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3 September 2003

Mississippi Space Services
A Joint Venture of CSC and Shaw Group
John C. Stennis Space Center
Building 2204
Stennis Space Center, Mississippi 39529-6000

Attention Ms. Terri L. Baker

Ladies and Gentlemen:

Geotechnical Investigation
John C. Stennis Space Center
NASA First Response Facility
Stennis Space Center
Hancock County, Mississippi
Eustis Engineering Project No. 18080

Transmitted are two copies (one bound and one unbound) of our engineering report covering a geotechnical investigation for the subject project.

Thank you for asking us to perform these services.

Yours very truly,

EUSTIS ENGINEERING COMPANY, INC.

GWENDOLYN P. SANDERS, P.E.

CLS:aln/mcp

GEOTECHNICAL INVESTIGATION

JOHN C. STENNIS SPACE CENTER

NASA FIRST RESPONSE FACILITY

STENNIS SPACE CENTER

HANCOCK COUNTY, MISSISSIPPI

EUSTIS ENGINEERING PROJECT NO. 18080

FOR
MISSISSIPPI SPACE SERVICES
STENNIS SPACE CENTER, MISSISSIPPI

By
Eustis Engineering Company, Inc.
Metairie, Louisiana

3 SEPTEMBER 2003

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NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER (HANCOCK COUNTY), MISSISSIPPI
EUSTIS ENGINEERING PROJECT NO. 18080

INTRODUCTION

1. This report contains the results of a geotechnical investigation performed for the proposed NASA First Response Facility at the Stennis Space Center in Hancock County, Mississippi. Authorization to proceed was given under Mississippi Space Services Task Order No. 22-MSS-48589.

2. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of Mississippi Space Services and their associates for specific application to the subject site. In the event of any changes in the nature, design, or location of the proposed First Response Facility, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified and verified in writing. Should these data be used by anyone other than Mississippi Space Services and their associates, they should contact Eustis Engineering for interpretation of data and to secure any other information pertinent to this project.

3. The analyses and recommendations contained in this report are based in part on data obtained from the soil borings. The nature and extent of variations in subsoil conditions between and away from the boring locations may not become evident until construction. If variations then appear, it will be necessary to reevaluate the recommendations contained in this report.

4. Recommendations and conclusions contained in this report are to some degree subjective and should be used for design purposes only. This report should not be included in the contract plans and specifications. However, the results of the soil borings and laboratory tests contained in the Appendix of this report may be included in the plans and specifications.

SCOPE

5. The original scope of work included the drilling of three undisturbed soil test borings (each to the 20 foot depth) and the drilling of three auger borings (each to the 5 foot depth.) The investigation included the drilling of soil test borings to determine subsoil conditions and stratification, and to obtain samples of the various substrata. Soil mechanics laboratory tests performed on samples obtained from the borings were used to evaluate the physical properties of the subsoils. Engineering analyses, based on the soil borings and laboratory test results, were made to determine recommendations regarding site preparation, drainage, placement and compaction of fill, allowable soil bearing values, estimates of settlement, and construction recommendations.

6. An additional scope of work included the drilling of two undisturbed soil test borings (each to the 75 foot depth). The additional scope of work was preformed in accordance with Eustis Engineering's proposal dated 1 August 2003. The additional scope of work was performed to assist with engineering analyses for allowable pile load capacities and estimates of settlement from placement of fill.

7. It should be noted that the scope of this work does not include the investigation or detection of biological pollutants in or around the structure. The term "biological pollutants", includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and the byproducts of any such biological organisms.

SOIL BORINGS

8. Five undisturbed sample type soil test borings, three 20 feet in depth (1, 2, and 3) and two 75 feet in depth (4 and 5) and three auger borings (A-1, A-2, and A-3) each 5 feet in depth were made at the site between 25 July and 11 August 2003 at the locations shown on Figure 1. Detailed descriptive logs of the borings and laboratory tests are shown in both tabular and graphical form in the Appendix.

Undisturbed Borings

9. The undisturbed soil borings were made with a truck mounted rotary type drill rig. Upon completion of drilling the borings, the holes were backfilled in accordance with current regulatory requirements.

10. Samples of cohesive or semi-cohesive subsoils were obtained at close intervals or changes in stratum using a 3-in. diameter thinwall Shelby tube sampling barrel. The samples were immediately extruded from the sampling barrel, inspected, and visually classified by Eustis Engineering's soil technician. Pocket penetrometer tests were performed on the soil samples to give a general indication of their shear strength or consistency. The results of these tests are shown on the boring logs under the column heading "PP." Representative portions were then promptly placed in moisture proof containers and sealed for preservation of their natural moisture content.

11. Samples of cohesionless and semi-cohesive materials were obtained during the performance of in situ Standard Penetration Tests. This test consists of driving a 2-in. diameter sampler 1 foot into the soil after first seating it 6 inches. A 140-lb weight dropped 30 inches is used to advance the sampler. The number of blows required to drive the sampler is indicative of the relative density of cohesionless soils and the consistency of cohesive soils. The samples were retained in moisture proof containers for preservation of their natural moisture content. The results of

the Standard Penetration Tests are shown on the boring logs under the column heading "SPT."

Auger Borings

12. The auger borings was made with a hand auger. The subsoils were sampled directly from the auger blades at close intervals or changes in strata. These samples were sealed in plastic bags to preserve their natural moisture content.

LABORATORY TESTS

13. Soil mechanics laboratory tests consisting of natural water content, unit weight, and unconfined compression shear (UC) were performed on samples obtained from the undisturbed borings. Samples obtained from the auger borings were visually classified and tested for their natural water content. In addition, Atterberg liquid and plastic limit tests were performed on selected representative samples to aid in classification of the subsoils and to give an indication of their relative compressibility. The results of the laboratory tests are summarized on the boring logs in the Appendix.

14. Grain size analyses were performed on selected representative cohesionless samples obtained from the undisturbed borings. The results of these tests are shown graphically on separate sheets following the boring logs in the Appendix.

DESCRIPTION OF SITE CONDITIONS

15. A compaction test (AASHTO T 99) was also performed on a composite sample made from bulk samples taken at the auger boring locations. The results of this test are shown on a separate sheet in the Appendix. A California Bear Ratio (CBR) test (AASHTO T 193-99) was also performed in the laboratory. The results of the CBR test are tabulated in the Appendix.

Surface Conditions

16. The site of the proposed building is essentially level with elevations varying between 26.0 and 26.5 feet NAVD 88. Brush and several small trees are present on the site and standing water was observed over portions of the site. Existing grades at the site are approximately 2 feet below the grade of the existing roadway at the perimeter of the site.

Stratigraphy

17. Reference to boring logs shows that the surficial soils at the boring locations consist of approximately 1 foot of medium compact dark gray clayey silt and very soft to medium stiff dark gray, tan and brown silty clay or sandy clay. These surficial materials are underlain by medium stiff to very stiff light gray, tan and brown sandy clay to the approximate 6 to 13-ft depths. Strata of very loose to medium dense light gray, tan and white clayey sand or sand was encountered between the approximate 6 to 17-ft depths. Very soft to stiff gray clay or sandy clay continues to the termination of Borings 1, 2, and 3 at the 20-ft depths below the existing ground surface and to the approximate 31-ft depths in Borings 4 and 5. A stratum of loose to very dense light gray and gray sand was encountered between the approximate 31 to 44-ft depths. Medium stiff to stiff light gray and gray clay and sandy clay continues to the approximate 61-ft depth. Very dense gray and brown sand continues to the termination of Borings 4 and 5 at approximate 75-ft depths below the existing ground surface.

Ground Water

18. In order to determine the ground water conditions at the time of the field investigation, observations were made during the drilling of A-3. The boring was drilled without the addition of water and ground water was initially encountered at a depth of 3.5 feet below the existing ground surface. The depth to ground water will vary with climatic conditions, drainage improvements, and other factors. The

depth to ground water should be determined by those persons responsible for construction immediately prior to beginning work.

FOUNDATION ANALYSIS

Furnished Information

19. Information provided by Mississippi Space Services indicates the proposed NASA First Response Facility will be constructed on either footings or a pile supported foundation. The proposed facility will have an approximate 43,000 square foot slab with plan dimensions of 155' x 275'. The building is currently designed as a cast-in-place reinforced concrete frame with precast concrete cladding. Wall loads of approximately 7 kips/ft are anticipated with approximately half of this load being dead load. Column loads of approximately 210 kips are anticipated with approximately 190 kips being dead load. The finished floor elevation of the building is approximately 4.5 feet above existing site grades.

20. Parking for a total of 206 vehicles will be provided in a total of four parking lots in the vicinity of the building. A 100' x 100' helipad will be constructed in the rear of the building.

Foundation Recommendations

21. Compressible clays are present at the site in two distinct groupings. The first group of clay strata is present between approximately 13 feet and 31 feet below the existing ground surface. The second grouping of clay strata is present between approximately 40 and 61 feet below the existing ground surface. These clays introduce significant settlement potential to shallow foundations constructed within a 4 to 4.5-ft fill pad at the site. We estimate a slab-on-grade will settle 3 to 4 inches due solely to the slab load and fill placed for the building pad.

22. For this reason, we recommend against supporting the structure by footings and a slab-on-grade. We have developed recommendations for allowable soil bearing values for mats, slabs, and footings in this report. However, we understand these parameters will only be used to design appurtenant structures (outside the main building) for which settlement is not a concern.

