REPORT ON SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING ANALYSIS

FOR THE

PROPOSED BOX CULVERT UNIVERSITY DRIVE, DURHAM, NC

SEPTEMBER, 2011

A1 Project No # 10-070

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September 07, 2011

Kenneth E. Trefzger, PE, CFM
HDR, Inc.
3733 National Drive, Suite 207
Raleigh, NC 27612-4845

Re: Revised Geotechnical Report of the Proposed Box Culvert at the University Drive, for the City of Durham, NC
(A1 Project No: 10- 070)

Dear Mr. Trefzger,

We are pleased to submit to you our revised report on the subsurface exploration and geotechnical recommendation for the proposed Box Culvert at the University Drive, for the City of Durham, NC

The purpose of the exploration was to evaluate the general subsurface conditions within the proposed box culvert and pavement areas with regard to the design and construction of the culvert and pavement systems. This report presents our findings, conclusions and recommendations for foundation design, as well as construction considerations for the proposed foundations and paved areas.

A1 appreciates the opportunity to assist you during this phase of the project. If you should have any questions concerning this report, or if we may be of further assistance, please contact us.

We look forward to our continued relationship.

Respectfully submitted,
A1 Consulting Group, Inc.

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PROJECT UNDERSTANDING

We understand that A1 Consulting Group, Inc was hired by the HDR Engineering Company to perform the following services on the proposed relocation of Box Culvert at the University Drive for the City of Durham, NC:

1. Evaluate the subsurface soil and groundwater information based on drilling work performed by us on site.
2. Assess Site Geology
3. Earthwork recommendations for site grading
4. Review construction procedures for site work
5. Recommendations on earth slopes and their stability.
6. Recommendations on seismic classification and active forces.
7. Evaluation of soil bearing pressure
8. Recommendations on footings for the box culvert.
9. Recommendations on concrete rigid pavement design above the box culvert.
10. Recommendations on retaining walls for the box culvert.
11. Recommendations on the temporary shoring for the box culvert.
12. Recommendations on structural fill materials.
13. Quality control measures for construction.

PROJECT DESIGN SUMMARY:

The proposed project consists of the site investigation for the relocation of a concrete box culvert at the University Drive for the City of Durham, NC. The purpose of this geotechnical exploration was to determine the subsurface conditions as an aid to the structural recommendations for a 350-linear feet long double 5'x5' reinforced concrete box culvert.

For this purpose, six (6) borings were drilled at locations determined in consultation with the client. The borings were all advanced to predetermined depths ranging from 20 to 25 feet. Rock coring was conducted in select borings. Groundwater was measurable at the time of drilling; however seasonal variations in the groundwater levels can be expected.

The following parameters may be used for the design of foundations:

Expected Loading	
Vehicles	8,000 vehicles per day on University Drive
Horizontal	Not provided
Foundation Type	Spread Footings / Thickened slab
Foundation Base	natural excavated ground*, engineered fill soils and #57 stone. Undercut and replace unsuitable soils.*
Average bearing pressure	1,500 PSF (to be field verified during construction)
Construction De-watering	may be necessary
Water level considered for design	varies
Minimum Foundation width:	84 inch (bottom of the box culvert)
Minimum depth of footings:	72 inch (below the concrete pavement)
Settlement, as per design parameters: **	
Maximum	¾ inch
Differential	¾ inch
Pavement	6" ABC base 2.5 /3.5 inch of asphalt pavement in parking lot
Handling of existing utilities	Will be required
Rock excavation	May be required
Floodplain & wetlands	May be applicable

***Note: structural foundation bases must be field verified by a professional Geotechnical Engineer for adequate bearing capacity prior to pouring concrete.**

****Note: settlements are estimated based upon design parameters and observed subsurface conditions. A professional Geotechnical Engineer should verify subsurface conditions during excavation in order to determine if undercutting is required.**

Assumptions and Exclusions/Limitations:

- Client supplied information is accurate
- Information in this report is to be used for planning and initial design purposes only
- Borehole observations are assumed to approximate actual subsurface conditions
- Information in this report is not intended to be used for bid purposes

Project Information

1.01 Project Overview:

This report presents the results of the geotechnical exploration conducted for the investigation of the proposed culvert relocation in the parking lot of Nana's Restaurant at 2514 University Drive, Durham, NC and then it routes under the University Drive to go across the street into the creek in woods. A new twin-box culvert is proposed to replace this culvert as shown in the Appendix. Geotechnical borings were explored within the parking lot of the restaurant as well as one on each side of the University Drive. .

1.02 Background & Site:

The site is located adjacent to the North-West corner of the intersection of University Drive and Woodridge Drive in Durham, North Carolina. The current culvert is a 72-inch corrugated metal pipe (CMP) that is in questionable condition and extends underneath the Nana's Restaurant and several other businesses. The building has flooded a few times due to overflow from the existing culvert. The City of Durham is managing the design and construction of a culvert rerouting project. The project is administered through the private drainage assistance program. The original plan was to use precast double 5'x5' box culverts. NCDOT will not allow the precast be used at the bends or at changes in vertical grade and therefore the Designer (Client) intends to design the culvert as cast-in-place structure at a minimum at the bends or through the entire system. Furthermore, in-place concrete construction may be necessary in order to accommodate struts or horizontal bracing for temporary sheeting

The length of the proposed culvert is approximately 350 linear feet. Total acreage for site under disturbance is expected to be less than an acre and is located on University Drive in Durham, NC. See the Appendix for site map and photographs. The majority of the site is currently in a concrete parking lot of Nana's Restaurant, which is currently a fully functional facility. All construction work will have to be phased to limit disruption to businesses in the area.

1.03 Loading Information:

We have assumed the following loading on the culvert (to be verified by the structural engineer):

Parking Lot:

We have estimated that the traffic volumes for the pavements on the site will be approximately 250 automobiles per day with ten-percent delivery trucks and service vehicles. For purpose of this report, we have estimated maximum weight of delivery truck to be 50,000 pounds.

University Drive:

Total vehicular traffic on the University Drive at the site: estimated 8,000 vehicles per day with 10% heavy trucks.

If actual traffic volumes are greater than these assumed and reported volumes, please notify us and we will review our recommendations for applicability to the higher traffic.

1.04 Purpose and Scope of Services:

The purpose of this geotechnical exploration was to determine the subsurface conditions in order to provide recommendations regarding the design of the concrete box culvert. The following activities were conducted during our investigations:

- Advanced a total of six (6) soil test at locations determined by the client. Four of these borings were in the concrete parking lot, one boring was in asphalt pavement at the shoulder of University Drive and one boring was in grass shoulder of the University Drive at the other end of the culvert near the creek.
- Prepared soil boring logs describing the types of soils encountered and other relevant information. Sealed samples were carried to A1 laboratories.
- Performed laboratory tests on selected soils samples (tests included determination of grain size distribution, Atterberg Limits, moisture content, soil resistivity, pH, rock quality determination, and California bearing ratio test).
- Conducted geotechnical engineering evaluation of the available data to provide recommendations regarding construction considerations such as subgrade preparation, excavation, earthwork and groundwater control.
- Geotechnical recommendations on box culvert foundation and walls, and pavements.
- Prepared a report presenting all data, soil boring logs, observations and recommendations.

Boring locations, boring quantities and target depths were all determined by the client. The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around the site.

1.05 Proposed Construction:

We understand that a new concrete box culvert will be built at the site. The box culvert could be a combination of precast and cast-in-place or it could be all cast-in-place.

The anticipated site grading may be minimal. As indicated by the design civil engineer, the top of the proposed box culvert at the inflow may be at elevation 307.0 LF and the top of the box culvert at the outflow may be at elevation 303.0 LF. Proposed invert-in is at 301.24 LF and invert-out is 297.0 LF.

Box culvert will have a reinforced cast-in-place concrete slab on top and then an engineered fill. We recommend asphalt pavement above that to accommodate any potential settlements within the culvert area due to varied soft soils underneath.

Widening and repaving of University Drive are also expected but are not part of our project.

