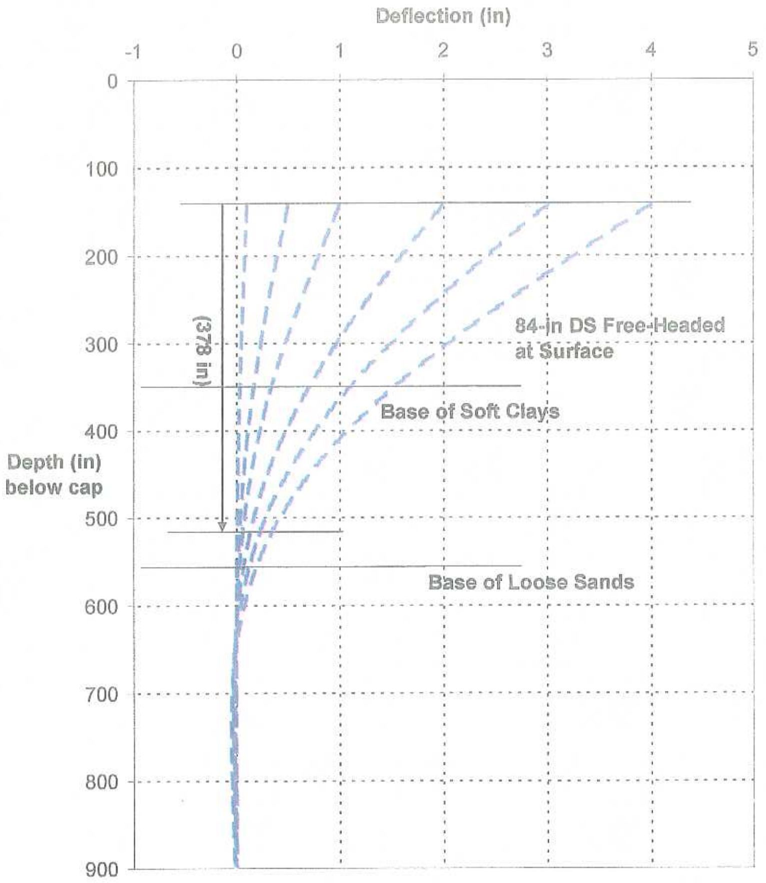
FIGURE NO. 42



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-5 PEE DEE OVERFLOW
72-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**INTERIOR PILE LATERAL DEFLECTION VS. DEPTH**	FIGURE NO.
CHECKED BY: JCL		**US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina	**43**
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

BORING B-5 PEE DEE OVERFLOW
84-IN. DIAMETER DRILLED SHAFT
(UNCRACKED SECTION)

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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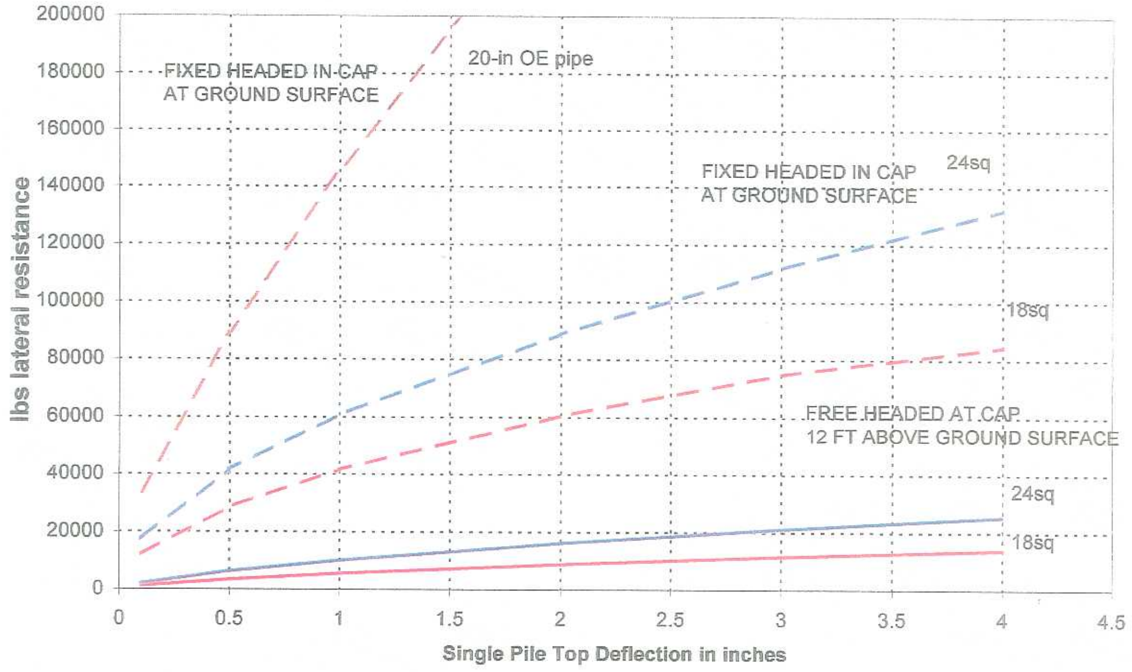
INTERIOR PILE LATERAL DEFLECTION VS. DEPTH

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. 44



YAUHANNAH LAKE

SCALE: NTS

CHECKED BY: JCL

DRAWN BY:

DATE: 4/4/2005

ENVIRONMENTAL SERVICES · ENGINEERING · TESTING

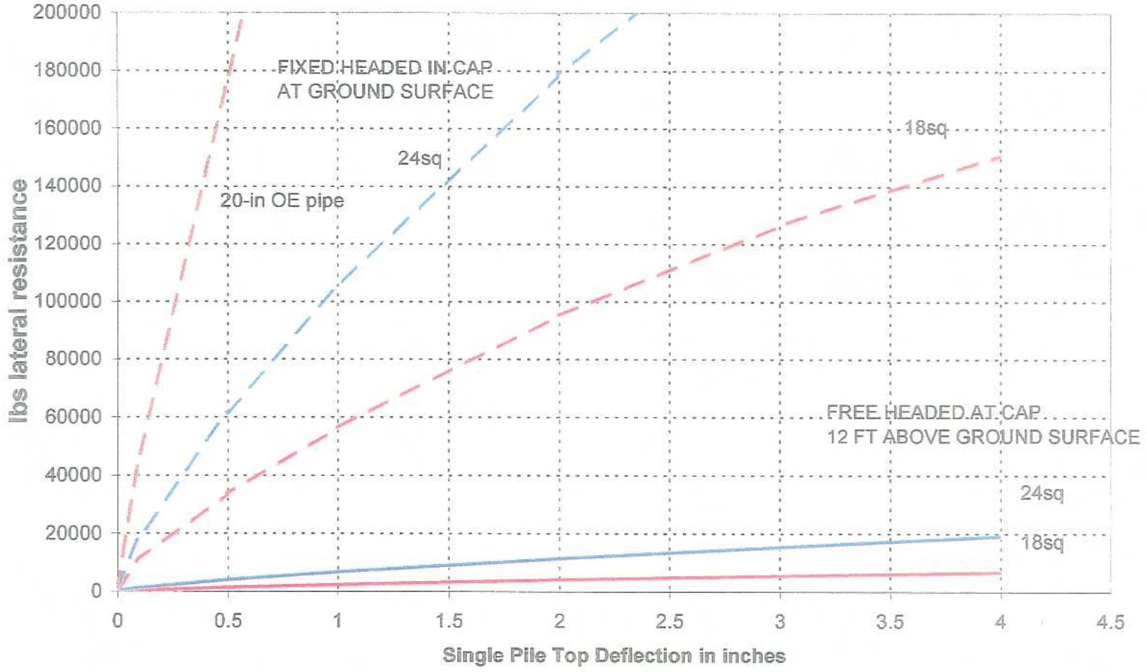
INTERIOR PILE LATERAL DEFLECTION VS. APPLIED SHEAR – YAUHANNAH LAKE

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

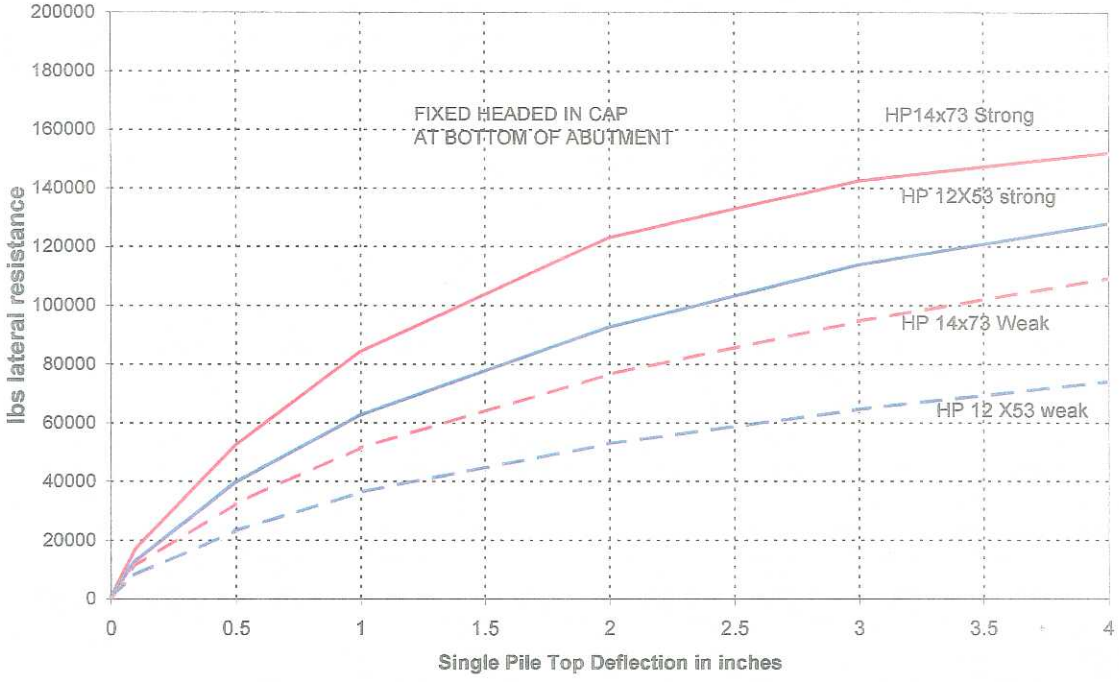
JOB NO. 1611-04-569

FIGURE NO. 45



PEE DEE OVERFLOW

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	INTERIOR PILE LATERAL DEFLECTION VS. APPLIED SHEAR – PEE DEE OVERFLOW US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	FIGURE NO. 46
CHECKED BY: JCL			
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



YAUHANNAH LAKE

SCALE: NTS
CHECKED BY: JCL
DRAWN BY:
DATE: 4/4/2005

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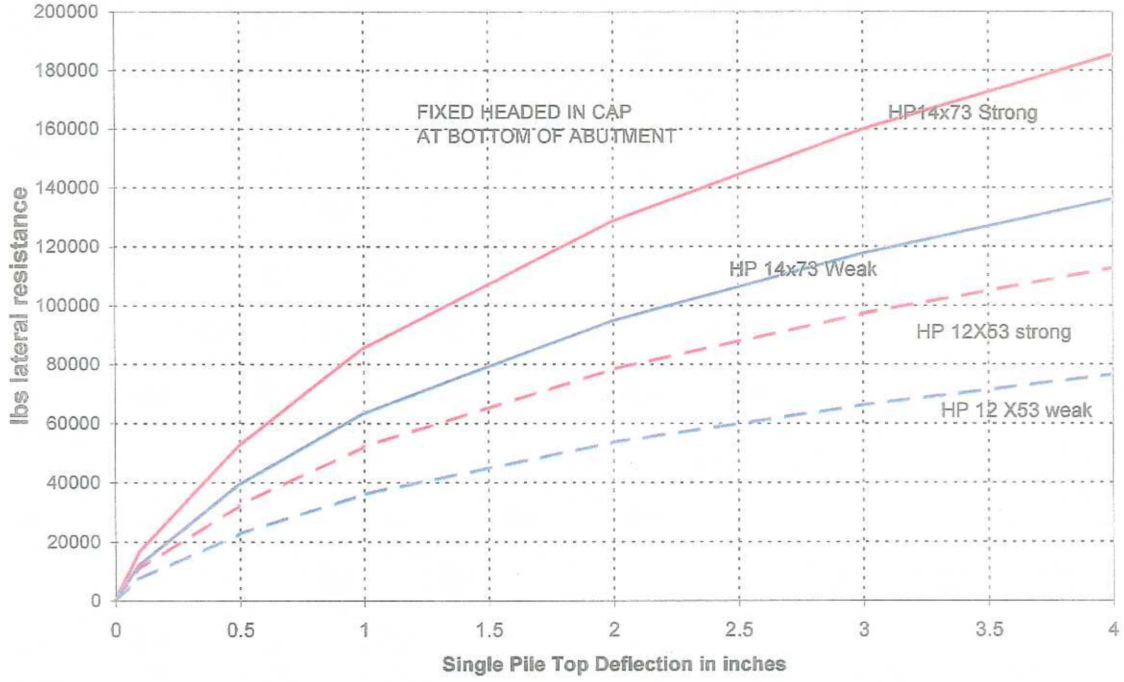
END BENT LATERAL DEFLECTION VS. APPLIED SHEAR – YAUHANNAH LAKE

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

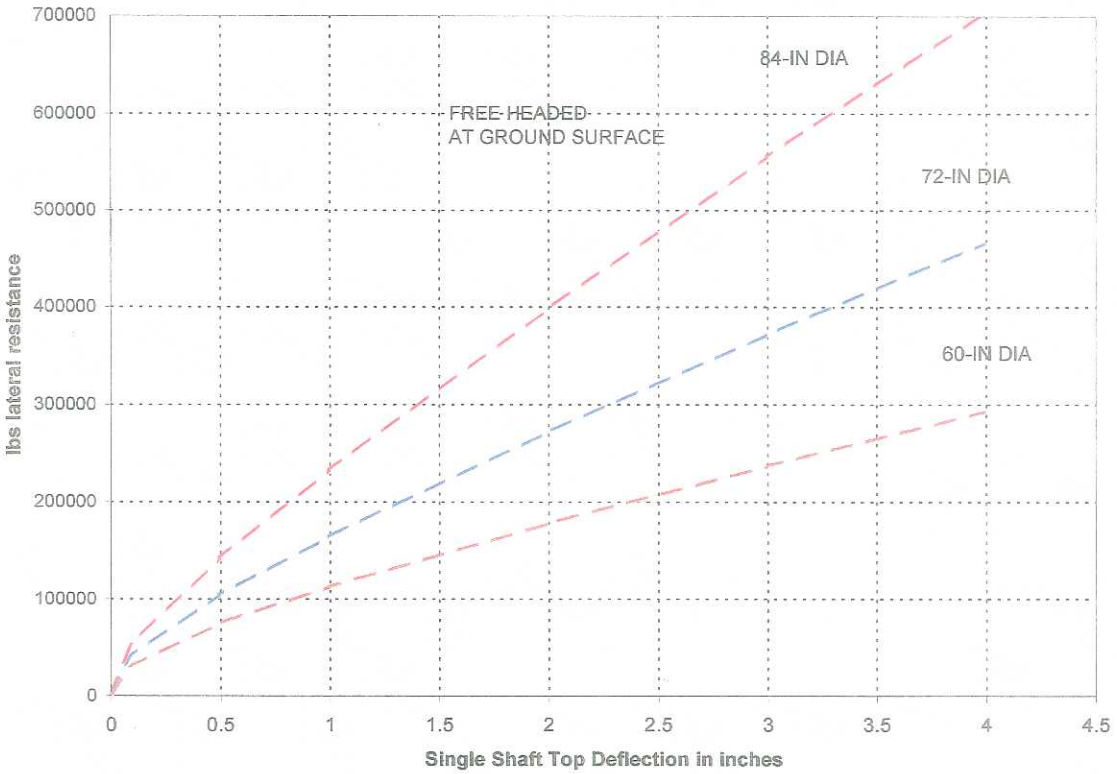
JOB NO. 1611-04-569

FIGURE NO. 47



PEE DEE OVERFLOW

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	END BENT LATERAL DEFLECTION VS. APPLIED SHEAR – PEE DEE OVERFLOW US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	FIGURE NO. 48
CHECKED BY: JCL			
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



YAUHANNAH LAKE

SCALE: NTS

CHECKED BY: JCL

DRAWN BY:

DATE: 4/4/2005

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ENVIRONMENTAL SERVICES · ENGINEERING · TESTING

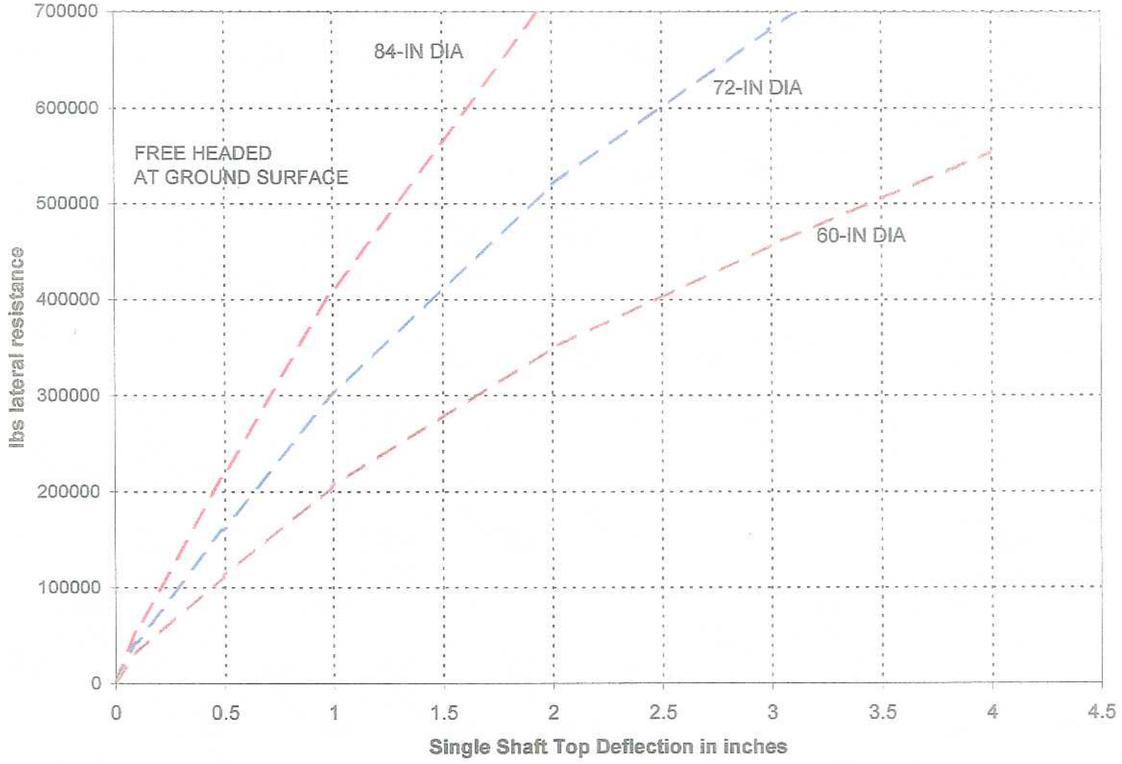
INTERIOR BENT LATERAL DEFLECTION VS. APPLIED SHEAR – YAUHANNAH LAKE DRILLED SHAFTS

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE

Yauhannah, South Carolina

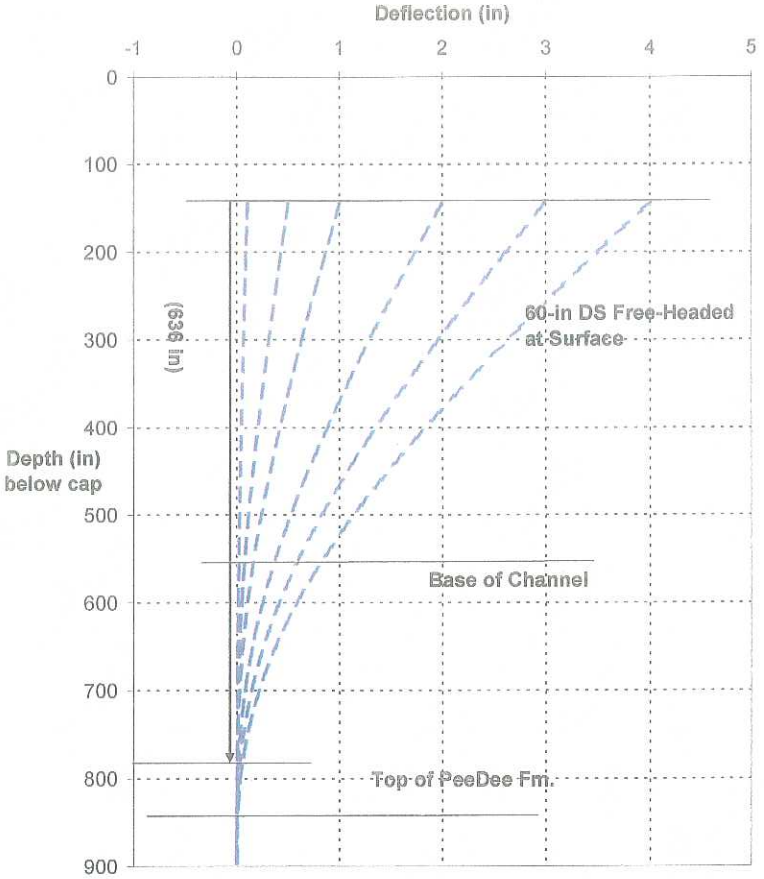
JOB NO. 1611-04-569

FIGURE NO. 49



PEE DEE OVERFLOW

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	INTERIOR BENT LATERAL DEFLECTION VS. APPLIED SHEAR – PEE DEE OVERFLOW DRILLED SHAFTS US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	FIGURE NO. 50
CHECKED BY: JCL			
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

PEE DEE RIVER MAIN BRIDGE
60-IN. DIAMETER DRILLED SHAFT

SCALE: NTS	**INTERIOR BENT LATERAL DEFLECTION VS. DEPTH**	FIGURE NO.
CHECKED BY: JCL	**US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE** Yauhannah, South Carolina	**51**
DRAWN BY:		
DATE: 4/4/2005	JOB NO. 1611-04-569	

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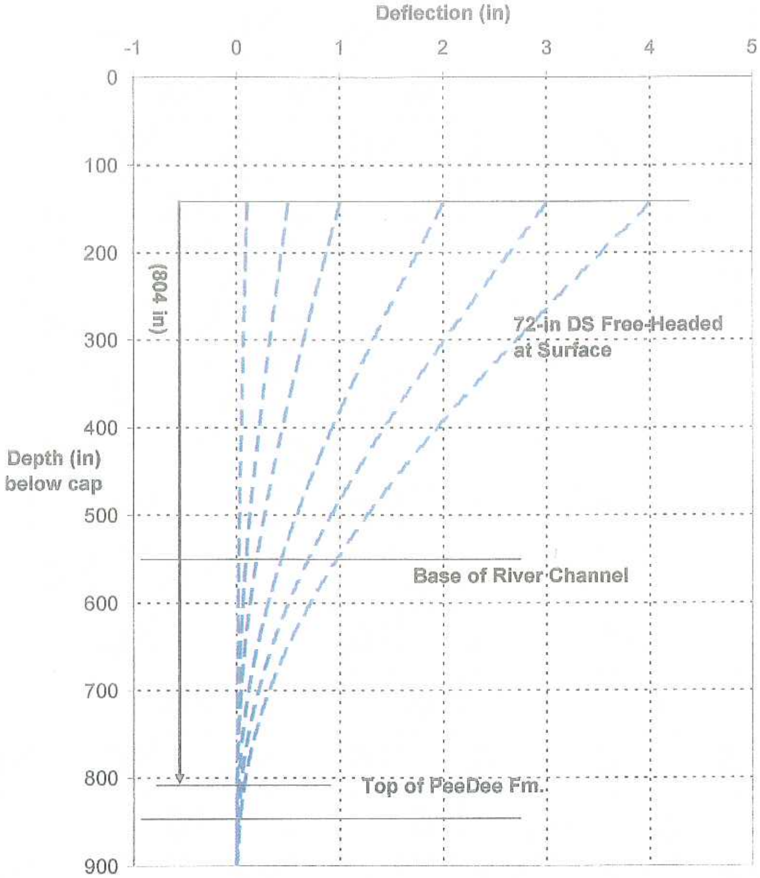
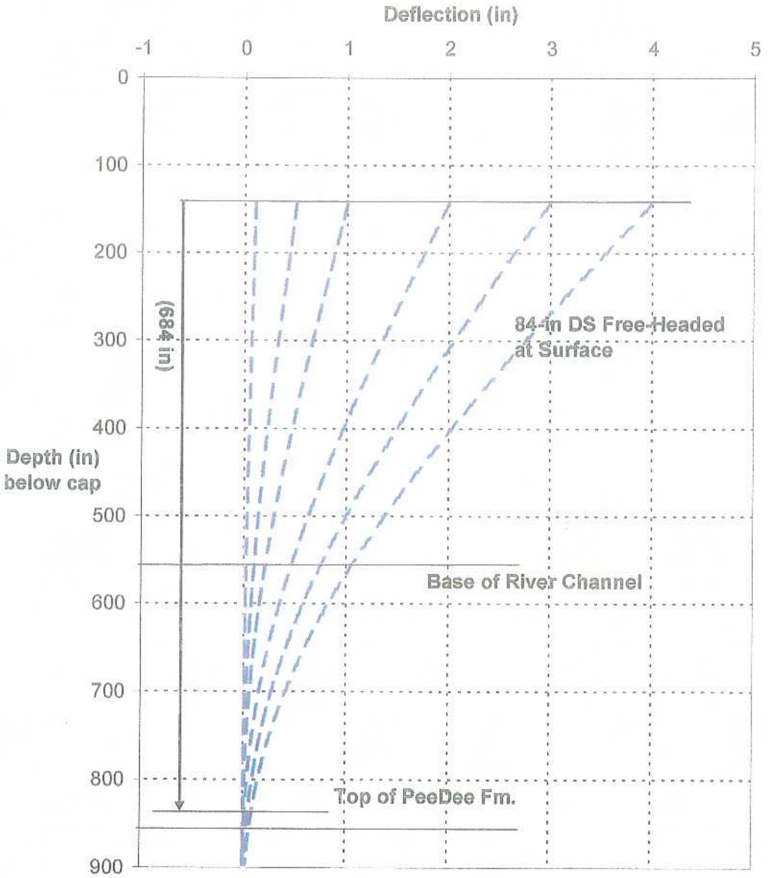


Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

PEE DEE RIVER MAIN BRIDGE
72-IN. DIAMETER DRILLED SHAFT

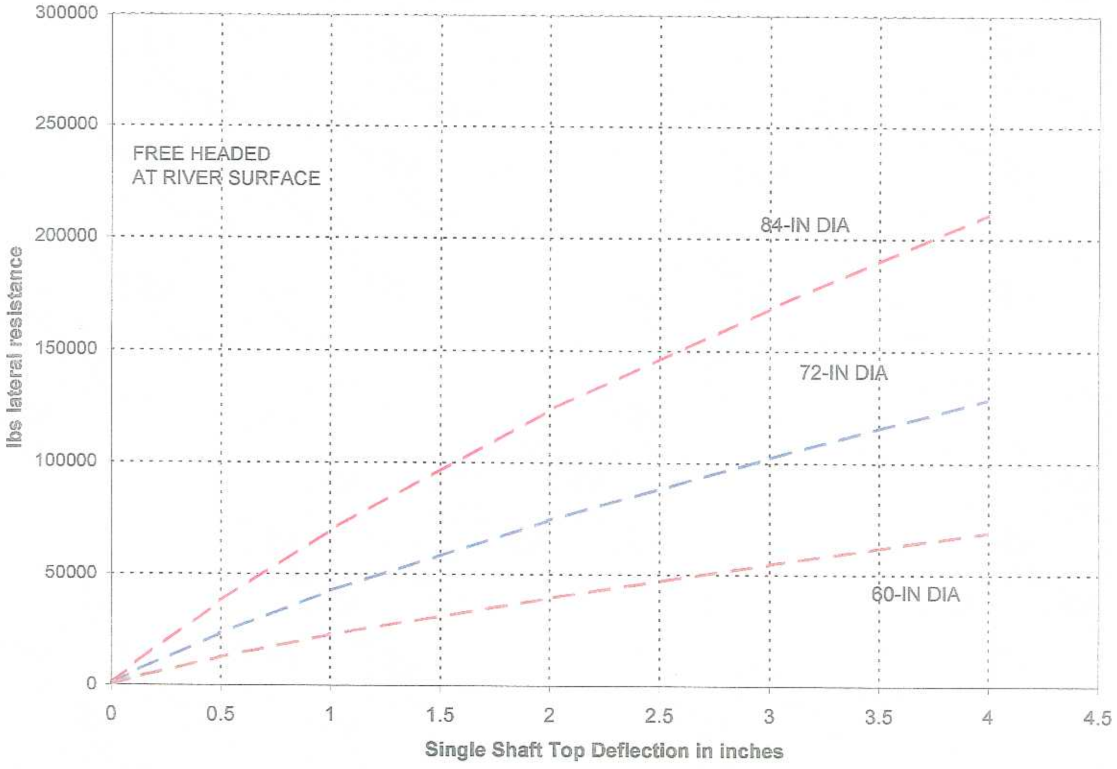
SCALE: NTS CHECKED BY: JCL DRAWN BY: DATE: 4/4/2005	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	INTERIOR BENT LATERAL DEFLECTION VS. DEPTH US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina JOB NO. 1611-04-569	FIGURE NO. 52



Equiv length of unsupported
pile giving same deflection vs load
as fully embedded pile.

PEE DEE RIVER MAIN BRIDGE
84-IN. DIAMETER DRILLED SHAFT

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	INTERIOR BENT LATERAL DEFLECTION VS. DEPTH	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	53
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



SCALE:	NTS
CHECKED BY:	JCL
DRAWN BY:	
DATE:	4/4/2005

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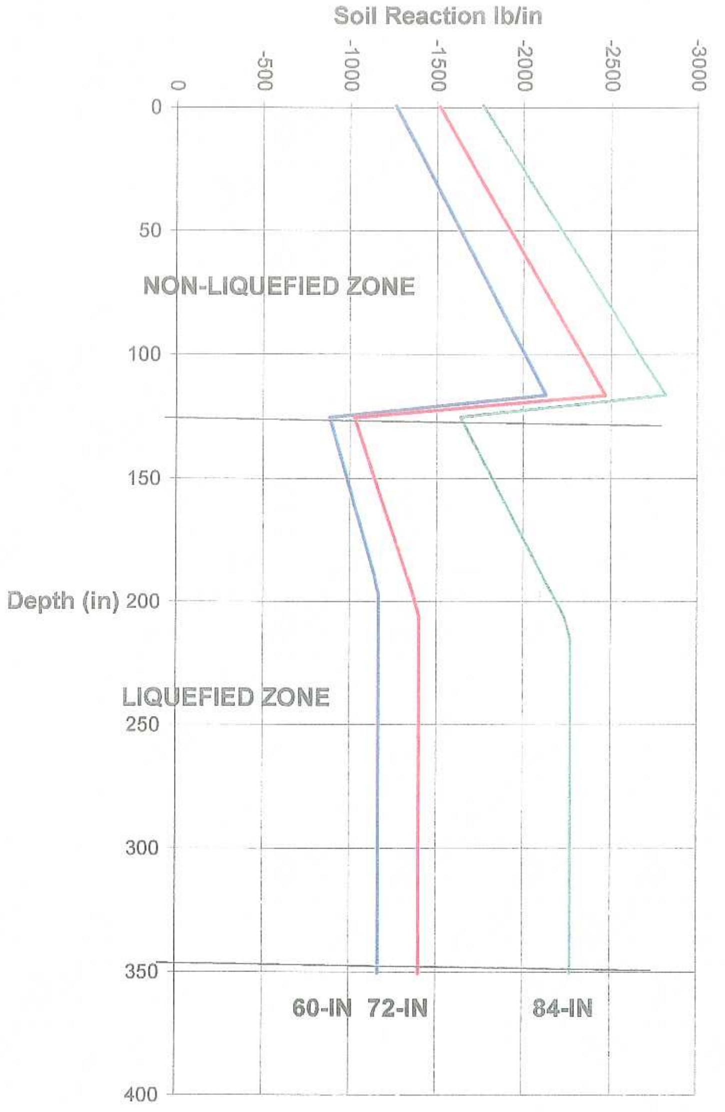
INTERIOR BENT LATERAL DEFLECTION VS. APPLIED SHEAR
PEE DEE RIVER MAIN BRIDGE DRILLED SHAFTS

US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE
Yauhannah, South Carolina

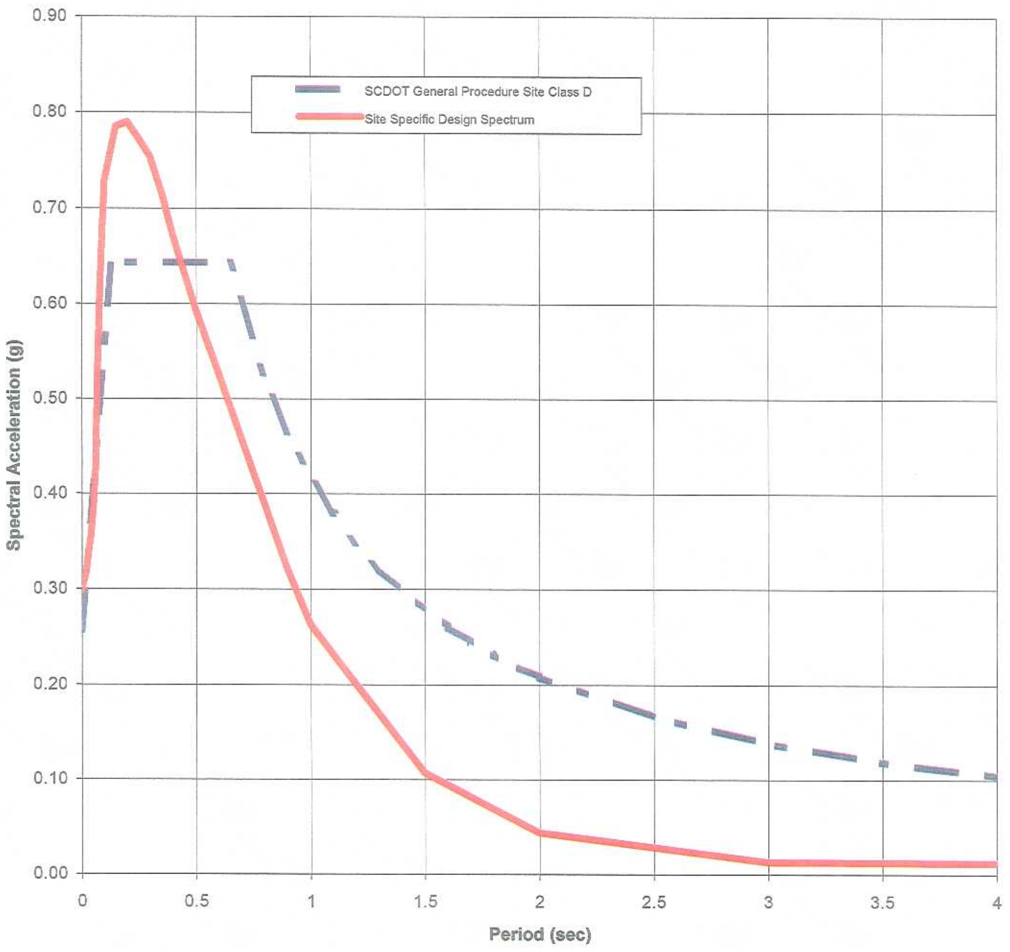
JOB NO. 1611-04-569

FIGURE NO. 54



PEE DEE RIVER MAIN BRIDGE
DRILLED SHAFT

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	SOIL REACTION APPLIED UNDER LATERAL SPREADING	FIGURE NO.
CHECKED BY: JCL		US 701 OVER GREAT PEEDEE RIVER, PEEDEE OVERFLOW AND YAUHANNAH LAKE Yauhannah, South Carolina	55
DRAWN BY:			
DATE: 4/4/2005		JOB NO. 1611-04-569	



Period	Spectral Acceleration (% g)		
(sec)	Raw Site Specific	SCDOT Site Class D	70% SCDOT Site Class D
0	0.3	0.26	0.18
0.02	0.32	0.32	0.22
0.03	0.34	0.35	0.24
0.04	0.36	0.38	0.26
0.05	0.39	0.41	0.28
0.06	0.43	0.44	0.31
0.07	0.54	0.47	0.33
0.08	0.62	0.5	0.35
0.09	0.68	0.53	0.37
0.1	0.73	0.56	0.39
0.13	0.77	0.65	0.45
0.15	0.79	0.65	0.45
0.2	0.79	0.65	0.45
0.25	0.77	0.65	0.45
0.3	0.75	0.65	0.45
0.35	0.72	0.65	0.45
0.4	0.67	0.65	0.45
0.5	0.59	0.65	0.45
0.6	0.53	0.65	0.45
0.65	0.5	0.65	0.45
0.7	0.46	0.6	0.42
0.8	0.39	0.52	0.37
0.9	0.32	0.47	0.33
1	0.26	0.42	0.29
1.5	0.11	0.28	0.2
2	0.04	0.21	0.15
3	0.01	0.14	0.1
4	0.01	0.1	0.07

Note: S_{D1} is limited in accordance with Section 3.4.5.1 of the SCDOT Seismic Design Specifications.