23. Without the benefit of a site preloading program, we recommend the building be supported by driven timber piles. As the piles will experience downdrag settlements attributable to construction of the fill pad, the lengths of the piles should be governed by the amount of settlement judged tolerable by the structural engineer. Regardless of the pile length selected, all building loads (wall, columns, and floors) should be supported by the piles. In addition, all piles should penetrate to the same tip embedment to minimize the potential for differential settlement. Pile supported features will settle significantly less than grade supported features and the differential settlements between pile supported and grade supported features should be considered in the project designs.

24. Site conditions are such that an influx of water, naturally or as a result of construction operations, could cause significant decreases in the strength of near surface soils. Degradation of site conditions due to excess water will cause decreases in bearing capacity and workability of the near surface soils and an increase in settlement potential of overlying structures. We recommend the site be properly prepared to rapidly direct water off site. We further recommend all site preparation, excavation, clearing, proofrolling, and backfilling be completed when the subgrade is dry and stable.

<u>Site Preparation</u>

25. <u>Drainage During Construction.</u> The initial step to prepare the site for construction should be to establish adequate temporary and permanent drainage to prevent ponding of water and ensure immediate runoff of all rainfall. It is strongly recommended the contractor maintain adequate surface drainage away from all

foundation and pavement areas during and after construction. This may be accomplished by utilizing existing drainage features and by setting grades to ensure positive drainage of water away from the foundation areas. Sumps and pumps may be required to remove rainfall and ground water from shallow excavations. During construction, the contractor should exercise caution during inclement weather to ensure subsoil support is not degraded by construction operations.

26. Permanent Drainage. The near surface soils are subject to a reduction in shear strength and excessive settlement if the moisture content of these soils increases (naturally or as a result of construction operations). It is strongly recommended adequate permanent drainage (including adequate surface and subsurface features as required) be provided to collect all rainfall away from the building foundation and pavement areas after completion of construction. All downspouts draining rainfall from the building roof should be connected to pipes which discharge away from the building or into a drainage system. Water should not be allowed to collect near the building foundation and pavement areas.

27. Clearing and Stripping. Within the areas of the proposed fill pad (including the building footprint, parking lots, roadways, and helipad), the existing ground surfaces should be stripped of vegetation, loose topsoil, debris, stumps, organic matter, and any other deleterious materials. Stripping should be to the minimum depth necessary to remove vegetation and roots and reach firm undisturbed soils. Based on the soil borings, we expect approximately 6 to 12 inches of surface materials to be removed. The site should not be stripped until construction drainage measures have been provided. The exact depth of stripping should be determined during construction.

28. Subgrade Preparation. After the stripping and clearing operations, the exposed surface should be proofrolled with a bulldozer or tracked vehicle having an operating weight between 75 and 90 kips and a ground pressure between 10 and 15 psi. Alternative proofrolling techniques may be proposed, but these methods should be approved by Eustis Engineering prior to their use at the site. Any

depressions, stump holes, or weak areas identified should be thoroughly cleaned out to the surface of firm undisturbed soil and backfilled with a select structural fill material placed and compacted under controlled conditions. All clearing, proofrolling, and compaction operations should be performed during periods of dry weather only. Motorized wheeled equipment should not be allowed within the foundation areas during periods of inclement weather to prevent rutting of the subgrade.

29. Structural Fill. A select granular material, such as hydraulically pumped river sand, should be used as backfill and/or fill required to reach design grade. Sand fill should be non-plastic and free of roots, clay lumps, and other deleterious materials with no more than 10% by weight of material passing a U.S. Standard No. 200 mesh sieve. Alternatively, clayey sand having a maximum liquid limit of 25 and a plasticity index of no more than 15 may be used as select fill. Prior to transporting structural fill to the site, a sample of the borrow material should be tested to verify its conformance to the specifications.

30. Placement and Compaction. In general, the fill should be placed in lifts of 6 to 8 inches loose measure and compacted to a dry density corresponding to 95% of the maximum dry density determined in accordance with ASTM D 698. Each lift within the uppermost 2 feet beneath pavements or slabs or within the uppermost 3 feet below footings should be compacted to 95% of the maximum dry density within ±2% of optimum water content in accordance with ASTM D 1557. The initial lift of backfill placed in depressions or stump holes may be compacted to 93% of the maximum dry density near optimum moisture in accordance with ASTM D 698 provided the top of this lift is more than 2 feet below the proposed pavements or slabs and more than 3 feet below any proposed footings.

31. Quality Control. Density tests should be performed on each lift of the compacted structural fill to determine if the contractor has achieved the recommended density. We recommend a minimum of one inplace density test be performed for every 25,000 square feet of material placed up to two levels 2 feet below proposed

pavements for 3 feet below proposed footings. The frequency of in situ testing should be increased to one test per 10,000 square feet for fill materials placed above a level 2 feet below proposed pavements or 3 feet below proposed footings. In any case, at least one density test should be performed during any shift in which compaction operations are completed. Additional testing may be required in areas where there is an apparent change in quality of fill, effectiveness of compaction, or moisture levels in the compacted material.

32. Fill Settlement. Consolidation of the subsoils can be expected due to placement of fill to reach finished grade. Fill placement may result in differential settlement between grade supported structures, such as pavements and pile supported structures. Your design should recognize this potential. Fill placement will also affect pile foundations as discussed subsequently in "Deep Foundations."

33. Based on the topographic survey data, approximately 4 feet of fill will be required to reach design grade at the proposed facility. We estimate ultimate consolidation settlement of the existing ground surface at the center of an approximate 300' x 300' filled area due to placement of 4 feet of fill will be 2½ to 3½ inches. At the edges of the filled area, settlements will be approximately equal to one-half the settlement at the center of the filled area. Approximately one-fourth of the center settlement will occur at the corners of the filled areas.

34. Utilities. We recommend that flexible type connections be specified for all piping and utilities going to and from the proposed structure. These connections should be designed to accommodate the settlements and differential settlements described in the previous paragraphs due to fill placement.

Shallow Foundations

35. Depth of Footings. Based on the boring logs, continuous grade beam footings and isolated square footing foundations for the proposed structure should be placed to bear at least 24 inches below final grade on compacted structural fill. Precautions

should be exercised so excavations for footings do not become wet prior to pouring concrete. Foundations should be poured immediately after the completion of the excavations. All footing excavations should be carefully inspected by qualified personnel to verify footings will be placed to bear on firm undisturbed soil or compacted structural fill at the recommended depth and the excavation is in a dry condition prior to pouring concrete. Eustis Engineering may be retained to observe the condition within footing excavations prior to concrete placement.

36. Allowable Soil Bearing Values. Analyses have been made to estimate the net allowable soil bearing values for continuous grade beam footing foundations and isolated square footing foundations. These recommendations are provided with the understanding that these footings will support isolated structures that are not connected to the main building and the footings may experience substantial settlements relative to pile supported structures. A shallow continuous footing foundation, placed to bear at least 24 inches below final grade and having a width of 1 to 3 feet, may be designed for a net allowable soil bearing value of 1,500 psf. A shallow isolated square footing foundation, placed to bear at least 24 inches below final grade and having a width between 2 and 5 feet, may be designed for a net allowable soil bearing value of 1,800 psf. These allowable soil bearing values contain estimated factors of safety of 3 against a soil shear failure.

37. Estimated Settlement of Footings. Assuming a long term dead load pressure intensity equal to 80% of the allowable soil bearing values, estimates of settlement were made for various footing types and sizes. We estimate settlement of continuous footing foundations having widths of 1 to 2 feet and isolated square footings with side dimensions of 3 feet to be ¼ inch or less. We estimate settlement of isolated square footings with side dimensions of 4 to 5 feet to be ¼ to ½ inch and continuous footings with a width of 3 feet and square footings with side dimensions of 6 to 7 feet to be ½ to ¾ inch. If the footings are constructed within the footprint of a slab, these settlement estimates should be considered additive to the settlement estimated for the slab supporting a separate uniform dead load. Ground

surface settlements due to site filling should also be added to individual footing settlements.

38. Our estimates of settlement assume the center to center spacing between continuous footings is not less than three times the footing width, and the center to center spacing between adjacent square footings is not less than twice the largest footing side dimension. We have also assumed the site has been prepared as recommended in this report and the foundation soils are not degraded or exposed to excess moisture prior to placing concrete for the footings, and no more than 6 feet of fill will be required to reach finished grade at the site. To decrease the potential of differential movements, concrete for footings should be placed integrally with grade beams and slabs. If any of our assumptions are not met, Eustis Engineering should be notified to reevaluate potential settlement.

Deep Foundations

39. Allowable Pile Load Capacities. Analyses have been made to determine the estimated allowable single pile load capacities in compression for various sizes of open end steel pipe piles and treated ASTM D 25 quality timber (or timber composite) piles for support of the proposed structure. The results of these analyses are shown on Figure 2. It should be noted that pile penetration is referenced from the existing ground surface instead of the proposed site elevation.

40. Ultimate pile load capacities represent the largest applied loads required to overcome the combined resistance of soil adhesion or friction on the surface area of the embedded portion of the pile and the penetration resistance of soils at the pile toe. Our allowable pile load capacities are obtained by dividing the ultimate load capacity by a factor of safety of 2 against failure of a single pile through the soil. These estimated allowable load capacities should be verified by pile load tests. In the event pile load tests are not performed, the factor of safety should be increased to 2.5, effectively decreasing the allowable pile load capacity.