1.06 Investigation Summary:

This phase of the subsurface investigation was conducted on July 20 and 21, 2011. The soil borings were advanced using a truck-mounted drill rig fitted with 3.25-inch inside diameter rotary augers. Split spoon samples (SPT) were taken, four in the top ten feet and one for every five feet thereafter. To obtain the samples, the sampler was placed on the bottom of the borehole and driven to penetrate to a depth of 18 inches (or to a maximum of 50 blows per 6 inches). The first 6 inches of sampler penetration is considered as seating, and the sum of second and third 6-inch increments of penetration (or a fraction thereof) is termed as the Standard Penetration Resistance (N-value). In the case of continuous sampling the penetration distance is 24 inches, and the sum of the third and fourth 6-inch increments are used to determine the N-value.

Boring locations were staked in the field by A1 from the client's boring location site plan. A joint site visit was conducted with the client's project manager to confirm boring locations. All boring locations were adjusted based on utilities interference and client's requests. A boring location diagram indicating the drilled boring locations is presented in the Appendix. All drilling and sampling operations were conducted in general compliance with ASTM D 1586.

A1 staff had contacted the North Carolina One-Call Center (ULOCO) 72 hours in advance and they had marked all the utilities on site.

Six (6) soil borings were advanced at staked locations as shown on the boring location plan. All borings were extended to the auger refusal or as authorized by the client. The location of all borings was as close as possible to the staked locations, subject to utilities interference and drilling convenience.

Field boring logs were prepared incorporating the details such as blow counts, occurrence of ground water, description of soils, rock, etc. Groundwater was measured at the time of drilling. The borings were back-filled using on site auger cuttings. The site was cleaned and restored prior to demobilization.

Standard Penetration Tests were performed at designated intervals in the soil test borings in general accordance with ASTM D 1586 in order to obtain data for estimating soil strength and consistency. In conjunction with the penetration testing, split-spoon soil samples were recovered for soil classification and potential laboratory testing. Water level measurements were attempted at the termination of drilling.

While in the field, a representative of the geotechnical engineer visually examined each sample to evaluate the type of soil encountered, soil plasticity, moisture condition, organic content, presence of lenses and seams, colors and apparent geological origin. The results of the visual soil classifications for the borings, as well as field test results, are presented on the individual "Test Boring Records," included in the Appendix. Similar soils were grouped into strata on the logs. The strata lines represent approximate boundaries between the soil types; however, the actual transition between soil types in the field may be gradual in both the horizontal and vertical directions.

All SPT samples were carried in sealed containers and submitted to the A1 laboratory, where selected samples were checked for moisture, gradation analysis, Atterberg limits, and USGS classification. Additional tests were performed for soil resistivity and pH values. All rock samples were carried in cardboard boxes and checked for RQD and rock integrity. The results of the laboratory tests are included in the Appendix, and are summarized below. Three bulk samples from borings along the present University Drive and parking lot were collected for pavement design purposes.

Four grain size analyses, six natural moisture content analyses, three soil resistivity tests, and three CBRs were performed. Grain size analysis was done on samples representative of the soils encountered. Results are incorporated into the report below.

Beneath the topsoil and concrete pavement, each of the test borings encountered 6-feet to 13-feet of silty SAND and/or sandy SILT Fill over sandy Clay / Clay SAND with Mica. Some of the borings encountered partially weathered rock (PWR) or Rock. Ground water was encountered in several borings (Boring # 4 at depth 8.4' and boring # 5 At depth 12.0'). Some of the soil descriptions include "plastic" silts. These materials exhibited standard penetration resistances of 2 to 4 blows per foot. The provided laboratory test data indicated that the soils had approximately 45% passing the #200 sieve. Liquid limits ranging from 20 to 26, and plasticity indices ranging from non-plastic to 12. Three bulk samples tested using Standard Proctor techniques resulted in maximum dry densities of 117 to 122 pounds per cubic foot (PCF) at optimum moisture contents of 9 to 12 percent. All of the moisture content tests indicated that the natural soil moisture content is above the optimum moisture content for compaction (except in boring B5).

Soil pH and resistivity were measured in three borings to test for corrosive influences on the proposed structure. The soil pH values as measured are generally not in the range to be considered corrosive to concrete. Soil pH values above 6.0 are considered to be of low corrosivity potential. Two of the pH values were slightly more acidic. Values in the range of 5.0 to 6.0 are considered to be moderate risk for corrosiveness to concrete. Since much of the existing overburden appears to be fill the pH values could vary within short distances.

Soil resistivity as measured indicates the soil to be corrosive to steel. Concrete is not as susceptible to corrosion due to resistivity, but any exposed steel may be. If any steel is to be exposed to the soils deeper than ten feet from the surface then some form of corrosion protection (such as an epoxy or bituminous coating) may be justified.

Ground water levels at 24-hours of 8.4 to 12 feet below ground surface (BGS) were noted. Ground water levels will fluctuate depending on seasonal variations of precipitation and other factors.

1.07 Report Overview:

The remainder of this report will discuss the surface and subsurface conditions at the project site (Section 2.0), a discussion of observations and collected data (Section 3.0), and Recommendations (Section 4.0). The Appendix contains site plans, maps, site photographs, tabulated data, boring logs and cross sections illustrating and documenting the information in the report body.

2.0 Site and Subsurface Description

2.01 Site Location and Description:

The site is located adjacent to the N-W corner of the intersection of University Drive and Woodridge Drive at Nana's restaurant, 2514 University Drive, Durham, NC , mainly in a populated urban commercial area of Durham, in Durham County, North Carolina.

The majority of the site is located under an existing concrete parking lot of the restaurant with a small segment running under the University Drive leading into the creek.
The overall topography of the project site can be described as flat to mildly sloping towards the culvert outfall. Total relief between the surface high point (311.0') and the low point (307.0') on the project is approximately 4 feet.

No wetland studies were performed by A1.

We were informed that the project site is not within the FEMA-designated 100-year floodplain.

See Appendix for photographs of site conditions.

2.02 Site Geology and Soils:

The site is located within Durham County, North Carolina and is in the piedmont geologic province. The piedmont is characterized by rolling topography with rounded hills and valleys.

The geological province of the site is the Durham Basin, characterized by sedimentary rocks. The geology underlying the site and vicinity has been mapped as the Chatham Group, consisting of conglomerate, sandstone and mudstone of Triassic age. The parent rock material is characterized by a wide variation in grain sizes, (*Geologic Map of North Carolina, North Carolina Geological Survey, Raleigh, North Carolina, 1985*).

The residual soils at the site are derived from the underlying sedimentary rocks, and are mapped as Cartecay Series (*Kirby, 1971)*. Cartecay soils, found in stream flood plains along smaller streams, are poorly drained and consist of grayish brown to brown silty loam, sandy loam or sandy clay loam. The soils have a moderate permeability and are moderate to strongly acidic. The shrink-swell potential is low. Much of the soil may be poorly suited for foundations, and is fair for use as road fill. *(Kirby, Robert M.., 1970, Soil Survey of Durham County, North Carolina. United States Department of Agriculture, Soil Conservation Service).* With the increased depth, the soil becomes less weathered, coarser grained and the structural character of underlying parent rock becomes more evident. Soils underneath may become partially weathered rock with blow counts of 100 per foot or greater. The bedrock depth is more than 5 feet.

2.03 Subsurface Conditions:

Details of soil conditions encountered in our field exploration program are shown on the individual soil boring logs (B1 through B6) included in the Appendix. The soils present were largely composed of silt, sand, or fine sandy silt. The following subsurface description is of a generalized nature, provided to highlight the major soil strata encountered at the site. Boring logs should be reviewed for specific information as to individual boring locations. The stratification of the soils, as shown on the soil boring logs represents the soil conditions in the actual boring locations; other variations may occur and should be expected between borings. Lines of demarcation represent the approximate boundary between subsurface materials; the transition may be gradual. The general stratification may be found as follows:

- A layer of organic topsoil. In the borings under pavement this layer is absent and replaced with disturbed soil of a gravelly sand/soil mix.
- A stratum consisting of very soft to very stiff dark brown, gray, orange mottled fine sandy silt. This layer could extend to depths of up to 5 feet. Typical SPT N values for this stratum range from 2 to 17.
- A layer of loose to medium dense brown to gray silty or clayey sand. In this stratum there may be lenses of clayey and silty material containing visible very fine mica. This stratum may extend to about 13 feet. Typical SPT N values for this stratum range from 2 to 18.