In accordance with Section 3.4.5.1, the site specific design spectrum may need to be reviewed by a third party or limited to no less than 70% of the Site Class D for Yauhannah Lake where liquefiable soils are not present.

S_{D1} = 0.29 S_{DS} = 0.79

Job No.: 1611-04-569

Date: May 2005

S&ME

SITE SPECIFIC ACCELERATION RESPONSE SPECTRA
2% PROBABILITY OF EXCEEDANCE IN 50 YRS.
US 701 BRIDGES OVER GREAT PEE DEE and OVERFLOW
HORRY and GEORGETOWN COUNTIES, SOUTH CAROLINA

Figure: 56

PLATE 1 - View of existing PeeDee River Bridge (Bridge 2) from upstream. Bridge is supported by cast-in-place concrete piers extending below the scour line, in turn supported on timber piles extending approx. 15-30 ft below the base of the piers. The central span was constructed in about 1996 and is supported by drilled shafts bearing in Pee Dee Formation at a depth of about 40-50 feet below the mudline. Piers in river channel beyond the existing bridge were part of old structure that was demolished when the current bridge was built in 1952.

SCALE:	NTS	**PHOTOGRAPHIC PLATE - 1** US 701 Bridges Yauhannah, South Carolina	FIGURE NO.
CKED BY:	JCL		P-1
DRAWN BY:	MG		
DATE:	4/1/2005	JOB NO. 1611-04-569	

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PLATE 2 - View of existing PeeDee Overflow Bridge from near south abutment. At time of visit area beneath bridge structure was almost entirely dry.

SCALE:	NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**PHOTOGRAPHIC PLATE - 2** US 701 Bridges Yauhannah, South Carolina	FIGURE NO. **P-2**
CHECKED BY:	JCL			
DRAWN BY:	MG			
DATE:	4/1/2005		JOB NO. 1611-04-569	

PLATE 3 - View of sag in bridge deck looking south on PeeDee Overflow bridge.

SCALE:	NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**PHOTOGRAPHIC PLATE - 3** US 701 Bridges Yauhannah, South Carolina	FIGURE NO.
CHECKED BY:	JCL			P-3
DRAWN BY:	MG			
DATE:	4/1/2005		JOB NO. 1611-04-569	

PLATE 4 - View of north abutment of the Pee Dee River bridge from the marina access road. Boring B-4 is located nearby. Height of embankment at abutment is approximately 20 feet.

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	**PHOTOGRAPHIC PLATE - 4** US 701 Bridges Yauhannah, South Carolina		FIGURE NO.
CHECKED BY: JCL				
DRAWN BY: MG				P-4
DATE: 4/1/2005		JOB NO.	1611-04-569	

PLATE 5 - View of PeeDee River Bridge looking south from entrance to marina on north bank. Approach spans are pile bents supported by 18-inch precast concrete piles which extend about 30 feet below the surface and terminate in a zone of liquefiable sands which underlie the alignment north of the river.

SCALE:	NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	**PHOTOGRAPHIC PLATE - 5** US 701 Bridges Yauhannah, South Carolina	FIGURE NO. **P-5**
CHECKED BY:	JCL			
DRAWN BY:	MG			
DATE:	4/1/2005		JOB NO. 1611-04-569	

PLATE 6 - View of south bank from on top of the PeeDee River Bridge. Wooded area beyond road is marshy.

SCALE: NTS
CHECKED E JCL
DRAWN BY: MG
ATE: 4/1/2005

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PHOTOGRAPHIC PLATE - 6
US 701 Bridges
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO.
P-6

PLATE 7 - View of south end of Yauhannah Lake Bridge from bluff at south margin of the PeeDee River floodplain. Pile bents in the foreground extend through waters of Yauhannah Lake.

SCALE:	NTS	S&ME	**PHOTOGRAPHIC PLATE - 7** US 701 Bridges Yauhannah, South Carolina	FIGURE NO.
CHECKED BY:	JCL			P-7
DRAWN BY:	MG			
DATE:	4/1/2005	ENVIRONMENTAL SERVICES • ENGINEERING • TESTING	JOB NO. 1611-04-569	

PLATE 8 - View of south approach to PeeDee River main bridge from approximate location of Boring B-2.

SCALE:	NTS		
CHECKED BY:	JCL		
DRAWN BY:	MG		
DATE:	4/1/2005		

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PHOTOGRAPHIC PLATE - 8
US 701 Bridges
Yauhannah, South Carolina

JOB NO. 1611-04-569

FIGURE NO. **P-8**

PLATE 9 - View of approach bents on south side of main PeeDee River Bridge. Bents are supported by 18-in square precast piles which bear in non-liquefiable soils on this side of the river.

SCALE: NTS	S&ME ENVIRONMENTAL SERVICES · ENGINEERING · TESTING	PHOTOGRAPHIC PLATE - 9 US 701 Bridges Yauhannah, South Carolina		FIGURE NO. P-9
CHECKED BY: JCL				
DRAWN BY: MG				
DATE: 4/1/2005		JOB NO.	1611-04-569	

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPES

(Shown in Graphic Log)

▩	Fill
▦	Asphalt
▒	Concrete
■	Topsoil
░	Gravel
░	Sand
▥	Silt
▨	Clay
▤	Organic
▥	Silty Sand
▨	Clayey Sand
▥	Sandy Silt
▥	Clayey Silt
▨	Sandy Clay
▨	Silty Clay
▒	Partially Weathered Rock
▦	Cored Rock

WATER LEVELS

(Shown in Water Level Column)

▽ = Water Level At Termination of Boring
▼ = Water Level Taken After 24 Hours
◄ = Loss of Drilling Water
HC = Hole Cave

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Soft	0 to 2
Soft	3 to 4
Firm	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	31 to 50
Very Hard	Over 50

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50

SAMPLER TYPES

(Shown in Samples Column)

■	Shelby Tube
☒	Split Spoon
I	Rock Core
☐	No Recovery

TERMS

Standard Penetration Resistance	The Number of Blows of 140 lb. Hammer Falling 30 in. Required to Drive 1.4 in. I.D. Split Spoon Sampler 1 Foot. As Specified in ASTM D-1588.
REC	Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100%.
RQD	Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks excluded) Divided by the Total Length of the Core Run Times 100%.

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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

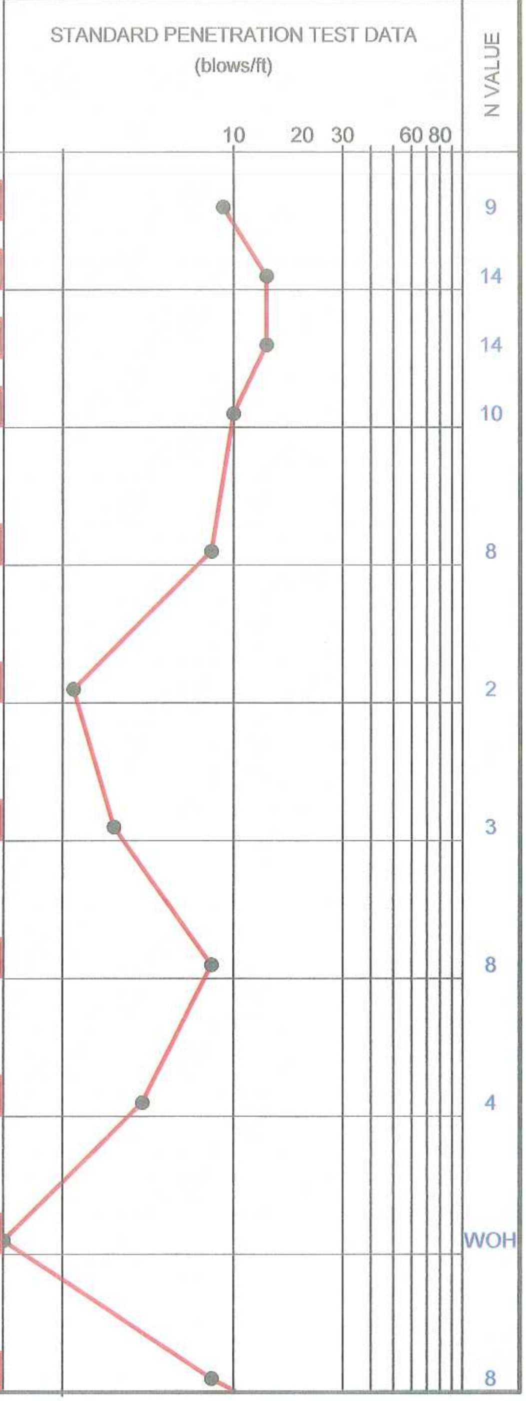
BORING LOG B-1

DATE DRILLED: 12/17/04
ELEVATION: 22.5
DRILLING METHOD: Mud Rotary
BORING DEPTH: 120.0
LOGGED BY: P. Skipper
WATER LEVEL: Water level at 20 ft after 24 hr.
DRILLER:
DRILL RIG: CME 45-B

NOTES:
Boring located at East end of bridge, 35 ft right of centerline of bridge on the SB shoulder.

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet)
	TOPSOIL	
	CLAYEY POORLY GRADED SAND (SP-SC)(FILL) - mostly medium to fine sands, some fines observed at varying depths, tan, wet to saturated, loose to medium dense.	
5		17.5
10		12.5
15		7.5
	FAT CLAY (CH) - medium to high plasticity fines, trace sands, few organics, medium toughness, high dry strength, gray, saturated, very soft to firm.	
20		2.5
25		-2.5
30		-7.5
	SILTY FAT CLAY (CH) - medium to low plasticity fines, trace sands, high toughness, very high dry strength, gray, saturated, very soft to soft.	
35		-12.5
40		-17.5
	SANDY CLAY (CL) -medium to low plasticity fines, some fine sands, medium tough, high dry strength, saturated, gray, firm.	

SAMPLE NO.	N VALUE
1	9
2	14
3	14
4	10
5	8
6	2
7	3
8	8
9	4
10	WOH
11	8

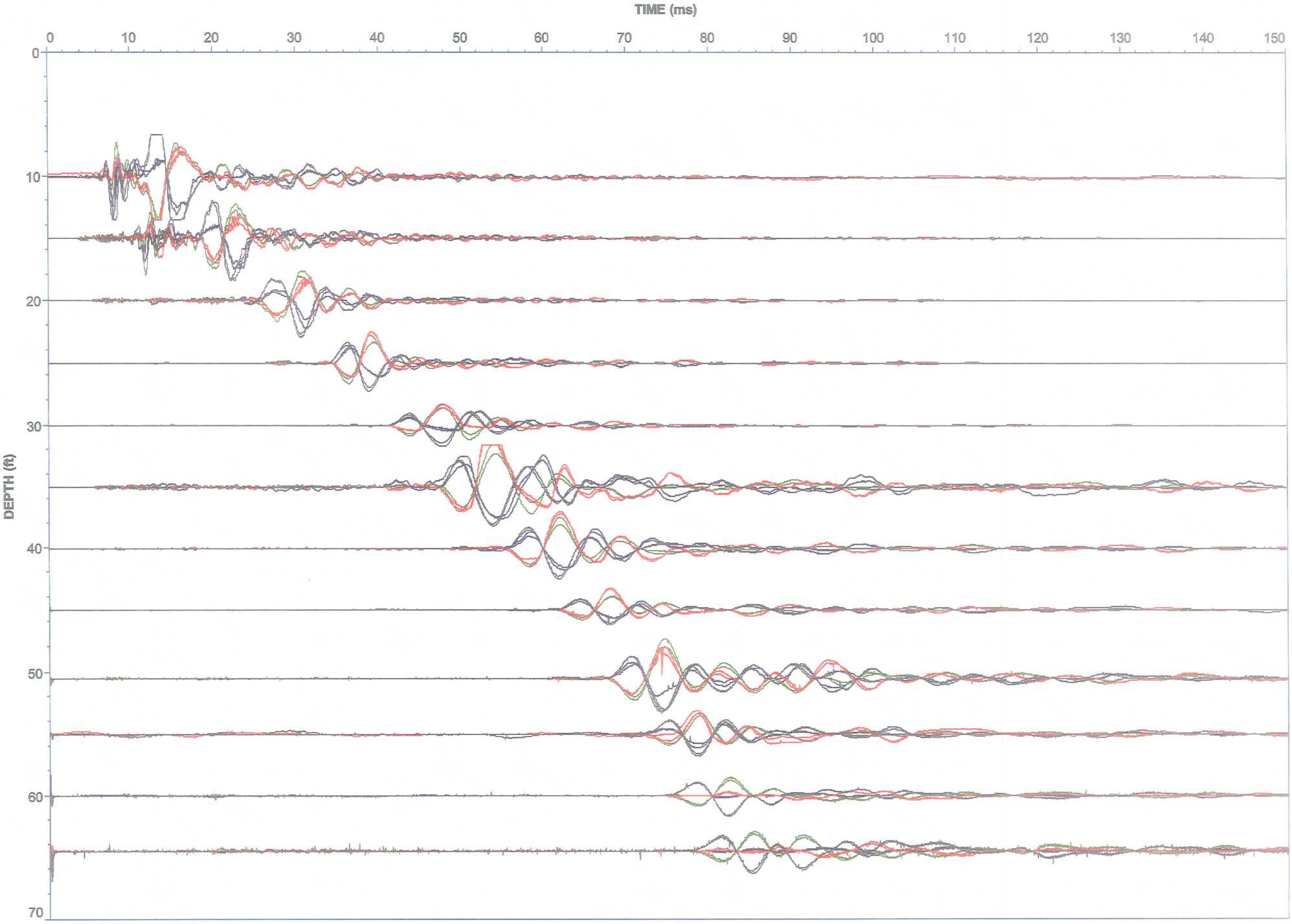


NEW LOG 1611-04-569.GPJ WITH CPT.GDT 4/4/05

NOTES:

1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

Job No: 04-205SC Client: S&ME, Inc. Project Title: 701 By-Pass Hole: CPT-C3 Date: 12/17/04 Oversite: P. Skipper



PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-1

DATE DRILLED: 12/17/04

ELEVATION: 22.5

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: P. Skipper

WATER LEVEL: Water level at 20 ft after 24 hr.

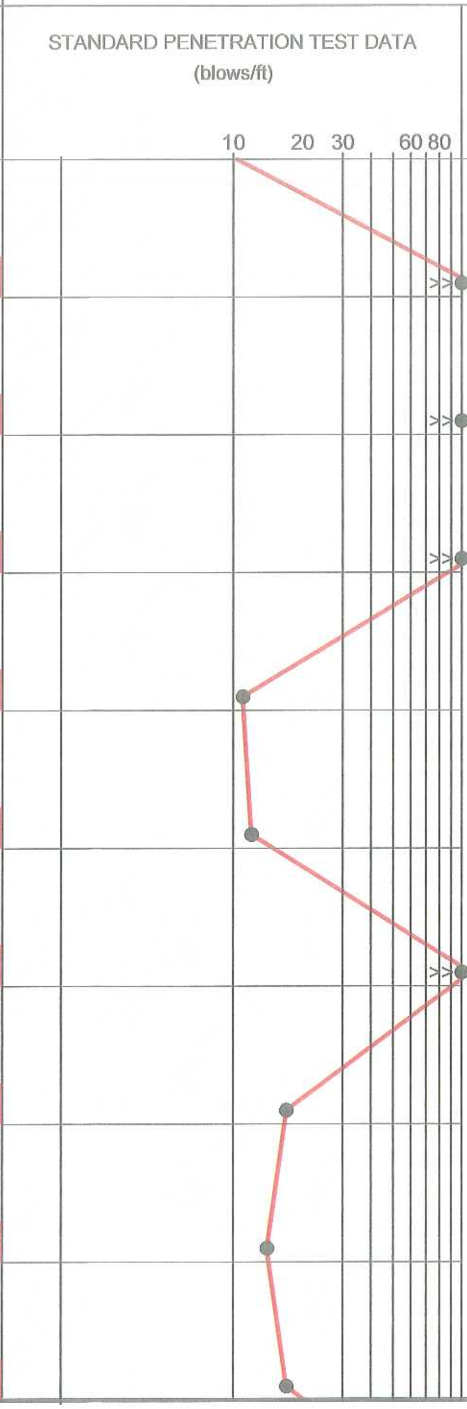
DRILLER:

DRILL RIG: CME 45-B

NOTES:
Boring located at East end of bridge, 35 ft right of centerline of bridge on the SB shoulder.

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	CLAYEY SAND (SC) - medium to fine sands, some plastic fines, cemented particles, shell fragments, dry to moist, dark gray, very dense. Sample crumbles in hand.				
50			-27.5	12	50/ 1"
55			-32.5	13	50/ 2"
	SANDY FAT CLAY (CH) - medium to high plasticity fines, some fine sands, shell observed, cemented particles, medium toughness, high dry strength, wet, dark gray, very dense.				
60			-37.5	14	50/ 3"
	SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard. Hard drilling encountered at 65 feet. Approximately 15 inches thick.				
65			-42.5	15	11
70			-47.5	16	12
75			-52.5	17	50/ 0"
	Hard drilling encountered at 75 feet. Approximately 8 inches thick.				
80			-57.5	18	17
85			-62.5	19	14
				20	17

GRAPHIC LOG



STANDARD PENETRATION TEST DATA (blows/ft)

NOTES:

1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-1

DATE DRILLED:	12/17/04	ELEVATION:	22.5
DRILLING METHOD:	Mud Rotary	BORING DEPTH:	120.0
LOGGED BY:	P. Skipper	WATER LEVEL:	Water level at 20 ft after 24 hr.
DRILLER:		DRILL RIG:	CME 45-B

NOTES:
Boring located at East end of bridge, 35 ft right of centerline of bridge on the SB shoulder.

MATERIAL DESCRIPTION

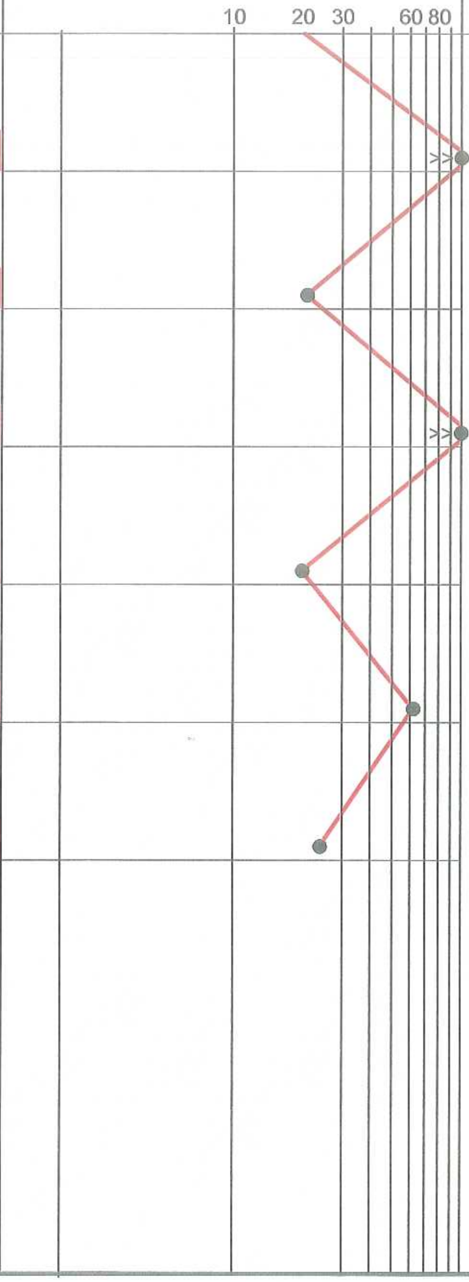
SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.
(continued)

Hard drilling encounterd at 95 feet. Approximately 10 inches thick.

Boring B-1 terminated at 120 feet.

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	N VALUE
95	-72.5	21	50/ 1"
100	-77.5	22	21
105	-82.5	23	50/ 1"
110	-87.5	24	20
115	-92.5	25	62
120	-97.5	26	24

STANDARD PENETRATION TEST DATA (blows/ft)



NOTES:

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2. BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.
3. STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.
4. WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.

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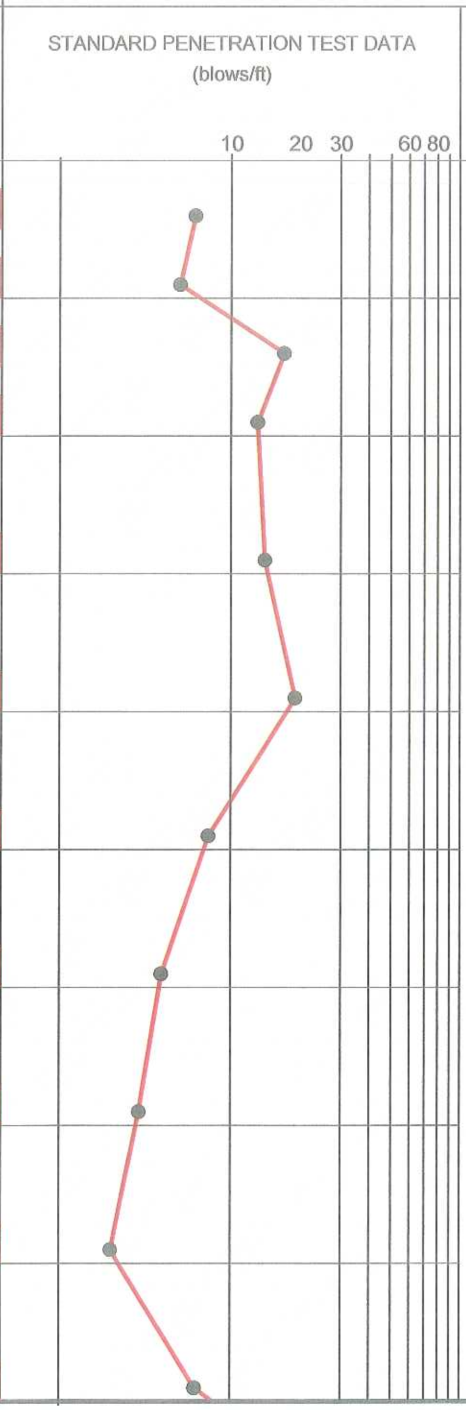
PROJECT:	US 701 Bridges at Yauhannah	**BORING LOG**	**B-2**
PROJECT NUMBER:	1611-04-569		
DATE DRILLED:	12/16/04	ELEVATION:	22.5
DRILLING METHOD:	Mud Rotary	BORING DEPTH:	120.0
LOGGED BY:	D. Hudspith	WATER LEVEL:	Water level at 21 ft after 24 hr.
DRILLER:		DRILL RIG:	CME 45-B

NOTES:
Boring located 100 ft South from SE corner of Bridge 2 at access road to river. Boring located on the SB shoulder, 30 ft right of CL.

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet)
	TOPSOIL	
	CLAYEY POORLY GRADED SAND (SP-SC)(FILL) - mostly medium to fine sands, some fines observed at varying depths, tan, wet to saturated, loose to medium dense.	
5		17.5
10		12.5
15		7.5
20		2.5
	FAT CLAY (CH) - medium to high plasticity fines, trace sands, few organics, medium toughness, high dry strength, gray, saturated, firm.	
25		-2.5
30		-7.5
	SANDY CLAY (CL) - medium to low plasticity, some fine sands, saturated, gray, soft.	
35		-12.5
	POORLY GRADED SAND (SP) - medium to fine sands, trace fines, saturated, gray, loose to very dense.	
40		-17.5

SAMPLE NO.	N VALUE
1	7
2	6
3	17
4	13
5	14
6	19
7	8
8	5
9	4
10	3
11	7



NOTES:

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2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-2

DATE DRILLED: 12/16/04
ELEVATION: 22.5
DRILLING METHOD: Mud Rotary
BORING DEPTH: 120.0
LOGGED BY: D. Hudspith
WATER LEVEL: Water level at 21 ft after 24 hr.
DRILLER:
DRILL RIG: CME 45-B

NOTES:
Boring located 100 ft South from SE corner of Bridge 2 at access road to river. Boring located on the SB shoulder, 30 ft right of CL.

MATERIAL DESCRIPTION

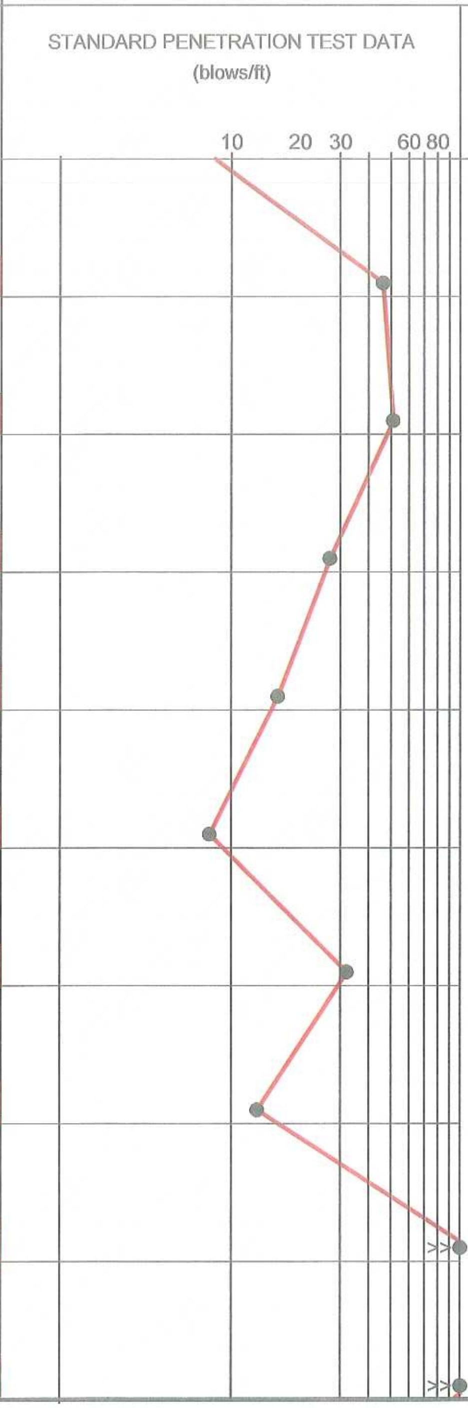
POORLY GRADED SAND (SP) - medium to fine sands, trace fines, saturated, gray, loose to very dense. *(continued)*

FAT CLAY (CH) - medium to high plasticity fines, trace sands, saturated, gray, medium toughness, high dry strength, firm.

POORLY GRADED SAND with CLAY SEAMS (SP) - medium to fine sands, some clay seams observed, saturated, gray, dense.

SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, very soft to very hard.

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	N VALUE
50	-27.5	12	46
55	-32.5	13	51
60	-37.5	14	27
65	-42.5	15	16
70	-47.5	16	8
75	-52.5	17	32
80	-57.5	18	13
85	-62.5	19	50/ 5"
		20	50/



NEW LOG 1611-04-569.GPJ WITH CPT.GDT 4/4/05

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4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-2

DATE DRILLED: 12/16/04

ELEVATION: 22.5

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: D. Hudspith

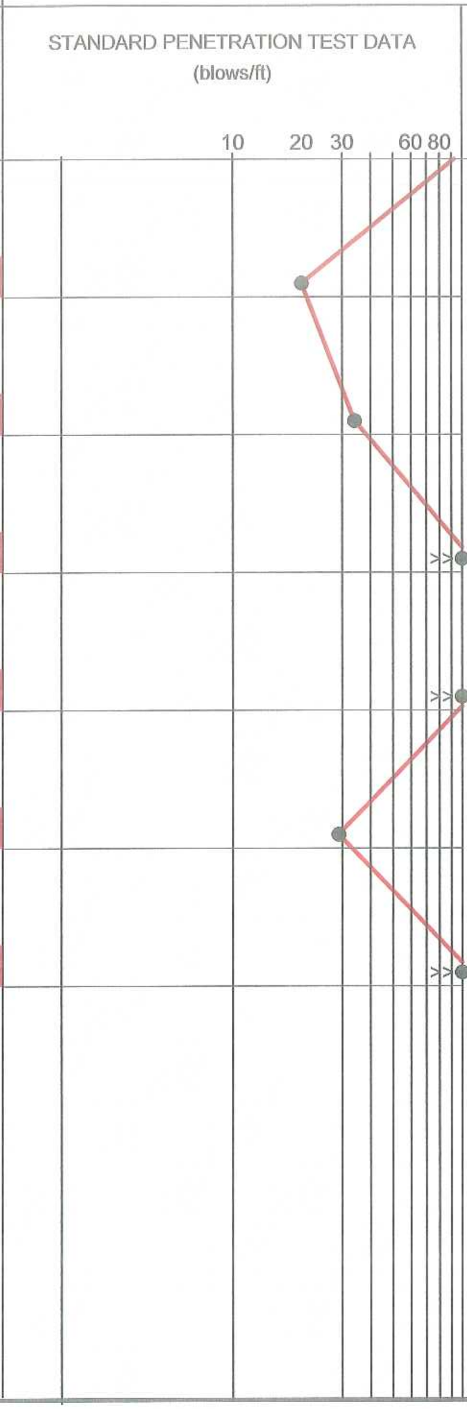
WATER LEVEL: Water level at 21 ft after 24 hr.

DRILLER:

DRILL RIG: CME 45-B

NOTES:
Boring located 100 ft South from SE corner of Bridge 2 at access road to river. Boring located on the SB shoulder, 30 ft right of CL.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, very soft to very hard.** ***(continued)***				5"
95	▨			-72.5	21 ☐	20
100	▨			-77.5	22 ☐	34
105	▨			-82.5	23 ☐	50/ 1"
110	▨			-87.5	24 ☐	50/ 1"
115	▨			-92.5	25 ☐	29
120	▨			-97.5	26 ☐	50/ 3"
		Boring B-2 terminated at 120 feet.				