41. Loads Due to Fill Placement. The piles recommended in this report will be affected by placement of fill on the site. As the fill settles from consolidation of the underlying deposits, negative skin friction (drag loads) will be induced on the piles as the soil settles along the pile. These drag loads may result in additional pile settlement and an increase in the load applied to the pile.

42. Piles should be structurally designed to resist the combination of sustained load and downdrag load. This is a separate loading case to be considered apart form selection of the appropriate pile load capacity required to resist the maximum applied loads. Piles should also be expected to settle under the application of downdrag loads. These pile settlements are discussed subsequently. Should the finished floor elevation of the building be changed and additional fill required, Eustis Engineering should be contacted to reevaluate the effect of fill placement on pile foundations.

43. Timber Piles. We recommend the timber piles meet specifications outlined in Section 719 of the Mississippi Standard Specifications for Road and Bridge Construction, 1990 edition (MSSRBC), for both preservative and quality assurance. Treatment should also follow Section 718 where applicable. The pile dimensions assumed in our analyses are provided on Figure 2. For longer pile lengths recommended in this report, treated timber piles may not be available or economical and timber composite piles may be required. However, composite piles should not be used to resist lateral or tensile loads.

44. Timber Composite Piles. Composite piles should consist of an untreated ASTM D 25 quality timber pile having a minimum 7-in. tip and 12-in. butt lower section and a 12-in. diameter concrete filled metal can upper section. The metal can upper section should extend a minimum distance of 10 feet below the existing ground surface to protect the untreated timber section. The metal can section should be of sufficient thickness to withstand handling stresses and soil and water pressures. We recommend a maximum can length of 18 feet. A mandrel impacting the timber pile butt should be used to install the metal can. Prior to placing concrete, the metal

can should be inspected to ensure it is free of water. Concrete placed in the metal can should have a minimum compressive strength of 4,000 psi. Timber composite piles should not be used to resist lateral or tensile loads.

45. <u>Open End Steel Pipe Piles.</u> We recommend the open end steel pipe piles meet the requirements outlined in Section 719.04 of the MSSRBC. The steel piles should be designed to have a wall thickness that is structurally sufficient to withstand handling and driving stresses. The pile dimensions assumed in our analyses are based on the outside pile diameter only. The actual wall thickness selected for the open end pile will not affect our estimated vertical capacities. However, we recommend your specifications require no protrusions past the outside diameter of the pipe piles. This includes the use of spiral welded pipes or segmented pile sections. If the welds protrude past the pile's outside diameter, the soil-pile friction is disturbed during installation of the pile and the pile's capacity may be reduced.

46. <u>Structural Capacity.</u> Analyses for pile capacities are based on a soil-pile relationship only. Therefore, the structural capacity of the piles and their connections to transmit these loads should be determined by a structural engineer.

47. <u>Pile Group Capacity.</u> All of the piles considered will derive the majority of their supporting capacity from skin friction; therefore, it is necessary to consider the effect of group action. In this regard, the supporting value of the friction piles driven in groups should be investigated on the basis of group perimeter shear by the formula shown on Figure 3.

48. <u>Pile Spacing.</u> The minimum spacing between individual piles should be determined by the formula given on Figure 3. The minimum spacing between rows or groups of piles should also meet the requirements discussed in the “Settlement” section of this report.

49. <u>Estimated Settlement Due to Structural Loads.</u> We recommend slabs be cast monolithically with grade beams and be rigidly connected to pile caps to minimize

the potential for differential settlements. We estimate piles driven within sand strata between 35 and 44 feet below the existing ground surface to a resistance indicative of the design load capacity will settle½ to ¾ inch due to structural loads. We estimate piles driven to a resistance indicative of the design load capacity within the sand strata present below the 61-ft depth will settle ½ inch or less due to structural loads. These estimates do not include elastic deformation of the piles which should be added to the settlement estimates. Elastic deformation of the piles may be estimated as 67% to 75% of the static column strain of a pile acting as a column. ***These estimates of settlement are due to structural loads only and should be considered additive to settlements due to the placement of fill.***

50. Our estimates of settlement are based on the assumptions piles will be driven in small groups or widely spaced rows. We have assumed the largest group dimension will be no greater than 20% of the pile length and the center to center spacing between groups will be no closer than twice the largest group dimension. We have assumed the center to center spacing between rows of single piles will be no closer than 8 feet. In the event any of our assumptions are not met, Eustis Engineering should be contacted to evaluate the potential settlement of the pile foundations. All piles should be installed to approximately the same tip embedment in order to minimize differential foundation settlements.

51. Estimated Settlement Due to Fill Placement. The placement of fill at the site will result in negative skin friction (drag loads) on the surface of the piles. These drag loads have the potential to increase settlement of piles. Based on our analyses, we estimate piles driven to a driving resistance indicative of the estimated allowable load capacity between 35 and 44 feet below the existing ground surface will experience 1¼ to 1¾ inches of settlement due to the placement of 4 feet of fill. We estimate piles driven to a resistance indicative of the design load capacity at a depth of 61 feet or more will experience ½ to ¾ inch of settlement due to the placement of 4 feet of fill. These settlements should be added to pile settlements estimated for the applied structural loads. If our assumptions are not met, Eustis Engineering should be contacted to reevaluate potential settlement of pile foundations.

52. Differential Settlement. Your design should recognize the potential for differential settlement between pile supported features and grade supported features. A joint considering these movements should be provided between any grade supported and pile supported features to accommodate potential differential settlement. In addition, the structure should be designed as rigidly as possible to minimize the potential for differential settlements.

Installation of Driven Piles

53. Quality Control. Close field supervision should be maintained by experienced personnel to ensure proper procedures are followed and accurate records are kept for all pile driving operations. The driving record should include, as a minimum, the date, pile type, overall length, tip and butt diameters, embedment below finished grade, depth and diameter of predrill, hammer type, and the number of blows per foot of penetration. An accurate driving record is especially important to verify the piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics which may indicate pile breakage.

54. Air Hammers. ASTM D 25 quality timber piles (installed singly or as part of a composite pile) should be driven with a single acting air hammer having a manufacturer's rated energy of 15,000 ft-lbs per blow. Open end steel pipe piles should be driven with a single acting air hammer with a manufacturer's rated energy of at least 19,500 ft-lbs per blow for allowable compressive capacities up to 60 tons.

55. Diesel Hammer. In lieu of an air hammer, a diesel hammer may also be used for the installation of the steel piles. We recommend the diesel hammer have a rated energy of 1.5 times the energy recommended for a comparable installation with a single acting air hammer.

56. Pile Refusal. Refusal criteria for the timber piles may be taken as 25 blows per foot with the recommended hammer to minimize damage to these piles. Refusal criteria for the steel piles should be determined based on the results of the test pile

program and dynamic analyses or testing. However, we anticipate these piles will be driven to their full penetration assuming adequate pile strength is present to prevent buckling of the pile during installation. If the piles are driven with the aid of a mandrel (composite piles) or a follower, or if the pile driving helmet is allowed to impact the ground surface, Eustis Engineering should be consulted to adjust these refusal criteria.

57. Alternate Installation Methods. We do not recommend vibratory methods be utilized for pile installation. If a vibratory hammer is selected for the project, Eustis Engineering should be contacted to evaluate the reduction in the estimated allowable pile load capacities presented. Also, we do not recommend the use of jetting to aid in the installation of the piles. Eustis Engineering should be consulted for additional installation criteria if jetting is required. If any other alternate installation methods are selected, Eustis Engineering should be contacted to evaluate the impact on the estimated capacities presented.

58. Prepunching or Predrilling. Prepunching or predrilling for timber piles with embedment depths greater than the 45-ft depth may be required to assist pile driving through medium dense to very dense sand layers present between the approximate 6 to 13-ft depths and the 31 to 44-ft depths. These materials were encountered at almost all of the boring locations so that predrilling should be anticipated for all piles penetrating these soils.

59. Predrilling through surficial materials to a depth of 15 feet may be by dry auger methods. Predrilling to greater depths should be accomplished by wet rotary techniques. In either case, the prepunch or predrill bit should be no larger than the pile tip diameter for timber piles or 75% of the pile diameter for steel piles. In no case should predrilling extend closer than 10 feet from the design pile tip embedment. Actual requirements should be determined during the test pile program. Eustis Engineering should be contacted if a deeper pilot hole is necessary.

60. Dynamic Analyses. The steel pipe piles should be designed by a structural engineer to have a cross-section which is structurally sufficient to facilitate driving of the piles without damage. Dynamic analyses (WEAP) can be performed to evaluate driving stresses, pile cushion, and driveability once the hammer and appurtenant equipment have been selected. Structural requirements can then be verified by a structural engineer and installation criteria can be established.

Test Piles and Load Tests

61. Eustis Engineering considers a test pile program and load test as an extension of our geotechnical investigation. Therefore, Eustis Engineering should be retained to perform these services. A series of test piles of each type and length proposed for use should be installed for the project. The actual number of test piles should be determined based on the number and type of piles used in any one area. Eustis Engineering should be consulted to develop a test pile program consistent with the project's scope.