- A layer of very dense gray/brown/red/black mottled weathered rock sampling as silty sand. Rock structure is clearly delineated and may contain abundant mica. This layer was typically dry throughout, with a typical SPT N value range 100+ to refusal.

If the actual site conditions are found to be different than the data provided to us, we would like to have an opportunity to review the site conditions to make any necessary adjustments to our recommendations contained herein.

2.04 Groundwater:

Groundwater observations were made at the completion of the borings. Measurable groundwater was found in B4 and B5 at depths of 8.4 to 12'. Other borings were immediately backfilled due to their location in a small active parking lot of a popular restaurant. Seasonal groundwater variations may be encountered during construction.

3.0 Structural Discussions

The recommendations made in this report are based on the data obtained during the field investigation program and laboratory testing.

3.01 Seismic Classification and Active Force on Culvert Wall:

Every structure should be evaluated to resist the effects of earthquake motions. The proposed site is located in Durham, North Carolina.

Based on the International Building code 2009, the site is classified as **Type E** table 1615.1.1. Active Force on Culvert Wall with Earthquake Forces was calculated using Mononobe-Okabe Equation and is enclosed in the Appendix. Active Force per unit length of wall was calculated to be 1,500 PLF, determined at a height of 2/3rd from bottom of culvert. Shear wave testing of the soils can be used to provide a more thorough evaluation of the seismic classification of site. A 100-foot deep boring was not required by the structural engineer due to size and height of the proposed structure.

3.02 Foundations:

Borings B1 through B6 show blow counts ranging from 3 to 80 blows per foot (bpf) approximately 9 feet below the ground surface. Based on the exploration, the soils encountered for this site consisted of mostly brown fine sandy silt or silty sand. Bearing capacity calculations indicated that a maximum allowable design soil bearing pressure of **1,500 PSF** (gross) should be used for the bottom of the box culvert footing. The footing base should be designed using a modulus of subgrade reaction (k) of 100 pounds per cubic inch. In addition, the footing and culvert slab should bear at a sufficient depth to provided adequate resistance to scour. A scour study was not part of this investigation.

3.03 Settlement:

Based on the estimated settlement calculations, general stratigraphy in the building areas, past experience with similar projects and the anticipated magnitude of the traffic loads, it is our opinion that the total immediate settlement could be over 1-inch in the low blow count silts and differential settlement potentials for the culvert should be on the order of ¾ of an inch. This conclusion is contingent upon compliance with the site preparation and fills placement recommendations outlined in this report.

4.0 Geotechnical Recommendations

4.01 Shrinkage and swell factors:

Based on our previous experience with similar soils, we recommend a shrinkage factor of 20 % and a swell factor of 10 % for all excavations on this project.

The following recommendations are based on the information available on the proposed construction, the subsurface data provided to us, and our experience with soils and subsurface conditions similar to those described in the test boring records provided to us. Conditions may be encountered during construction that are substantially different than those indicated by the borings. In these instances, adjustments to the design and construction may be necessary depending on actual conditions.

4.02 General Site Development Considerations

Any trees, underbrush, topsoil, roots, and other deleterious materials should be removed from the proposed construction area. Special attention should be given to the removal of tree stumps within the proposed construction area. Site clearing, grubbing, and stripping, should be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive mixing of topsoil, humus and organic debris with clean underlying soils.

All relocation of existing underground utilities should be completed before site grading begins. The ends of abandoned underground utilities should be permanently sealed to prevent the inadvertent introduction of fluids into the construction area. Any septic tanks and drain fields within proposed construction areas and 20 feet outside the construction limits should be excavated and removed.

We recommend that areas to receive structural fill be proofrolled prior to placement of structural fill. Areas of proposed excavation should be proofrolled after rough finished subgrade is achieved. Proofrolling should be performed using a loaded dump truck weighing at least 15 tons. Proofrolling should be accomplished by performing at least 3 passes in each of two perpendicular directions within entire construction areas, and 10 feet beyond. Any unsuitable materials that may be present, and any low consistency soils that are encountered which cannot be adequately densified in place, should be removed and replaced with well compacted fill material placed in accordance with the

Structural Fill section of this report. Proofrolling should be observed by our representative to determine if remedial measures are necessary. Proofrolling should facilitate the identification of soft surficial soils, but should not be expected to reveal soft conditions more than 2 feet below the ground surface at the time of proofrolling. Footing examinations will be required to evaluate the presence of deeper soft soils, which could adversely affect foundation support. Footing examinations will be discussed later in this report.

Based on our experience on similar sites, there may also be buried obstructions such a boulders or construction debris. On sites located in developed areas this is not an unusual occurrence. Such buried materials occur in isolated areas which may not be detected by the soil test borings. Any buried waste construction debris or trash which is found during the construction operation should be thoroughly excavated, and the waste material should be removed from the site prior to placement of fill soils.

After excavation, some heave of the soils should be expected. Additionally, creating an excavation of this size could also allow inflow of ground water. Remedial measures should be expected in this area at the time of construction to provide a suitable base for support of the culvert structure.

Site grading is expected in the planned road widening of University Drive. No test data has been provided regarding subsurface conditions in the area of University Drive and is not within our scope of services.

4.03 Highly Plastic Soils:

Soil boring logs indicate the presence of some moderately plastic soils on the site. If any highly plastic soils occur at or near design subgrades their characteristic to shrink and swell with changes in moisture can cause structural problems. In addition, these soils lose strength when wet. All highly plastic soils within 3 feet of design subgrades under structures and within 2 feet of design subgrade in areas to be paved should be removed and replaced with low plasticity materials. The low plasticity materials should have a plastic index (PI) of less than 30.

4.04 Utilities:

We recommend any / all utility lines be located outside of planned construction areas, the trenches cleaned of backfill soils, and, after utility emplacement, the trench backfilled with compacted fill as recommended in this report. Past experience indicates utility trench backfill is often poorly compacted. Also, cracked or deteriorated pipes can collapse, leak or serve as conduits for subsurface erosion. Any of these conditions can result in excessive settlement of foundations and pavements.

4.05 Construction Dewatering

The presence of ground water was encountered during drilling operation. Therefore, *we* do anticipate that ground water control will be essentially required. However, ground water levels can fluctuate. In areas where excavations are greater than 3 or 4 feet below the ground water level, or in areas where trench dewatering proves to be ineffective, it may be necessary to use a well point system or other methods to efficiently perform construction dewatering.

We must emphasize that dewatering requirements will be dictated by ground water conditions at the time of construction. The contractor should use a technique or combination of techniques which achieves the desired result under actual field conditions.

4.06 Earth Slopes

Because of the confined nature of the site, the depth of the excavation required for culvert installation and the close proximity of the existing masonry building it will be necessary to install some type of temporary earth support to protect both the workers and the building.
The contractor will be required to submit the proposed system to the owner for review by the client prior to the start of construction.

There may be some locations where temporary construction slopes can be installed. If this option becomes available then the temporary construction slopes should be designed in strict compliance with the most recent OSHA regulations. The test borings indicate that most soils at the site are Type C as defined in the *Occupational Safety and Health Standards for the Construction Industry (29 CFR, Part 1926, Subpart P), July 1, 2001.* This dictates that temporary construction slopes in residual soils be no steeper than 1.5 horizontal to 1 vertical for excavation depths of up to 20 feet. Flatter slopes may be required due to the presence of large volumes of mica within the soils, or in areas where softer soils are encountered. We recommend that a “competent person” as defined in the OSHA Regulations be present on site during excavations. Temporary construction slopes should be closely observed for signs of mass movement: tension cracks near the crest, bulging at the toe of the slope, etc. If potential stability problems are observed, the Geotechnical Engineer should be immediately contacted. The responsibility for excavation safety and stability of construction slopes should be solely with the contractor.

We recommend that permanent cut or fill slopes be approximately 3 (H) to1.0(V) to maintain long term stability and to provide ease of maintenance. Slopes constructed steeper than 3(H) to 1.0(V) could be highly susceptible to erosion, will be difficult to maintain, and could experience large scale slope failure in some instances. However, steeper slopes under tight site conditions can be built using the following stabilization methods. For fill slopes, use geogrid reinforcement. For cut slopes, use soil nails or pile panel wall or other approved method. The crest or toe of cut or fill slopes should be no closer than 15 feet to any building foundation. The crest or toe should be no closer than 10 feet to the edge of any pavements. For slope stability designs, contractor shall engage a professional engineer to design alternates and get it approved from the owner.