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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-3

DATE DRILLED: 12/15/04

ELEVATION: 5.0

DRILLING METHOD: Mud Rotary

BORING DEPTH: 100.0

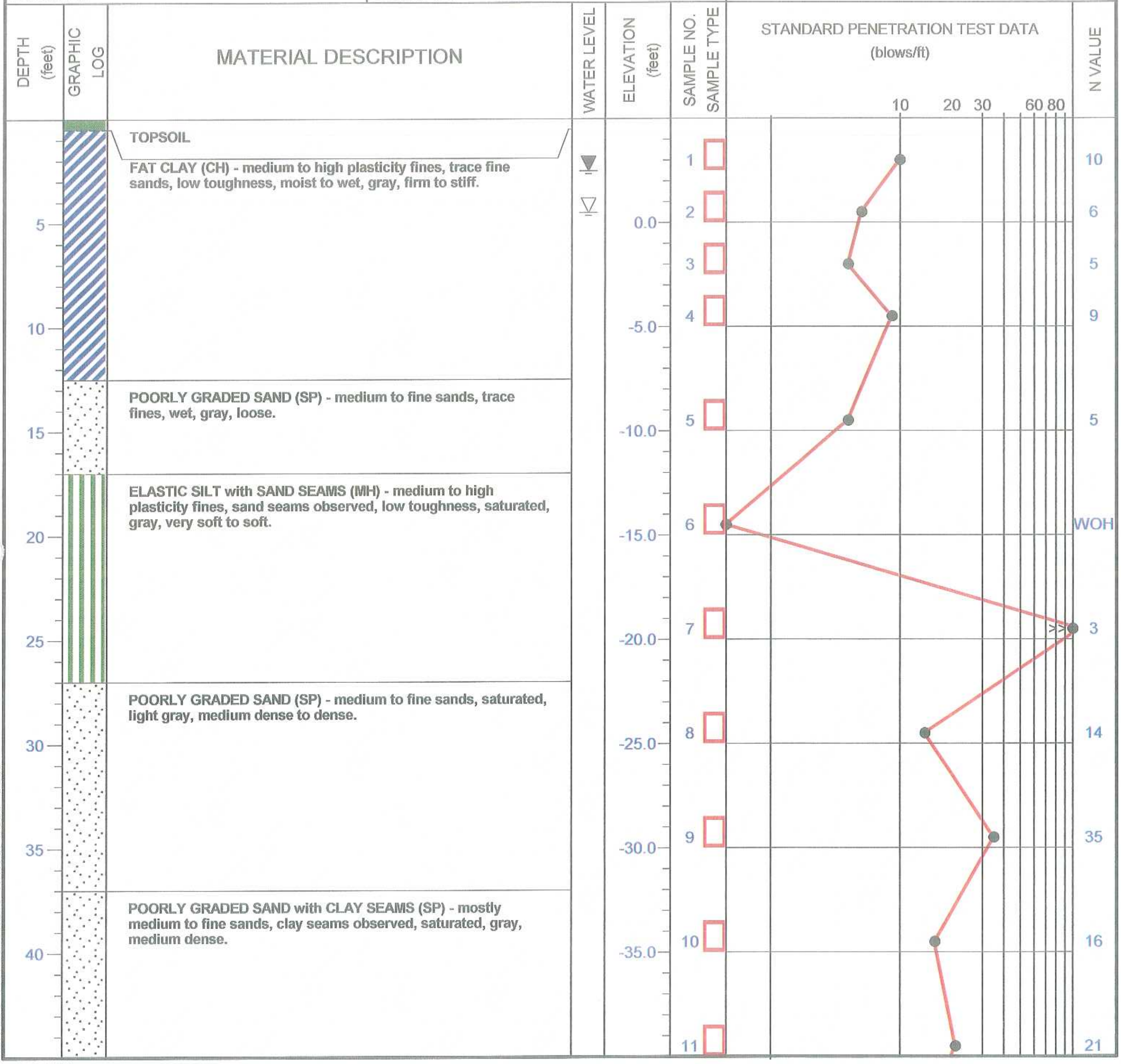
LOGGED BY: D. Hudspith

WATER LEVEL: Water level at 2.5 ft after 24 hr.

DRILLER:

DRILL RIG: CME 45-B

NOTES:



TOPSOIL

FAT CLAY (CH) - medium to high plasticity fines, trace fine sands, low toughness, moist to wet, gray, firm to stiff.

POORLY GRADED SAND (SP) - medium to fine sands, trace fines, wet, gray, loose.

ELASTIC SILT with SAND SEAMS (MH) - medium to high plasticity fines, sand seams observed, low toughness, saturated, gray, very soft to soft.

POORLY GRADED SAND (SP) - medium to fine sands, saturated, light gray, medium dense to dense.

POORLY GRADED SAND with CLAY SEAMS (SP) - mostly medium to fine sands, clay seams observed, saturated, gray, medium dense.

SAMPLE NO.	N VALUE
1	10
2	6
3	5
4	9
5	5
6	WOH
7	3
8	14
9	35
10	16
11	21

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PROJECT:	US 701 Bridges at Yauhannah	BORING LOG	B-3
PROJECT NUMBER:	1611-04-569		
DATE DRILLED:	12/15/04	ELEVATION:	5.0
DRILLING METHOD:	Mud Rotary	BORING DEPTH:	100.0
LOGGED BY:	D. Hudspith	WATER LEVEL:	Water level at 2.5 ft after 24 hr.
DRILLER:		DRILL RIG:	CME 45-B

NOTES:

MATERIAL DESCRIPTION

POORLY GRADED SAND with CLAY SEAMS (SP) - mostly medium to fine sands, clay seams observed, saturated, gray, medium dense. *(continued)*

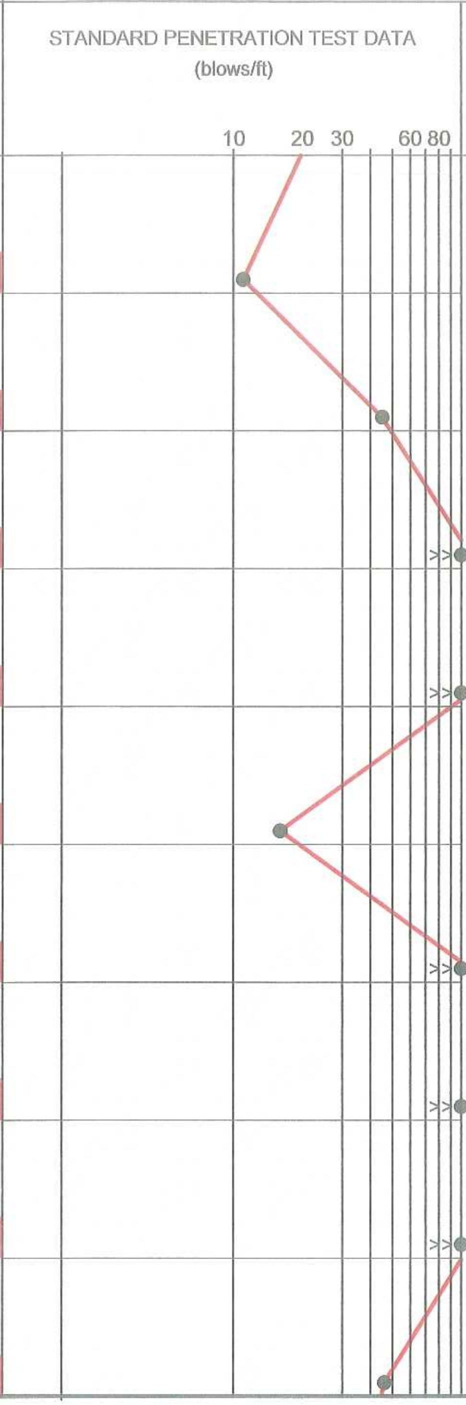
POORLY GRADED SAND (SP) - medium to fine sands, trace fines, saturated, gray, dense to very dense.

SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, very stiff to very hard.

Hard drilling encountered at 76 feet. Approximately 8 inches thick.

Hard drilling encountered from approximately 83 to 100 feet.

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	N VALUE
50	-45.0	12	11
55	-50.0	13	45
60	-55.0	14	50/ 1"
65	-60.0	15	50/ 0"
70	-65.0	16	16
75	-70.0	17	50/ 0"
80	-75.0	18	50/ 1"
85	-80.0	19	50/ 1"
		20	46



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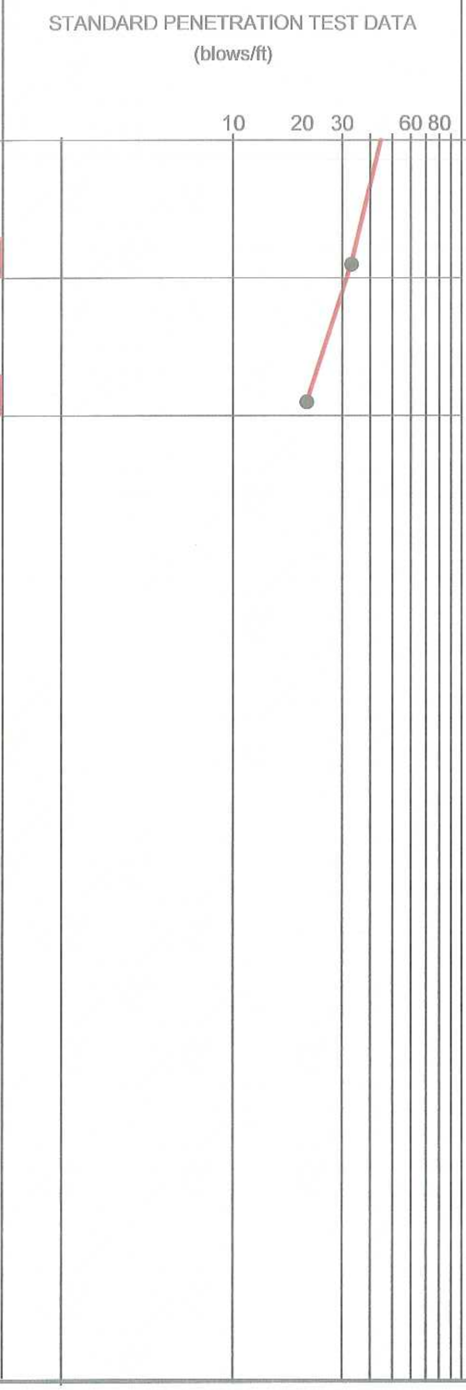
PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-3

DATE DRILLED: 12/15/04	ELEVATION: 5.0
DRILLING METHOD: Mud Rotary	BORING DEPTH: 100.0
LOGGED BY: D. Hudspith	WATER LEVEL: Water level at 2.5 ft after 24 hr.
DRILLER:	DRILL RIG: CME 45-B

NOTES:



DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, very stiff to very hard.** ***(continued)***				
95	▨			-90.0	21 ☐	33
100		**Boring B-3 terminated at 100 feet.**		-95.0	22 ☐	21

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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-4

DATE DRILLED: 12/14/04

ELEVATION: 4.0

DRILLING METHOD: Mud Rotary

BORING DEPTH: 110.0

LOGGED BY: D. Hudspith

WATER LEVEL: Water level at 2.5 ft after 24 hr.

DRILLER:

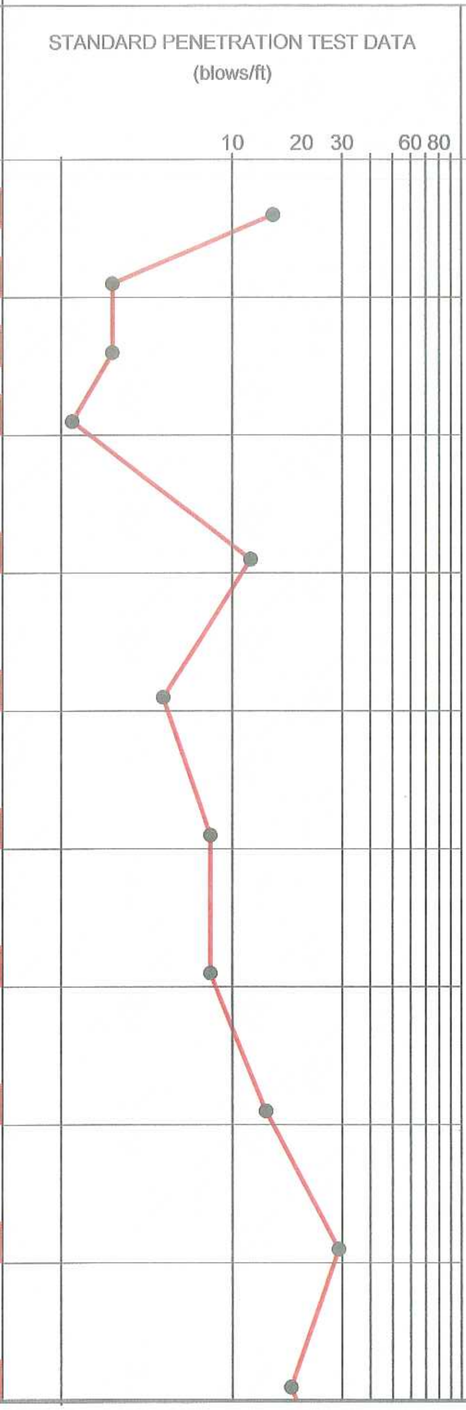
DRILL RIG: CME 45-B

NOTES:
Boring located between Bent 3 & Bent 4 on middle bridge, 35 ft right of centerline of bridge on the SB shoulder.

MATERIAL DESCRIPTION
TOPSOIL
CLAYEY SAND (SC) - medium to fine sands, some fines, gray , wet, very loose.
FAT CLAY (CH) - medium to high plasticity fines, trace sands, wet, gray, very soft.
CLAYEY SAND (SC) - medium to fine sands, some fines, few organics observed, wet, gray, very loose.
POORLY GRADED SAND (SP) - medium to fine sands, trace fines, some organics observed, wet, gray, loose to dense.

DEPTH (feet)	ELEVATION (feet)
5	-1.0
10	-6.0
15	-11.0
20	-16.0
25	-21.0
30	-26.0
35	-31.0
40	-36.0

SAMPLE NO.	N VALUE
1	15
2	3
3	3
4	2
5	12
6	5
7	8
8	8
9	14
10	29
11	18



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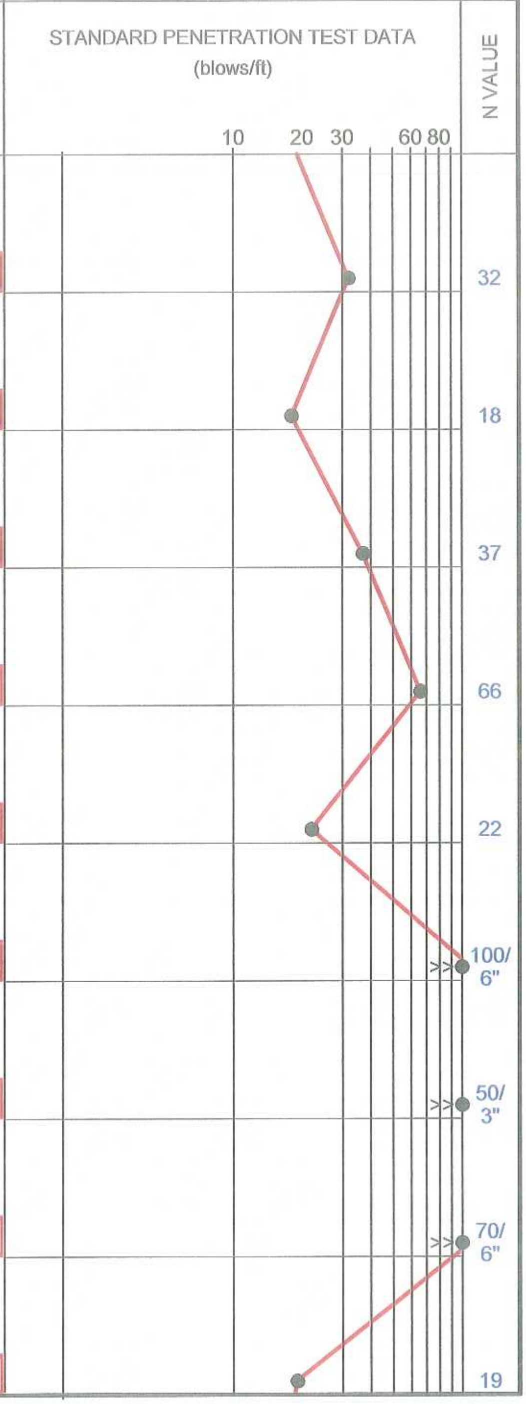
PROJECT: US 701 Bridges at Yauhannah	BORING LOG **B-4**
PROJECT NUMBER: 1611-04-569	

DATE DRILLED: 12/14/04	ELEVATION: 4.0	NOTES: Boring located between Bent 3 & Bent 4 on middle bridge, 35 ft right of centerline of bridge on the SB shoulder.
DRILLING METHOD: Mud Rotary	BORING DEPTH: 110.0	
LOGGED BY: D. Hudspith	WATER LEVEL: Water level at 2.5 ft after 24 hr.	
DRILLER:	DRILL RIG: CME 45-B	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
		POORLY GRADED SAND (SP) - medium to fine sands, trace fines, some organics observed, wet, gray, loose to dense. ***(continued)***				
50				-46.0	12	32
55				-51.0	13	18
60				-56.0	14	37
65				-61.0	15	66
		SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.				
70				-66.0	16	22
		Hard drilling encountered at approximately 72 feet.				
75				-71.0	17	100/ 6"
80				-76.0	18	50/ 3"
		Hard drilling encountered at approximately 83 feet. Approximately 12 inches thick.				
85				-81.0	19	70/ 6"
		Hard drilling encountered at approximately 85 feet. Approximately 15 inches thick.				
					20	19



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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-4

DATE DRILLED: 12/14/04

ELEVATION: 4.0

DRILLING METHOD: Mud Rotary

BORING DEPTH: 110.0

LOGGED BY: D. Hudspith

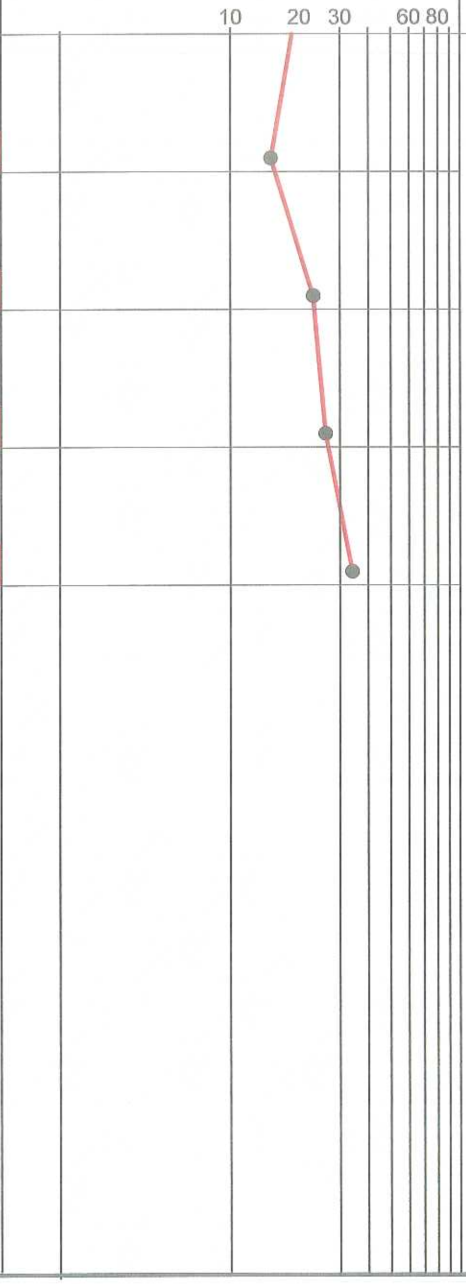
WATER LEVEL: Water level at 2.5 ft after 24 hr.

DRILLER:

DRILL RIG: CME 45-B

NOTES:
Boring located between Bent 3 & Bent 4 on middle bridge, 35 ft right of centerline of bridge on the SB shoulder.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft)	N VALUE
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.** ***(continued)***					
95	▨			-91.0	21 ☐		15
100	▨			-96.0	22 ☐		23
	▨	**Hard drilling encountered at approximately 101 feet. Approximately 6 inches thick.**					
105	▨			-101.0	23 ☐		26
110	▨			-106.0	24 ☐		34
		Boring B-4 terminated at 110 feet.					



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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-5

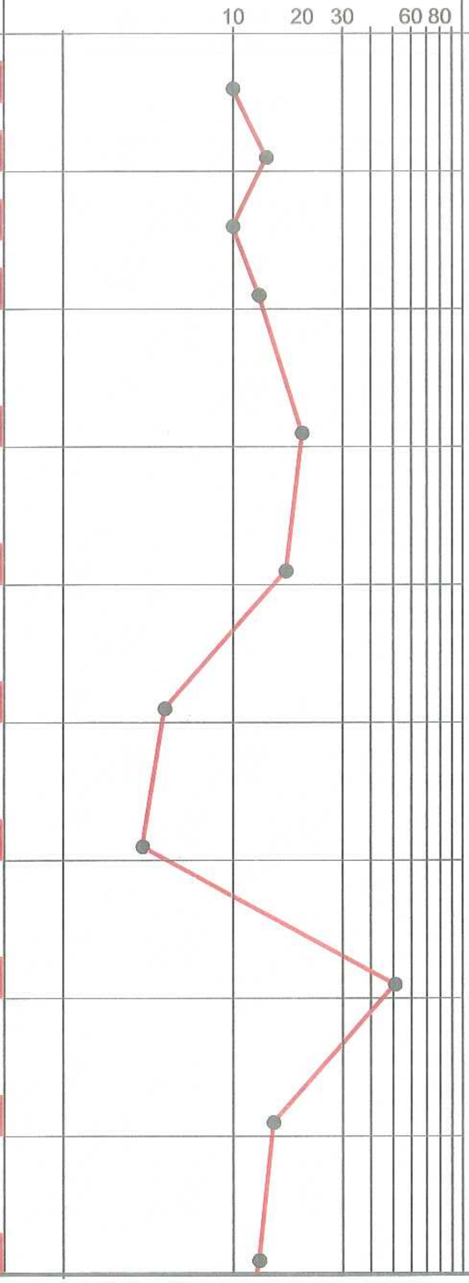
DATE DRILLED: 12/13/04
ELEVATION: 22.5
DRILLING METHOD: Mud Rotary
BORING DEPTH: 120.0
LOGGED BY: D. Hudspith
WATER LEVEL: Water level at 20 ft after 24 hr.
DRILLER:
DRILL RIG: CME 45-B

NOTES:
Boring located at West end of bridge.

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)
0	**TOPSOIL**		
	CLAYEY POORLY GRADED SAND (SP-SC)(FILL) - mostly medium to fine sands, some fines observed at varying depths, tan, wet to saturated, loose to medium dense.		
5			17.5
10			12.5
		∇	
15			7.5
20		▼	2.5
	FAT CLAY (CH) - residual soil, medium to high plasticity fines, trace sands, organics observed, medium toughness, saturated, gray, soft to firm.		
25			-2.5
30			-7.5
	POORLY GRADED SAND (SP) - mdeium to fine sands, trace fines, saturated, light gray, loose to very dense.		
35			-12.5
40			-17.5

SAMPLE NO.	N VALUE
1	10
2	14
3	10
4	13
5	20
6	17
7	5
8	4
9	51
10	15
11	13

STANDARD PENETRATION TEST DATA (blows/ft)



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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-5

DATE DRILLED: 12/13/04

ELEVATION: 22.5

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: D. Hudspith

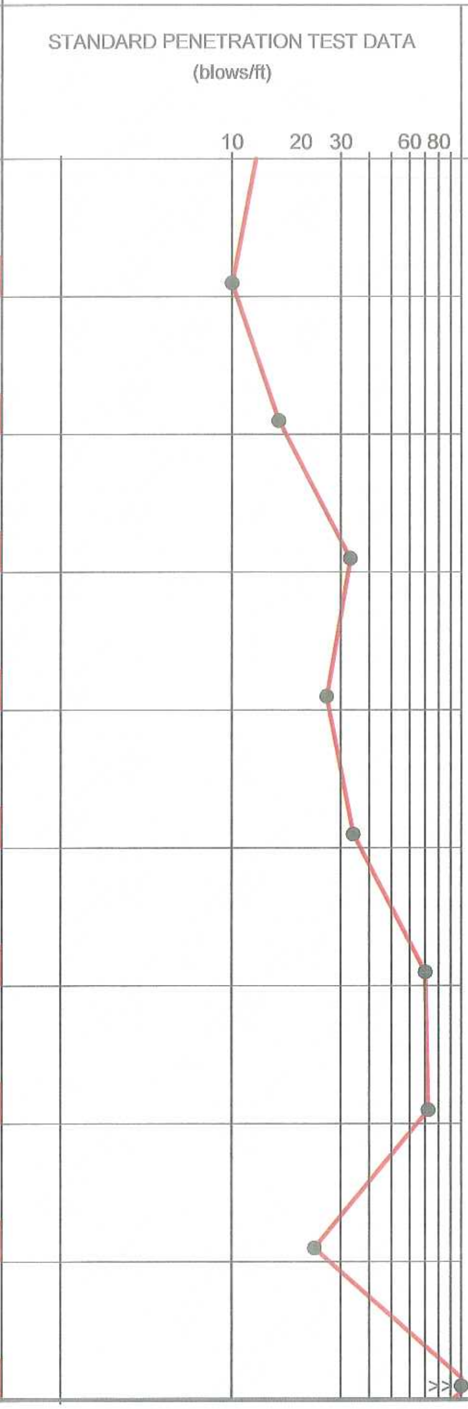
WATER LEVEL: Water level at 20 ft after 24 hr.

DRILLER:

DRILL RIG: CME 45-B

NOTES:
Boring located at West end of bridge.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	⁘	**POORLY GRADED SAND (SP) - mdeium to fine sands, trace fines, saturated, light gray, loose to very dense.** *(continued)*				
50	⁘			-27.5	12 ☐	10
55	⁘			-32.5	13 ☐	16
60	⁘			-37.5	14 ☐	33
65	⁘			-42.5	15 ☐	26
70	⁘			-47.5	16 ☐	34
75	⁘			-52.5	17 ☐	70
80	⁘			-57.5	18 ☐	72
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.**				
85	▨			-62.5	19 ☐	23
	▨				20 ☐	100/



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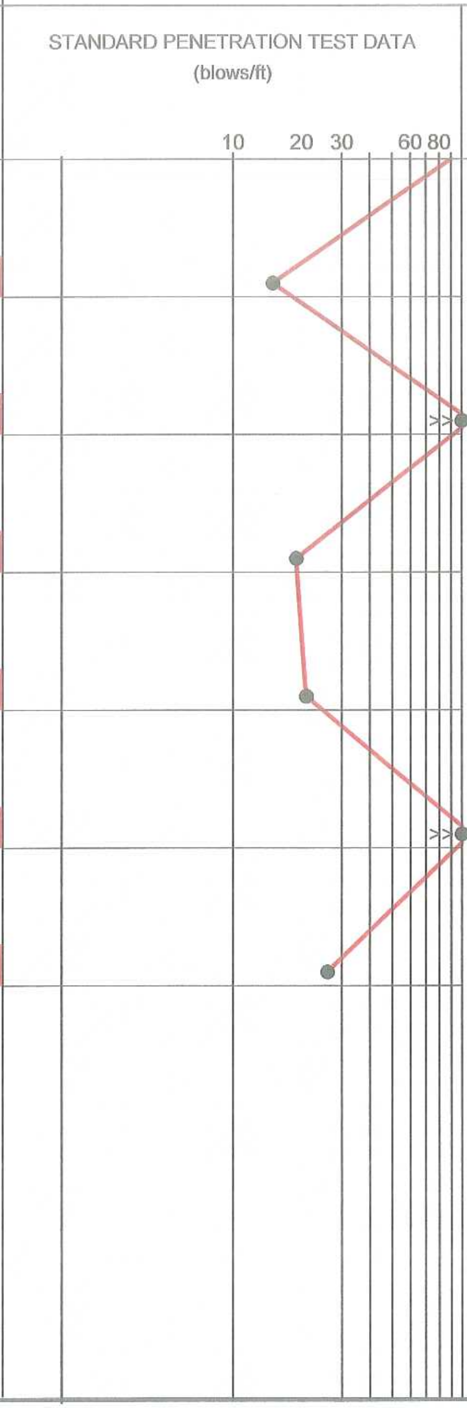
PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-5

DATE DRILLED: 12/13/04	ELEVATION: 22.5	NOTES: Boring located at West end of bridge.
DRILLING METHOD: Mud Rotary	BORING DEPTH: 120.0	
LOGGED BY: D. Hudspith	WATER LEVEL: Water level at 20 ft after 24 hr.	
DRILLER:	DRILL RIG: CME 45-B	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.** ***(continued)***				2"
95	▨			-72.5	21 ☐	15
	▨	**Hard drilling observed at 98 feet. Approximately 6 inches thick.**				
100	▨			-77.5	22 ☐	100/ 1"
105	▨			-82.5	23 ☐	19
110	▨			-87.5	24 ☐	21
115	▨			-92.5	25 ☐	50/ 2"
	▨	**Hard drilling observed at 118 feet. Approximately 6 inches thick.**				
120		**Boring B-5 terminated at 120 feet.**		-97.5	26 ☐	26



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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-6

DATE DRILLED: 12/27/04

ELEVATION: 22.5

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: C. Doulton

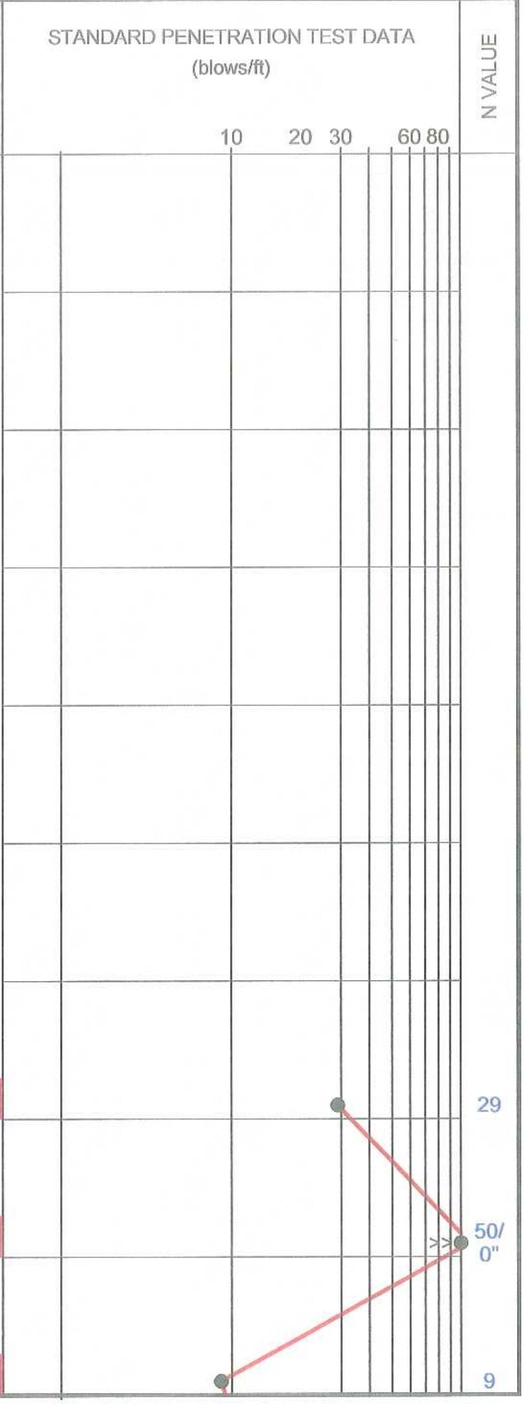
WATER LEVEL: Water level at 11 feet after 24 hr.

DRILLER: Mid-Atlantic Drilling

DRILL RIG: CME 45-B

NOTES:
Boring located at south end of the southernmost bridge, on the north bound side shoulder, approximately 15 feet from the edge of pavement. This boring is a continuance of sounding C-1 which refused at 35 feet.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
		No sampling performed due to previous exploration by CPT rig.				
5				17.5		
10			▼	12.5		
15				7.5		
20				2.5		
25				-2.5		
30				-7.5		
35		SILTY SAND (SM) - mostly fine sands, some low plasticity fines, dark gray, wet. - Hard drilling encountered from approximately 35 to 120 feet due to stratified cemented lenses.		-12.5	1 ☐	29
40				-17.5	2 ☐	50/ 0"
					3 ☐	9



NEW LOG 1611-04-569.GPJ WITH CPT.GDT 4/4/05

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PROJECT: US 701 Bridges at Yauhannah

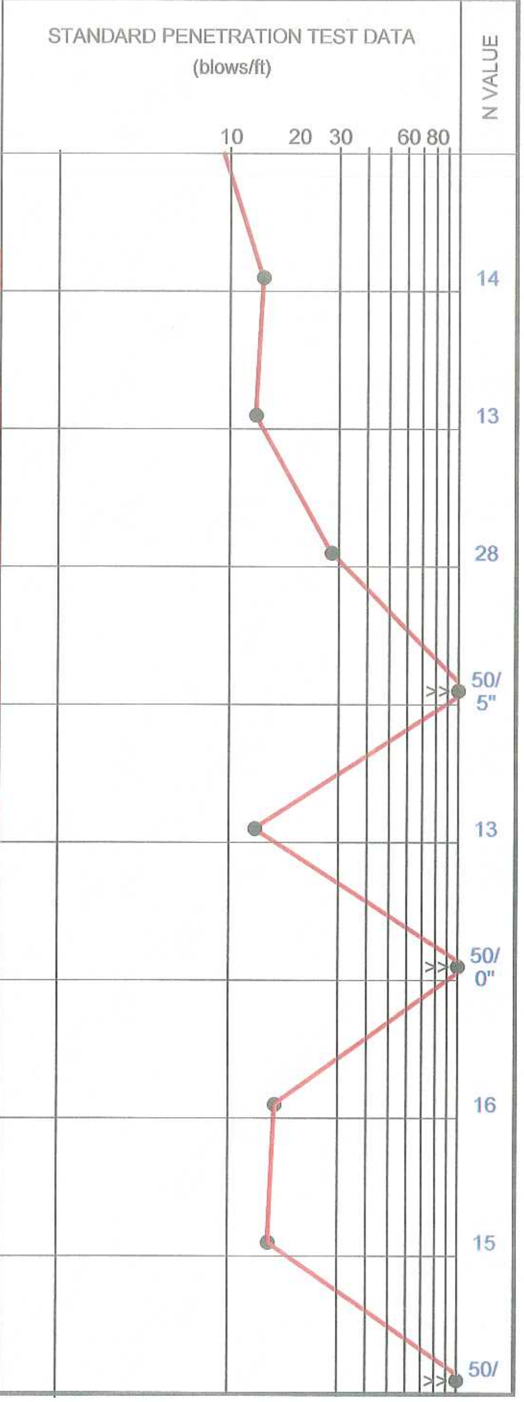
PROJECT NUMBER: 1611-04-569

BORING LOG B-6

DATE DRILLED:	12/27/04	ELEVATION:	22.5
DRILLING METHOD:	Mud Rotary	BORING DEPTH:	120.0
LOGGED BY:	C. Doulton	WATER LEVEL:	Water level at 11 feet after 24 hr.
DRILLER:	Mid-Atlantic Drilling	DRILL RIG:	CME 45-B

NOTES: Boring located at south end of the southernmost bridge, on the north bound side shoulder, approximately 15 feet from the edge of pavement. This boring is a continuance of sounding C-1 which refused at 35 feet.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	SAMPLE TYPE	N VALUE
		SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.					
50				-27.5	4	☐	14
55				-32.5	5	☐	13
60				-37.5	6	☐	28
65				-42.5	7	☐	50/ 5"
70				-47.5	8	☐	13
75				-52.5	9	☐	50/ 0"
80				-57.5	10	☐	16
85				-62.5	11	☐	15
					12	☐	50/



NOTES:

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2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

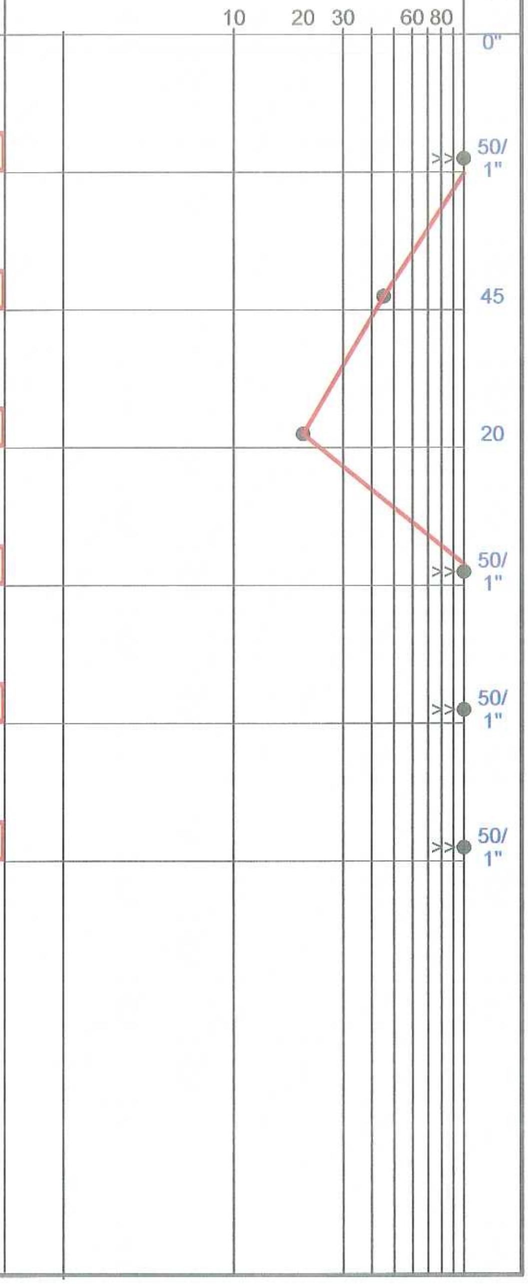
BORING LOG B-6

DATE DRILLED: 12/27/04
ELEVATION: 22.5
DRILLING METHOD: Mud Rotary
BORING DEPTH: 120.0
LOGGED BY: C. Doulton
WATER LEVEL: Water level at 11 feet after 24 hr.
DRILLER: Mid-Atlantic Drilling
DRILL RIG: CME 45-B

NOTES:
Boring located at south end of the southernmost bridge, on the north bound side shoulder, approximately 15 feet from the edge of pavement. This boring is a continuance of sounding C-1 which refused at 35 feet.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	N VALUE
	▨	**SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.** ***(continued)***				0"
95	▨			-72.5	13 ☐	50/ 1"
100	▨			-77.5	14 ☐	45
105	▨			-82.5	15 ☐	20
110	▨			-87.5	16 ☐	50/ 1"
115	▨			-92.5	17 ☐	50/ 1"
120	▨			-97.5	18 ☐	50/ 1"
		Boring B-6 terminated at 120 feet.				