62. The test piles should be the same type and embedment anticipated for the job piles and installed with the same equipment and techniques proposed for the job piles. The test piles can be used to evaluate installation methods. Driven test piles will provide more definitive information regarding the anticipated driving resistance and vibrations from pile driving.

63. We recommend a minimum of four probe piles be installed across the building to delineate installation requirements and confirm the minimum embedments required. These probe piles should be installed across the building foundation equally spaced across the building footprint. Additional probe piles may be required depending on the results of these tests or the number and types of piles selected for the project. In general, the probe pile showing the least resistance for the greatest embedment should be selected for performance of the static load test. The selection of the probe pile to test should be based on a consensus between the structural and geotechnical engineers.

64. The test piles should be allowed to set for at least 28 days subsequent to the installation of the reaction system. The test piles should then be load tested to failure in accordance with ASTM D 1143. The results should be evaluated by Eustis Engineering to verify the estimated pile load capacities presented in this report. As an alternate, open end steel pipe piles may be evaluated using dynamic pile testing methods to evaluate capacity.

65. Dynamic Pile Testing. The initial installation of open end pipe piles should be considered for monitoring with a Pile Driving Analyzer®. A PDA can monitor driving stresses during installation and evaluate pile integrity during or after installation. A PDA can also monitor energy transferred to the pile by the hammer to evaluate pile installation efficiency. In order to evaluate pile capacity, a "restrike" DPT should be performed a minimum of 28 days after its initial installation. Shorter restrike set times may be considered, but a test may not indicate the full ultimate capacity. In any case, we do not recommend a restrike set time less than 14 days to evaluate capacity. Data from this restrike should be further evaluated by CAPWAP® computer analyses. Eustis Engineering is available to perform and evaluate the results of DPT and CAPWAP analyses.

Pavement Analysis

66. Method of Analysis. The pavement components and thicknesses were determined using methods presented in the AASHTO Guide for Design of Pavement Structures. In addition, the resilient soil modulus (M_r) of the subgrade was estimated based on the type of soil, probable drainage conditions, and engineering experience.

67. Traffic. Furnished information indicates that approximately 206 parking spaces will be provided in the new parking lots. No traffic volumes were furnished. Therefore, Eustis Engineering has made assumptions necessary to provide rigid and flexible pavement recommendations. For the parking areas, we have assumed that they will experience approximately 412 cars and 412 light duty trucks (i.e., pickup trucks, vans, and sport utility vehicles) per day. We have assumed that the driveways and

serviceways will experience twice the traffic of the parking areas per day, a small delivery truck per day, and three garbage trucks per week. We have assumed that the driveway to the firehouse will experience approximately 10 pickup trucks per day, three garbage trucks per week, and four fire trucks per day. The axle loads assumed in our analyses are tabulated below. ***These traffic assumptions should be verified prior to implementation of our recommendations.*** If traffic conditions are significantly different than those presented, Eustis Engineering should be contacted to reevaluate the pavement recommendations.

TYPE OF VEHICLE	SINGLE AXLE LOAD IN KIPS	
	FRONT	REAR
Passenger Cars	2	2
Pickup Trucks, Vans, or Sport Utility Vehicles	2	5
Delivery Truck	12	20
Garbage Truck	24	30
Fire Truck	18	30

68. Our analyses assume the site is prepared in accordance with the recommendations provided in this report. Our analyses assume all paving materials will conform to the MSSRBC. These traffic data assumptions were converted to equivalent 18-kip single axle loads (E_{18}) using AASHTO equivalency factors for flexible and rigid pavements. A 20-year design life and a terminal serviceability index (P_t) of 2.0 were used for the analyses of rigid and flexible pavements.

69. <u>Rigid Pavement.</u> Based on our analyses, we recommend the parking areas be comprised of 5 inches of Portland Cement Concrete. Driveways, serviceways, and fire truck areas should be comprised of 8 inches of Portland Cement Concrete.

70. Portland Cement Concrete should conform to the material requirements for pavement concrete as specified in Section 501 of the MSSRBC. The concrete should have a specified 28-day compressive strength of 4,000 psi to give the pavement adequate flexural strength. The concrete pavement should also be wire mesh reinforced against temperature and shrinkage, and should be constructed in

accordance with the provisions of the MSSRBC, Section 501. The concrete should be underlain by at least 8 inches of compacted sand fill. The sand fill should conform to the material requirements given in Section 703.21 of the MSSRBC for Class B3 borrow excavation material. The sand subbase should be compacted to 95% of its maximum dry density near optimum water content using ASTM D 1557. Placement and compaction of the base materials should also conform with Section 304 of the MSSRBC.

71. Grades should provide for adequate drainage to prevent saturation of sand fill beneath the pavement. All joints should be sealed to prevent infiltration of water. All pavement details, such as wire mesh, reinforcement, dowels, joints, curbs, etc., should be designed by a pavement design engineer.

72. Flexible Pavement. In the parking areas, the pavement section should have an overall thickness of 17 inches. This pavement section should consist of 8 inches of sand subbase, 6 inches of stone base course, 1.5 inches of asphaltic binder course, and 1.5 inches of asphaltic wearing course. Driveways, serviceways, and fire truck areas should have an overall thickness of 24 inches. This pavement section should consist of 12 inches of sand subbase, 8 inches of stone base course, 1.5 inches of asphaltic binder course, and 2.5 inches of asphaltic wearing course.

73. The asphaltic binder and wearing courses should have a minimum Marshall Stability of 1,500 pounds and conform with Section 401 of the MSSRBC. The material for the crushed stone base course should conform to the requirements of Section 703.07 of the MSSRBC for Class I or 2, Group A or B coarse aggregates. This material should be obtained from an approved source in accordance with Section 106 of the MSSRBC. Eustis Engineering should be contacted to evaluate alternate proposed gradations available for the project. Sand subbase should follow the recommendations given in Section 703.21 of the MSSRBC for Class B3 borrow excavation material. Structural fill used as subbase should be compacted to 95% of its maximum dry density in accordance with ASTM D 1557. Placement and

compaction of the base and subbase materials should also conform with Section 304 of the MSSRBC.

74. Grades should provide for adequate drainage to prevent saturation of the subgrade and base course materials. If the type and thickness of pavement components are changed, Eustis Engineering should be consulted to determine the suitability of these materials and the structural number of the pavement.

<u>Helipad Slab</u>

75. Our analyses assume that maximum loading of the helipad will be induced by a Chinook CH47-234 helicopter. Furnished information indicates that this helicopter has a gross vehicle weight of 50,000 pounds. Approximately 58% of the gross helicopter weight is transmitted by the four front wheels and the remaining 42% is transmitted by two rear wheels.

76. Based on our analyses, we recommend the helipad be comprised of 8 inches of Portland Cement Concrete. Our analyses assume a 20-year design life and a flexural strength of the concrete of 700 psi. Portland Cement Concrete should conform to the material requirements for pavement concrete as specified in Section 501 of the MSSRBC. The concrete pavement should also be wire mesh reinforced against temperature and shrinkage, and should be constructed in accordance with the provisions of the MSSRBC, Section 501. The concrete should be underlain by at least 12 inches of compacted sand fill. The sand fill should conform to the material requirements given in Section 703.21 of the MSSRBC for Class B3 borrow excavation material. The sand subbase should be compacted to 95% of its maximum dry density near optimum water content using ASTM D 1557. Placement and compaction of the base materials should also conform with Section 304 of the MSSRBC.

77. Grades should provide for adequate drainage to prevent saturation of sand fill beneath the pavement. All joints should be sealed to prevent infiltration of water. All pavement details, such as wire mesh, reinforcement, dowels, joints, curbs, markings, lighting, etc., should be designed by a pavement design engineer.

78. Estimated Settlement for Slabs. Analyses were made to determine the estimated settlement near the center of a 100' x 100' floor slab for the proposed helipad. Based on a uniform dead load pressure intensity of 100 psf from an 8-in. concrete slab, we estimate settlement near the center of a slab will be approximately ½ to ¾ inch. Actual settlements may vary ±20% from the values indicated due to variations in subsoil conditions. Settlement at the corners and midpoint of the sides is estimated to be one-quarter and one-half of these values, respectively.

79. The settlement estimates assume the loading intensity is applied by a flexible foundation. As the rigidity of the slab increases, settlement will be more uniform with less settlement at the center and greater settlement at the sides and corners. Uniform settlement of a rigid slab may be estimated as 85% of the center settlement of a flexible slab. Settlement of the slab should be considered additive to that estimated for the fill materials beneath the slab as presented previously in this report.

Vibrations

80. Pile driving, as well as other construction activities, have the potential to generate vibrations that may affect nearby structures, pavements, and underground utilities. Eustis Engineering recommends vibrations be monitored during the test pile program and during subsequent construction activities of concern. This monitoring should evaluate peak particle velocities during pile driving at critical structures with a seismograph, as well as other construction activities generating vibrations (hauling of fill, moving heavy equipment, etc.). The record of peak particle velocities will provide information in assessing potential damage and the need for changes in construction operations.