4.07 Foundation Recommendations

After site preparation and site grading are complete, it is our opinion that the proposed box culvert may be supported on conventional shallow foundations. We recommend the use of a design allowable soil bearing pressure of 1,500 pounds per square foot (PSF).

Portions of the footings for this project may bear on new structural fill material. For this reason, we must emphasize the importance of quality control during the placement of structural fill. Performance of building foundations which are supported by structural fill material will depend largely on achieving the recommended level of compaction on fill materials. Compacted soil densities less than the recommended percentage of the standard Proctor maximum dry density could result in excessive foundation settlement.

There may be locations where undercut of the existing soil is required in order to achieve the necessary 1500 psf bearing capacity. Access to these may not allow for the placement of the standard structural fill. In these areas the undercut material can be replaced with # 57 stone wrapped in filter fabric.

It is possible that during construction, some of the existing soils at the project site will have an allowable soil bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification of such soils and to enable the development of remedial measures.

Detailed foundation examinations should be performed in each foundation excavation prior to placement of reinforcing steel. These examinations should be performed by our representative to confirm that the design allowable soil bearing pressure is available. The footing examinations should be performed using a combination of visual observation, hand rod probing, and dynamic cone penetrometer testing. Dynamic cone penetrometer testing, as described in ASTM STP-399, should be performed in the entire length of culvert foundation at no greater than 50 foot intervals in staggered locations. If the soil is found to have an unsatisfactory bearing capacity, our inspector will review the problem with our project Geotechnical Engineer.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement techniques to be the primary remedial measure.

Exposure to the environment may weaken the soils at the foundation bearing surface, if they are exposed for extended periods of time. If the foundation bearing surface becomes softened due to exposure, the soft soils should be removed prior to placement of concrete.

4.08 Retaining Structures:

The following lateral earth pressure coefficients, assuming a level backfill surface and no excess hydrostatic pressure are given below.

Description	#57 Stone	On-Site Silty SAND or Sandy SILT Soils	
		Ø = 34°	**Ø = 32°**
At-rest Coefficient, K_o	0.33	0.44	0.47
Active Coefficient, K_a	0.20	0.28	0.31
Passive Coefficient, K_p	5.00	3.54	3.25
Moist Unit Weight	105 pcf	118 pcf	
Soil/Concrete Friction Factor	0.50	0.3	

Friction angle for soils can be used as 33° for design; and friction angle between soil and concrete is 21°

For soils of this nature, the fill slopes should be compacted to 95% relative density and should be protected from erosion by vegetation or other means. Cut and fill slopes of 3H: 1V or flatter may be desirable for mowing and other maintenance purposes. Should constructed slopes be required for this project, it is recommended that a geotechnical engineer be contacted to perform slope stability analysis.

4.09 Earth Pressure

For the design of the lateral pressures acting on the sides of the culvert box, Ko *for the on-site soils* should be used.

If cantilever sheeting is used to protect the adjacent buildings during construction. Ka of the on site soils should be used for the lateral pressure design.

If horizontal braced sheeting is used Ko of the on-site soils should used. for the lateral pressure design.

The ground water *unit weight s*hould be used if no drainage system is incorporated behind retaining walls. The development of excessive water pressure is a common cause of retaining wall failures. Drainage systems should be carefully designed to insure that long term permanent drainage is accomplished.

For the evaluation of the resistance of soil to lateral loads, which is frequently necessary for evaluating the stability of retaining walls, and laterally loaded foundations, the passive earth pressure must be calculated. The passive earth pressure can be calculated using the same basic equation described above with the coefficient of passive pressure, Kp, as follows:

Kp = *4.02* for soil types SP and SP-SM
Kp = 3.*54* for soil type SM
Kp = 3.25 for soil types ML
Kp= 2.76 for soil type CL

It should be noted that full development of passive pressure requires deflections toward the soil mass on the order of 1.0% to 4.0% of total wall height.

For analysis of sliding resistance of the base of a retaining wall, the coefficient of friction may be taken as 0.3 for the micaceous silts reported to be present at the project site. The force which resists the base sliding is calculated by multiplying the normal force on the base by the coefficient of friction. Full development of the frictional force could require deflection of the base of roughly ¼ to ½ inch. Active force on culvert wall due to earthquake forces is calculated and attached separately.

The above design recommendations are based on the following assumptions:

(1) Horizontal backfill
(2) 95% standard Proctor compaction effort on backfill (ASTM D-698)
(3) Direct soil pressure based on soil characteristics is used for computations
(4) Uniform surcharge (if any)
(5) Negligible wall friction. We recommend that no wall friction be used since the value of wall friction is highly dependent on the degree of compaction immediately adjacent to the wall.

For sloping backfill conditions, the same basic approach may be used with some adjustment to the earth pressure coefficients. The earth pressure coefficients for cases of sloping backfill depend on several factors, not the least of which is the actual slope of the backfill. Earth pressure coefficients for cases of sloping backfill can be provided at your request.

4.10 Reinforced Soil Structures:

Reinforced soil structures may not apply in this situation.

4.11 Pavement Design Recommendations

Based on the above described site preparation recommendations, we anticipate that the pavement area subgrade soils in the planned pavement areas will consist of micaceous silts. We understand that the existing pavement is PC concrete. However bituminous c*oncrete pavement should be considered because of the soft soils, potential settlement in the area of the culvert, and possible* *need to access the top of culvert in future.* However, the design engineer can use other options for aesthetic reasons.

We have used a design CBR value for these soils of 4

We recommend that the pavement be designed as a flexible pavement using guidelines established by the Asphalt Institute for Full Depth Asphalt Pavement Structures. Based on laboratory tests on similar material, a California Bearing Ratio of four was selected for on-site soil compacted to 95 percent of the maximum dry density determined in accordance with ASTM Specification D-698, Standard Proctor Method. For the general parking area we recommend that the pavements be designed for two and a half inches of asphalt overlying six inches of compacted crushed stone. For general access roadways (University Drive section) and in truck loading areas we recommend the design consist of a minimum of three and a half inches of asphalt over eight inches of compacted crushed stone. These designs are based on the Asphalt Institute's MS-1 Thickness Design Manual. However, if the actual road section at University Drive is higher than these recommendations, contractor shall follow that standard section.

Regardless of the section and type of construction utilized, saturation of the subgrade materials and asphalt pavement areas results in a softening of the subgrade material and shortened life span for the pavement. Therefore we recommend that both the surface and subsurface materials for the pavement be properly graded to enhance surface and subgrade drainage. By quickly removing surface and subsurface water, softening of the subgrade can be reduced and the performance of the parking area can be improved. Site preparation for the parking areas should be similar to that for the building area including stripping, proofrolling, and the placement of compacted structural fill.

Proofrolling the parking and drive areas may identify portions of the pavement subgrade that contain soft unsuitable soils. (*HOWEVER PROOFROLLING IS NOT RECCOMENDED WITHIN FIVE FEET OF THE CULVERT).* In such sections, we recommend use of Dynamic Cone Penetrometer. CBR values greater than 10 represent good subgrade. If significant areas with low strength are discovered, we can provide specific recommendations for repairs which may include undercutting, placement of reinforcing grid and additional stone base or replacement with properly compacted fill soils. Making these recommendations requires observing conditions during construction and addressing the problems if they occur.

5.0 Construction Recommendations

5.01 Site Grading & Earthwork

All trees, underbrush, roots and other organic matter should be removed before construction. Soils on site should be easily graded using typical earth moving equipment.

The vertical and horizontal extent of loose soils should be determined in the field by a soils engineer at the time of excavations. All excavations should be performed during dry weather.

We recommend that excavations for footings bearing in virgin soil be examined by a qualified geotechnical engineer or his representative prior to placement of concrete. The purpose of this inspection would be to establish that the exposed materials are similar to those encountered by the borings. Any soft or loose soils should be undercut and backfilled with suitable fill materials. The undercutting should extend laterally beyond the footing perimeter a minimum distance of 9 inches for each 1 ft. of vertical excavation below the base of the footing. The backfill should be compacted as described above. Foundation excavation bearing in properly compacted fill need only be checked and cleaned of any loose soil prior to concrete placement.