STANDARD PENETRATION TEST DATA (blows/ft)



NOTES:

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2. ***BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.***
3. ***STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.***
4. ***WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.***

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PROJECT: US 701 Bridges at Yauhannah PROJECT NUMBER: 1611-04-569	**BORING LOG B-7**

DATE DRILLED: **12/30/04**	ELEVATION: **22.5**	NOTES: **Boring located at the north end of the northernmost bridge, on the north bound side shoulder, approximately 30 feet from the edge of pavement. This boring is a continuance of sounding C-3, which refused at 65 feet.**
DRILLING METHOD: **Mud Rotary**	BORING DEPTH: **120.0**	
LOGGED BY: **C. Doulton**	WATER LEVEL: **Water level at 13.5 feet after 24 hr.**	
DRILLER: **Mid-Atlantic Drilling**	DRILL RIG: **CME 45-B**	

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft) 10 20 30 60 80	N VALUE
		No sampling performed due to previous exploration by CPT rig.					
5				17.5			
10				12.5			
			▼				
15				7.5			
20				2.5			
25				-2.5			
30				-7.5			
35				-12.5			
40				-17.5			

NEW LOG 1611-04-569.GPJ WITH CPT.GDT 4/4/05

NOTES:

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3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-7

DATE DRILLED: 12/30/04

ELEVATION: 22.5

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: C. Doulton

WATER LEVEL: Water level at 13.5 feet after 24 hr.

DRILLER: Mid-Atlantic Drilling

DRILL RIG: CME 45-B

NOTES:
Boring located at the north end of the northernmost bridge, on the north bound side shoulder, approximately 30 feet from the edge of pavement. This boring is a continuance of sounding C-3, which refused at 65 feet.

MATERIAL DESCRIPTION

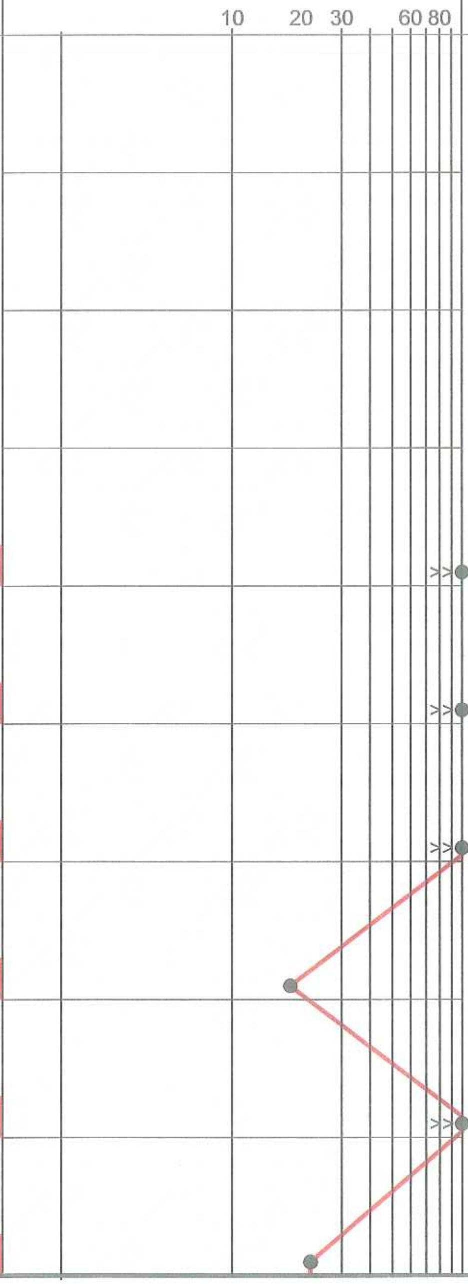
No sampling performed due to previous exploration by CPT rig.
(continued)

SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.

- Hard drilling encountered from approximately 65 to 120 feet due to stratified cemented lenses.

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	N VALUE
50	-27.5		
55	-32.5		
60	-37.5		
65	-42.5	1	50/ 2"
70	-47.5	2	50/ 2"
75	-52.5	3	50/ 0"
80	-57.5	4	18
85	-62.5	5	50/ 4"
		6	22

STANDARD PENETRATION TEST DATA (blows/ft)



NOTES:

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2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

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ENVIRONMENTAL SERVICES

PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-7

DATE DRILLED: 12/30/04

DRILLING METHOD: Mud Rotary

LOGGED BY: C. Doulton

DRILLER: Mid-Atlantic Drilling

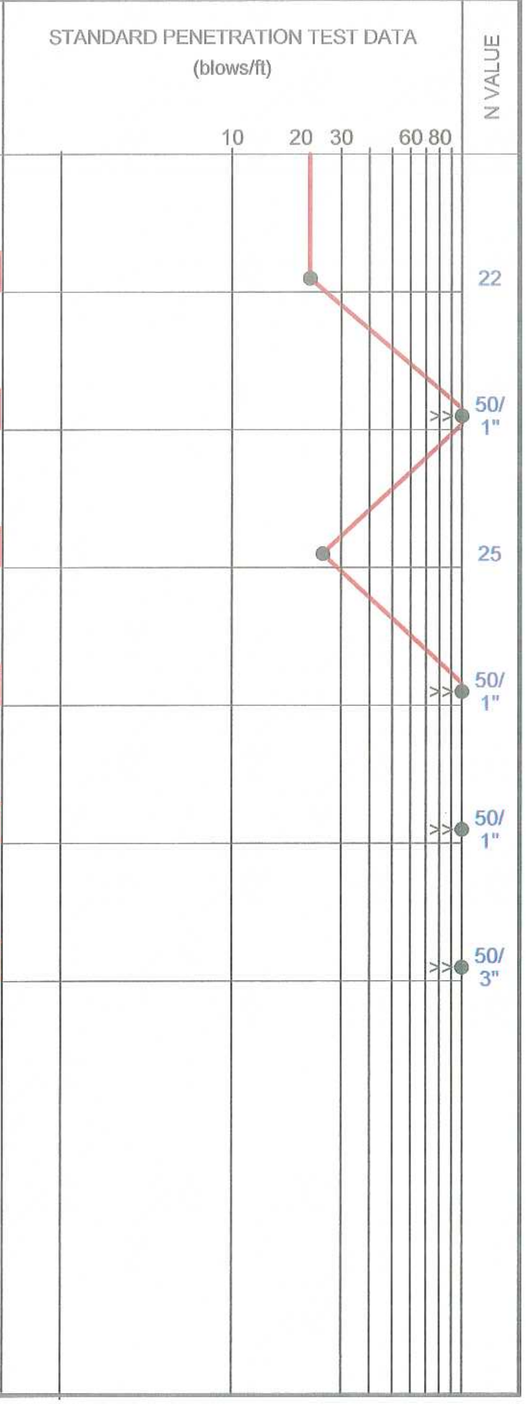
ELEVATION: 22.5

BORING DEPTH: 120.0

WATER LEVEL: Water level at 13.5 feet after 24 hr.

DRILL RIG: CME 45-B

NOTES:
Boring located at the north end of the northernmost bridge, on the north bound side shoulder, approximately 30 feet from the edge of pavement. This boring is a continuance of sounding C-3, which refused at 65 feet.



DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	SAMPLE TYPE	N VALUE
	▨	SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muriatic acid, stiff to very hard. *(continued)*					
95	▨			-72.5	7	☐	22
100	▨			-77.5	8	☐	50/ 1"
105	▨			-82.5	9	☐	25
110	▨			-87.5	10	☐	50/ 1"
115	▨			-92.5	11	☐	50/ 1"
120	▨			-97.5	12	☐	50/ 3"
		Boring B-7 terminated at 120 feet.					

NOTES:

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4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

PROJECT:	US 701 Bridges at Yauhannah	**BORING LOG**	**B-8**
PROJECT NUMBER:	1611-04-569		

DATE DRILLED:	12/31/04	ELEVATION:	5.0
DRILLING METHOD:	Mud Rotary	BORING DEPTH:	120.0
LOGGED BY:	C. Doulton	WATER LEVEL:	Water level at 3.5 feet after 24 hr.
DRILLER:	Mid-Atlantic Drilling	DRILL RIG:	CME 45-B

NOTES:
Boring located at Bent 15 beneath the middle bridge. This boring is a continuance of sounding C-2, which refused at 60 feet.

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO. SAMPLE TYPE	STANDARD PENETRATION TEST DATA (blows/ft) 10 20 30 60 80	N VALUE
		No sampling performed due to previous exploration by CPT rig.					
			▼				
5				0.0			
10				-5.0			
15				-10.0			
20				-15.0			
25				-20.0			
30				-25.0			
35				-30.0			
40				-35.0			

NOTES:

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3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-8

DATE DRILLED: 12/31/04
ELEVATION: 5.0
DRILLING METHOD: Mud Rotary
BORING DEPTH: 120.0
LOGGED BY: C. Doulton
WATER LEVEL: Water level at 3.5 feet after 24 hr.
DRILLER: Mid-Atlantic Drilling
DRILL RIG: CME 45-B

NOTES:
Boring located at Bent 15 beneath the middle bridge. This boring is a continuance of sounding C-2, which refused at 60 feet.

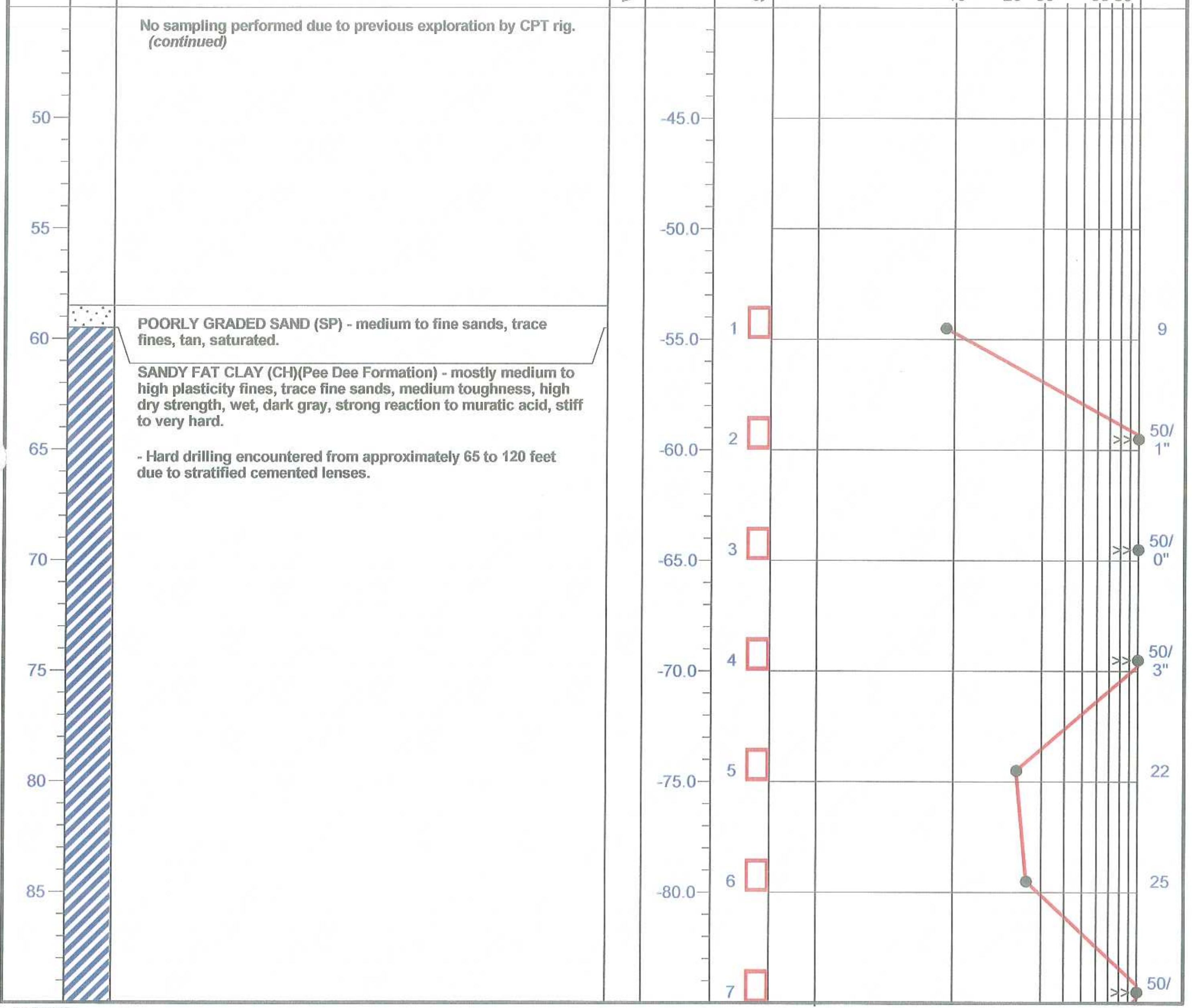
DEPTH (feet) | GRAPHIC LOG | MATERIAL DESCRIPTION | WATER LEVEL | ELEVATION (feet) | SAMPLE NO. SAMPLE TYPE | STANDARD PENETRATION TEST DATA (blows/ft) | N VALUE

No sampling performed due to previous exploration by CPT rig. *(continued)*

POORLY GRADED SAND (SP) - medium to fine sands, trace fines, tan, saturated.

SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.

- Hard drilling encountered from approximately 65 to 120 feet due to stratified cemented lenses.



1	9
2	50/ 1"
3	50/ 0"
4	50/ 3"
5	22
6	25
7	50/

NOTES:

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3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

PROJECT: US 701 Bridges at Yauhannah

PROJECT NUMBER: 1611-04-569

BORING LOG B-8

DATE DRILLED: 12/31/04

ELEVATION: 5.0

DRILLING METHOD: Mud Rotary

BORING DEPTH: 120.0

LOGGED BY: C. Doulton

WATER LEVEL: Water level at 3.5 feet after 24 hr.

DRILLER: Mid-Atlantic Drilling

DRILL RIG: CME 45-B

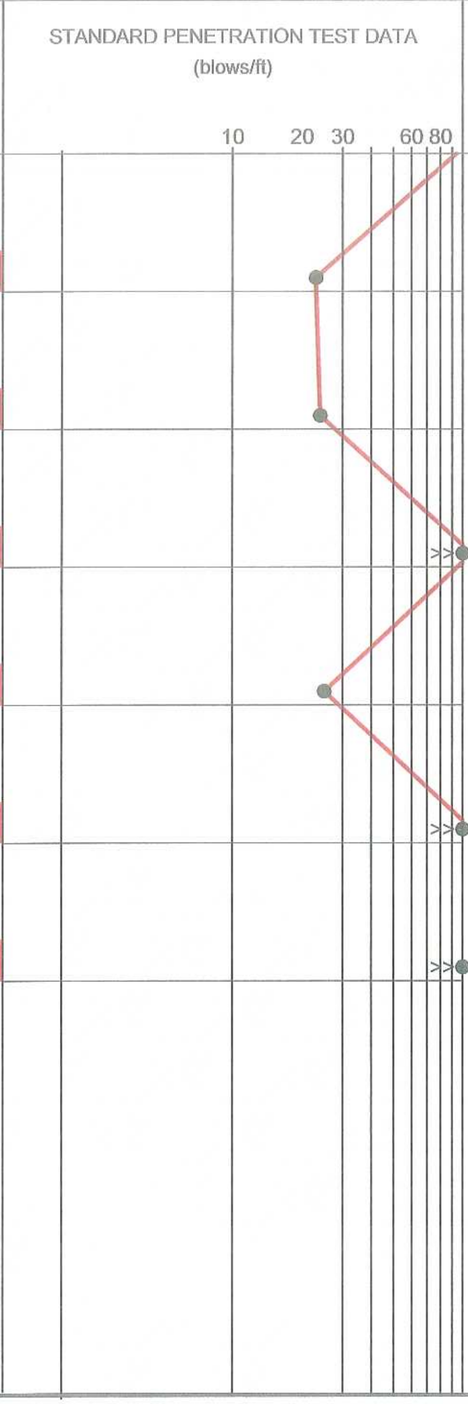
NOTES:
Boring located at Bent 15 beneath the middle bridge. This boring is a continuance of sounding C-2, which refused at 60 feet.

MATERIAL DESCRIPTION

SANDY FAT CLAY (CH)(Pee Dee Formation) - mostly medium to high plasticity fines, trace fine sands, medium toughness, high dry strength, wet, dark gray, strong reaction to muratic acid, stiff to very hard.
(continued)

Boring B-8 terminated at 120 feet.

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	N VALUE
			0"
95	-90.0	8	23
100	-95.0	9	24
105	-100.0	10	50/ 0"
110	-105.0	11	25
115	-110.0	12	50/ 0"
120	-115.0	13	50/ 5"


STANDARD PENETRATION TEST DATA (blows/ft)

NOTES:

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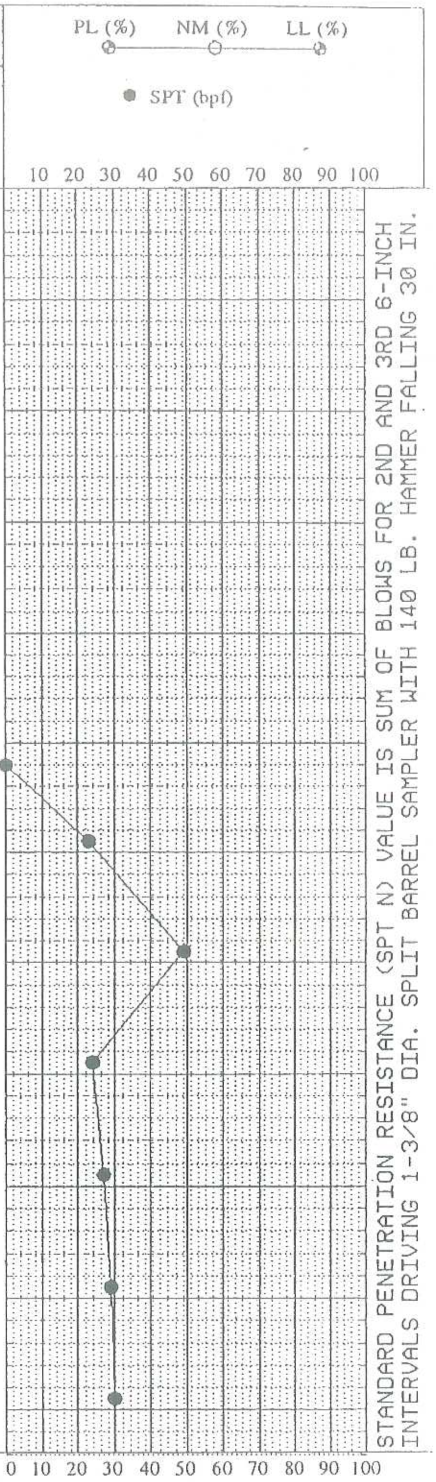
ENVIRONMENTAL SERVICES

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.	LEGEND	ELEV (ft)	IDENT	TYPE	N-COUNT 1ST 6" 2ND 6" 3RD 6"	% Fines
0	GREAT PEE DEE RIVER. (25.0')	■	5.2				
5		■	0.2				
10		■	-4.8				
15		■	-9.8				
20		■	-14.8				
25	POORLY GRADED SAND (SP) - mostly fine-med sand, very loose, tan; - COASTAL PLAIN RESIDUUM	░	-19.8	SS-1	▨	W.O.H.	
	POORLY GRADED SAND (SP) - mostly fine-med sand, firm and dense, light gray - COASTAL PLAIN RESIDUUM	░		SS-2	▨	9-15-6	
30		░	-24.8				
35		░	-29.8	SS-3	▨	6-30-19	
40		░	-34.8	SS-4	▨	9-12-12	
45		░	-39.8	SS-5	▨	8-12-15	
50		░	-44.8	SS-6	▨	10-15-14	
55		░	-49.8	SS-7	▨	5-12-18	



STANDARD PENETRATION RESISTANCE (SPT N) VALUE IS SUM OF BLOWS FOR 2ND AND 3RD 6-INCH INTERVALS DRIVING 1-3/8" DIA. SPLIT BARREL SAMPLER WITH 140 LB. HAMMER FALLING 30 IN.

DRILLER:	J.ALEXANDER
EQUIPMENT:	TRI-CONE ROLLER
METHOD:	ROTARY WASH
HOLE DIA.:	2 7/8"

REMARKS:

US 701 BRIDGE OVER GREAT PEE DEE RIVER - Boring is located 33.0' West of East abut and 24.0' North of center line. File No. 22.56961

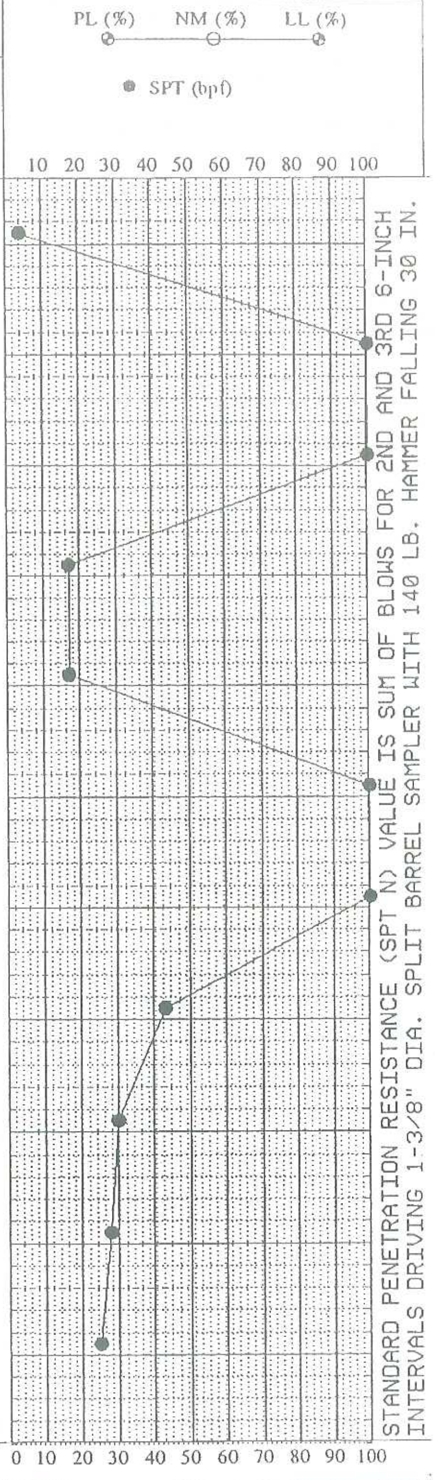
THIS RECORD IS A REASONABLE INTERPRETATION OF SOIL CONDITIONS AT THE BORING LOCATION. SOIL CONDITIONS AT OTHER LOCATIONS MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

PROJECT: US 701 BRIDGE REPLACEMENT
OFFSET: 24.5000 BORING NO.: B-1
STATION: 79914.7000 B-101
DRILLED: April 3, 1996
PROJ. NO.: 30510-6-6549 PAGE 1 OF 2

LAW ENGINEERING, INC.

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.	ELEV (ft)	IDENT	N-COUNT 1ST 6" 2ND 6" 3RD 6"
60	CLAYEY SAND (SC) - mostly fine sand, some med-plastic fines, loose, dark gray - COASTAL PLAIN RESIDUUM	-54.8	SS-8	2-2-2
65	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, interbedded with fine cemented sand seams, very hard, dark gray - COASTAL PLAIN RESIDUUM	-59.8	SS-9	50/2"
70		-64.8	SS-10	50/2"
75	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, very stiff, dark grey - COASTAL PLAIN RESIDUUM	-69.8	SS-11	6-8-9
80	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, interbedded with fine cemented sand seams, very stiff, dark grey - COASTAL PLAIN RESIDUUM	-74.8	SS-12	6-7-10
	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, very stiff, dark gray - COASTAL PLAIN RESIDUUM			
85	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, interbedded with fine cemented sand seams, very hard, dark grey - COASTAL PLAIN RESIDUUM	-79.8	SS-13	50/5.5"
90		-84.8	SS-14	50/2"
95	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, very stiff to hard, dark gray - COASTAL PLAIN RESIDUUM	-89.8	SS-15	17-23-20
100		-94.8	SS-16	18-15-15
105		-99.8	SS-17	7-12-16
110		-104.8	SS-18	10-12-13
	Boring Terminated Field crew noted difficult drilling, hard material at depths of 61-63 and 76-78 feet.			



STANDARD PENETRATION RESISTANCE (SPT N) VALUE IS SUM OF BLOWS FOR 2ND AND 3RD 6-INCH INTERVALS DRIVING 1-3/8" DIA. SPLIT BARREL SAMPLER WITH 140 LB. HAMMER FALLING 30 IN.

DRILLER:	J.ALEXANDER
EQUIPMENT:	TRI-CONE ROLLER
METHOD:	ROTARY WASH
HOLE DIA.:	2 7/8"

REMARKS:
US 701 BRIDGE OVER GREAT PEE DEE RIVER - Boring is located 33.0' West of East abut and 24.0' North of center line. File No. 22.56961

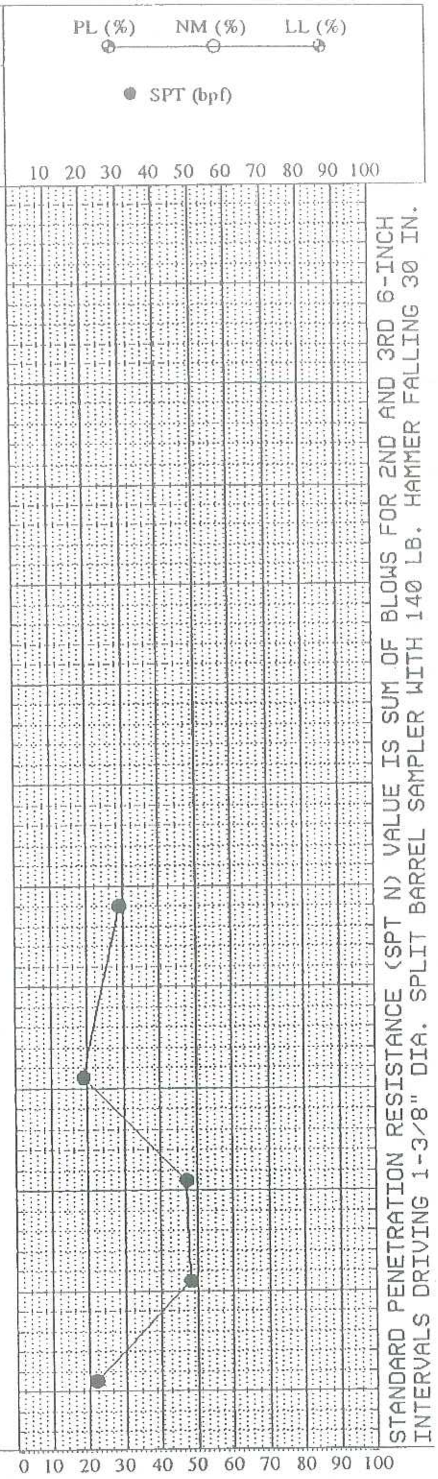
THIS RECORD IS A REASONABLE INTERPRETATION OF SOIL CONDITIONS AT THE BORING LOCATION. SOIL CONDITIONS AT OTHER LOCATIONS MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

PROJECT: US 701 BRIDGE REPLACEMENT
OFFSET: 24.5000 BORING NO.: B-1
STATION: 79914.7000 B-101
DRILLED: April 3, 1996
PROJ. NO.: 30510-6-6549 PAGE 2 OF 2

LAW ENGINEERING, INC.

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.	LEGEND	ELEV (ft)	IDENT	TYPE	N-COUNT 1ST 6" 2ND 6" 3RD 6"	% Fines
0	GREAT PEE DEE RIVER. (35.0')	■	5.2				
5		■	0.2				
10		■	-4.8				
15		■	-9.8				
20		■	-14.8				
25		■	-19.8				
30		■	-24.8				
35	POORLY GRADED SAND (SP) - mostly fine-med sand, firm, light gray - COASTAL PLAIN RESIDUUM	░	-29.8	SS-1	▨	6-12-17	
		░		SS-2	▨	8-35/3"	
40		░	-34.8				
		░		SS-3	▨	4-8-11	
45		░	-39.8				
	POORLY GRADED SAND (SP) - mostly fine-med sand, dense and firm, light gray - COASTAL PLAIN RESIDUUM	░					
		░		SS-4	▨	23-23-24	
50		░	-44.8				
		░		SS-5	▨	10-20-28	
55		░	-49.8				
		░		SS-6	▨	15-15-7	
60		░	-54.8				



STANDARD PENETRATION RESISTANCE (SPT N) VALUE IS SUM OF BLOWS FOR 2ND AND 3RD 6-INCH INTERVALS DRIVING 1-3/8" DIA. SPLIT BARREL SAMPLER WITH 140 LB. HAMMER FALLING 30 IN.

DRILLER:	J.ALEXANDER
EQUIPMENT:	TRI-CONE ROLLER
METHOD:	ROTARY WASH
HOLE DIA.:	2 7/8"

REMARKS:
US 701 BRIDGE OVER GREAT PEE DEE RIVER - Boring is located 10.0' East of West abut and 22.5' South of center line. File No. 22.56961

THIS RECORD IS A REASONABLE INTERPRETATION OF SOIL CONDITIONS AT THE BORING LOCATION. SOIL CONDITIONS AT OTHER LOCATIONS MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

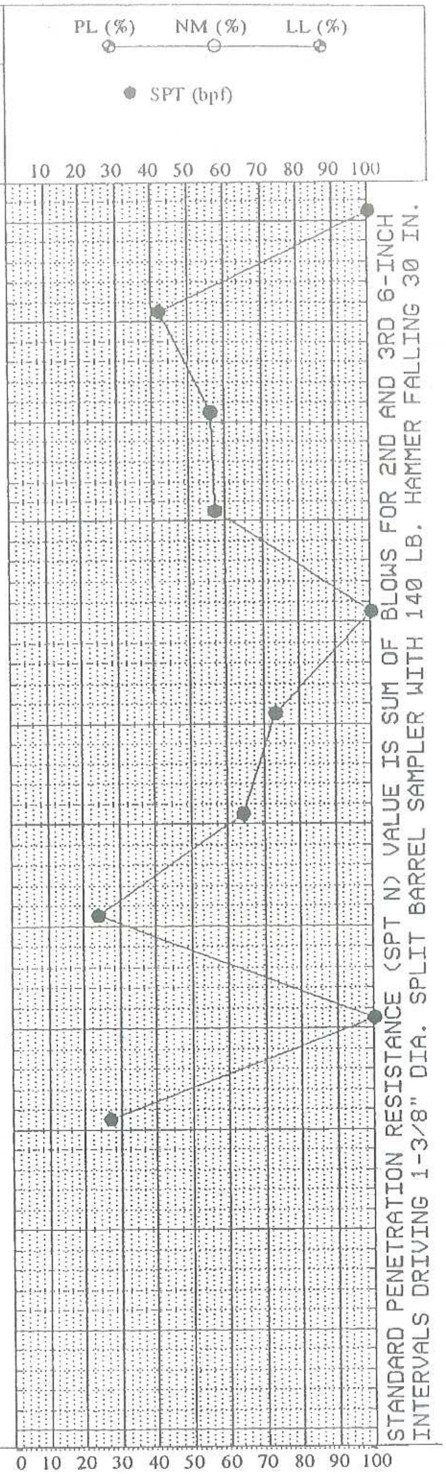
SOIL TEST BORING RECORD

PROJECT: US 701 BRIDGE REPLACEMENT
OFFSET: -22.5000
BORING NO.: B-2
B-102
STATION: 80050.4500
DRILLED: April 5, 1996
PROJ. NO.: 30510-6-6549
PAGE 1 OF 2

LAW ENGINEERING, INC.