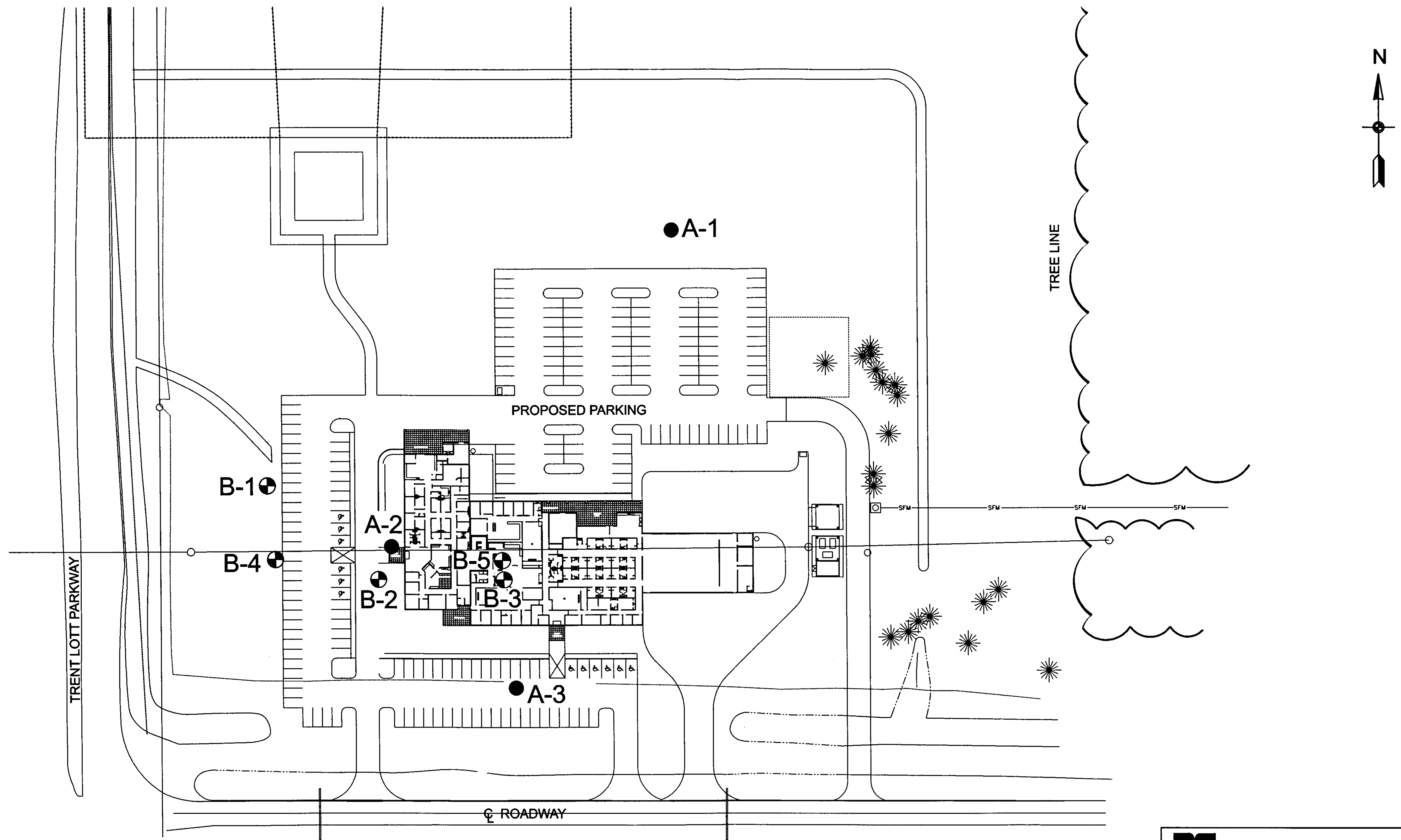
81. Peak particle velocities (measured at a structure) exceeding 0.5 in./sec may induce damage to the structure, particularly when this structure has been previously stressed by settlement or other movements. Peak particle velocities between 0.25 and 0.5 in./sec may be sensed as being detrimental by human perception.

Furthermore, peak particle velocities of 0.25 in./sec may densify loose surficial fill materials such as those encountered at the site. If sustained vibration levels of 0.25 in./sec are measured at a structure, pavement, or utility of concern, Eustis Engineering should be notified and the construction operations generating these vibrations should be terminated and consideration given to altering these procedures.

ADDITIONAL GEOTECHNICAL SERVICES

82. To provide continuity between the investigation, design, and construction phases, Eustis Engineering should be retained to provide additional services during completion of the project. These services may include consultation during design and construction, reviewing geotechnical aspects of plans and specifications, providing inspection of excavations, reviewing site drainage plans and construction sequences proposed by the contractor, testing and approval of proposed fill and pavement materials, logging the installation of test piles and job piles, performing load tests and evaluating their results, asphalt and concrete testing and inspection, steel inspection, and any other soil and materials testing services. Eustis Engineering offers a complete range of materials testing services which will provide quality control during construction and conformance to design specifications. Eustis Engineering can also perform DPT during installation and evaluate PDA data with respect to driving stresses, load capacity, and pile integrity.

83. In summary, Eustis Engineering should be retained to monitor all geotechnical related work performed by the contractor. If construction problems arise, Eustis Engineering should be notified to participate in the development of solutions. This participation permits the geotechnical engineer to evaluate the effects of unanticipated conditions and propose solutions on the geotechnical design assumptions particular to the project. The design geotechnical engineer may also be able to judge how site specific soil and ground water conditions will affect the success of a proposed construction alternative.



DENOTES UNDISTURBED BORINGS

DENOTES AUGER BORINGS

BORINGS DRILLED 25 JULY AND 8 AND 11 AUGUST 2003

100' 0 100'

GRAPHIC SCALE

EUSTIS ENGINEERING COMPANY, INC.
GEOTECHNICAL ENGINEERS
3011 28TH STREET METAIRIE, LOUISIANA

LOCATION OF BORINGS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER
HANCOCK COUNTY, MISSISSIPPI

DRAWN BY: J. SMITH	PLOT DATE: 26 AUG. 03	CADD FILE: FIGURE 1.DGN
CHECKED BY: S.R.S.	JOB NO.: 18080	FIGURE 1

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER
HANCOCK COUNTY, MISSISSIPPI

ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITIES

PILE TYPE	PILE DIAMETER IN INCHES	PILE TIP EMBEDMENT BELOW EXISTING GROUND SURFACE IN FEET	ESTIMATED ALLOWABLE SINGLE PILE LOAD CAPACITY IN TONS[2] FACTOR OF SAFETY ≈ 2	
			COMPRESSION	TENSION
Treated ASTM D 25 Quality Timber	6 (Tip) 8 (Butt)	35-38[1]	15	10
	7 (Tip) 12 (Butt)	35-38[1]	20	14
Treated ASTM D 25 Quality Timber or Timber Composite	7 (Tip) 13 (Butt)	61 - 64[1]	30	22[3]
Open End Steel Pipe Piles	12[4]	35-38[1] 61-64[1]	27 50	15 29
Open End Steel Pipe Piles	14[4]	35-38[1] 61-64[1]	33 60	18 33
Open End Steel Pipe Piles	16[4]	35-38[1] 61-64[1]	40 70	20 38

Notes:

[1] Assumes piles are firmly embedded in the dense sand strata to a penetration resistance indicative of the capacity.

[2] Pile load capacities are based solely on resistance present at the soil to pile interface. Applicable load capacities based on structural capacity or code limitations should be considered by the structural engineer.

[3] Timber composite piles should not be used to resist tensile loadings.

[4] Pile diameter refers to the outside diameter of the pipe.

CAPACITY OF PILE GROUPS

The maximum allowable load carrying capacity of a pile group is no greater than the sum of the single pile load capacities, but may be limited to a lower value if so indicated by the result of the following formula.

$$Q_a = \frac{P \ x \ L \ x \ c}{(FSF)} + \frac{2.6 \ q_u \ (1 + 0.2 \ \frac{w}{b}) \ A}{(FSB)}$$

In Which:

Q_a	=	Allowable load carrying capacity of pile group, lb
P	=	Perimeter distance of pile group, ft
L	=	Length of pile, ft
c	=	Average (weighted) cohesion or shear strength of material between surface and depth of pile tip, psf
q_u	=	Average unconfined compressive strength of material in the zone immediately below pile tips, psf (unconfined compressive strength = cohesion x 2)
w	=	Width of base of pile group, ft
b	=	Length of base of pile group, ft
A	=	Base area of pile group, sq ft
(FSF)	=	Factor of safety for the friction area = 2
(FSB)	=	Factor of safety for the base area = 3

The values of c and q_u used in this formula should be based on applicable soil data shown on the Log of Boring and Test Results for this report. In the application of this formula, the weight of the piles, pile caps and mats, considering the effect of buoyancy, should be included.