Due to the nature of the native soils, it will be necessary to protect the site from runoff during rain events. Runoff should be prevented from accumulating on the graded surface by ditching or other suitable means. In addition, construction traffic should be limited across the slab/foundation area. Access route around the perimeter should be created.

5.02 Temporary Shoring:

During partial removal of the existing culvert *and construction of the new culvert* temporary shoring of the excavation walls may be necessary. Construction slopes should be designed in strict compliance with current OSHA regulations. Results of the test borings indicate that most soils are Type C as defined by OSHA. Type C soils will require construction slopes no steeper the 1.5 H to 1 V for excavations up to 20 feet. Depending upon the actual conditions encountered, including groundwater, flatter slopes may be necessary. A competent person as defined by OSHA guidelines should be present during excavation to determine the type of material encountered, and to monitor the slopes for any signs of mass movement (such as tension cracks near the crest, bulging at the toe of the slope, etc.). If potential stability problems are encountered the geotechnical engineer should be contacted immediately, for additional services. Note that the contractor bears sole responsibility for excavation safety and the stability of construction slopes.

If a shoring system is used, it should be designed by a registered Professional Engineer. The system design should include all calculations, plans, specifications, assumptions, and testing requirements. The system should stabilize all boulders and/or soft soils behind the shoring system.

This shoring should be evaluated as the excavation proceeds in order to assure the integrity of the excavation walls.

5.03 TEMPORARY EXCAVATION STABILITY

Excavations greater than four feet in depth should be sloped or shored in accordance with local, state, and federal regulations, including OSHA "Construction Standard for Excavations" (29 CFR Part 1926.650-652). The contractor is usually solely responsible for site safety. This information is provided only as a service and under no circumstances should A1 be assumed to be responsible for construction site safety.

5.04 Structural Fill

In order to achieve high density structural fill, the following recommendations are offered:

(1) Materials selected for use as structural fill should be free of vegetable matter, waste, construction debris, mica, and other deleterious materials. The material should not contain rocks having a diameter over 3 inches. On-site soils with silt and mica are not recommended for structural fill. It is our opinion that the following soils represented by their USCS group symbols will typically be suitable for use as structural fill: (SW), (SP), (SM), (SC), (ML),(CL), (GC), and (GM). The following soil types are considered unsuitable for use as structural fill at this site (MH), (CH), (OL), (OH), and (PT).Please refer to Type-III soils for select backfill in NCDOT specification.

(2) Laboratory Proctor compaction tests and classification tests should be performed on representative samples obtained from the proposed borrow material to provide data necessary to determine acceptability and for quality control. The moisture content of suitable borrow soils should generally not be more than 3 percentage points above or more than 3 percentage points below optimum at the time of compaction. Tighter moisture limits may be necessary with the micaceous silts present at this site.

(3) Suitable fill material should be placed in thin lifts (lift thickness depends on type of compaction equipment, but in general, lifts of 8 inches loose measurement are

recommended). The soil should be compacted by mechanical means such as steel drum or sheepsfoot rollers. Proofrolling with rubber tired, heavily loaded vehicles may be desirable at approximately every third lift to bind the lifts together and to seal the surface of the compacted area thus reducing potential for absorption of surface water following a rain. This sealing operation is particularly important at the end of the work day and at the end of the week.

Within small excavations such as behind retaining walls or in footing excavations, we recommend the use of “wacker packers” or diesel sled tamps to achieve the specified compaction. Loose lift thicknesses of 4 to 6 inches are recommended in small area fills.

(4) We recommend that structural fill be compacted to a minimum of 98% of the standard Proctor maximum dry density (ASTM Specification D-698). Additionally, the in-place maximum dry density of structural fill should be no less than 90 pcf. Pavement area fill soils and pavement subgrades should be compacted in accordance with the current NCDOT Standard Specifications.

(5) An experienced soil engineering technician should take adequate density tests throughout the fill placement operation to verify that the specified compaction is achieved. It is particularly important that this be accomplished during the initial stages of the compaction operation to enable adjustments to the compaction operation, if necessary.

5.05 EFFECTS OF CONSTRUCTION METHODS

Several aspects of construction at this site could adversely affect the adjacent streets, utilities and nearby facilities. Therefore, proper design and special care during construction will be needed to protect the adjoining properties. These items are discussed below.

Jackhammering, blasting, pile driving and other construction activities can generate vibrations that travel off-site. These vibrations can cause damage to adjacent structures if not properly controlled. Care must be taken to prevent damage of newly placed structures, Especially fresh concrete. Any blasting charges that are used must be properly sized and timed to prevent structural damage. We recommend that vibration monitoring be performed for structures located nearby during the construction activities that generate a large amount of vibration. This will reduce the potential for large magnitude vibrations and subsequent damage claims.

6.0 General Conditions And Notes

6.01 General Conditions:

The analysis, conclusions, and recommendations submitted in this report are based on the exploration previously outlined and the data obtained from the client and collected at the points shown on the attached locations plan. This report does not reflect specific variations that may occur between test locations, and therefore can not be used for bid quantity determination. The full nature and extent of variations between borings and of subsurface conditions may not become evident until the course of construction. If variations become evident at any time before or during the course of construction, it will be necessary to make a re-evaluation of the conclusions and recommendations of this report and further exploration, observation, and/or testing may be required.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices and makes no other warranties, either expressed or implied, as to the professional advice under the terms of our agreement and included in this report. The recommendations contained herein are made with the understanding that the contract documents between the owner and foundation or earthwork contractor or excavating and earthwork subcontractors, if any, shall require that the contractor certify that all the work in connection with foundations, compacted fills and other elements of the foundation or other supporting components are in place at the locations, with proper dimensions and plumb, as shown on the plans and specifications for the project.

Further, it is understood the contract documents will specify that the contractor will, upon becoming aware of apparent or latent subsurface conditions differing from those disclosed by the original soil investigation work, promptly notify the owner, both verbally to permit immediate verification of the change and in writing, as to the nature and extent of the differing conditions and that no claim by the contractor for any conditions differing from those anticipated in the plans and specifications and disclosed by the soil studies will be allowed under the contract unless the contractor has so notified the owner both verbally and in writing, as required above, of such changed conditions. The owner will, in turn, promptly notify this firm of the existence of such unanticipated conditions and will authorize such further investigation as may be required to properly evaluate these conditions.

Further, it is understood that any specific recommendation made in this report as to on-site construction review by this firm will be authorized and funds and facilities for such review will be provided at the times recommended if we are to be held responsible for the design recommendations.

During all below ground construction activities a geotechnical engineer should be present to make specific evaluations of the actual conditions encountered.

6.02 Procedures Regarding Field Logs, Laboratory Data Sheets and Samples:

In the process of obtaining and testing samples and preparing this report, procedures are followed that represent reasonable and accepted practice in the field of soil and foundation engineering.

Specifically, field logs are prepared during performance of the drilling and sampling operations that are intended to portray essentially field occurrences, sampling locations and other information.

The engineer preparing the report reviews the field and laboratory logs, classifications and tests data, and in his judgement in interpreting this data, may make further changes.

Samples taken in the field, some of which are later subjected to laboratory tests, are retained in our laboratory for sixty (60) days and are then destroyed unless special disposition is requested by the client. Samples retained over a long period of time even in sealed jars are subject to moisture loss, which changes the apparent strength of cohesive soil generally increasing the strength from what was originally encountered in the field. Since they are no longer representative of the moisture conditions initially encountered, an inspection of these samples should recognize this factor.

The subsurface information contained herein has been obtained for planning and design purposes only and not for construction or pay purposes. Soil strata, soil moisture, groundwater levels and rock strata descriptions and indicated boundaries are based on interpretation and may not necessarily reflect the actual subsurface conditions within a borehole, beyond or between borehole locations.

This report is prepared for the exclusive use of **HDR Engineers** and the **City of Durham** for the specific application to the proposed construction project. This report is for design purposes only and is not sufficient to prepare an accurate bid of quantities. No other warranties expressed or implied are made.

6.03 Additional Services:

As the project progresses, site conditions may dictate revising certain design parameters. All recommendations made here were arrived at through field information and project information supplied by the Engineer.