SEE KEY SYMBOL SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS BELOW.

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS	ELEV (ft)	SAMPLES / TESTS IDENT	TYPE	N-COUNT 1ST 6" 2ND 6" 3RD 6"	% Fines
65	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, interbedded with fine cemented sand seams, hard to very hard, dark gray - COASTAL PLAIN RESIDUUM	-59.8	SS-7	▨	4-35-50/2"	
70		-64.8	SS-8	▨	6-8-34	
75		-69.8	SS-9	▨	35-22-34	
80		-74.8	SS-10	▨	14-22-25	
85		-79.8	SS-11	▨	14-50/2.5"	
90		-84.8	SS-12	▨	13-13-50/5"	
95	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, very stiff, dark gray - COASTAL PLAIN RESIDUUM	-89.8	SS-13	▨	40-30-34	
100	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, interbedded with fine sand seams, very hard, dark gray - COASTAL PLAIN RESIDUUM	-94.8	SS-14	▨	8-10-14	
105	LEAN CLAY WITH SAND (CL) - some fine sand, mostly med-plastic fines, very stiff, dark gray - COASTAL PLAIN RESIDUUM	-99.8	SS-15	▨	50/4.5"	
110	Boring Terminated Field crew noted difficult drilling, hard material at depths of 64-65.5, 70-75, and 91-93.5 feet.	-104.8	SS-16	▨	10-12-15	
115		-109.8				
120		-114.8				
125		-119.8				



STANDARD PENETRATION RESISTANCE (SPT N) VALUE IS SUM OF BLOWS FOR 2ND AND 3RD 6-INCH INTERVALS DRIVING 1-3/8" DIA. SPLIT BARREL SAMPLER WITH 140 LB. HAMMER FALLING 30 IN.

DRILLER:	J.ALEXANDER
EQUIPMENT:	TRI-CONE ROLLER
METHOD:	ROTARY WASH
HOLE DIA.:	2 7/8"

REMARKS:

US 701 BRIDGE OVER GREAT PEE DEE RIVER - Boring is located 10.0' East of West abut and 22.5' South of center line. File No. 22.56961

THIS RECORD IS A REASONABLE INTERPRETATION OF SOIL CONDITIONS AT THE BORING LOCATION. SOIL CONDITIONS AT OTHER LOCATIONS MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

PROJECT: US 701 BRIDGE REPLACEMENT
OFFSET: -22.5000 BORING NO.: B-2
STATION: 80050.4500
DRILLED: April 5, 1996
PROJ. NO.: 30510-6-6549

B-102

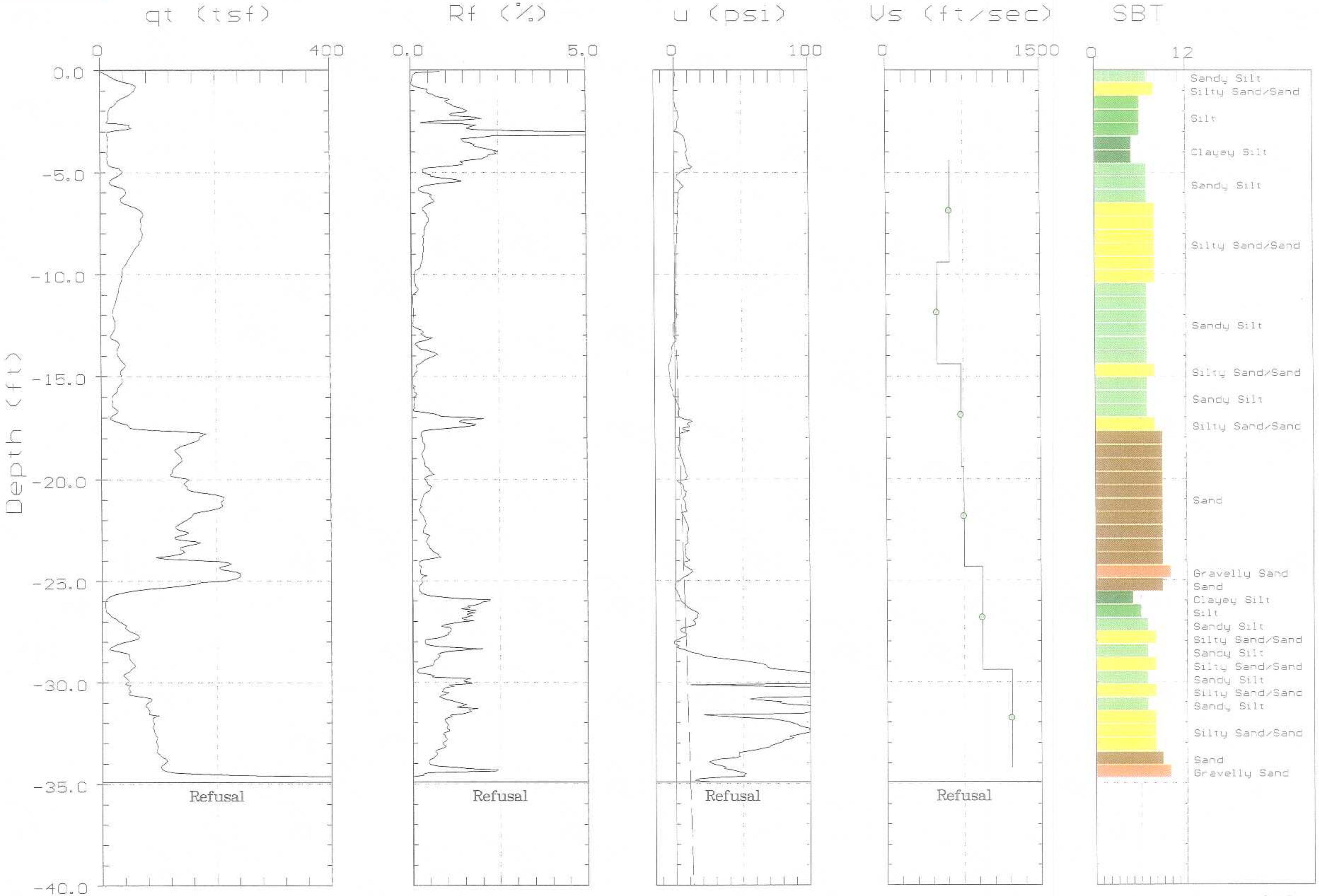
PAGE 2 OF 2

LAW ENGINEERING, INC.

S&ME, Inc.

Sounding: CPT-C1
Location: Conway, SC

Oversight: P. Skipper
Date: 12:17:04 09:02



Max. Depth: 34.91 (ft)
Depth Inc.: 0.066 (ft)

SBT: Soil Behavior Type (Robertson 1990)
Estimated Phreatic Surface

GREGG

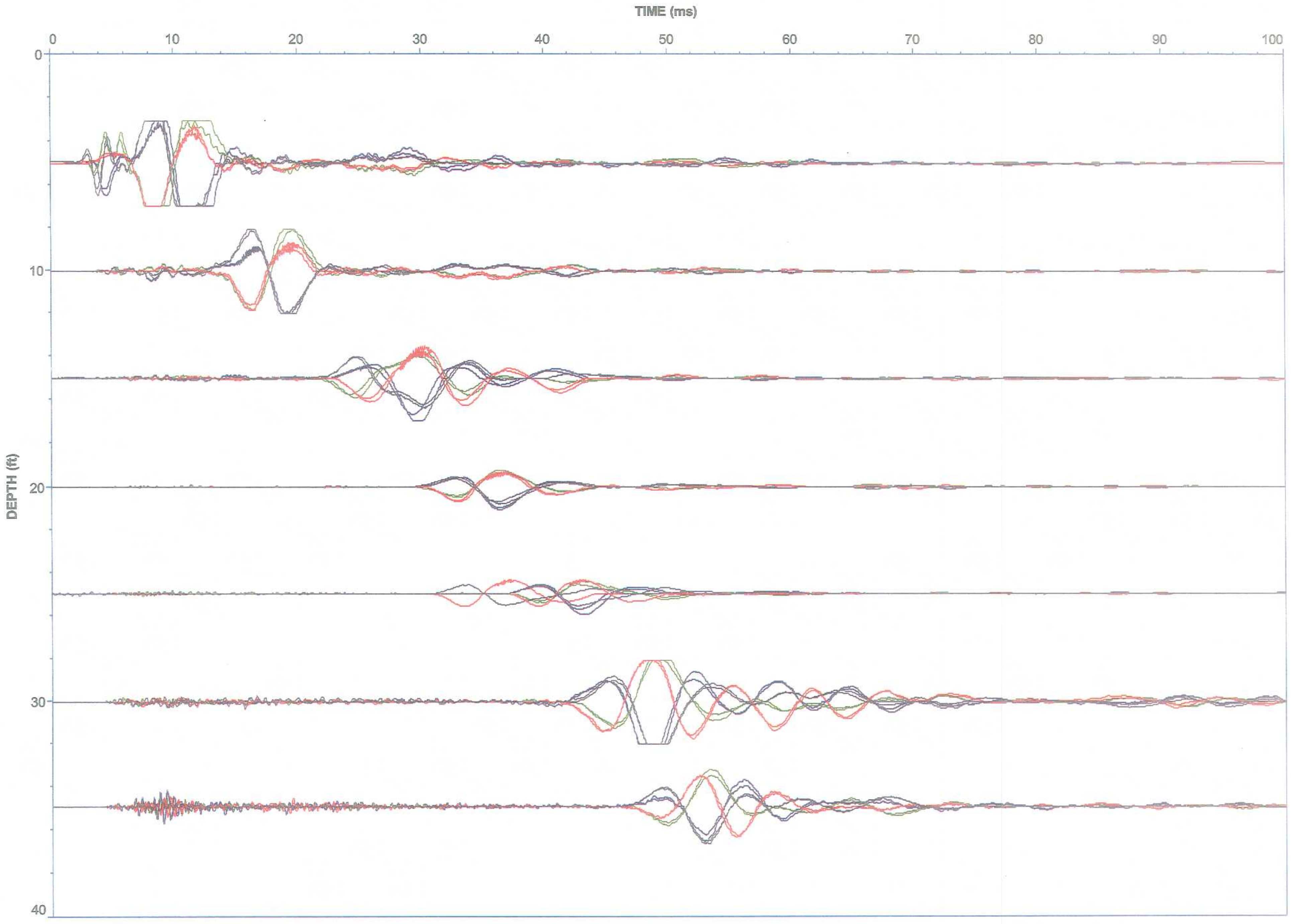
Shear Wave Velocity Calculations

701 By Pass
Conway, South Carolina

Geophone Offset:	0.66	Feet	Sounding:	**CPT-C1**
Source Offset:	1.67	Feet	Date:	12/17/04

Test Depth (feet)	Geophone Depth (feet)	Waveform Ray Path (feet)	Incremental Distance (feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (ft/s)	Interval Mid-Depth (feet)
5.05	4.39	4.70	4.70	9.87			
10.04	9.38	9.53	4.83	17.62	7.75	623.4	6.89
15.02	14.36	14.46	4.93	27.49	9.87	499.4	11.87
20.01	19.35	19.43	4.97	34.30	6.81	729.1	16.86
24.99	24.33	24.39	4.97	40.88	6.58	754.6	21.84
30.04	29.38	29.43	5.04	46.29	5.41	931.6	26.86
34.90	34.24	34.28	4.85	50.28	3.99	1216.4	31.81

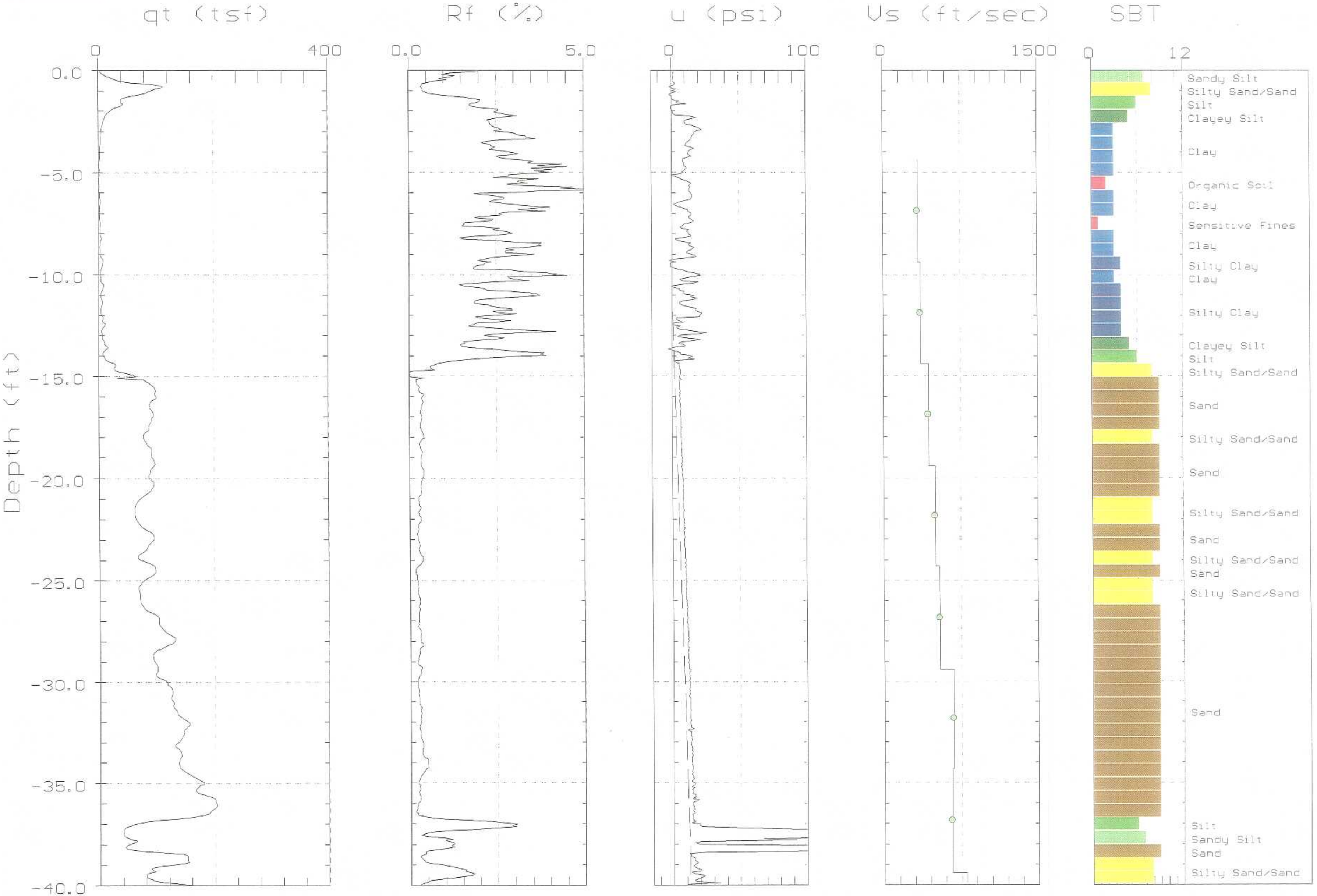
Job No: 04-205SC Client: S&ME, Inc. Project Title: 701 By-Pass Hole: CPT-C1 Date: 12/17/04 Oversite: P. Skipper



S&ME, Inc.

Sounding: CPT-C2
Location: Conway, SC

Oversight: P. Skipper
Date: 12:17:04 08:11



Max. Depth: 60.24 (ft)
Depth Inc.: 0.066 (ft)

SBT: Soil Behavior Type (Robertson 1990)
⊻ Estimated Phreatic Surface

GREGG

Shear Wave Velocity Calculations

701 By Pass

Conway, South Carolina

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

Sounding: **CPT-C2**

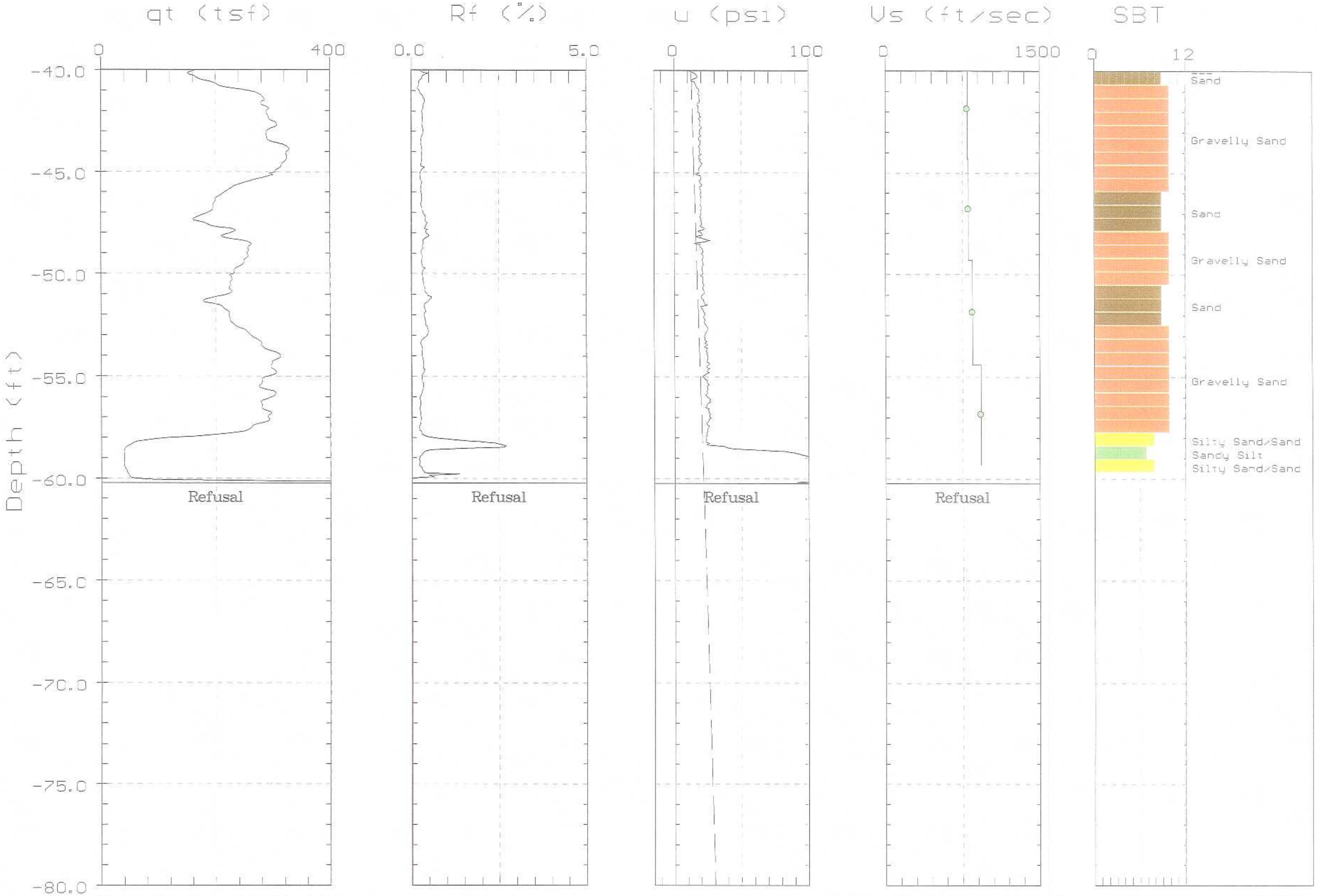
Date: 12/17/04

Test Depth (feet)	Geophone Depth (feet)	Waveform Ray Path (feet)	Incremental Distance (feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (ft/s)	Interval Mid-Depth (feet)
5.05	4.39	4.70	4.70	11.28			
10.04	9.38	9.53	4.83	25.66	14.38	336.0	6.89
15.02	14.36	14.46	4.93	39.19	13.53	364.3	11.87
20.01	19.35	19.43	4.97	50.47	11.28	440.2	16.86
24.99	24.33	24.39	4.97	60.34	9.87	503.1	21.84
30.04	29.38	29.43	5.04	69.64	9.30	542.0	26.86
35.00	34.34	34.38	4.95	76.97	7.33	675.7	31.86
40.02	39.36	39.40	5.01	84.59	7.62	658.1	36.85
44.97	44.31	44.35	4.95	90.79	6.20	797.8	41.84
49.99	49.33	49.36	5.02	96.99	6.20	809.2	46.82
55.01	54.35	54.38	5.02	102.91	5.92	847.5	51.84
59.99	59.33	59.36	4.98	108.27	5.36	928.7	56.84

S&ME, Inc.

Sounding: CPT-C2
Location: Conway, SC

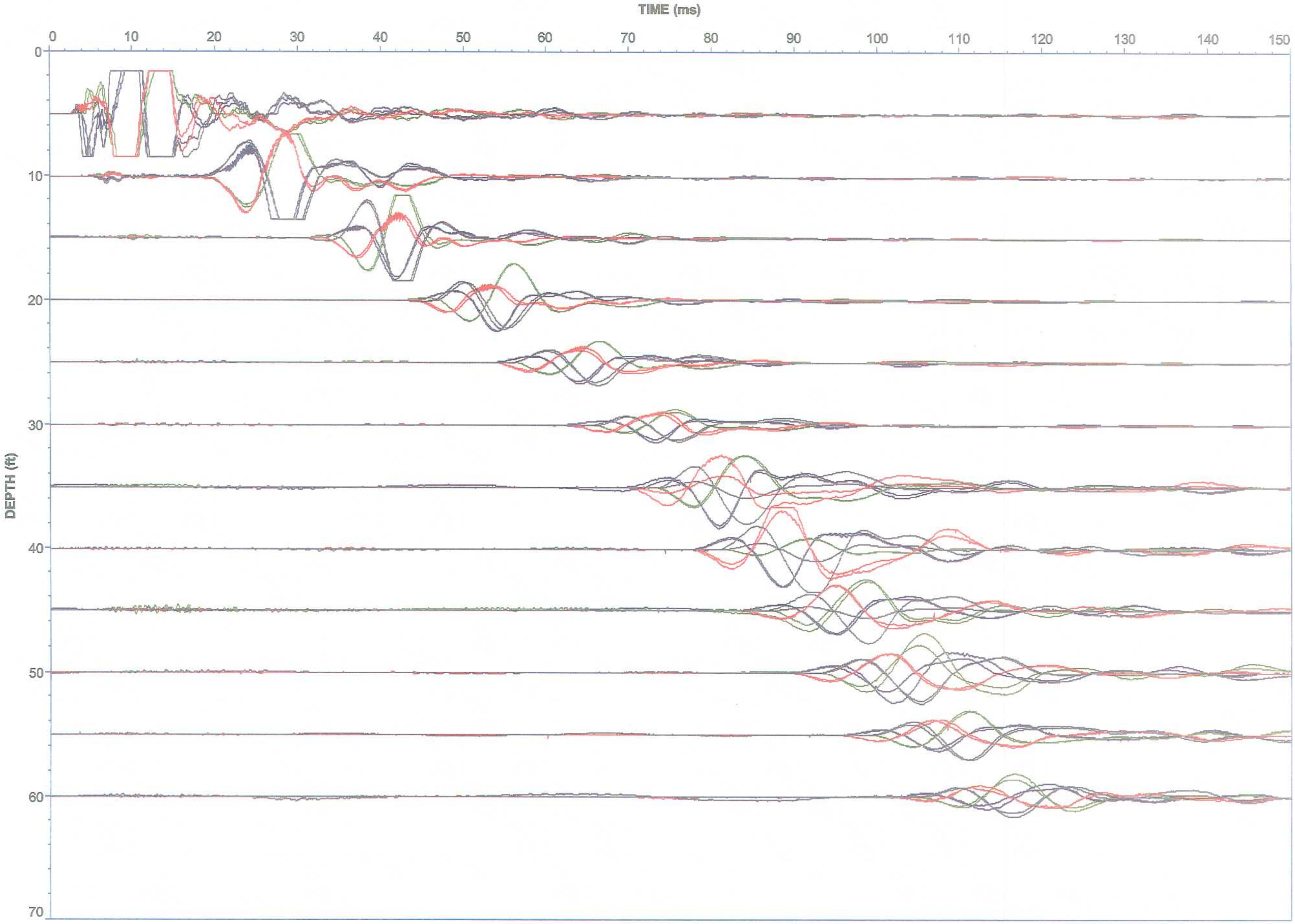
Oversight: P. Skipper
Date: 12:17:04 08:11



Max. Depth: 60.24 (ft)
Depth Inc.: 0.066 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Job No: 04-205SC Client: S&ME, Inc. Project Title: 701 By-Pass Hole: CPT-C2 Date: 12/17/04 Oversite: P. Skipper

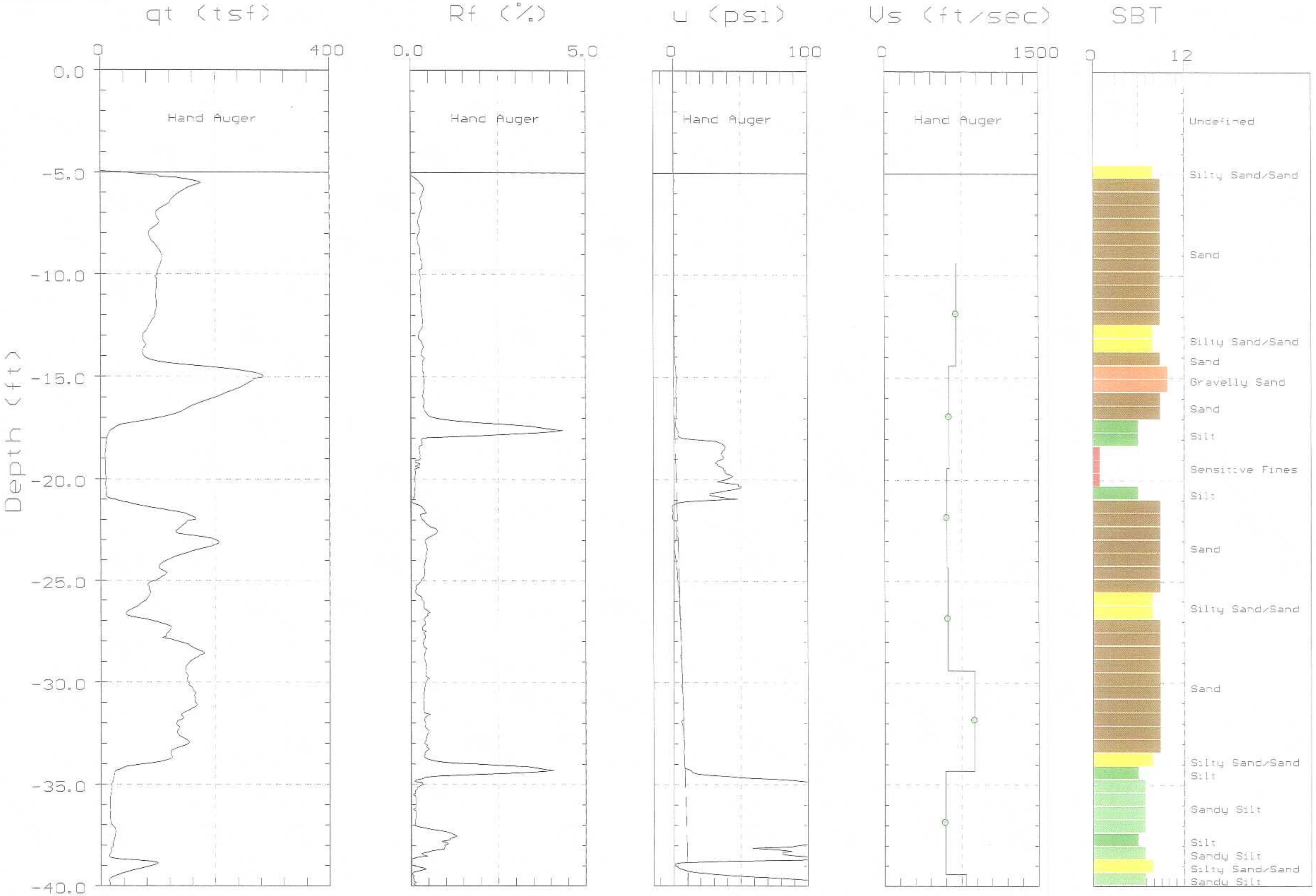


GREGG

S&ME, Inc.

Sounding: CPT-C3
Location: Conway, SC

Oversight: P. Skipper
Date: 12:16:04 13:53



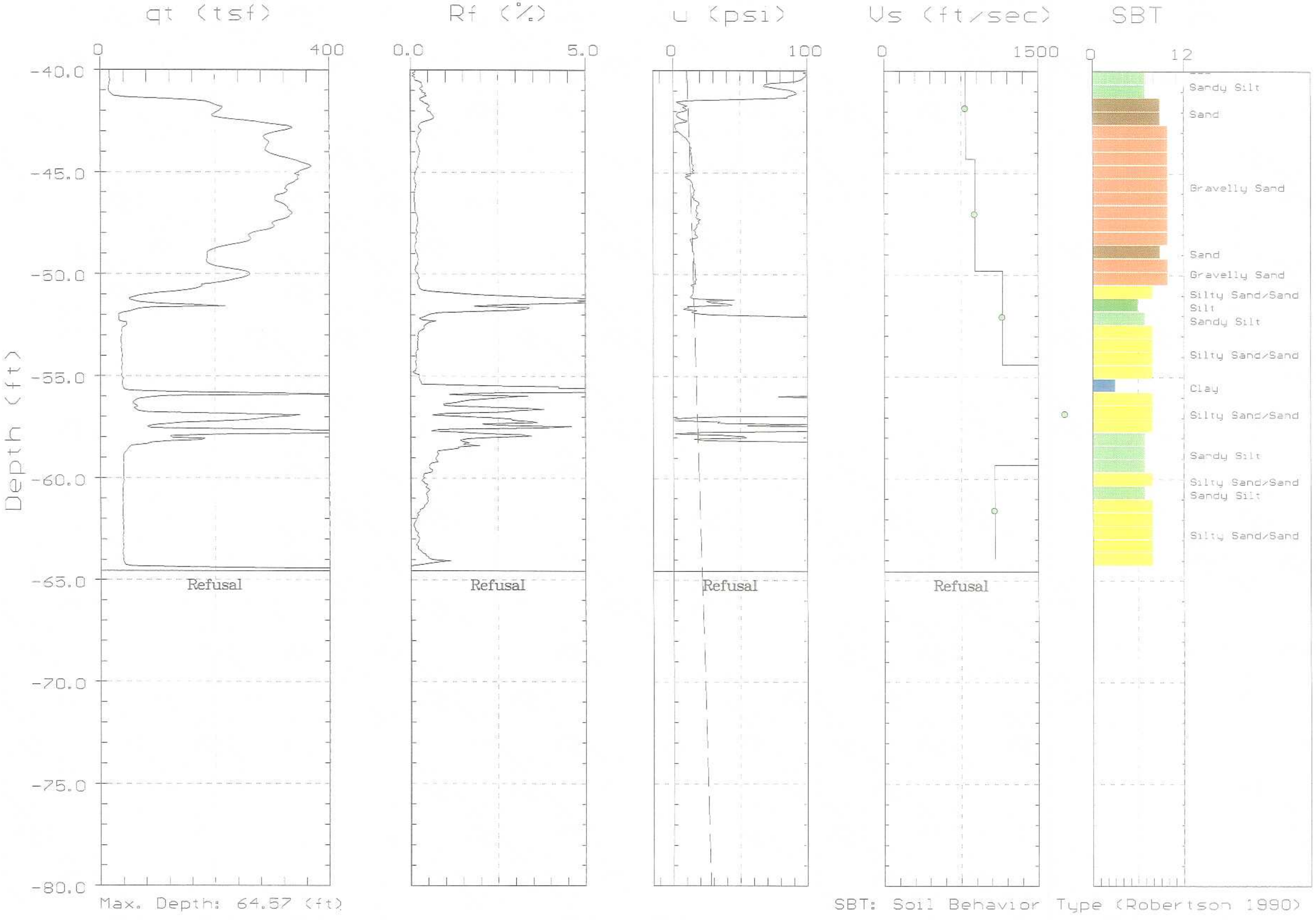
Max. Depth: 64.57 (ft)
Depth Inc.: 0.066 (ft)

SBT: Soil Behavior Type (Robertson 1990)
Estimated Phreatic Surface

S&ME, Inc.