SPACING WITHIN PILE GROUPS

$$SPAC = 0.05 \ (L_1) + 0.025 \ (L_2) + 0.0125 \ (L_3)$$

In Which:

SPAC	=	Center to center of piles, feet
L_1	=	Pile penetration up to 100 feet
L_2	=	Pile penetration from 101 to 200 feet
L_3	=	Pile penetration beyond 200 feet

NOTE: Minimum pile spacing = 3 feet or 3 pile diameters, whichever is greater

APPENDIX

LEGEND AND NOTES FOR LOG OF BORING AND TEST RESULTS

PP Pocket penetrometer resistance in tons per square foot

TV Torvane shear strength in tons per square foot

SPT Standard Penetration Test. Number of blows of a 140-lb. hammer dropped 30 inches required to drive 2-in O.D., 1.4-in. I.D. sampler a distance of one foot into the soil, after first seating it 6 inches

SPLR Type of Sampling ■ Shelby ☒ SPT ▱ Auger ☐ No Sample

SYMBOL Clay Silt Sand Humus — Predominant type shown heavy; Modifying type shown light

DENSITY Unit weight in pounds per cubic foot

USC Unified Soil Classification

TYPE	UC	Unconfined compression shear
	OB	Unconsolidated undrained triaxial compression shear on one specimen confined at the approximate overburden pressure
	UU	Unconsolidated undrained triaxial compression shear
	CU	Consolidated undrained triaxial compression shear
	DS	Direct shear
	CON	Consolidation
	PD	Particle size distribution
	k	Coefficient of permeability in centimeters per second
	SP	Swelling pressure in pounds per square foot

ϕ Angle of internal friction in degrees

c Cohesion in pounds per square foot

Other laboratory test results reported on separate figure

Ground Water Measurements ▼ Initial ▽ Final

GENERAL NOTES

(1) At the time the borings were made, ground water levels were measured below existing ground surface. These observations are shown on the boring logs. However, ground water levels may vary due to seasonal and other factors. If important to construction, the depth to ground water should be determined by those persons responsible for construction, immediately prior to beginning work.

(2) While the individual logs of borings are considered to be representative of subsurface conditions at their respective locations on the dates shown, it is not warranted that they are representative of subsurface conditions at other locations and times.

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 7/25/03 **Boring:** 1 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Soft dark gray silty clay w/roots	CL	1	0-0.5	31										
	0.50				Medium stiff brown sandy clay w/roots	CL	2	2-3	22	101	123	UC	--	895	32	15	17		
	3.75				Very stiff light gray & tan sandy clay w/fine sand lenses	CL	3	5-6	18	109	128	UC	--	2985					
					Stiff light gray & tan sandy clay w/fine sand layers	CL	4	8-9	19	107	127	UC	--	1640					
10					Loose white fine sand	SP	5	11-12											PD
					Loose gray fine sand w/clay lenses	SP	6	14-15	22										
					Medium stiff gray clay w/wood	CH													
20	0.75						7	19-20											
30																			
40																			
50																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 7/25/03 **Boring:** 2 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Medium stiff gray & tan sandy clay w/roots	CL	1	0-0.5	25										
	0.50				Stiff dark gray & tan sandy clay w/roots	CL	2	2-3	21	102	123	UC	--	1015					
	1.00				Very stiff light gray & tan sandy clay w/trace of fine sand	CL	3	5-6	22	102	125	UC	--	2555					
					Stiff light gray & tan sandy clay w/fine sand layers	CL	4	8-9	21	104	125	UC	--	1155					
10		3			Very loose gray clayey sand	SC	5	9-10.5											PD
		5			Loose brown fine sand	SP	6	12-13.5											
					Soft gray clay	CH	7	14-15	77										
20							8	19-20											
30																			
40																			
50																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 7/25/03 **Boring:** 3 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Medium stiff brown & gray sandy clay w/roots	CL	1	0-0.5	30										
	1.00					CL	2	2-3	20	104	125	UC	--	1020					
					Stiff brown & tan sandy clay w/trace of fine sand	CL													
	3.20				Stiff light gray & tan sandy clay		3	5-6	21	103	125	UC	--	1635					
							4	8-9	16	109	126	UC	--	1275					
10					Very soft brown & tan sandy clay	CL	5	9-12	26										
					Loose white fine sand	SP	6	12-15	26										
		3			Very loose white fine sand	SP	7	15-16.5											
		4			Soft light gray clay w/organic matter, fine sand lenses & pockets, & wood	CH	8	18.5-20	31										
20																			
30																			
40																			
50																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 8/08/03 **Boring:** 4 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	S P L R	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Very soft dark gray silty clay w/organic matter	CL	1	0-0.5											
	1.50				Stiff gray & tan sandy clay w/trace of fine sand	CL	2	2-3	24	102	126	UC	--	1120					
	4.25				Very stiff tan & light gray sandy clay w/clayey sand layers	CL	3	5-6											
	4.25						4	8-9	15	112	129	UC	--	1235					
10		27			Medium dense white & light gray fine sand	SP	5	9-9.5											
							6	9.5-11											
		8			Stiff gray clay w/sand pockets & concretions	CH	7	12.5-14	55										
					Medium stiff gray clay w/wood & organic matter	CH													
20	0.85						8	19.0-20	86	49	91	UC	--	620					
					Medium stiff light gray sandy clay w/roots	CL													
	1.00						9	24-25	21	106	129	UC	--	635					
					Very soft light gray sandy clay w/roots	CL													
30							10	29-30	22	103	125	UC	--	200					
					Medium dense light gray fine sand w/wood	SP	11	31-32.5	35										
		25					12	32.5-34											
					Very dense light gray fine sand w/clayey sand layers	SP													
		50=6"					13	35.5-37											
		50=3"			Very dense light gray fine sand		14	38.5-40											
40					Stiff gray clay w/fine sand lenses	CH													
		10					15	41.5-43											
	0.75				Medium stiff gray clay w/fine sand pockets	CH	16	44-45	48	73	107	UC	--	545					
50	0.50						17	49-50	43										

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 8/08/03 **Boring:** 4 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50					Medium stiff gray clay w/fine sand pockets	CH													
					Stiff light gray clay w/wood	CH													
	1.25						18	54-55	35	86	116	UC	--	1040					
					Stiff light gray sandy clay	CL													
60	1.70						19	59-60	20	106	127	UC	--	1405					
					Very dense gray fine sand	SP	20	61-62.5											
		50 = Seat					21	62.5-64											
		50 = Seat			w/trace of medium dense sand		22	65.5-67											
70		50 = Seat					23	68.5-70											
					Very dense gray medium sandy clay	SP													
		50 = Seat					24	73.5-75											
80																			
90																			
100																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 8/11/03 **Boring:** 5 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Medium compact dark gray clayey silt w/silty sand lenses	ML	1	0-0.5											
	1.25					CL	2	2-3	20	102	123	UC	--	905					
					Medium stiff dark gray sandy clay w/silty sand lenses	CL													
	4.00				Stiff to very stiff tan and gray sandy clay		3	5-6	27	95	121	UC	--	1840					
					Medium dense light gray fine sand	SP	4	6-7.5											
		17					5	8.5-10											
10																			
		23					6	11.5-13											
	0.10				Very soft gray clay w/fine sand pockets & organic clay layers	CH	7	14-15											
					Soft dark gray clay w/organic matter & wood	CH													
20	0.50						8	19-20	92	47	90	UC	--	325					
					Medium stiff dark gray clay w/wood & organic matter	CH													
							9	24-25	104										
					Medium stiff light gray sandy clay	CL													
30							10	29-30	22	103	125	UC	--	740					
					Loose light gray fine sand w/wood & clay pockets	SP													
		Dense					11	34-35	20										
		45			Dark gray fine sand	SP	12	35.5-37											
		32					13	38.5-40											
40					Very dense gray fine sand	SP													
		50					14	41.5-43											
					Medium stiff gray clay w/fine sand layers	CH	15	44-45	39										
50	0.50				w/trace of wood		16	49-50	49	72	108	UC	--	540					

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 8/11/03 **Boring:** 5 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
50			Medium stiff gray clay w/trace of wood	CH													
			Medium stiff light gray clay w/wood & fine sand lenses	CH													
	0.75				17	54-55	27	94	120	UC	--	810					
			Medium stiff light gray clay w/fine sand pockets	CH													
60	0.75				18	59-60	44	75	109	UC	--	705					
		50=6"	Very dense light gray fine sand	SP	19	60.5-62											
		50=Seat			20	63.5-65											
			Very dense gray medium to fine sand	SW													
		50=Seat			21	66.5-68											
70		50=Seat			22	69.5-71											
			Very dense brown medium to fine sand w/gravel	SW													
		50=Seat			23	73.5-75											
80																	
90																	
100																	

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 7/25/03 **Boring:** A-1 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Soft light gray & tan sandy clay	CL	1	0-1	20										
					Soft tan & light gray clay w/silty sand lenses	CH	2	1-2											
							3	2-3	32										
					Stiff tan & light gray clay w/silty sand lenses & pockets	CH	4	3-4											
							5	4-5	31										
10																			
20																			
30																			
40																			
50																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** 7/25/03 **Boring:** A-2 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density		Shear Tests			Atterberg Limits				Other Tests
										Dry	Wet	Type	ø	C	LL	PL	PI		
0					Very soft black clay w/organic matter	CH	1	0-1											
					Very soft gray, brown, & tan sandy clay	CL	2	1-2	21						23	13	10		
					Medium stiff tan sandy clay	CL	3	2-3											
					Stiff light gray, tan, & reddish-brown sandy clay	CL	4	3-4	23										
							5	4-5											
10																			
20																			
30																			
40																			
50																			

Comments:

EUSTIS ENGINEERING COMPANY, INC.