Since the final design may deviate from the design on which these recommendations are made, A1 would be pleased to review the revised design and make additional recommendations as an additional consulting service. Items that may require additional services include:

- Review of plans and specifications for foundations, drainage, etc.
- Site preparation observation for grading work.
- Footing and slab evaluation
- Review of locations requiring subgrade improvement for the foundation system.
- Review of earthwork quantities and unsuitable soils or rock removal quantities from foundation and utility trench areas.
- Pavement construction observation

In addition, A1 will provide construction review, field engineering, inspection and material testing services during the earthwork foundation, concrete and pavement phases of the project. A1 assumes no responsibility for compliance with the design, recommendations or performance of the foundation or pavement systems unless we have been authorized to perform these services during construction.

APPENDIX

DURHAM

1 8 6 9

CITY OF MEDICINE

CITY OF DURHAM

Contract Drawings For

City of Durham

University Drive
Culvert Replacement

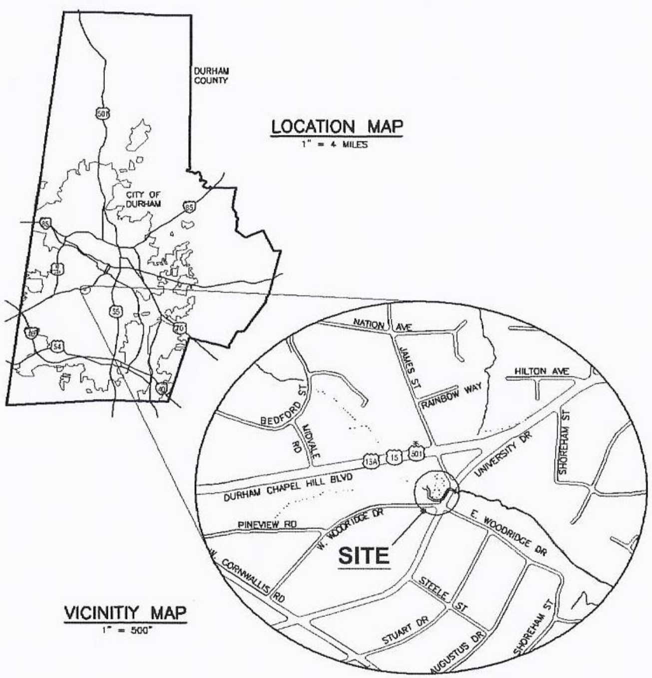
Civil/Structural

Contract No.
SD-2009-02

Project No.
007592-42950-018

Durham, North Carolina
December 2008

INDEX OF DRAWINGS

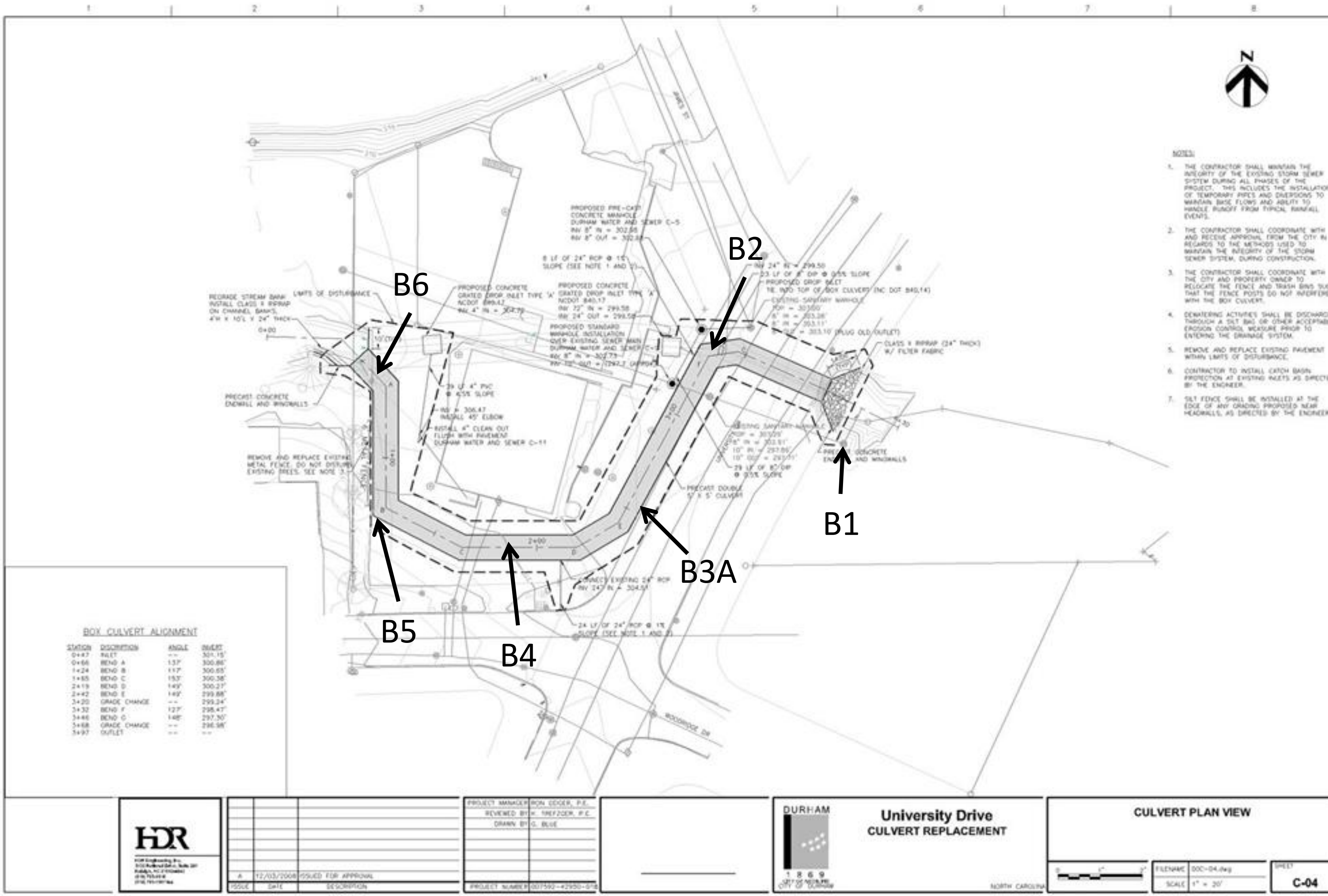


LOCATION MAP
1" = 4 MILES

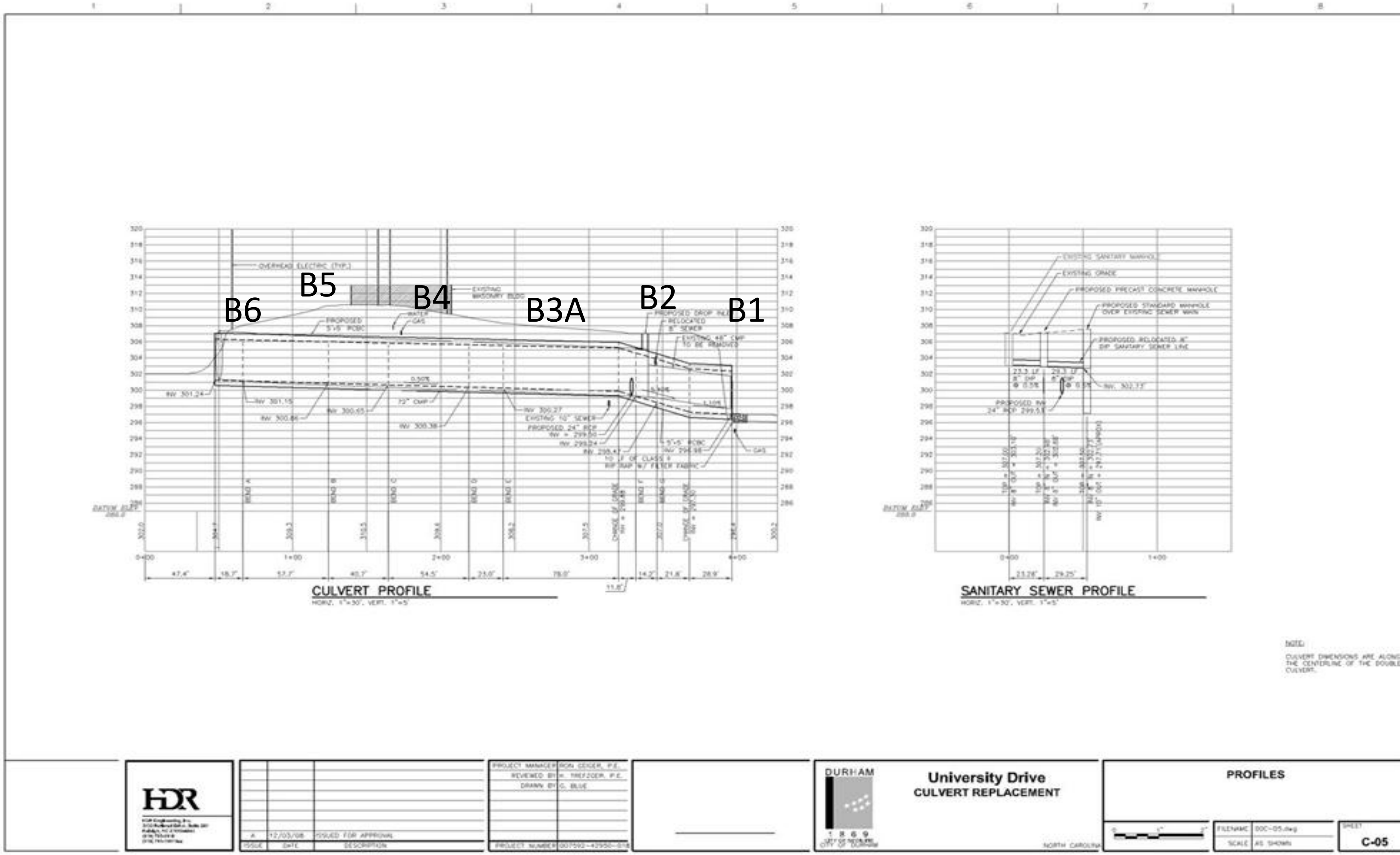
VICINITIY MAP
1" = 500'

North Carolina ONE-CALL CENTER
1 800 632-4949

University Drive Culvert Replacement Boring Locations – Plan View



University Drive Culvert Replacement Boring Locations- Profile



University Drive Culvert Replacement Site Conditions

Boring B6

Boring B3 and B2 locations

Boring B1location

Restored B4

Laboratory Results Summary

Boring No.	Sample No.	Sample Depth (ft)	USCS Class.	Description	Liquid Limit	Plastic Limit	Plasticity Index	Natural Moisture	pH	Min Soil Resistivity