Sounding: CPT-C3
Location: Conway, SC

Oversight: P. Skipper
Date: 12:16:04 13:53



Max. Depth: 64.57 (ft)
Depth Inc.: 0.066 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

Shear Wave Velocity Calculations

701 By Pass

Conway, South Carolina

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

Sounding: **CPT-C3**
Date: 12/16/04

Test Depth (feet)	Geophone Depth (feet)	Waveform Ray Path (feet)	Incremental Distance (feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (ft/s)	Interval Mid-Depth (feet)
10.04	9.38	9.53	9.53	14.38			
15.02	14.36	14.46	4.93	21.43	7.05	699.2	11.87
20.01	19.35	19.43	4.97	29.32	7.89	629.3	16.86
24.99	24.33	24.39	4.97	37.50	8.18	607.0	21.84
30.04	29.38	29.43	5.04	45.68	8.18	616.2	26.86
35.00	34.34	34.38	4.95	51.32	5.64	878.2	31.86
40.02	39.36	39.40	5.01	59.77	8.45	593.5	36.85
44.97	44.31	44.35	4.95	65.98	6.21	796.5	41.84
50.45	49.79	49.82	5.48	72.18	6.20	883.3	47.05
55.01	54.35	54.38	4.56	76.13	3.95	1153.8	52.07
59.99	59.33	59.36	4.98	78.95	2.82	1765.2	56.84
64.55	63.89	63.92	4.56	83.18	4.23	1077.6	61.61

Project: US 701 Bridges
Location: Yauhannah, S.C.
Project Number: 1611-04-569

TABLE 1
SUMMARY OF LABORATORY TESTING

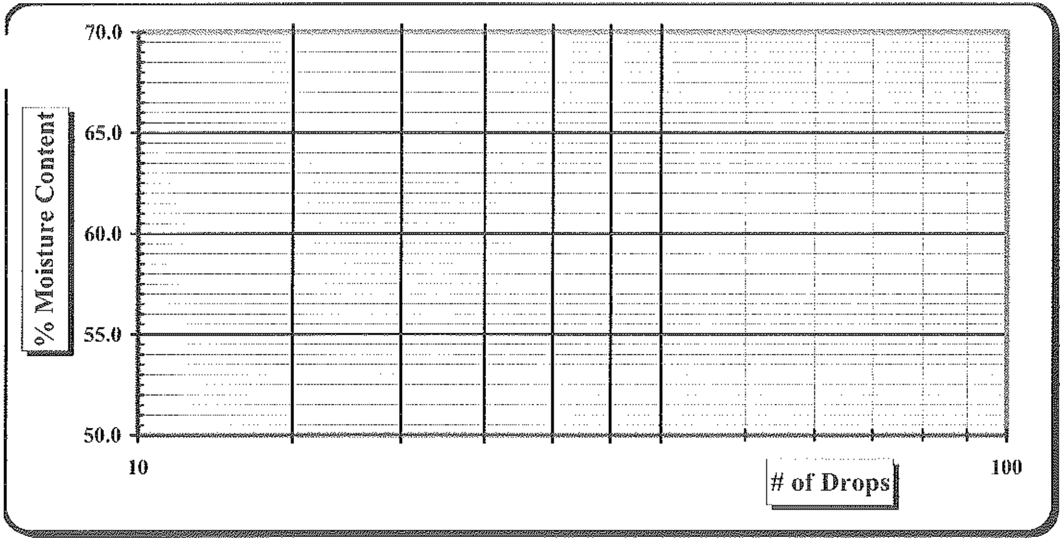
Boring	Sample Number	Depth	Moisture Content	Liquid Limit	Plastic Limit	Plastic Index	% Fines	ASTM Classification	Void Ratio	C_u	C_c
		(ft)	(%)				(<#200 Sieve)				
B-1	6	20	21.8	NP	NP	NP	18	MH	0.59		
	8	30	36.9	59	27	32	95.3	CH	0.99		
	11	45	48.1	36	28	8	63.7	CL	1.30		
	15	65	29.9	28	23	5	29.6	SC-SM	0.80		
	19	80	25.5	35	17	18	56	CL	0.69		
B-2	7	25	34.2				91.5	CL	0.93		
	9	25	24.1	26	21	5	14.1	SC-SM	0.65		
	12	50	19.4				5.4	SP-SC	0.52		
	14	60	33.6				7.2	SP-SC	0.90		
	17	75	13.8				4.8	SP	0.37		
B-3	2	5		49	25	24	83.4	CL-CH			
	4	10		34	20	14	71.2	CL			
	6	20		33	20	13	82.2	CL			
	8	30	24.9				4.6	SP	0.66		
	10	40	15.7				7.2	SP-SC	0.42		
B-4	3	7.5		36	20	16	72	CL			
	5	15	32.5				8	SP-SC	0.87	2.14	1.05
	7	25	21.2				2.3	SP	0.56	2.22	0.94
	9	35	17.2				2.2	SP	0.46	3.53	1.20
B-5	7	25	36.7	54	23	31	90.8	MH	0.99		
	9	35	17.9				6.2	SP-SC	0.48		
	11	45	21.2				2.5	SP	0.56		
	13	55	19.4				4.4	SP	0.51		
	15	65	25.3				4.7	SP	0.67		
	19	85	16.1	NP	NP	NP	3.1	SP	0.43		
	21	95	24.8	39	15	24	61.7	CL	0.67		

Liquid Limit, Plastic Limit, and Plastic Index

Project #: **1611-04-569**
Report Date:
Project Name: US 701 Bridges
Test Date(s): 1/28/05
Client Name:
Client Address:
Boring #: B-1
Sample #: SS-6
Sample Date: 1/11/05
Depth: 20 feet

Sample Description: Gray Elastic Silt (MH)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #									
A	Tare Weight									
B	Wet Soil Weight + A									
C	Dry Soil Weight + A									
D	Water Weight (B-C)									
E	Dry Soil Weight (C-A)									
F	% Moisture Content (D/E)*100									
N	# OF DROPS							*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average									



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:
Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒	NP, Non-Plastic	☒
Liquid limit Test:	Multipoint Method ☐	One-point Method ☐		Liquid Limit	
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐		Plastic Limit	
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐		Plastic Index	
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐		Group Symbol	

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-1
Sample #: SS-6
Sample Date: 1/11/05
Depth: 20 feet

Sample Description: Light Brown Silty Clayey Sand (SC-SM)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	K
	Tare Number		A	Tare Weight	85.50
A	Tare Weight	85.5	B	Wet Weight + Tare Wt.	226.70
B	Total Sample Dry Wt. + Tare Wt.	201.4	C	Dry Weight + Tare Wt.	201.40
C	Total Sample Dry Weight (B-A)	115.9	D	Water Wt. (B-C)	25.30
D	Total Sample Wt. After #200 Wash	95.0	E	Dry Wt.(C-A)	115.90
E	Percent Passing #200 (1-D/C)x100	18.0%	Moisture Content (100 x D/E) (%)		21.8%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	--
Plastic Index	--	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

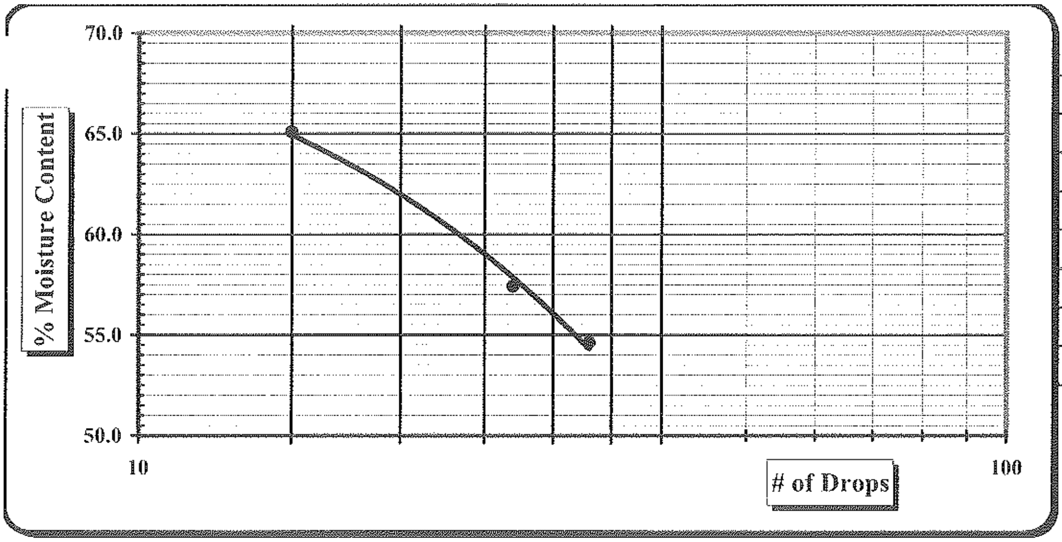
Technical Responsibility: R FOREST
Signature
Position

Liquid Limit, Plastic Limit, and Plastic Index

Project #: **1611-04-569**
Project Name: US 701 Bridges
Client Name:
Client Address:
Boring #: B-1
Sample #: SS-8
Sample Date: 1/11/05
Depth: 30 feet
Report Date:
Test Date(s): 1/28/05

Sample Description: Gray Fat Clay with Sand (CH)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	18	10	23				L99	L24	L6
A	Tare Weight	10.88	11.03	10.82				4.23	4.21	4.20
B	Wet Soil Weight + A	21.47	20.08	21.22				5.84	6.56	6.39
C	Dry Soil Weight + A	17.73	16.78	17.12				5.50	6.05	5.93
D	Water Weight (B-C)	3.74	3.30	4.10				0.34	0.51	0.46
E	Dry Soil Weight (C-A)	6.85	5.75	6.30				1.27	1.84	1.73
F	% Moisture Content (D/E)*100	54.6%	57.4%	65.1%				26.8%	27.7%	26.6%
N	# OF DROPS	33	27	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							27.0%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:
Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒
Liquid limit Test: Multipoint Method ☐ One-point Method ☐
Classification: ASTM D 2487 ☒ AASHTO M 145 ☐
Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐
Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐

NP, Non-Plastic ☐
Liquid Limit 59
Plastic Limit 27
Plastic Index 32
Group Symbol CH

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-1
Sample #: SS-8
Sample Date: 1/11/05
Depth: 30 feet

Sample Description: Gray Elastic Silt (MH)

	Particle Size Analysis / Without Hydrometer Analysis			Moisture Content	Natural
				Tare #	G
	Tare Number		A	Tare Weight	86.60
A	Tare Weight	86.6	B	Wet Weight + Tare Wt.	177.50
B	Total Sample Dry Wt. + Tare Wt.	153.0	C	Dry Weight + Tare Wt.	153.00
C	Total Sample Dry Weight (B-A)	66.4	D	Water Wt. (B-C)	24.50
D	Total Sample Wt. After #200 Wash	3.1	E	Dry Wt.(C-A)	66.40
E	Percent Passing #200 (1-D/C)x100	95.3%	Moisture Content (100 x D/E) (%)		36.9%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes: Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature *Position*

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: **1611-04-569**

Project Name: US 701 Bridges

Client Name:

Client Address:

Report Date:

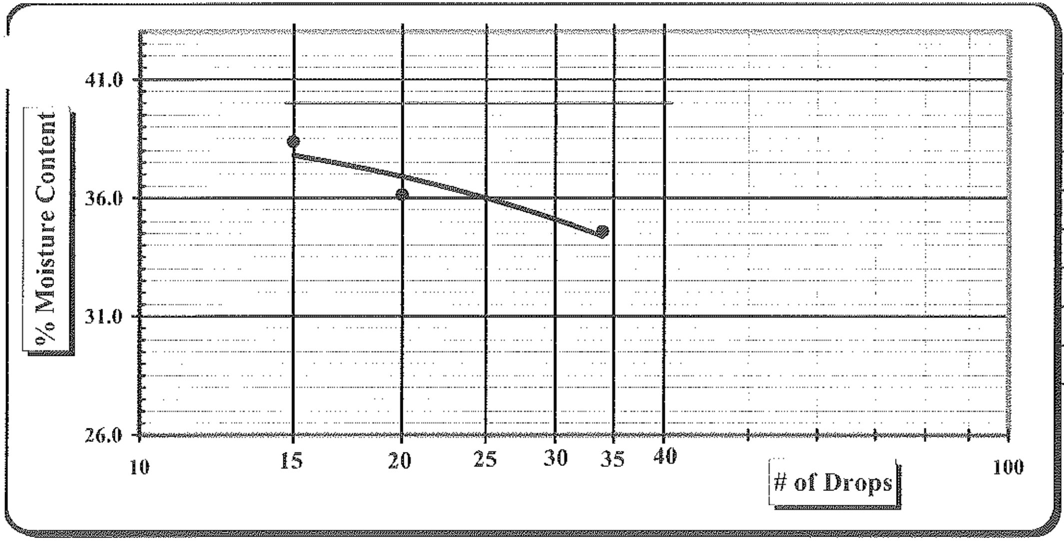
Test Date(s): 1/28/05

Boring #: B-1 Sample #: SS-11 Sample Date: 1/11/05

Depth: 45 feet

Sample Description: Gray Sandy Lean Clay (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	5	2	3				T4	L6	T12
A	Tare Weight	10.95	10.98	11.02				4.21	4.20	4.28
B	Wet Soil Weight + A	18.89	27.94	26.17				5.71	5.41	6.30
C	Dry Soil Weight + A	16.85	23.44	21.97				5.39	5.14	5.85
D	Water Weight (B-C)	2.04	4.50	4.20				0.32	0.27	0.45
E	Dry Soil Weight (C-A)	5.90	12.46	10.95				1.18	0.94	1.57
F	% Moisture Content (D/E)*100	34.6%	36.1%	38.4%				27.1%	28.7%	28.7%
N	# OF DROPS	34	20	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							28.2%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒

Liquid limit Test: Multipoint Method ☐ One-point Method ☐

Classification: ASTM D 2487 ☒ AASHTO M 145 ☐

Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐

Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐

NP, Non-Plastic	☐
Liquid Limit	36
Plastic Limit	28
Plastic Index	8
Group Symbol	CL

Technician Name: E NITZ ________
Certification #

Technical Responsibility: R FOREST ________ ________
Signature *Position*

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-1
Sample #: SS-11
Sample Date: 1/11/05
Depth: 45 feet

Sample Description: Gray Sandy Lean Clay (CL)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	I
	Tare Number		A	Tare Weight	85.80
A	Tare Weight	85.8	B	Wet Weight + Tare Wt.	187.40
B	Total Sample Dry Wt. + Tare Wt.	154.4	C	Dry Weight + Tare Wt.	154.40
C	Total Sample Dry Weight (B-A)	68.6	D	Water Wt. (B-C)	33.00
D	Total Sample Wt. After #200 Wash	24.9	E	Dry Wt.(C-A)	68.60
E	Percent Passing #200 (1-D/C)x100	63.7%	Moisture Content (100 x D/E) (%)		48.1%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	36	Fineness Modulus	--
Plastic Limit	38	Cu = D60/D10:	--
Plastic Index	8	$Cc = (D30)^2 / (D10xD60)$:	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	
Description of Sand & Gravel	Rounded ☐	Angular ☐
Hard & Durable ☐	Soft ☐	Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

echnician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature

Position

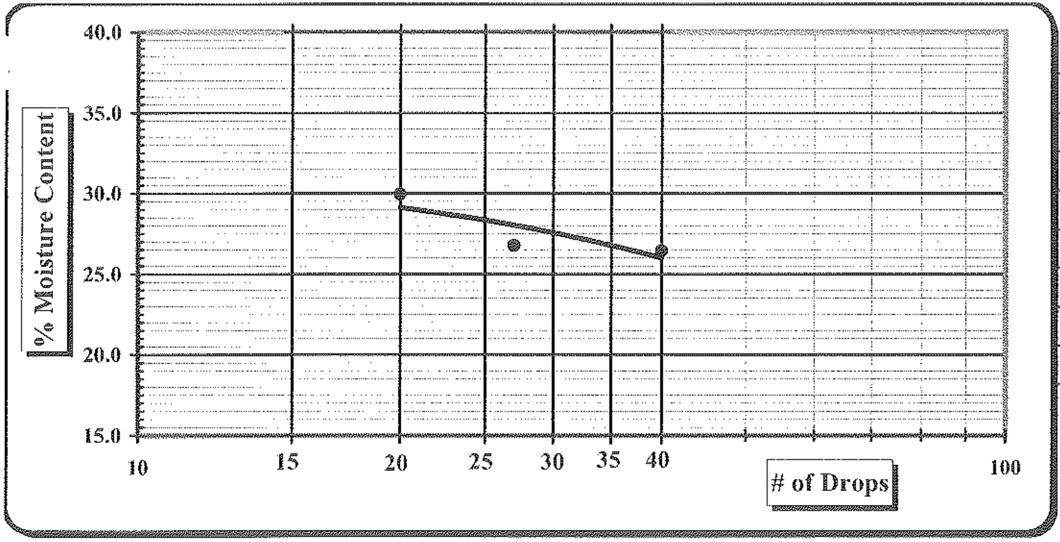
Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: **1611-04-569**
Project Name: US 701 Bridges
Client Name:
Client Address:
Report Date:
Test Date(s): 1/28/05
Boring #: B-1
Sample #: SS-15
Sample Date: 1/11/05
Depth: 65 feet

Sample Description: Gray Silty Clayey Sand (SC-SM)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	5	20	7				L14	L10	L51
A	Tare Weight	10.95	10.75	11.07				4.29	4.23	4.23
B	Wet Soil Weight + A	23.18	23.90	23.94				6.08	6.13	5.68
C	Dry Soil Weight + A	20.62	21.12	20.97				5.75	5.78	5.40
D	Water Weight (B-C)	2.56	2.78	2.97				0.33	0.35	0.28
E	Dry Soil Weight (C-A)	9.67	10.37	9.90				1.46	1.55	1.18
F	% Moisture Content (D/E)*100	26.4%	26.8%	29.9%				22.7%	22.8%	23.4%
N	# OF DROPS	40	27	20				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							23.0%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:

Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation	☐	Dry Preparation	☐	Air Dried	☒	NP, Non-Plastic	☐
Liquid limit Test:	Multipoint Method	☐	One-point Method	☐			Liquid Limit	28
Classification:	ASTM D 2487	☒	AASHTO M 145	☐			Plastic Limit	23
Liquid limit Test:	ASTM D 4318	☒	AASHTO T 89	☐			Plastic Index	5
Plastic limit Test:	ASTM D 4318	☒	AASHTO T 90	☐			Group Symbol	CL-ML

Technician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature

Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

S&ME

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-1
Sample #: SS-15
Sample Date: 1/11/05
Depth: 65 feet

Sample Description: Gray Silty Clayey Sand (SC-SM)

Particle Size Analysis / Without Hydrometer Analysis		
	Tare Number	
A	Tare Weight	85.9
B	Total Sample Dry Wt. + Tare Wt.	143.4
C	Total Sample Dry Weight (B-A)	57.5
D	Total Sample Wt. After #200 Wash	40.5
E	Percent Passing #200 (1-D/C)x100	29.6%

	Moisture Content	Natural
	Tare #	F
A	Tare Weight	85.90
B	Wet Weight + Tare Wt.	160.60
C	Dry Weight + Tare Wt.	143.40
D	Water Wt. (B-C)	17.20
E	Dry Wt.(C-A)	57.50
Moisture Content (100 x D/E) (%)		29.9%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	28	Fineness Modulus	--
Plastic Limit	23	Cu = D60/D10:	--
Plastic Index	5	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature *Position*

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-1, SS-15 grain size

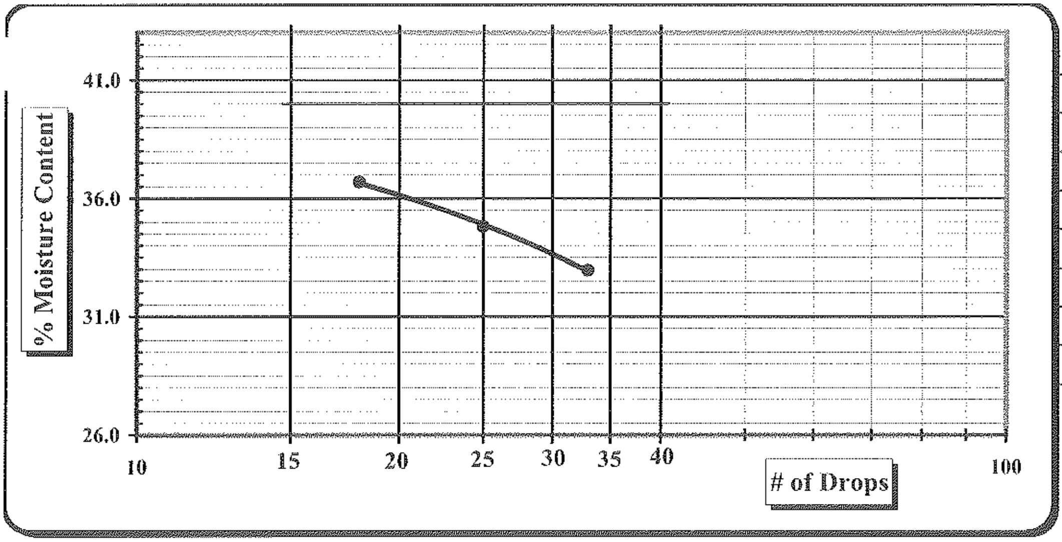
S&ME

Liquid Limit, Plastic Limit, and Plastic Index

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:
Report Date:
Test Date(s): 1/28/05
Boring #: B-1
Sample #: SS-19
Sample Date: 1/11/05
Depth: 80 feet

Sample Description: Gray Sandy Lean Clay (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	20	4	16				L7	L14	T21
A	Tare Weight	10.75	11.06	10.92				4.21	4.28	4.29
B	Wet Soil Weight + A	21.08	22.68	22.77				5.84	5.72	5.52
C	Dry Soil Weight + A	18.52	19.68	19.59				5.61	5.52	5.34
D	Water Weight (B-C)	2.56	3.00	3.18				0.23	0.20	0.18
E	Dry Soil Weight (C-A)	7.77	8.62	8.67				1.40	1.24	1.05
F	% Moisture Content (D/E)*100	32.9%	34.8%	36.7%				16.4%	16.1%	17.1%
N	# OF DROPS	33	25	18				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							16.6%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒	NP, Non-Plastic ☐
Liquid limit Test:	Multipoint Method ☐	One-point Method ☐		Liquid Limit 35
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐		Plastic Limit 17
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐		Plastic Index 18
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐		Group Symbol CL

Technician Name: _____E NITZ_____ __________ *Certification #*

Technical Responsibility: ____R FOREST____ ________________ *Signature* ________________ *Position*

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-1
Sample #: SS-19
Sample Date: 1/11/05
Depth: 80 feet

Sample Description: Gray Sandy Lean Clay (CL)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	O
	Tare Number		A	Tare Weight	84.50
A	Tare Weight	84.5	B	Wet Weight + Tare Wt.	201.70
B	Total Sample Dry Wt. + Tare Wt.	177.9	C	Dry Weight + Tare Wt.	177.90
C	Total Sample Dry Weight (B-A)	93.4	D	Water Wt. (B-C)	23.80
D	Total Sample Wt. After #200 Wash	41.1	E	Dry Wt.(C-A)	93.40
E	Percent Passing #200 (1-D/C)x100	56.0%	Moisture Content (100 x D/E) (%)		25.5%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	35	Fineness Modulus	--
Plastic Limit	17	Cu = D60/D10:	--
Plastic Index	18	$Cc = (D30)^2 / (D10xD60)$:	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature

Position

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-1, SS-19 grain size

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-2
Sample #: SS-7
Sample Date: 1/11/05
Depth: 25 feet

Sample Description: Gray Fat Clay (CH)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Natural
	Tare Number		A	Tare Weight	86.60
A	Tare Weight	86.6	B	Wet Weight + Tare Wt.	215.20
B	Total Sample Dry Wt. + Tare Wt.	182.4	C	Dry Weight + Tare Wt.	182.40
C	Total Sample Dry Weight (B-A)	95.8	D	Water Wt. (B-C)	32.80
D	Total Sample Wt. After #200 Wash	8.1	E	Dry Wt.(C-A)	95.80
E	Percent Passing #200 (1-D/C)x100	91.5%	Moisture Content (100 x D/E) (%)		34.2%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:	Maximum Particle Size	--	
	Apparent Relative Density	--	
Liquid Limit --	Fineness Modulus	--	
Plastic Limit --	Cu = D60/D10:	--	
Plastic Index --	Cc =$(D30)^2$/ (D10xD60):	--	

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

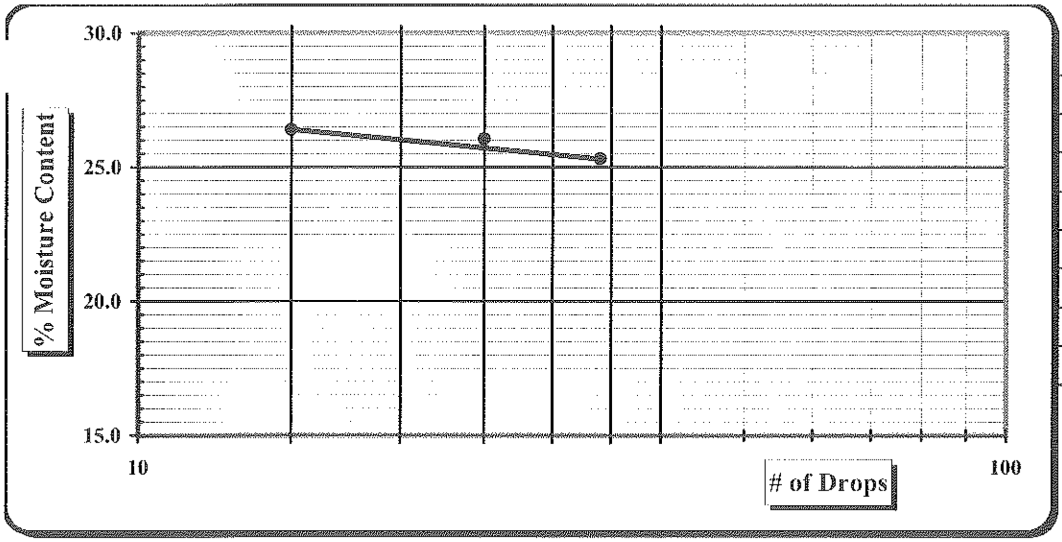
Laboratory Record Version 4.2

Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:
Report Date:
Test Date(s): 1/28/05
Boring #: B-2
Sample #: SS-9
Sample Date: 1/11/05
Depth: 25 feet
Sample Description: Gray Silty Clayey Sand (SC-SM)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	2	9	16				L5	L7	L11
A	Tare Weight	10.97	11.07	10.91				4.23	4.21	4.21
B	Wet Soil Weight + A	22.06	21.91	29.92				6.09	5.79	5.93
C	Dry Soil Weight + A	19.82	19.67	25.95				5.75	5.52	5.65
D	Water Weight (B-C)	2.24	2.24	3.97				0.34	0.27	0.28
E	Dry Soil Weight (C-A)	8.85	8.60	15.04				1.52	1.31	1.44
F	% Moisture Content (D/E)*100	25.3%	26.0%	26.4%				22.4%	20.6%	19.4%
N	# OF DROPS	34	25	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							20.8%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:
Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒
Liquid limit Test: Multipoint Method ☐ One-point Method ☐
Classification: ASTM D 2487 ☒ AASHTO M 145 ☐
Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐
Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐

NP, Non-Plastic ☐
Liquid Limit 26
Plastic Limit 21
Plastic Index 5
Group Symbol CL-ML

Technician Name: E NITZ
Certification #
Technical Responsibility: R FOREST
Signature
Position

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-2,SS-9 Limits

Laboratory Record Version 4.2

S&ME

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
'oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-2
Sample #: SS-9
Sample Date: 1/11/05
Depth: 25 feet

Sample Description: Gray Silty Clayey Sand (SC-SM)

Particle Size Analysis / Without Hydrometer Analysis		
	Tare Number	
A	Tare Weight	85.6
B	Total Sample Dry Wt. + Tare Wt.	216.7
C	Total Sample Dry Weight (B-A)	131.1
D	Total Sample Wt. After #200 Wash	112.6
E	Percent Passing #200 (1-D/C)x100	14.1%

Moisture Content		Natural
	Tare #	M
A	Tare Weight	85.60
B	Wet Weight + Tare Wt.	248.30
C	Dry Weight + Tare Wt.	216.70
D	Water Wt. (B-C)	31.60
E	Dry Wt.(C-A)	131.10
Moisture Content (100 x D/E) (%)		24.1%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	26	Fineness Modulus	--
Plastic Limit	21	Cu = D60/D10:	--
Plastic Index	5	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-2
Sample #: SS-12
Sample Date: 1/11/05
Depth: 50 feet
Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)

	Particle Size Analysis / Without Hydrometer Analysis			Moisture Content	Natural
				Tare #	P
	Tare Number		A	Tare Weight	85.50
A	Tare Weight	85.5	B	Wet Weight + Tare Wt.	229.00
B	Total Sample Dry Wt. + Tare Wt.	205.7	C	Dry Weight + Tare Wt.	205.70
C	Total Sample Dry Weight (B-A)	120.2	D	Water Wt. (B-C)	23.30
D	Total Sample Wt. After #200 Wash	113.7	E	Dry Wt.(C-A)	120.20
E	Percent Passing #200 (1-D/C)x100	5.4%	Moisture Content (100 x D/E) (%)		19.4%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes: Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2/(D10xD60)$: --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-2
Sample #: SS-14
Sample Date: 1/11/05
Depth: 60 feet

Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)

Particle Size Analysis / Without Hydrometer Analysis

	Tare Number	
A	Tare Weight	86.0
B	Total Sample Dry Wt. + Tare Wt.	195.0
C	Total Sample Dry Weight (B-A)	109.0
D	Total Sample Wt. After #200 Wash	101.2
E	Percent Passing #200 (1-D/C)x100	7.2%

	Moisture Content	Natural
	Tare #	L
A	Tare Weight	86.00
B	Wet Weight + Tare Wt.	231.60
C	Dry Weight + Tare Wt.	195.00
D	Water Wt. (B-C)	36.60
E	Dry Wt.(C-A)	109.00
	Moisture Content (100 x D/E) (%)	33.6%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	--
Plastic Index	--	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

S&ME

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-2
Sample #: SS-17
Sample Date: 1/11/05
Depth: 75 feet

Sample Description: Gray Poorly Graded Sand (SP)

Particle Size Analysis / Without Hydrometer Analysis				Moisture Content	Natural
				Tare #	J
	Tare Number		A	Tare Weight	84.20
A	Tare Weight	84.2	B	Wet Weight + Tare Wt.	229.70
B	Total Sample Dry Wt. + Tare Wt.	212.0	C	Dry Weight + Tare Wt.	212.00
C	Total Sample Dry Weight (B-A)	127.8	D	Water Wt. (B-C)	17.70
D	Total Sample Wt. After #200 Wash	121.7	E	Dry Wt.(C-A)	127.80
E	Percent Passing #200 (1-D/C)x100	4.8%		Moisture Content (100 x D/E) (%)	13.8%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes: Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature *Position*

S&ME

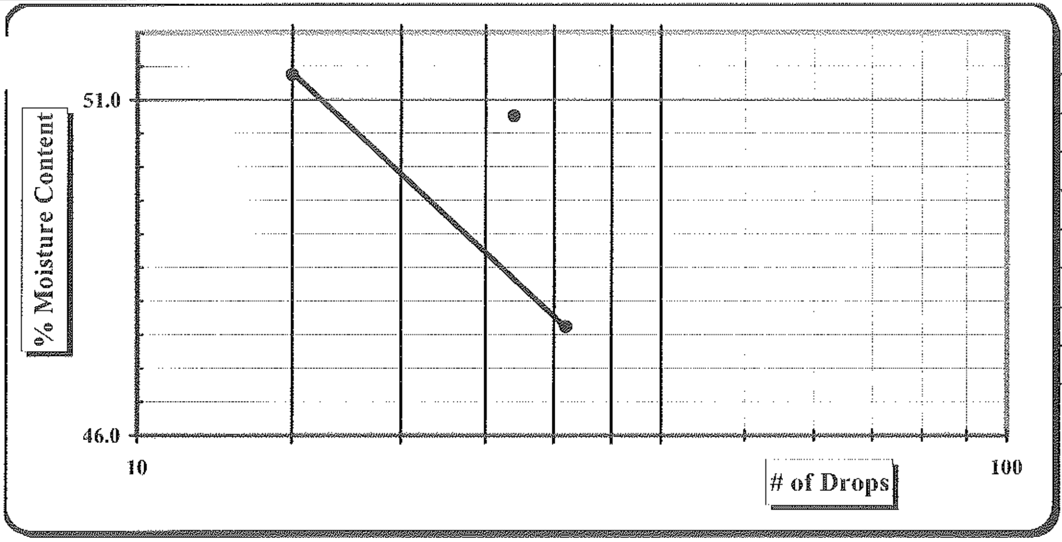
Liquid Limit, Plastic Limit, and Plastic Index

Project #: **1611-04-569**
Project Name: US 701 Bridges
Client Name:
Client Address:
Boring #: B-3
Sample #: SS-2

Report Date:
Test Date(s): 1/28/05
Sample Date: 1/11/05
Depth: 5 feet

Sample Description: Gray Lean Clay with Sand (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	23	21	10				L7	L5	T20
A	Tare Weight	10.85	10.59	11.04				4.20	4.21	4.28
B	Wet Soil Weight + A	21.05	18.61	19.88				5.83	5.85	6.63
C	Dry Soil Weight + A	17.76	15.91	16.88				5.51	5.52	6.15
D	Water Weight (B-C)	3.29	2.70	3.00				0.32	0.33	0.48
E	Dry Soil Weight (C-A)	6.91	5.32	5.84				1.31	1.31	1.87
F	% Moisture Content (D/E)*100	47.6%	50.8%	51.4%				24.4%	25.2%	25.7%
N	# OF DROPS	31	27	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							25.1%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation	☐	Dry Preparation	☐	Air Dried	☒	NP, Non-Plastic	☐
Liquid limit Test:	Multipoint Method	☐	One-point Method	☐			Liquid Limit	49
Classification:	ASTM D 2487	☒	AASHTO M 145	☐			Plastic Limit	25
Liquid limit Test:	ASTM D 4318	☒	AASHTO T 89	☐			Plastic Index	24
Plastic limit Test:	ASTM D 4318	☒	AASHTO T 90	☐			Group Symbol	CL-ML

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-3
Sample #: SS-2
Sample Date: 1/11/05
Depth: 5 feet

Sample Description: Gray Lean Clay with Sand (CL)

Particle Size Analysis / Without Hydrometer Analysis		
	Tare Number	
A	Tare Weight	0.0
B	Total Sample Dry Wt. + Tare Wt.	50.0
C	Total Sample Dry Weight (B-A)	50.0
D	Total Sample Wt. After #200 Wash	8.3
E	Percent Passing #200 (1-D/C)x100	83.4%

	Moisture Content	Natural
	Tare #	
A	Tare Weight	
B	Wet Weight + Tare Wt.	
C	Dry Weight + Tare Wt.	
D	Water Wt. (B-C)	
E	Dry Wt.(C-A)	
	Moisture Content (100 x D/E) (%)	

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.00	0.0%	**100.0%**
0.60	#30	5.00	10.0%	**90.0%**
0.43	#40	5.00	10.0%	**90.0%**
0.25	#60	1.70	3.4%	**96.6%**
0.15	#100	3.20	6.4%	**93.6%**
0.075	#200	8.10	16.2%	**83.8%**

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	49	Fineness Modulus	--
Plastic Limit	25	Cu = D60/D10:	--
Plastic Index	24	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.0%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**10.0%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**6.2%**
% Silt and Clay	< 0.075 mm	**83.8%**

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

echnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-3, SS-2 grain size

Laboratory Record Version 4.0

Report Date: 1/05

S&ME

Particle Size Analysis of Soils

Project #: **1611-04-569**
Test Date(s): 1/11/05
Project Name: **US 701 Bridges**
Client Name:
Client Address:
Boring #: B-3
Sample #: SS-2
Sample Date:
Location:
Offset:
Elevation: 5 feet
Sample Description: Gray Lean Clay with Sand (CL)

Liquid Limit: 49
Plastic Limit: 25
Plastic Index: 24

Sieve	Retained Wt.	Percent Passing	
3.0"	0.0	Soil Mortar (portion passing the #10 sieve)	100.0%
1.5"	0.0		100.0%
1.0"	0.0		100.0%
3/4"	0.0		100.0%
1/2"	0.0		100.0%
3/8"	0.0		100.0%
#4	0.0		100.0%
#10	0.0	100.0%	100.0%
#30	5.0	95.2%	95.2%
#40	5.0	95.2%	95.2%
#60	1.7	98.4%	98.4%
#100	3.2	96.9%	96.9%
#200	8.1	92.2%	92.2%