LOG OF BORING AND TEST RESULTS

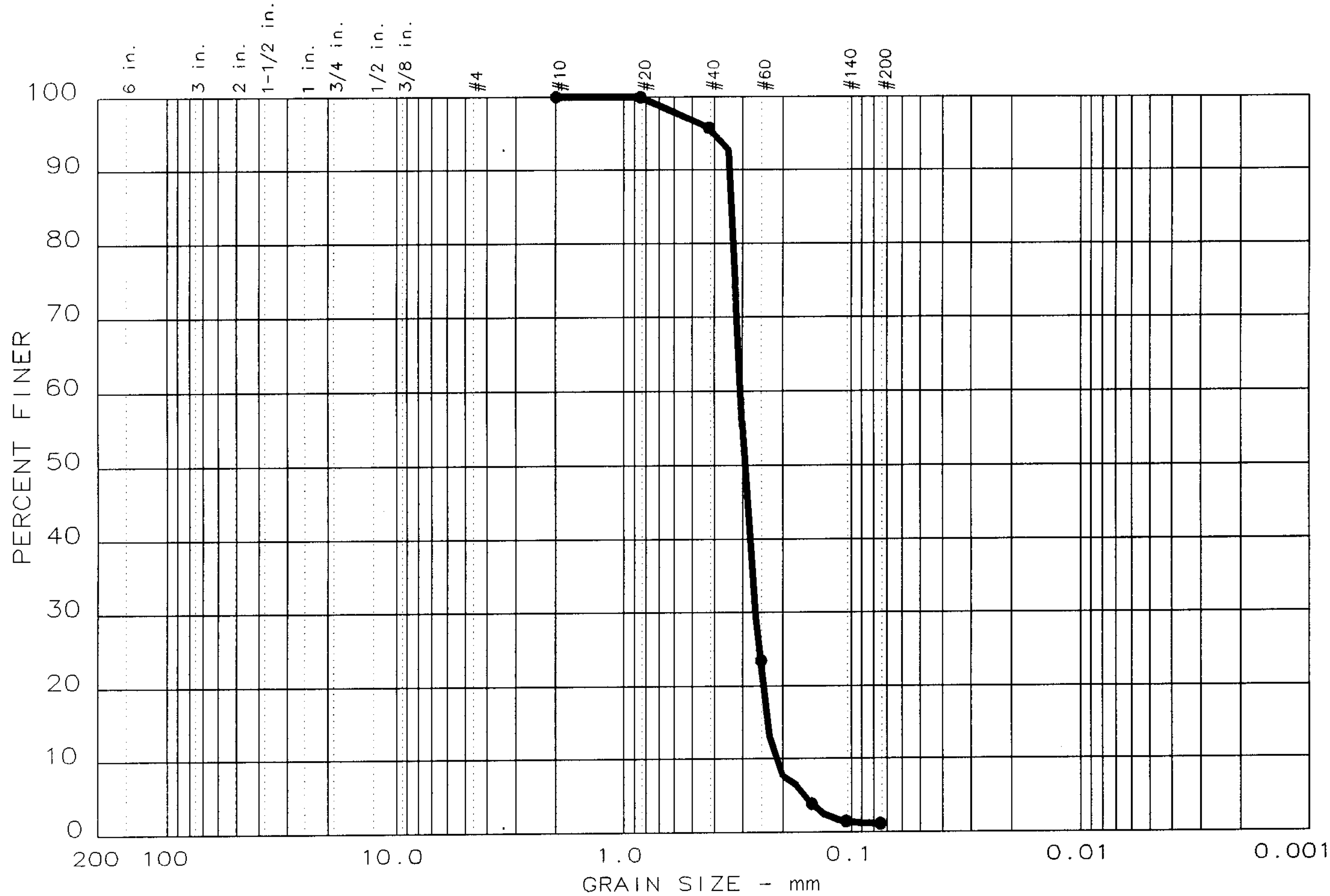
JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER, MISSISSIPPI
TASK ORDER NO. 48589

Ground Elev.: **Datum:** **Gr. Water Depth:** See Text **Job No.:** 18080 **Date Drilled:** See Text **Boring:** A-3 **Refer to "Legends & Notes"**

Scale In Feet	PP	SPT	SPLR	Symbol	Visual Classification	USC	Sample Number	Depth In Feet	Water Content Percent	Density Dry	Density Wet	Shear Tests Type	Shear Tests ø	Shear Tests C	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI		Other Tests
0					Soft tan & light gray sandy clay w/roots	CL	1	0-1	23										
							2	1-2											
					Soft dark gray sandy clay	CL	3	2-3	18										
					Medium stiff gray & tan clay	CH	4	3-4											
					Stiff tan & dark gray clay w/silt lenses & pockets	CH	5	4-5	30										
10																			
20																			
30																			
40																			
50																			

Comments:

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	98.9	1.1		SP		

SIEVE inches size	PERCENT FINER ●		
	GRAIN SIZE		
D_{60}	0.31		
D_{30}	0.26		
D_{10}	0.21		
	COEFFICIENTS		
C_c	1.04		
C_u	1.4		

SIEVE number size	PERCENT FINER ●		
10	100.0		
20	99.9		
40	95.8		
60	23.5		
100	3.8		
140	1.4		
200	1.1		

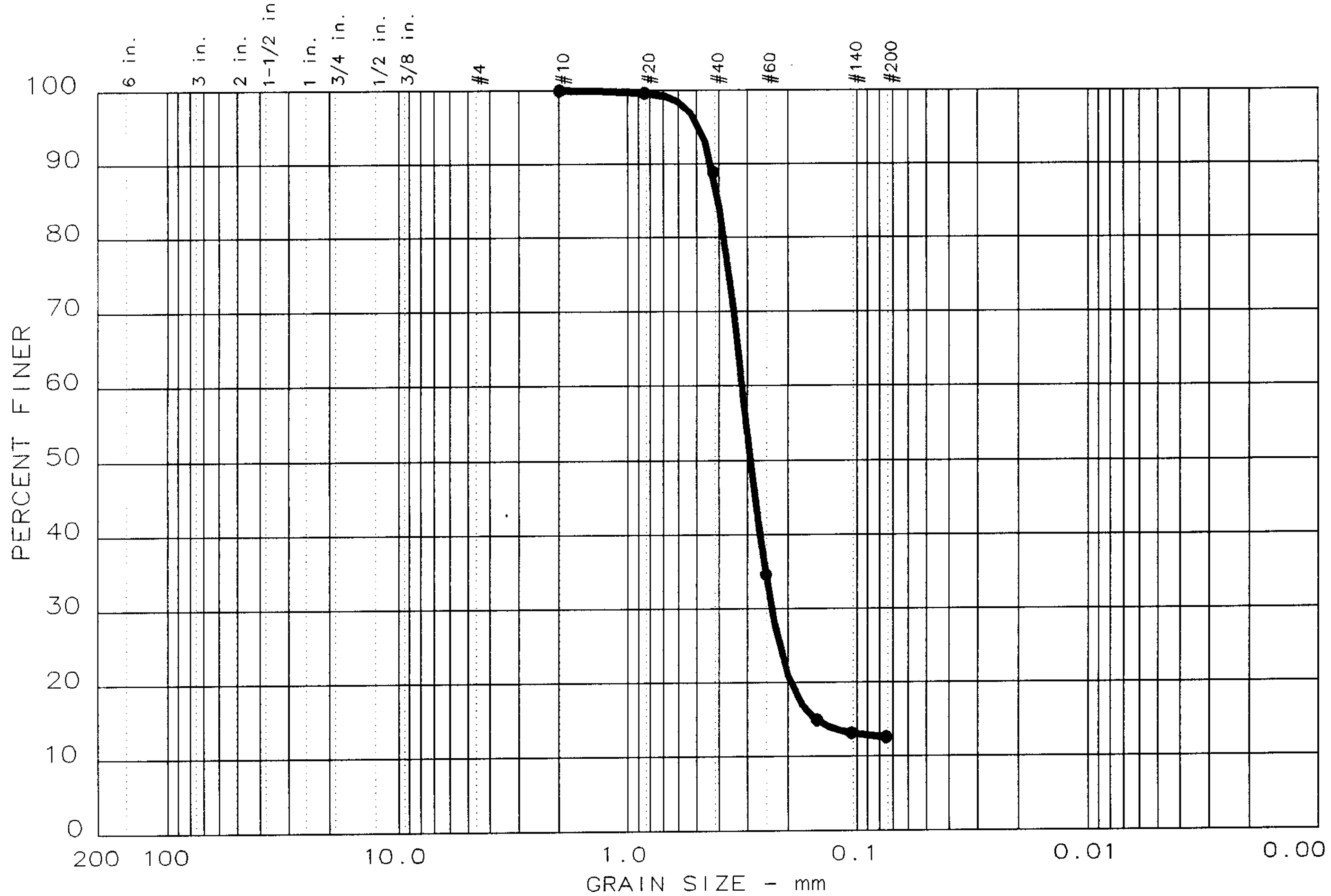
Sample information:
● Boring 1, Sample 5
Loose white FINE SAND

Remarks:
Sample depth 11'-12'

Eustis Engineering Company, Inc.