1	3	6.0-7.5						26		
	4	8.5-10.0	ML	sandy silt	20	NP	NP		5.3	
2	3	6.0-7.5							8.5	
	5	14.0-15.5						19.0		
3A	2	3.5-5.0							7.2	
	3	6.0-7.5	SC-SM	silty clayey sand	23	18	5			
	4	8.5-10.0						30.0		
B4	2	3.5-5.0						16.0		
	4	8.5-10.0						26.0	6.5	
B5	3	6.0-7.5							5.3	
	4	8.5-10.0	SM	silty sand	26	17	9	8.0		
B6	3	6.0-7.5							5.1	
	4	8.5-10.0						22.0		
	5	14.0-15.5	SW-SC	well graded sand with clay	25	13	12			
B!	BLK1			Max dry density 117.4 pcf @ 12.5% moisture: CBR 13% (unsoaked), 4% (soaked)						4.650 k ohm/cm @ 16% moisture
B3A	BLK3A			Max dry density 122.1 pcf @ 9% moisture						2.750 k ohm/cm @ 17% moisture
B6	BLK6			Max dry density 117.6 pcf @ 12% moisture: CBR 12% (unsoaked) 7% (soaked)						3.000 k ohm/cm @ 15% moisture

Table of SPT Values
University Drive Culvert
Durham, NC

BORING #	DEPTH (FT.)	GROUND ELEV. (FT)*	SPT 1 (1.0-2.5')	SPT 2 (3.5-5")	SPT 3 (6-7.5')	SPT 4 (8.5-10	SPT 5 (13.5-15')	SPT 6 (18.5-20')	SPT 7 (23.5-25)
1	15.5		4	4	4	9	100*	Begin RockCoring	
2	20		17	1	WH	5	14	100+	
3	25		5	4	2	3	4	100+	100+
4	25		8	6	4	5	100+	100+	100+
5	20		13	8	8	30	100+	100+	
6	17		8	100+	8	10	18	Begin RockCoring	

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SIEVE ANALYSIS

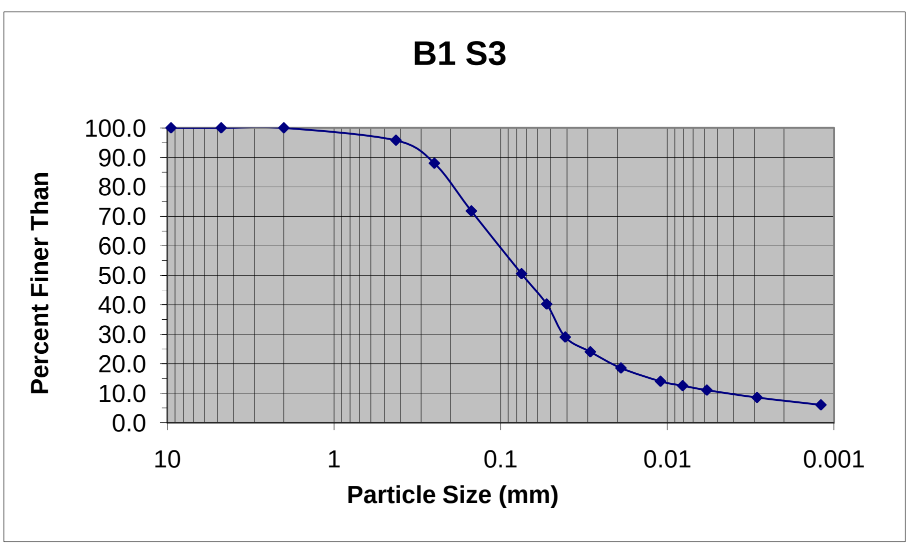
Project #	10-070	Date:	8/1/2011
Project Name:	University Drive Culvert Replacement	Depth:	6.0-7.5
Sample #	B1 S3	Tested by:	Bill Boyd
Weight of Oven-dried sample(gms)	100	Checked by:	V. Goel

#	Sieve Size	Sieve Size (mm)	Weight Retained	Weight Passed	Total Percent
1	3/8 in	9.5		100.0	100.0
2	No. 4	4.75	0	100.0	100.0
3	No. 10	2	0	100.0	100.0
4	No. 40	0.425	4.2	95.8	95.8
5	No. 60	0.25	7.8	88.0	88.0
6	No. 100	0.15	16.2	71.8	71.8
7	No. 200	0.075	21.3	50.5	50.5
8	No. 270	0.053	10.3	40.2	40.2
		0.041	(Hydrometer Values)		29.0
		0.029			24.0
		0.019			18.5
		0.011			14.0
		0.0081			12.5
		0.0058			11.0
		0.0029			8.5
		0.0012			6.0