Pan #:		Beaker #:	
Hydrometer Jar #:			
Pan Tare Weight (grams):			85.70
Total Sample Wet Wt. + tare wt. (grams):			106.00
Weight of Total Sample Air Dried:			20.30
Weight of Air Dried Hydrometer Sample (g):			105.60
Total Sample Oven Dried:			19.90
Hydrometer Sample Oven Dried (W):			103.52
% Passing #10:			100.0%
Correction Factor a (Table 1):			0.99

Specific Gravity:		2.708	
Pan # (washed sample):			
Moisture Content		Hygroscopic	Natural
	Tare #	M	
A	Tare Wt.	85.70	
B	Wet Wt. + A	106.00	
C	Dry Wt. + A	105.60	
D	Water Wt. (B-C)	0.40	
E	Dry Wt.(C-A)	19.90	
% Moisture (100 x D/E)		2.01%	

Notes: Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Maximum Particle Size:			Total Sand:	7.8%	< 4.74 mm and > 0.075 mm	Type: Mechanical Stirring Apparatus (A)
% Gravel:	0.0%	< 75 mm and > 4.75 mm	Silt & Clay:	92.2%	< 0.075	Dispersion Time: 1 min.
Coarse Sand:	0.0%	< 4.75 mm and > 2.00 mm	Silt:	64.2%	< 0.075 and > 0.005 mm	Sodium Hexametaphosphate: 40 g./ Liter
Medium Sand:	4.8%	< 2.00 mm and > 0.425 mm	Clay:	28.0%	< 0.005 mm	
Fine Sand:	3.0%	< 0.425 mm and > 0.075 mm	Colloids:	19.0%	< 0.001 mm	Hydrometer: 151H ☐ 152H ☒

Time		Temp.	Hydrometer	Corrections		Hydrometer	Percent Passing		Table 2	Table 3	Diameter
				Control Cylinder	Composite Correction		P(-#10) =	P (total) =			D =
Clock	T (Min.)	(°C)	Reading			R	(R x a / W) x 100	P x % Passing #10	L	K	K x ((L/T)$^{1/2}$
13:10	2	20.0	37.0		3.9	33.1	31.7%	31.7%	10.9	0.01344	0.03138
13:13	5	20.0	35.0		3.9	31.1	29.7%	29.7%	11.2	0.01344	0.02012
13:23	15	20.0	32.0		3.9	28.1	26.9%	26.9%	11.7	0.01344	0.01187
13:38	30	19.4	27.0		4.2	22.8	21.8%	21.8%	12.7	0.01361	0.00886
14:08	60	19.4	23.0		4.2	18.8	18.0%	18.0%	13.3	0.01361	0.00641
17:18	250	19.4	18.0		4.2	13.8	13.2%	13.2%	14.2	0.01361	0.00324
13:08	1440	20.0	14.0		3.9	10.1	9.7%	9.7%	14.7	0.01344	0.00136
13:08	2880	20.0	13.0		3.9	9.1	8.7%	8.7%	14.8	0.01344	0.00096

Technician Name / Certification #

E. Nitz

References:

ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: J. Lessley
Position: Chief Engineer

Laboratory Record Version 4.2

S&ME

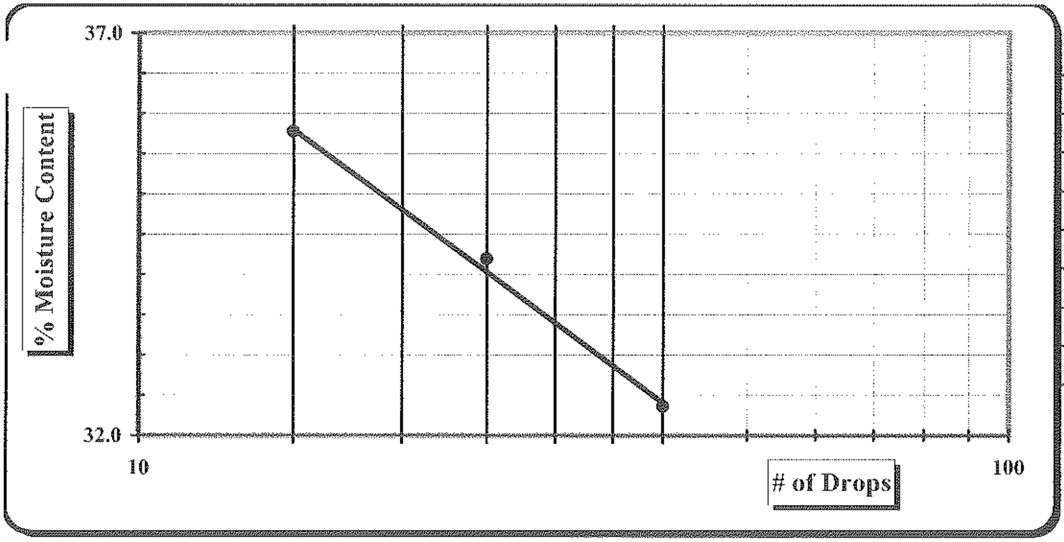
Liquid Limit, Plastic Limit, and Plastic Index

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:
Report Date:
Test Date(s): 1/28/05

Boring #: B-3
Sample #: SS-4
Sample Date: 1/11/05
Depth: 10 feet

Sample Description: Gray Lean Clay with Sand (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	15	5	8				T21	T2	L10
A	Tare Weight	10.84	10.95	11.04				4.29	4.26	4.23
B	Wet Soil Weight + A	23.07	22.96	20.87				5.56	5.26	5.34
C	Dry Soil Weight + A	20.08	19.90	18.28				5.34	5.09	5.16
D	Water Weight (B-C)	2.99	3.06	2.59				0.22	0.17	0.18
E	Dry Soil Weight (C-A)	9.24	8.95	7.24				1.05	0.83	0.93
F	% Moisture Content (D/E)*100	32.4%	34.2%	35.8%				21.0%	20.5%	19.4%
N	# OF DROPS	40	25	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							20.3%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:

Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒ NP, Non-Plastic ☐
Liquid limit Test: Multipoint Method ☐ One-point Method ☐ Liquid Limit 34
Classification: ASTM D 2487 ☒ AASHTO M 145 ☐ Plastic Limit 20
Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐ Plastic Index 14
Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐ Group Symbol CL-ML

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Particle Size Analysis of Soils

S&ME

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-3
Sample #: SS-4
Sample Date: 1/11/05
Depth: 10 feet

Sample Description: Gray Lean Clay with Sand (CL)

	Particle Size Analysis / Without Hydrometer Analysis	
	Tare Number	Carla
A	Tare Weight	82.0
B	Total Sample Dry Wt. + Tare Wt.	50.0
C	Total Sample Dry Weight (B-A)	50.0
D	Total Sample Wt. After #200 Wash	14.4
E	Percent Passing #200 (1-D/C)x100	71.2%

	Moisture Content	Natural
	Tare #	
A	Tare Weight	
B	Wet Weight + Tare Wt.	
C	Dry Weight + Tare Wt.	
D	Water Wt. (B-C)	
E	Dry Wt.(C-A)	
Moisture Content (100 x D/E) (%)		

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.00	0.0%	**100.0%**
0.60	#30	0.70	1.4%	**98.6%**
0.43	#40	1.00	2.0%	**98.0%**
0.25	#60	1.80	3.6%	**96.4%**
0.15	#100	4.00	8.0%	**92.0%**
0.075	#200	14.40	28.8%	**71.2%**

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	34	Fineness Modulus	--
Plastic Limit	20	Cu = D60/D10:	--
Plastic Index	14	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.0%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**2.0%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**26.8%**
% Silt and Clay	< 0.075 mm	**71.2%**

Description of Sand & Gravel Rounded ☐ Angular ☐

Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________

Certification #

Technical Responsibility: R FOREST ____________ ____________

Signature *Position*

S&ME

Laboratory Record Version 4.0

Report Date: 1/05

Particle Size Analysis of Soils

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:
Test Date(s): 1/11/05

Liquid Limit: 34
Plastic Limit: 20
Plastic Index: 14

Boring #: B-3
Sample #: SS-4
Sample Date:
Location:
Offset:
Elevation: 10 feet
Sample Description: Gray Lean Clay with Sand (CL)

Pan #:		Beaker #:	
Hydrometer Jar #:			
Pan Tare Weight (grams):			78.00
Total Sample Wet Wt. + tare wt. (grams):			100.00
Weight of Total Sample Air Dried:			22.00
Weight of Air Dried Hydrometer Sample (g):			99.60
Total Sample Oven Dried:			21.60
Hydrometer Sample Oven Dried (W):			97.79
% Passing #10:			100.0%
Correction Factor a (Table 1):			0.99

	Specific Gravity:	2.708	
	Pan # (washed sample):		
	Moisture Content	Hygroscopic	Natural
	Tare #	Gina	
A	Tare Wt.	78.00	
B	Wet Wt. + A	100.00	
C	Dry Wt. + A	99.60	
D	Water Wt. (B-C)	0.40	
E	Dry Wt.(C-A)	21.60	
	% Moisture (100 x D/E)	1.85%	

Sieve	Retained Wt.	Percent Passing	
3.0"	0.0	Soil Mortar (portion passing the #10 sieve)	100.0%
1.5"	0.0		100.0%
1.0"	0.0		100.0%
3/4"	0.0		100.0%
1/2"	0.0		100.0%
3/8"	0.0		100.0%
#4	0.0		100.0%
#10	0.0	100.0%	100.0%
#30	0.7	99.3%	99.3%
#40	1.0	99.0%	99.0%
#60	1.8	98.2%	98.2%
#100	4.0	95.9%	95.9%
#200	14.4	85.3%	85.3%

Notes: Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Maximum Particle Size:			Total Sand:	14.7%	< 4.74 mm and > 0.075 mm	Type: Mechanical Stirring Apparatus (A)
% Gravel:	0.0%	< 75 mm and > 4.75 mm	Silt & Clay:	85.3%	< 0.075	Dispersion Time: 1 min.
Coarse Sand:	0.0%	< 4.75 mm and > 2.00 mm	Silt:	57.3%	< 0.075 and > 0.005 mm	Sodium Hexametaphosphate: 40 g./ Liter
Medium Sand:	1.0%	< 2.00 mm and > 0.425 mm	Clay:	28.0%	< 0.005 mm	
Fine Sand:	13.7%	< 0.425 mm and > 0.075 mm	Colloids:	19.0%	< 0.001 mm	Hydrometer: 151H ☐ 152H ☒

Time		Temp.	Hydrometer	Corrections		Hydrometer	Percent Passing		Table 2	Table 3	Diameter
Clock	T (Min.)	(°C)	Reading	Control Cylinder	Composite Correction	R	P(-#10) = (R x a / W) x 100	P (total) = P x % Passing #10	L	K	D = K x ((L/T)$^{1/2}$
12:32	2	20.0	23.0		3.9	19.1	19.3%	19.3%	13.2	0.01344	0.03453
12:35	5	20.0	20.0		3.9	16.1	16.3%	16.3%	13.7	0.01344	0.02225
12:45	15	20.0	17.0		3.9	13.1	13.3%	13.3%	14.2	0.01344	0.01308
13:00	30	20.0	16.0		3.9	12.1	12.2%	12.2%	14.3	0.01344	0.00928
13:30	60	20.0	14.0		3.9	10.1	10.2%	10.2%	14.7	0.01344	0.00665
16:40	250	20.0	12.0		3.9	8.1	8.2%	8.2%	15.0	0.01344	0.00329
12:30	1440	19.8	9.0		4.0	5.0	5.1%	5.1%	15.5	0.01361	0.00141
12:30	2880	20.0	9.0		3.9	5.1	5.2%	5.2%	15.5	0.01344	0.00099

Technician Name / Certification #

E. Nitz

References:
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 422: Particle Size Analysis of Soils
ASTM D 854: Specific Gravity of Soils
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

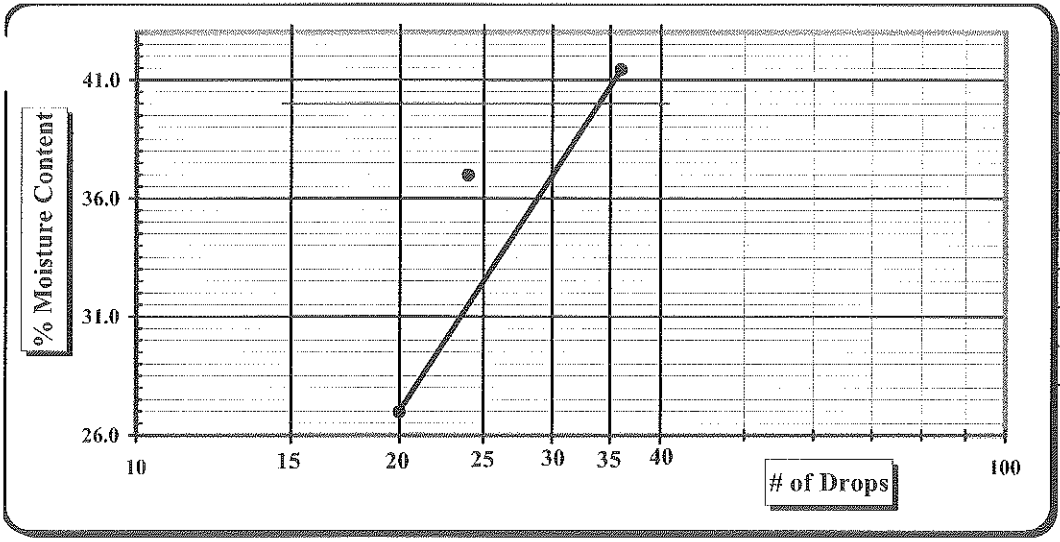
Technical Responsibility: J. Lessley
Position: Chief Engineer

Liquid Limit, Plastic Limit, and Plastic Index

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:
Report Date:
Test Date(s): 1/28/05
Boring #: B-3
Sample #: SS-6
Sample Date: 1/11/05
Depth: 20 feet

Sample Description: Gray Lean Clay with Sand (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	15	18	7				L24	L11	L99
A	Tare Weight	10.84	10.08	11.07				4.20	4.21	4.22
B	Wet Soil Weight + A	19.58	19.01	21.57				5.59	5.50	5.28
C	Dry Soil Weight + A	17.02	16.60	19.34				5.35	5.28	5.11
D	Water Weight (B-C)	2.56	2.41	2.23				0.24	0.22	0.17
E	Dry Soil Weight (C-A)	6.18	6.52	8.27				1.15	1.07	0.89
F	% Moisture Content (D/E)*100	41.4%	37.0%	27.0%				20.9%	20.6%	19.1%
N	# OF DROPS	36	24	20				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							20.2%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes:
Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒		NP, Non-Plastic	☐
Liquid limit Test:	Multipoint Method ☐	One-point Method ☐			Liquid Limit	33
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐			Plastic Limit	20
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐			Plastic Index	13
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐			Group Symbol	CL

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-3
Sample #: SS-6
Sample Date: 1/11/05
Depth: 20 feet

Sample Description: Gray Lean Clay with Sand (CL)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	
	Tare Number	G	A	Tare Weight	
A	Tare Weight	86.9	B	Wet Weight + Tare Wt.	
B	Total Sample Dry Wt. + Tare Wt.	50.0	C	Dry Weight + Tare Wt.	
C	Total Sample Dry Weight (B-A)	50.0	D	Water Wt. (B-C)	
D	Total Sample Wt. After #200 Wash	8.9	E	Dry Wt.(C-A)	
E	Percent Passing #200 (1-D/C)x100	82.2%	Moisture Content (100 x D/E) (%)		

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.00	0.0%	**100.0%**
0.60	#30	0.00	0.0%	**100.0%**
0.43	#40	0.00	0.0%	**100.0%**
0.25	#60	2.50	5.0%	**95.0%**
0.15	#100	6.00	12.0%	**88.0%**
0.075	#200	8.80	17.6%	**82.4%**

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	33	Fineness Modulus	--
Plastic Limit	20	Cu = D60/D10:	--
Plastic Index	13	Cc =$(D30)^2/(D10xD60)$:	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.0%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**0.0%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**17.6%**
% Silt and Clay	< 0.075 mm	**82.4%**

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature Position

S&ME

Laboratory Record Version 4.0

Report Date: [illegible]1/05

Particle Size Analysis of Soils

Project #: **1611-04-569**
Test Date(s): 1/11/05
Project Name: **US 701 Bridges**
Client Name:
Client Address:

Liquid Limit: 33
Plastic Limit: 20
Plastic Index: 13

Boring #: B-3
Sample #: SS-6
Sample Date:
Location:
Offset:
Elevation: 20 feet
Sample Description: Gray Lean Clay with Sand (CL)

Pan #:		Beaker #:	
Hydrometer Jar #:			
Pan Tare Weight (grams):			86.70
Total Sample Wet Wt. + tare wt. (grams):			106.00
Weight of Total Sample Air Dried:			19.30
Weight of Air Dried Hydrometer Sample (g):			105.70
Total Sample Oven Dried:			19.00
Hydrometer Sample Oven Dried (W):			104.06
% Passing #10:			100.0%
Correction Factor a (Table 1):			0.99

	Specific Gravity:	2.708	
	Pan # (washed sample):		
	Moisture Content	Hygroscopic	Natural
	Tare #	Natural	
A	Tare Wt.	86.70	
B	Wet Wt. + A	106.00	
C	Dry Wt. + A	105.70	
D	Water Wt. (B-C)	0.30	
E	Dry Wt.(C-A)	19.00	
	% Moisture (100 x D/E)	1.58%	

Sieve	Retained Wt.	**Percent Passing**	
3.0"	0.0	Soil Mortar (portion passing the #10 sieve)	100.0%
1.5"	0.0		100.0%
1.0"	0.0		100.0%
3/4"	0.0		100.0%
1/2"	0.0		100.0%
3/8"	0.0		100.0%
#4	0.0		100.0%
#10	0.0	100.0%	100.0%
#30	0.0	100.0%	100.0%
#40	0.0	100.0%	100.0%
#60	2.5	97.6%	97.6%
#100	6.0	94.2%	94.2%
#200	8.8	91.5%	91.5%

Notes: Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Maximum Particle Size:			Total Sand:	8.5%	< 4.74 mm and > 0.075 mm	Type: Mechanical Stirring Apparatus (A)
% Gravel:	0.0%	< 75 mm and > 4.75 mm	Silt & Clay:	91.5%	< 0.075	Dispersion Time: 1 min.
Coarse Sand:	0.0%	< 4.75 mm and > 2.00 mm	Silt:	63.5%	< 0.075 and > 0.005 mm	Sodium Hexametaphosphate: 40 g./ Liter
Medium Sand:	0.0%	< 2.00 mm and > 0.425 mm	Clay:	28.0%	< 0.005 mm	
Fine Sand:	8.5%	< 0.425 mm and > 0.075 mm	Colloids:	19.0%	< 0.001 mm	Hydrometer: 151H ☐ 152H ☒

Time		Temp.	Hydrometer	Corrections		Hydrometer	Percent Passing		Table 2	Table 3	Diameter
				Control Cylinder	Composite Correction		P(-#10) =	P (total) =			D =
Clock	T (Min.)	(°C)	Reading			R	(R x a / W) x 100	P x % Passing #10	L	K	K x ((L/T)$^{1/2}$
13:27	2	19.0	38.0		4.2	33.8	32.2%	32.2%	10.9	0.01361	0.03177
13:30	5	19.0	36.0		4.2	31.8	30.3%	30.3%	11.2	0.01361	0.02037
13:40	15	19.0	34.0		4.2	29.8	28.4%	28.4%	11.5	0.01361	0.01192
13:55	30	19.0	31.0		4.2	26.8	25.5%	25.5%	12.0	0.01361	0.00861
14:25	60	19.4	27.0		4.2	22.8	21.7%	21.7%	12.7	0.01361	0.00626
17:35	250	19.4	21.0		4.2	16.8	16.0%	16.0%	13.7	0.01361	0.00319
13:25	1440	19.8	15.0		4.0	11.0	10.5%	10.5%	14.5	0.01361	0.00137
13:25	2880	20.0	13.0		3.9	9.1	8.7%	8.7%	14.8	0.01344	0.00096

Technician Name / Certification #

E. Nitz

References:
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 422: Particle Size Analysis of Soils
ASTM D 854: Specific Gravity of Soils
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: J. Lessley
Position: Chief Engineer

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-3
Sample #: SS-8
Sample Date: 1/11/05
Depth: 30 feet

Sample Description: Gray Poorly Graded Sand (SP)

	Particle Size Analysis / Without Hydrometer Analysis	
	Tare Number	
A	Tare Weight	87.8
B	Total Sample Dry Wt. + Tare Wt.	215.9
C	Total Sample Dry Weight (B-A)	128.1
D	Total Sample Wt. After #200 Wash	122.2
E	Percent Passing #200 (1-D/C)x100	4.6%

Moisture Content		Natural
	Tare #	C
A	Tare Weight	87.80
B	Wet Weight + Tare Wt.	247.80
C	Dry Weight + Tare Wt.	215.90
D	Water Wt. (B-C)	31.90
E	Dry Wt.(C-A)	128.10
Moisture Content (100 x D/E) (%)		24.9%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:
Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

echnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature *Position*

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-3
Sample #: SS-10
Sample Date: 1/11/05
Depth: 40 feet

Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	B
	Tare Number		A	Tare Weight	84.70
A	Tare Weight	84.7	B	Wet Weight + Tare Wt.	231.70
B	Total Sample Dry Wt. + Tare Wt.	211.7	C	Dry Weight + Tare Wt.	211.70
C	Total Sample Dry Weight (B-A)	127.0	D	Water Wt. (B-C)	20.00
D	Total Sample Wt. After #200 Wash	117.8	E	Dry Wt.(C-A)	127.00
E	Percent Passing #200 (1-D/C)x100	7.2%	Moisture Content (100 x D/E) (%)		15.7%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	--
Plastic Index	--	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-3, SS-10 grain size

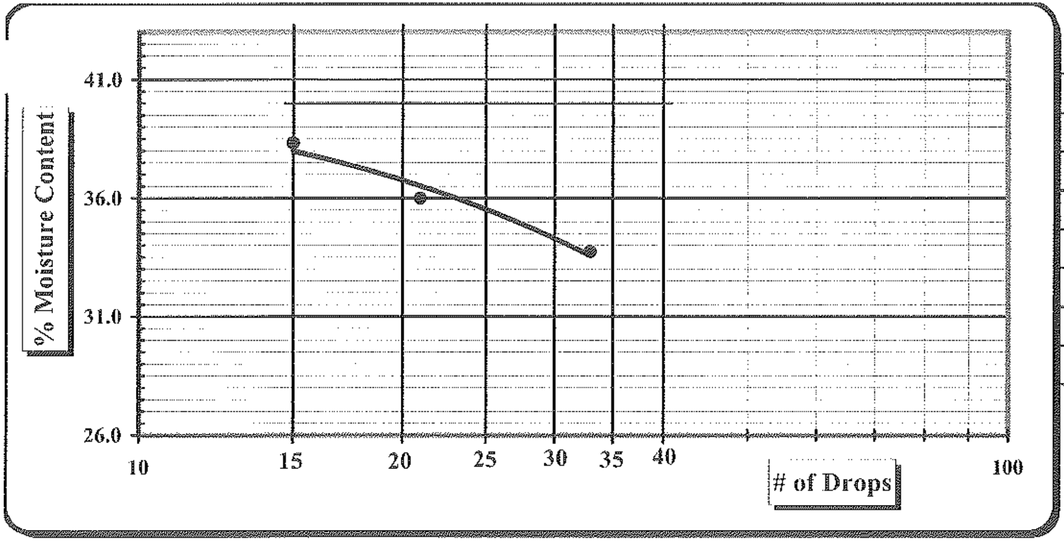
Liquid Limit, Plastic Limit, and Plastic Index

S&ME

Project #: **1611-04-569** Report Date:
Project Name: US 701 Bridges Test Date(s): 1/28/05
Client Name:
Client Address:
Boring #: B-4 Sample #: SS-3 Sample Date: 1/11/05
Depth: 7.5 feet

Sample Description: Gray Lean Clay with Sand (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	2	16	9				L24	L6	L99
A	Tare Weight	10.96	10.91	11.06				4.20	4.20	4.22
B	Wet Soil Weight + A	23.76	19.98	24.09				6.67	5.76	6.53
C	Dry Soil Weight + A	20.53	17.58	20.48				6.25	5.49	6.14
D	Water Weight (B-C)	3.23	2.40	3.61				0.42	0.27	0.39
E	Dry Soil Weight (C-A)	9.57	6.67	9.42				2.05	1.29	1.92
F	% Moisture Content (D/E)*100	33.7%	36.0%	38.3%				20.5%	20.9%	20.3%
N	# OF DROPS	33	21	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							20.6%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒
Liquid limit Test: Multipoint Method ☐ One-point Method ☐
Classification: ASTM D 2487 ☒ AASHTO M 145 ☐
Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐
Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐

NP, Non-Plastic	☐
Liquid Limit	36
Plastic Limit	20
Plastic Index	16
Group Symbol	CL

Technician Name: E NITZ _______ *Certification #*

Technical Responsibility: R FOREST _______ *Signature* _______ *Position*

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-4
Sample #: SS-3
Sample Date: 1/11/05
Depth: 7.5 feet

Sample Description: Gray Lean Clay with Sand (CL)

Particle Size Analysis / Without Hydrometer Analysis		
	Tare Number	
A	Tare Weight	87.4
B	Total Sample Dry Wt. + Tare Wt.	50.0
C	Total Sample Dry Weight (B-A)	50.0
D	Total Sample Wt. After #200 Wash	14.0
E	Percent Passing #200 (1-D/C)x100	72.0%

	Moisture Content	Natural
	Tare #	
A	Tare Weight	
B	Wet Weight + Tare Wt.	
C	Dry Weight + Tare Wt.	
D	Water Wt. (B-C)	
E	Dry Wt.(C-A)	
Moisture Content (100 x D/E) (%)		

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.00	0.0%	**100.0%**
0.60	#30	0.00	0.0%	**100.0%**
0.43	#40	0.00	0.0%	**100.0%**
0.25	#60	0.60	1.2%	**98.8%**
0.15	#100	3.30	6.6%	**93.4%**
0.075	#200	13.90	27.8%	**72.2%**

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	36	Fineness Modulus	--
Plastic Limit	20	Cu = D60/D10:	--
Plastic Index	16	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.0%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**0.0%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**27.8%**
% Silt and Clay	< 0.075 mm	**72.2%**

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature *Position*

S&ME

Laboratory Record Version 4.0

Report Date: 1/05

Particle Size Analysis of Soils

Project #: 1611-04-569
Test Date(s): 1/11/05
Project Name: US 701 Bridges
Client Name:
Client Address:
Boring #: B-4
Sample #: SS-3
Sample Date:
Location:
Offset:
Elevation: 7.5 feet
Sample Description: Gray Lean Clay with Sand (CL)

Liquid Limit: 36
Plastic Limit: 20
Plastic Index: 16

Pan #: / Beaker #:	
Hydrometer Jar #:	
Pan Tare Weight (grams):	86.50
Total Sample Wet Wt. + tare wt. (grams):	106.00
Weight of Total Sample Air Dried:	19.50
Weight of Air Dried Hydrometer Sample (g):	105.30
Total Sample Oven Dried:	18.80
Hydrometer Sample Oven Dried (W):	101.52
% Passing #10:	100.0%
Correction Factor a (Table 1):	0.99

	Specific Gravity:	2.708	
	Pan # (washed sample):		
	Moisture Content	Hygroscopic	Natural
	Tare #	D	
A	Tare Wt.	86.50	
B	Wet Wt. + A	106.00	
C	Dry Wt. + A	105.30	
D	Water Wt. (B-C)	0.70	
E	Dry Wt.(C-A)	18.80	
	% Moisture (100 x D/E)	3.72%	

Sieve	Retained Wt.	Percent Passing	
3.0"	0.0	Soil Mortar (portion passing the #10 sieve)	100.0%
1.5"	0.0		100.0%
1.0"	0.0		100.0%
3/4"	0.0		100.0%
1/2"	0.0		100.0%
3/8"	0.0		100.0%
#4	0.0		100.0%
#10	0.0	100.0%	100.0%
#30	0.0	100.0%	100.0%
#40	0.0	100.0%	100.0%
#60	0.6	99.4%	99.4%
#100	3.3	96.7%	96.7%
#200	13.9	86.3%	86.3%

Notes: Description of Sand & Gravel Particles: Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Maximum Particle Size:			Total Sand:	13.7%	< 4.74 mm and > 0.075 mm
% Gravel:	0.0%	< 75 mm and > 4.75 mm	Silt & Clay:	86.3%	< 0.075
Coarse Sand:	0.0%	< 4.75 mm and > 2.00 mm	Silt:	58.3%	< 0.075 and > 0.005 mm
Medium Sand:	0.0%	< 2.00 mm and > 0.425 mm	Clay:	28.0%	< 0.005 mm
Fine Sand:	13.7%	< 0.425 mm and > 0.075 mm	Colloids:	19.0%	< 0.001 mm

Type: Mechanical Stirring Apparatus (A)
Dispersion Time: 1 min.
Sodium Hexametaphosphate: 40 g./ Liter
Hydrometer: 151H ☐ 152H ☒

Time		Temp.	Hydrometer	Corrections		Hydrometer	Percent Passing		Table 2	Table 3	Diameter
				Control Cylinder	Composite Correction		P(-#10) =	P (total) =			D =
Clock	T (Min.)	(°C)	Reading			R	(R x a / W) x 100	P x % Passing #10	L	K	K x ((L/T))$^{1/2}$
12:47	2	19.8	25.0		4.0	21.0	20.5%	20.5%	12.9	0.01361	0.03457
12:50	5	20.0	22.0		3.9	18.1	17.7%	17.7%	13.3	0.01344	0.02192
13:00	15	20.0	20.0		3.9	16.1	15.7%	15.7%	13.7	0.01344	0.01284
13:15	30	20.0	17.0		3.9	13.1	12.8%	12.8%	14.2	0.01344	0.00925
13:45	60	20.0	15.0		3.9	11.1	10.8%	10.8%	14.5	0.01344	0.00661
16:55	250	20.0	13.0		3.9	9.1	8.9%	8.9%	14.8	0.01344	0.00327
12:45	1440	20.0	10.0		3.9	6.1	5.9%	5.9%	15.3	0.01344	0.00139
12:45	2880	20.0	10.0		3.9	6.1	5.9%	5.9%	15.3	0.01344	0.00098

Technician Name / Certification #

E. Nitz

References:

ASTM D 422: Particle Size Analysis of Soils
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: J. Lessley
Position: Chief Engineer

Laboratory Record Version 4.2

Particle Size Analysis of Soils

S&ME

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-4
Sample #: SS-5
Sample Date: 1/11/05
Depth: 15 feet

Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)

Particle Size Analysis / Without Hydrometer Analysis				Moisture Content	Natural
				Tare #	Angular
	Tare Number		A	Tare Weight	87.30
A	Tare Weight	87.3	B	Wet Weight + Tare Wt.	284.00
B	Total Sample Dry Wt. + Tare Wt.	235.8	C	Dry Weight + Tare Wt.	235.80
C	Total Sample Dry Weight (B-A)	148.5	D	Water Wt. (B-C)	48.20
D	Total Sample Wt. After #200 Wash	136.6	E	Dry Wt.(C-A)	148.50
E	Percent Passing #200 (1-D/C)x100	8.0%	Moisture Content (100 x D/E) (%)		32.5%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.40	0.3%	**99.7%**
0.60	#30	2.80	1.9%	**98.1%**
0.43	#40	15.00	10.1%	**89.9%**
0.25	#60	90.40	60.9%	**39.1%**
0.15	#100	129.30	87.1%	**12.9%**
0.075	#200	136.50	91.9%	**8.1%**

Notes:

	Maximum Particle Size	--	
	Apparent Relative Density	--	
Liquid Limit --	Fineness Modulus	--	
Plastic Limit --	Cu = D60/D10:	2.21	
Plastic Index --	Cc =$(D30)^2$/ (D10xD60):	1.12	

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.3%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**9.8%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**81.8%**
% Silt and Clay	< 0.075 mm	**8.1%**
Description of Sand & Gravel	Rounded ☐	Angular ☐
Hard & Durable ☐	Soft ☐	Weathered & Friable ☐

Organic Content

D10 = **0.14** D30 = **0.22** D60 = **0.31** D50 = **0.29** D90 = **0.43**

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
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ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ______ *Certification #*

Technical Responsibility: R FOREST ______ *Signature* ______ *Position*

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-4
Sample #: SS-7
Sample Date: 1/11/05
Depth: 25 feet

Sample Description: Gray Poorly Graded Sand (SP)

	Particle Size Analysis / Without Hydrometer Analysis			Moisture Content	Natural
				Tare #	H
	Tare Number		A	Tare Weight	84.50
A	Tare Weight	84.5	B	Wet Weight + Tare Wt.	223.80
B	Total Sample Dry Wt. + Tare Wt.	199.4	C	Dry Weight + Tare Wt.	199.40
C	Total Sample Dry Weight (B-A)	114.9	D	Water Wt. (B-C)	24.40
D	Total Sample Wt. After #200 Wash	112.3	E	Dry Wt.(C-A)	114.90
E	Percent Passing #200 (1-D/C)x100	2.3%	Moisture Content (100 x D/E) (%)		21.2%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	0.00	0.0%	**100.0%**
2.00	#10	0.40	0.3%	**99.7%**
0.60	#30	45.30	39.4%	**60.6%**
0.43	#40	70.50	61.4%	**38.6%**
0.25	#60	104.10	90.6%	**9.4%**
0.15	#100	110.50	96.2%	**3.8%**
0.075	#200	112.20	97.7%	**2.3%**

Notes:		Maximum Particle Size	--	Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
		Apparent Relative Density	--	Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**0.3%**
Liquid Limit	--	Fineness Modulus	--	Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**61.0%**
Plastic Limit	--	Cu = D60/D10:	2.31	Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**36.3%**
Plastic Index	--	Cc =$(D30)^2$/ (D10xD60):	0.93	% Silt and Clay	< 0.075 mm	**2.3%**
				Description of Sand & Gravel	Rounded ☐	Angular ☐
				Hard & Durable ☐	Soft ☐	Weathered & Friable ☐
				Organic Content		

D10 = **0.26** D30 = **0.38** D60 = **0.6** D50 = **0.5** D90 = **1.5**

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

echnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature *Position*

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-4
Sample #: SS-9
Sample Date: 1/11/05
Depth: 35 feet