Project No.: 18080
Project: John C. Stennis Space Center
Date: 7-31-03
Data Sheet No. ——

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.0	87.5	12.5		SC		

SIEVE inches size	PERCENT FINER ●		
	GRAIN SIZE		
D_{60}	0.32		
D_{30}	0.24		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ●		
10	100.0		
20	99.6		
40	88.9		
60	34.6		
100	14.9		
140	13.1		
200	12.5		

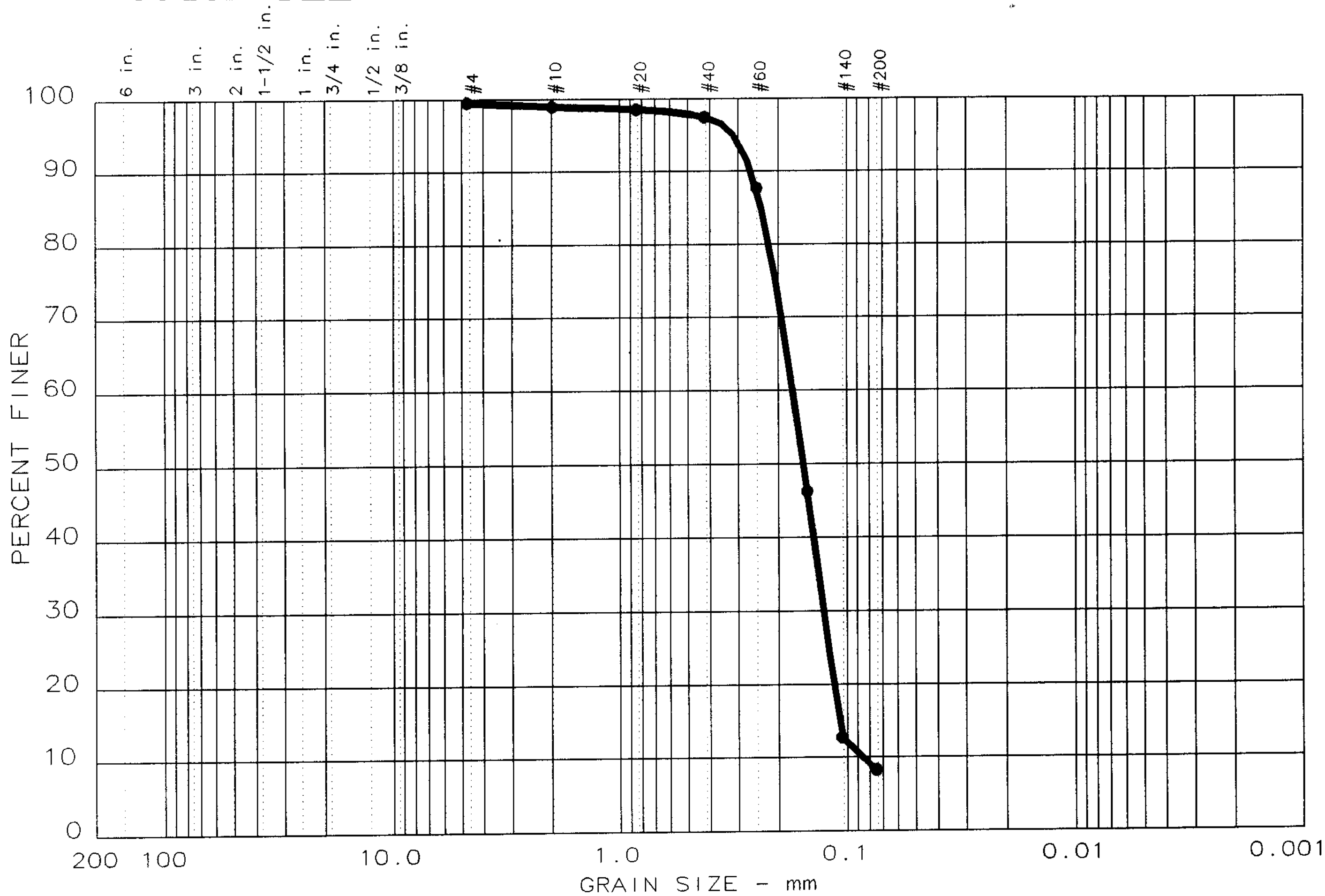
Sample information:
● Boring 2, Sample 5
Very Loose gray CLAYEY SAND

Remarks:
Sample depth 9'-10.5'

Eustis Engineering Company, Inc.

Project No.: 18080
Project: John C. Stennis Space Center
Date: 7-31-03
Data Sheet No. ____

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	0.5	91.2	8.3		SP-SM		

SIEVE inches size	PERCENT FINER ●		

	GRAIN SIZE		
D_{60}	0.17		
D_{30}	0.13		
D_{10}	0.08		

	COEFFICIENTS		
C_c	1.09		
C_u	2.0		

SIEVE number size	PERCENT FINER ●		
4	99.5		
10	98.9		
20	98.5		
40	97.4		
60	87.7		
100	46.2		
140	12.7		
200	8.3		

Sample information:

● Boring 6, Sample 12
Dense gray FINE SAND w/ silt

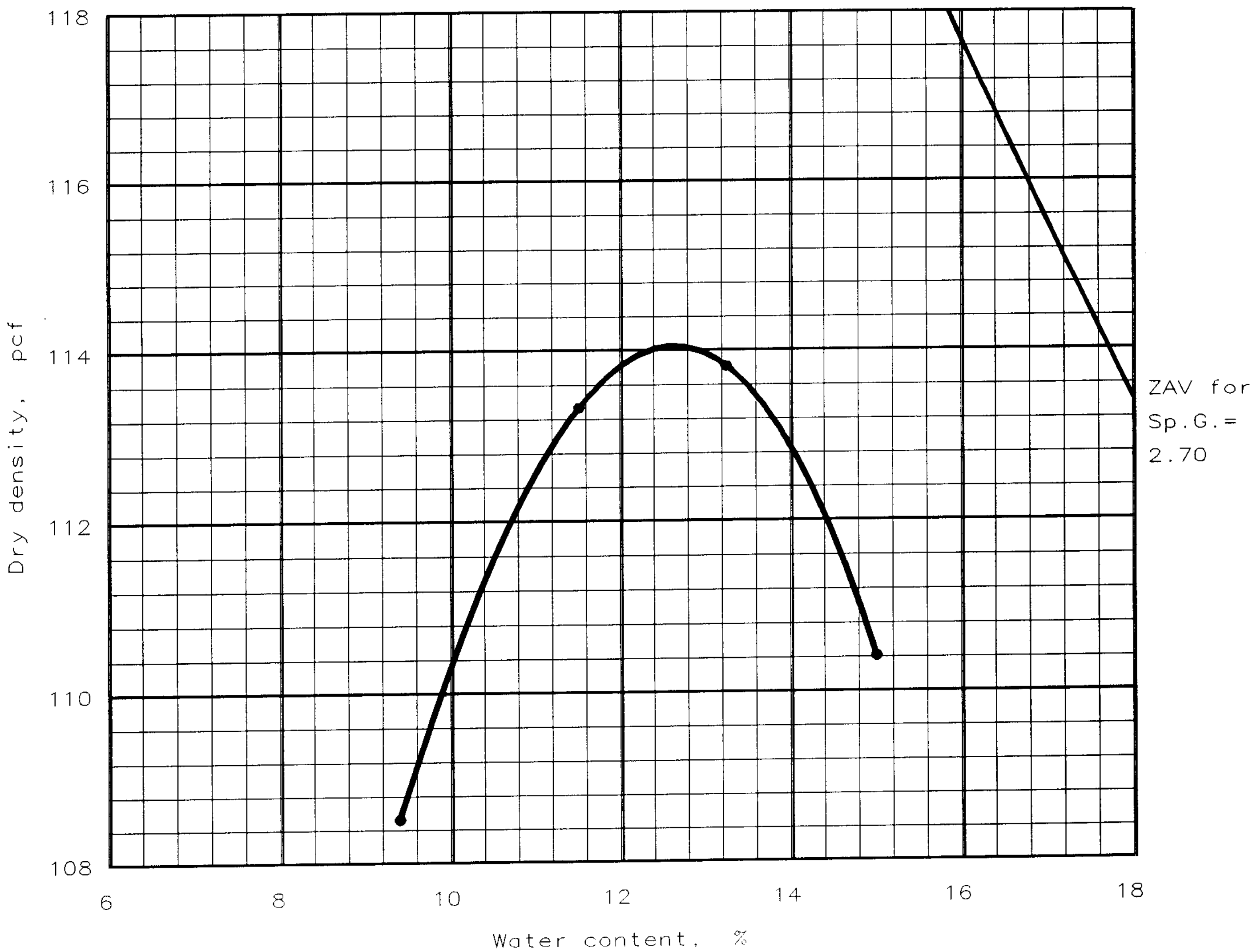
Remarks:
Sample depth 35.5'-37.0'

Eustis Engineering Company, Inc.

Project No.: 18080
Project: Stennis Space Center

Date: 8-15-03 Data Sheet No. ____

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: AASHTO T 99 Method A, Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	CL			2.70				

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 114.1 pcf Optimum moisture = 12.6 %	Tan & Gray SILTY CLAY

Project No.: 18080

Project: Stennis Space Center

Location: Hancock County, Mississippi

Date: 8-5-03

Remarks:

Composite Sample from

Auger Borings 1,2,3

MOISTURE-DENSITY RELATIONSHIP TEST

EUSTIS ENGINEERING COMPANY, INC.

ENC. NO.: ______

JOHN C. STENNIS SPACE CENTER
NASA FIRST RESPONSE FACILITY
STENNIS SPACE CENTER
HANCOCK COUNTY, MISSISSIPPI

ESTIMATED CALIFORNIA BEARING RATIO (CBR)
(ESTIMATED AT DRY DENSITY OF 110.9 PCF)

PENETRATION DEPTH IN INCHES	CBR
0.1	4.0
0.2	3.7