Sample Description: Gray Poorly Graded Sand (SP)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	E
	Tare Number		A	Tare Weight	84.50
A	Tare Weight	84.5	B	Wet Weight + Tare Wt.	307.90
B	Total Sample Dry Wt. + Tare Wt.	275.1	C	Dry Weight + Tare Wt.	275.10
C	Total Sample Dry Weight (B-A)	190.6	D	Water Wt. (B-C)	32.80
D	Total Sample Wt. After #200 Wash	186.5	E	Dry Wt.(C-A)	190.60
E	Percent Passing #200 (1-D/C)x100	2.2%	Moisture Content (100 x D/E) (%)		17.2%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"	0.0	0.0%	**100.0%**
25.00	1.0"	0.00	0.0%	**100.0%**
19.00	3/4"	0.00	0.0%	**100.0%**
12.50	1/2"	0.00	0.0%	**100.0%**
9.50	3/8"	0.00	0.0%	**100.0%**
4.75	#4	10.00	5.2%	**94.8%**
2.00	#10	31.80	16.7%	**83.3%**
0.60	#30	150.20	78.8%	**21.2%**
0.43	#40	162.70	85.4%	**14.6%**
0.25	#60	177.40	93.1%	**6.9%**
0.15	#100	183.60	96.3%	**3.7%**
0.075	#200	186.50	97.8%	**2.2%**

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	4.33
Plastic Index	--	Cc =(D30)^2/(D10xD60):	1.26

Gravel	< 75 mm and > 4.75 mm (#4)	**5.2%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**11.4%**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**68.7%**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**12.5%**
% Silt and Clay	< 0.075 mm	**2.2%**

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = **0.3** D30 = **0.7** D60 = **1.3** D50 = **1** D90 = **2.8**

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

echnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature *Position*

Liquid Limit, Plastic Limit, and Plastic Index

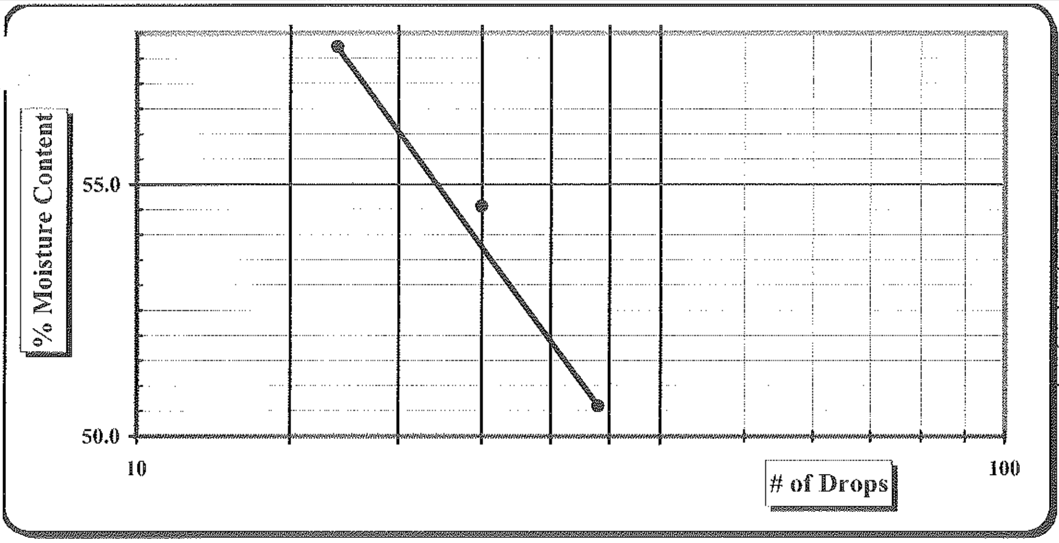
Project #:	1611-04-569	Report Date:	
Project Name:	US 701 Bridges	Test Date(s):	1/28/05
Client Name:			
Client Address:			

Boring #: B-5 Sample #: SS-7 Sample Date: 1/11/05

Depth: 25 feet

Sample Description: Dark Gray Elastic Silt with Sand (MH)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	4	13	24				T4	T9	T21
A	Tare Weight	11.06	10.76	10.89				4.21	4.20	4.29
B	Wet Soil Weight + A	19.90	20.93	20.18				5.64	5.79	6.26
C	Dry Soil Weight + A	16.93	17.34	16.78				5.38	5.49	5.88
D	Water Weight (B-C)	2.97	3.59	3.40				0.26	0.30	0.38
E	Dry Soil Weight (C-A)	5.87	6.58	5.89				1.17	1.29	1.59
F	% Moisture Content (D/E)*100	50.6%	54.6%	57.7%				22.2%	23.3%	23.9%
N	# OF DROPS	34	25	17				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							23.1%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒	NP, Non-Plastic ☐
Liquid limit Test:	Multipoint Method ☐	One-point Method ☐		Liquid Limit 54
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐		Plastic Limit 23
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐		Plastic Index 31
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐		Group Symbol MH

Technician Name: ____E NITZ_____ ________
Certification #

Technical Responsibility: ____R FOREST____ ____________ ____________
Signature *Position*

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-7
Sample Date: 1/11/05
Depth: 25 feet

Sample Description: Dark Gray Elastic Silt with Sand (MH)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Hayley
	Tare Number		A	Tare Weight	100.90
A	Tare Weight	100.9	B	Wet Weight + Tare Wt.	204.80
B	Total Sample Dry Wt. + Tare Wt.	176.9	C	Dry Weight + Tare Wt.	176.90
C	Total Sample Dry Weight (B-A)	76.0	D	Water Wt. (B-C)	27.90
D	Total Sample Wt. After #200 Wash	7.0	E	Dry Wt.(C-A)	76.00
E	Percent Passing #200 (1-D/C)x100	90.8%	Moisture Content (100 x D/E) (%)		36.7%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:

		Maximum Particle Size	--	
		Apparent Relative Density	--	
Liquid Limit	54	Fineness Modulus	--	
Plastic Limit	23	Cu = D60/D10:	--	
Plastic Index	31	Cc =$(D30)^2$/ (D10xD60):	--	

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature *Position*

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569

oject Name: US 701 Bridges

Client Name:

Client Address:

Test Date(s):

Report Date: 1/28/05

Boring #: B-5

Sample #: SS-9

Sample Date: 1/11/05

Depth: 35 feet

Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Midge
	Tare Number		A	Tare Weight	100.80
A	Tare Weight	100.8	B	Wet Weight + Tare Wt.	249.60
B	Total Sample Dry Wt. + Tare Wt.	227.0	C	Dry Weight + Tare Wt.	227.00
C	Total Sample Dry Weight (B-A)	126.2	D	Water Wt. (B-C)	22.60
D	Total Sample Wt. After #200 Wash	118.4	E	Dry Wt.(C-A)	126.20
E	Percent Passing #200 (1-D/C)x100	6.2%	Moisture Content (100 x D/E) (%)		17.9%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:

Maximum Particle Size --

Apparent Relative Density --

Liquid Limit -- Fineness Modulus --

Plastic Limit -- Cu = D60/D10: --

Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐

Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

ASTM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature

Position

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-5, SS-9 grain size

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
roject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-11
Sample Date: 1/11/05
Depth: 45 feet

Sample Description: Gray Poorly Graded Sand (SP)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Oliva
	Tare Number		A	Tare Weight	102.40
A	Tare Weight	102.4	B	Wet Weight + Tare Wt.	221.50
B	Total Sample Dry Wt. + Tare Wt.	200.7	C	Dry Weight + Tare Wt.	200.70
C	Total Sample Dry Weight (B-A)	98.3	D	Water Wt. (B-C)	20.80
D	Total Sample Wt. After #200 Wash	95.8	E	Dry Wt.(C-A)	98.30
E	Percent Passing #200 (1-D/C)x100	2.5%	Moisture Content (100 x D/E) (%)		21.2%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes: Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-13
Sample Date: 1/11/05
Depth: 55 feet

Sample Description: Gray Poorly Graded Sand (SP)

	Particle Size Analysis / Without Hydrometer Analysis			Moisture Content	Natural
				Tare #	Linda
	Tare Number		A	Tare Weight	99.50
A	Tare Weight	99.5	B	Wet Weight + Tare Wt.	266.40
B	Total Sample Dry Wt. + Tare Wt.	239.3	C	Dry Weight + Tare Wt.	239.30
C	Total Sample Dry Weight (B-A)	139.8	D	Water Wt. (B-C)	27.10
D	Total Sample Wt. After #200 Wash	133.6	E	Dry Wt.(C-A)	139.80
E	Percent Passing #200 (1-D/C)x100	4.4%	Moisture Content (100 x D/E) (%)		19.4%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes: Maximum Particle Size --
Apparent Relative Density --
Liquid Limit -- Fineness Modulus --
Plastic Limit -- Cu = D60/D10: --
Plastic Index -- Cc =$(D30)^2$/ (D10xD60): --

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐
Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

chnician Name: E NITZ ____________
Certification #

Technical Responsibility: R FOREST ____________ ____________
Signature *Position*

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-15
Sample Date: 1/11/05
Depth: 65 feet

Sample Description: Gray Poorly Graded Sand (SP)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Lois
	Tare Number		A	Tare Weight	100.10
A	Tare Weight	100.1	B	Wet Weight + Tare Wt.	230.00
B	Total Sample Dry Wt. + Tare Wt.	203.8	C	Dry Weight + Tare Wt.	203.80
C	Total Sample Dry Weight (B-A)	103.7	D	Water Wt. (B-C)	26.20
D	Total Sample Wt. After #200 Wash	98.8	E	Dry Wt.(C-A)	103.70
E	Percent Passing #200 (1-D/C)x100	4.7%	Moisture Content (100 x D/E) (%)		25.3%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	--
Plastic Index	--	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	
Description of Sand & Gravel	Rounded ☐	Angular ☐
Hard & Durable ☐	Soft ☐	Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils
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echnician Name: E NITZ ____________ *Certification #*

Technical Responsibility: R FOREST ____________ *Signature* ____________ *Position*

Liquid Limit, Plastic Limit, and Plastic Index

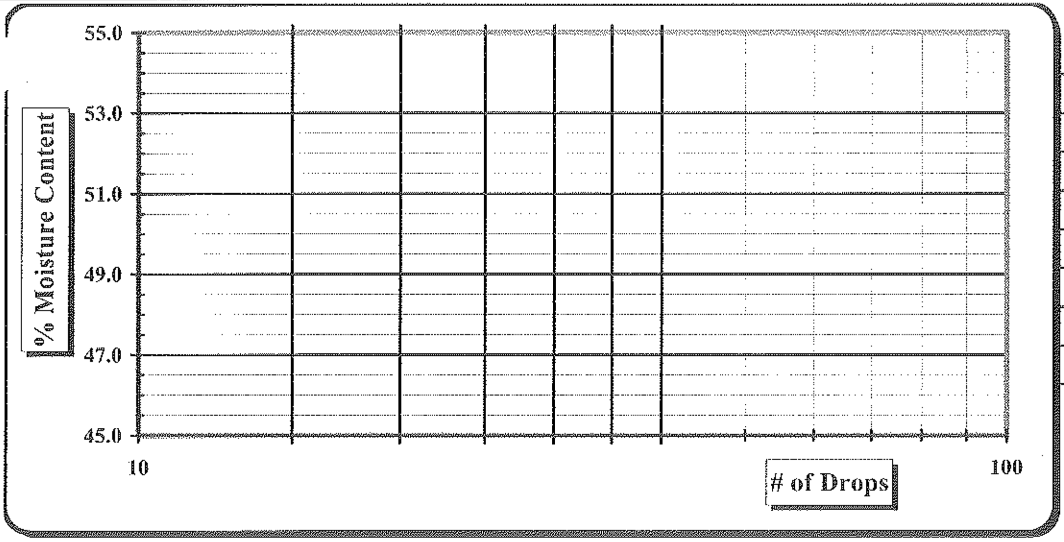
S&ME

Project #: **1611-04-569**
Project Name: US 701 Bridges
Client Name:
Client Address:
Boring #: B-5 Sample #: SS-19

Report Date:
Test Date(s): 1/28/05
Sample Date: 1/11/05
Depth: 85 feet

Sample Description: Gray Poorly Graded Sand (SP)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #									
A	Tare Weight									
B	Wet Soil Weight + A									
C	Dry Soil Weight + A									
D	Water Weight (B-C)									
E	Dry Soil Weight (C-A)									
F	% Moisture Content (D/E)*100									
N	# OF DROPS							*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average									



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation: Wet Preparation ☐ Dry Preparation ☐ Air Dried ☒
Liquid limit Test: Multipoint Method ☐ One-point Method ☒
Classification: ASTM D 2487 ☒ AASHTO M 145 ☐
Liquid limit Test: ASTM D 4318 ☒ AASHTO T 89 ☐
Plastic limit Test: ASTM D 4318 ☒ AASHTO T 90 ☐

NP, Non-Plastic ☒
Liquid Limit
Plastic Limit
Plastic Index
Group Symbol

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-19
Sample Date: 1/11/05
Depth: 85 feet

Sample Description: Gray Poorly Graded Sand (SP)

Particle Size Analysis / Without Hydrometer Analysis			Moisture Content		Natural
				Tare #	Chris
	Tare Number		A	Tare Weight	100.60
A	Tare Weight	100.6	B	Wet Weight + Tare Wt.	223.50
B	Total Sample Dry Wt. + Tare Wt.	206.5	C	Dry Weight + Tare Wt.	206.50
C	Total Sample Dry Weight (B-A)	105.9	D	Water Wt. (B-C)	17.00
D	Total Sample Wt. After #200 Wash	102.6	E	Dry Wt.(C-A)	105.90
E	Percent Passing #200 (1-D/C)x100	3.1%	Moisture Content (100 x D/E) (%)		16.1%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:

		Maximum Particle Size	--
		Apparent Relative Density	--
Liquid Limit	--	Fineness Modulus	--
Plastic Limit	--	Cu = D60/D10:	--
Plastic Index	--	Cc =$(D30)^2/(D10xD60)$:	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
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chnician Name: E NITZ

Certification #

Technical Responsibility: R FOREST

Signature *Position*

S&ME, INC.

231 Labonte Street
Conway, SC 29526

B-5, SS-19 grain size

Liquid Limit, Plastic Limit, and Plastic Index

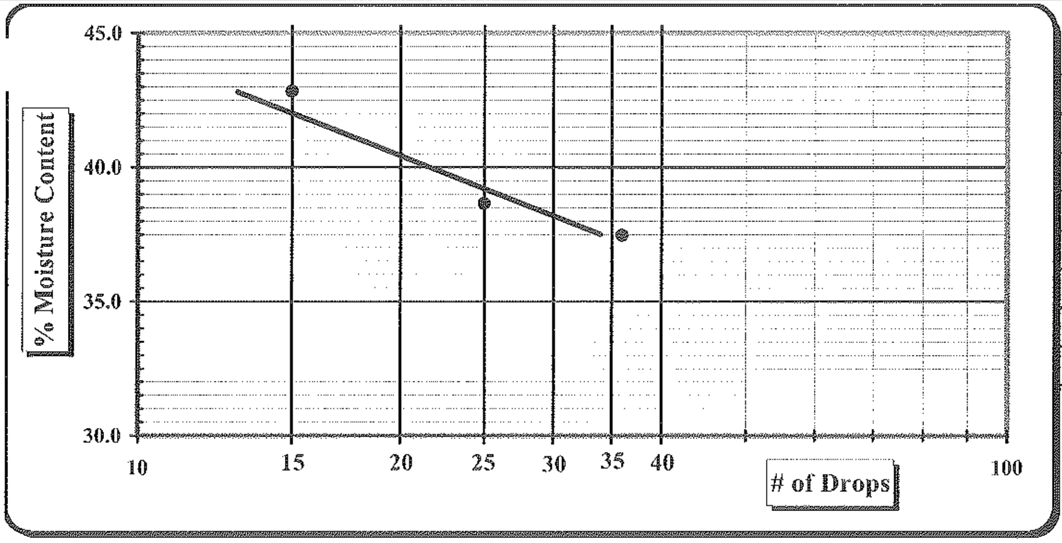
S&ME

Project #: **1611-04-569**
Project Name: US 701 Bridges
Client Name:
Client Address:
Boring #: B-5 Sample #: SS-21

Report Date:
Test Date(s): 1/28/05
Sample Date: 1/11/05
Depth: 95 feet

Sample Description: Gray Sandy Lean Clay (CL)

Pan #		Liquid Limit						Plastic Limit		
	Test #	1	2	3	4	5	6	1	2	3
	Tare #	6	18	14				3	13	5
A	Tare Weight	11.06	10.88	10.84				11.01	10.73	10.95
B	Wet Soil Weight + A	24.49	24.48	23.78				13.36	13.06	13.24
C	Dry Soil Weight + A	20.83	20.69	19.90				13.04	12.76	12.93
D	Water Weight (B-C)	3.66	3.79	3.88				0.32	0.30	0.31
E	Dry Soil Weight (C-A)	9.77	9.81	9.06				2.03	2.03	1.98
F	% Moisture Content (D/E)*100	37.5%	38.6%	42.8%				15.8%	14.8%	15.7%
N	# OF DROPS	36	25	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = F * FACTOR									
Ave.	Average							15.4%		



One Point Liquid Limit			
N	Factor	N	Factor
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.990	29	1.018
24	0.995	30	1.022
25	1.000		

Notes: Estimate the % Retained on the #40 Sieve

Special Sampling Methods:

Sample Preparation:	Wet Preparation ☐	Dry Preparation ☐	Air Dried ☒	NP, Non-Plastic	☐
Liquid limit Test:	Multipoint Method ☐	One-point Method ☐		Liquid Limit	39
Classification:	ASTM D 2487 ☒	AASHTO M 145 ☐		Plastic Limit	15
Liquid limit Test:	ASTM D 4318 ☒	AASHTO T 89 ☐		Plastic Index	24
Plastic limit Test:	ASTM D 4318 ☒	AASHTO T 90 ☐		Group Symbol	CL

Technician Name: E NITZ ________
Certification #

Technical Responsibility: R FOREST ________ ________
Signature Position

Laboratory Record Version 4.2

Particle Size Analysis of Soils

ASTM D 422

Project #: 1611-04-569
Project Name: US 701 Bridges
Client Name:
Client Address:

Test Date(s):
Report Date: 1/28/05

Boring #: B-5
Sample #: SS-21
Sample Date: 1/11/05
Depth: 95 feet

Sample Description: Gray Sandy Lean Clay (CL)

	Particle Size Analysis / Without Hydrometer Analysis	
	Tare Number	
A	Tare Weight	101.8
B	Total Sample Dry Wt. + Tare Wt.	184.9
C	Total Sample Dry Weight (B-A)	83.1
D	Total Sample Wt. After #200 Wash	31.8
E	Percent Passing #200 (1-D/C)x100	61.7%

	Moisture Content	Natural
	Tare #	Karen
A	Tare Weight	101.80
B	Wet Weight + Tare Wt.	205.50
C	Dry Weight + Tare Wt.	184.90
D	Water Wt. (B-C)	20.60
E	Dry Wt.(C-A)	83.10
	Moisture Content (100 x D/E) (%)	24.8%

Sieve Size (mm)	Sieve Size	Retained Weight	Percent Retained	**Percent Passing Total Sample**
37.50	1.5"		0.0%	
25.00	1.0"		0.0%	
19.00	3/4"		0.0%	
12.50	1/2"		0.0%	
9.50	3/8"		0.0%	
4.75	#4		0.0%	
2.00	#10		0.0%	
0.60	#30		0.0%	
0.43	#40		0.0%	
0.25	#60		0.0%	
0.15	#100		0.0%	
0.075	#200		0.0%	

Notes:
Maximum Particle Size --
Apparent Relative Density --

Liquid Limit	39	Fineness Modulus	--
Plastic Limit	15	Cu = D60/D10:	--
Plastic Index	24	Cc =$(D30)^2$/ (D10xD60):	--

Gravel	< 75 mm and > 4.75 mm (#4)	**0.0%**
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	**#VALUE!**
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	**#VALUE!**
Fine Sand	< 0.425 mm and > 0.075 mm (#200)	**#VALUE!**
% Silt and Clay	< 0.075 mm	

Description of Sand & Gravel Rounded ☐ Angular ☐
Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

Organic Content

D10 = D30 = D60 = D50 = D90 =

ASTM D 422: Particle Size Analysis of Soils
Hydrometer portion of test method not utilized.
ASTM D 421: Dry Preparation of Soil Samples
ASTM D 854: Specific Gravity of Soils
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ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technician Name: E NITZ
Certification #

Technical Responsibility: R FOREST
Signature
Position

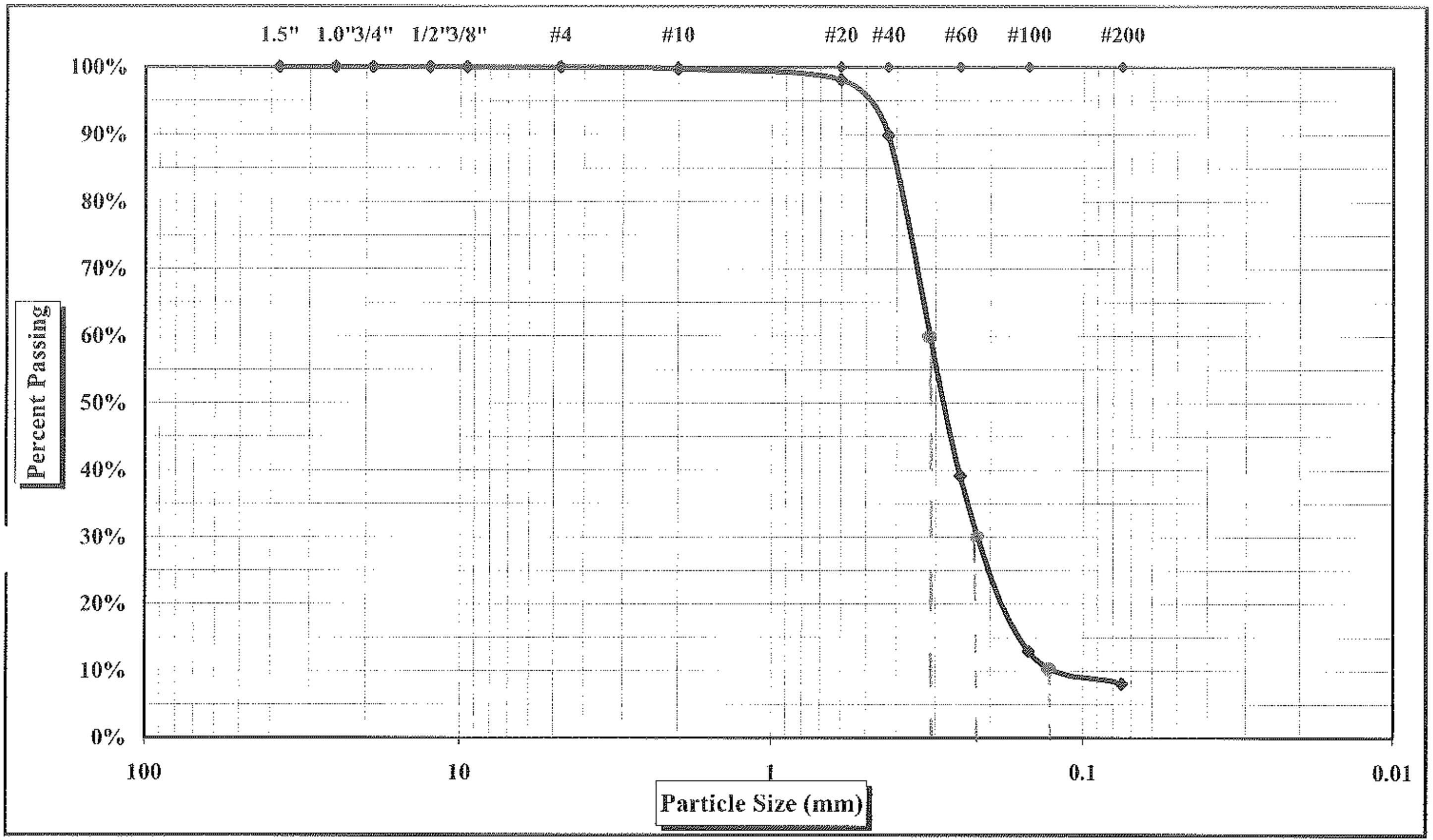
Particle Size Analysis of Soils

ASTM D 422

S&ME Project #:	**1611-04-569**	Report Date:	1/28/05
oject Name:	**US 701 Bridges**	Test Date(s):	
Client Name:			
Client Address:			

Boring #:	B-4	Sample #:	SS-5	Sample Date:	1/11/05
				Depth:	15 feet

Sample Description: Gray Poorly Graded Sand with Clay (SP-SC)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	--	Gravel	0%	Medium Sand	10%
Silt & Clay (% Passing #200)	8.1%	Coarse Sand	0%	Fine Sand	82%
Apparent Relative Density	--	Moisture Content	32.5%	Organic Content	
Liquid Limit	--	Plastic Limit	--	Plastic Index	--

Description of Sand & Gravel

Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

References: ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

'M D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

ıM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: R FOREST

Signature

Position

Particle Size Analysis of Soils

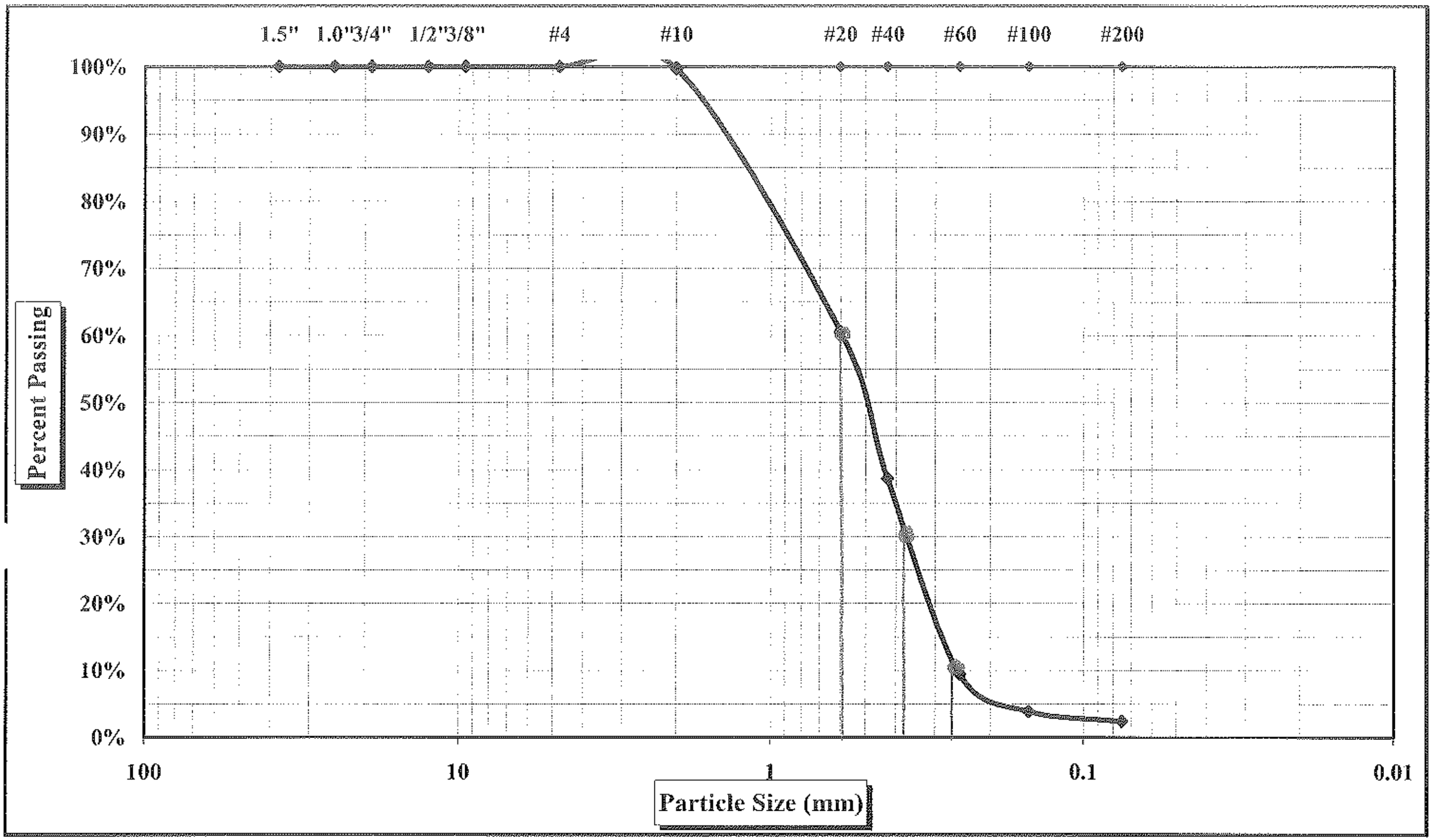
ASTM D 422

S&ME Project #: **1611-04-569**
oject Name: **US 701 Bridges**
Client Name:
Client Address:

Report Date: 1/28/05
Test Date(s):

Boring #: B-4
Sample #: SS-7
Sample Date: 1/11/05
Depth: 25 feet

Sample Description: Gray Poorly Graded Sand (SP)



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	--	Gravel	0%	Medium Sand	61%
Silt & Clay (% Passing #200)	2.3%	Coarse Sand	0%	Fine Sand	36%
Apparent Relative Density	--	Moisture Content	21.2%	Organic Content	
Liquid Limit	--	Plastic Limit	--	Plastic Index	--

Description of Sand & Gravel

Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

References: ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

M D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

TM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: ____R FOREST____ ____________________ ____________________

Signature *Position*

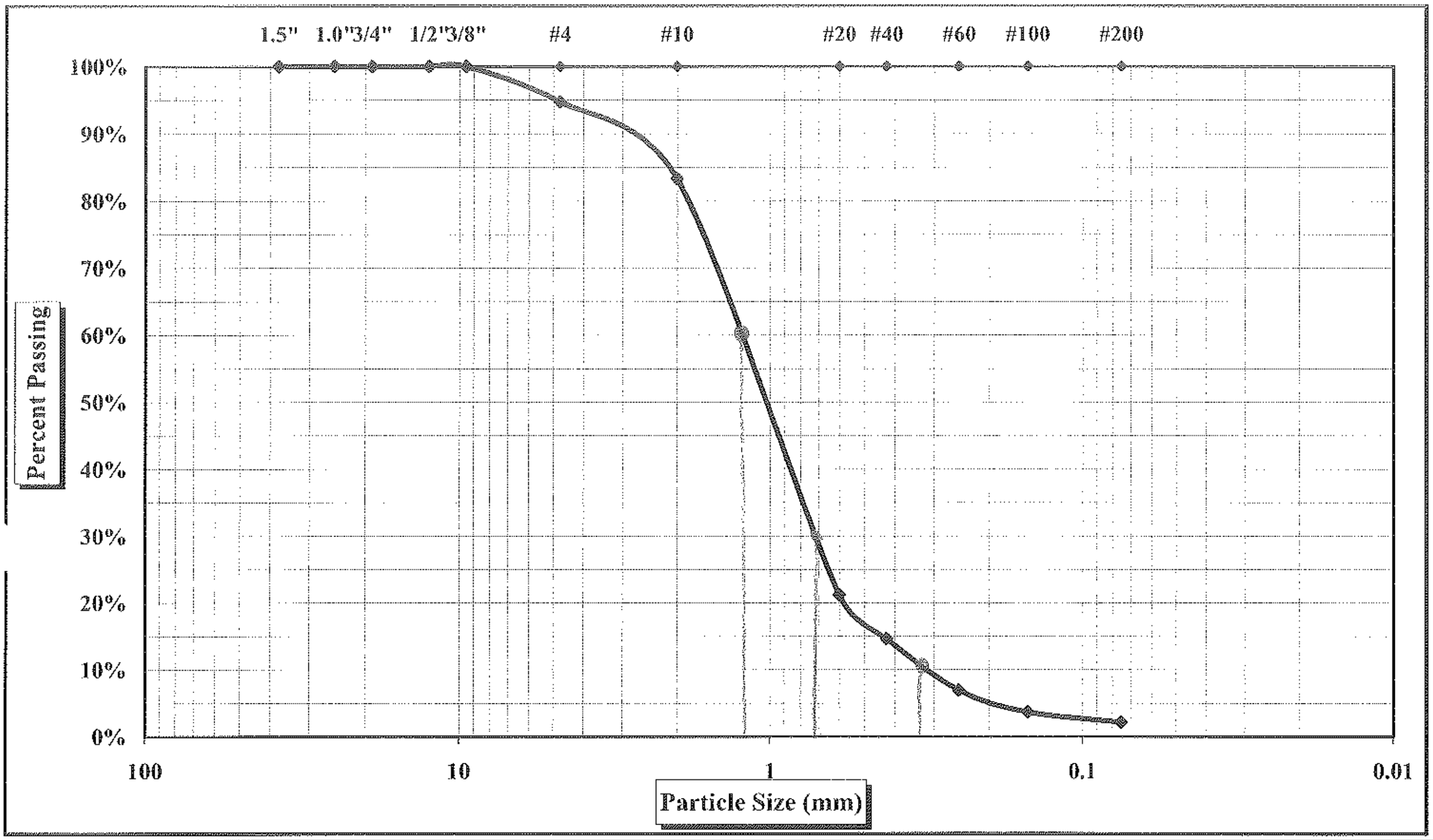
Particle Size Analysis of Soils

ASTM D 422

S&ME Project #: 1611-04-569
oject Name: US 701 Bridges
Client Name:
Client Address:

Report Date: 1/28/05
Test Date(s):

Boring #:	B-4	Sample #:	SS-9	Sample Date:	1/11/05
				Depth:	35 feet
Sample Description:	Gray Poorly Graded Sand (SP)				



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Maximum Particle Size	--	Gravel	5%	Medium Sand	69%
Silt & Clay (% Passing #200)	2.2%	Coarse Sand	11%	Fine Sand	12%
Apparent Relative Density	--	Moisture Content	17.2%	Organic Content	
Liquid Limit	--	Plastic Limit	--	Plastic Index	--

Description of Sand & Gravel

Rounded ☐ Angular ☐ Hard & Durable ☐ Soft ☐ Weathered & Friable ☐

References: ASTM D 422: Particle Size Analysis of Soils

Hydrometer portion of test method not utilized.

TM D 421: Dry Preparation of Soil Samples

ASTM D 854: Specific Gravity of Soils

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ASTM D 2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System)

Technical Responsibility: R FOREST

Signature

Position