Liquid Limit: 20
Plastic Index: NP

USCS Classification: ML

sandy silt



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SIEVE ANALYSIS

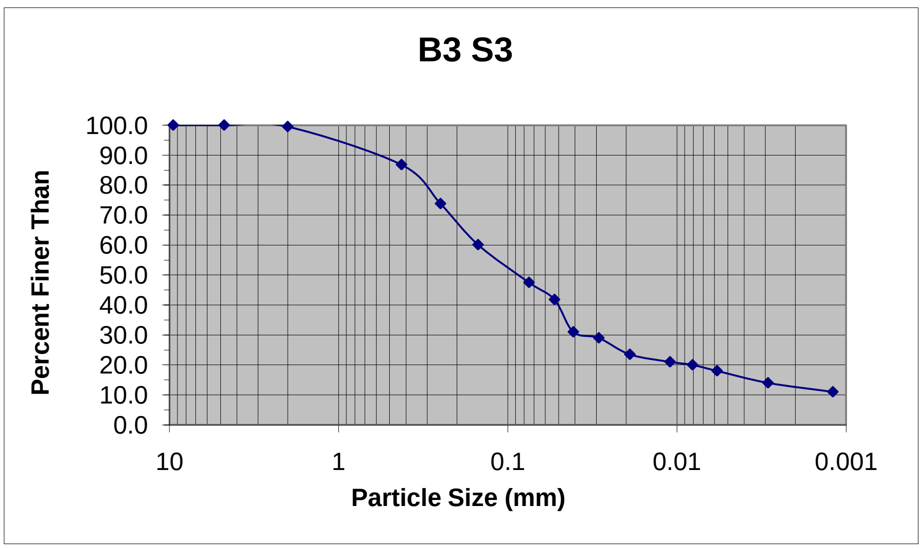
Project #	10-070	Date:	8/1/2011
Project Name:	University Drive Culvert Replacement	Depth:	6.0-7.5
Sample #	B3 S3	Tested by:	Bill Boyd
Weight of Oven-dried sample(gms)	100	Checked by:	V. Goel

#	Sieve Size	Sieve Size (mm)	Weight Retained	Weight Passed	Total Percent
1	3/8 in	9.5		100.0	100.0
2	No. 4	4.75	0	100.0	100.0
3	No. 10	2	0.5	99.5	99.5
4	No. 40	0.425	12.7	86.8	86.8
5	No. 60	0.25	13	73.8	73.8
6	No. 100	0.15	13.7	60.1	60.1
7	No. 200	0.075	12.6	47.5	47.5
8	No. 270	0.053	5.7	41.8	41.8
		0.041	(Hydrometer Values)		31.0
		0.029			29.0
		0.019			23.5
		0.011			21.0
		0.0081			20.0
		0.0058			18.0
		0.0029			14.0
		0.0012			11.0

Liquid Limit: 23
Plastic Index: 5

USCS Classification: **SC-SM**

clayey silty sand



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SIEVE ANALYSIS

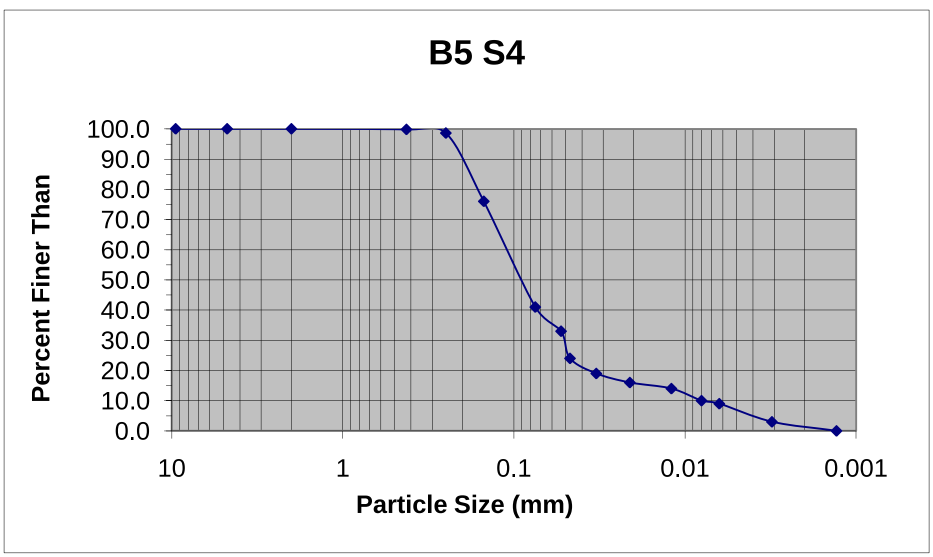
Project #	10-070	Date:	8/1/2011
Project Name:	University Drive Culvert Replacement	Depth:	13.5-15.0
Sample #	B5 S4	Tested by:	Bill Boyd
Weight of Oven-dried sample(gms)	50	Checked by:	V. Goel

#	Sieve Size	Sieve Size (mm)	Weight Retained	Weight Passed	Total Percent
1	3/8 in	9.5		50.0	100.0
2	No. 4	4.75	0	50.0	100.0
3	No. 10	2	0	50.0	100.0
4	No. 40	0.425	0.1	49.9	99.8
5	No. 60	0.25	0.6	49.3	98.6
6	No. 100	0.15	11.3	38.0	76.0
7	No. 200	0.075	17.5	20.5	41.0
8	No. 270	0.053	4.0	16.5	33.0
		0.047	(Hydrometer Values)		24.0
		0.033			19.0
		0.021			16.0
		0.012			14.0
		0.008			10.0
		0.0063			9.0
		0.0031			3.0
		0.0013			0.0

Liquid Limit: 26
Plastic Index: 9

USCS Classification: SM

silty sand



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SIEVE ANALYSIS

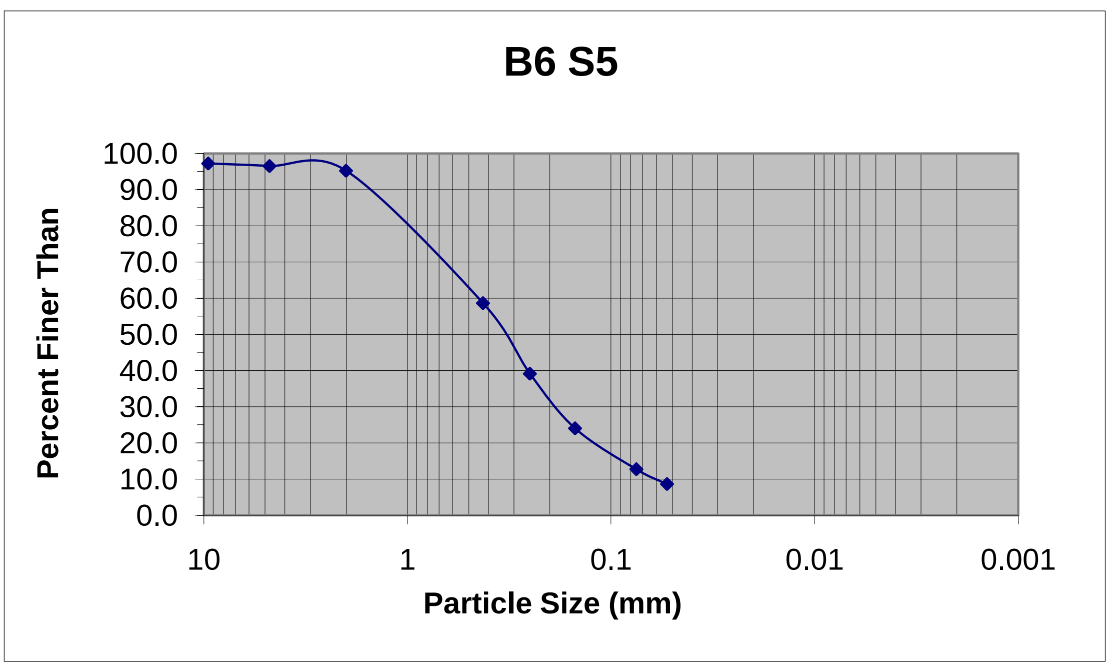
Project #	*10-070*	*Date:*	*8/1/2011*
Project Name:	*University Drive Culvert Replacement*	*Depth:*	*13.5-15.0*
Sample #	*B6 S5*	*Tested by:*	*Bill Boyd*
Weight of Oven-dried sample(gms)	*100*	*Checked by:*	*V. Goel*

#	Sieve Size	Sieve Size (mm)	Weight Retained	Weight Passed	Total Percent
1	3/8 in	9.5	2.8	97.2	97.2
2	No. 4	4.75	0.7	96.5	96.5
3	No. 10	2	1.3	95.2	95.2
4	No. 40	0.425	36.6	58.6	58.6
5	No. 60	0.25	19.5	39.1	39.1
6	No. 100	0.15	15.1	24.0	24.0
7	No. 200	0.075	11.3	12.7	12.7
8	No. 270	0.053	4.1	8.6	8.6

Liquid Limit: **25**
Plastic Index: **12**

USCS Classification:	**SW-SC**

well graded clayey sand



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STANDARD PROCTOR TEST

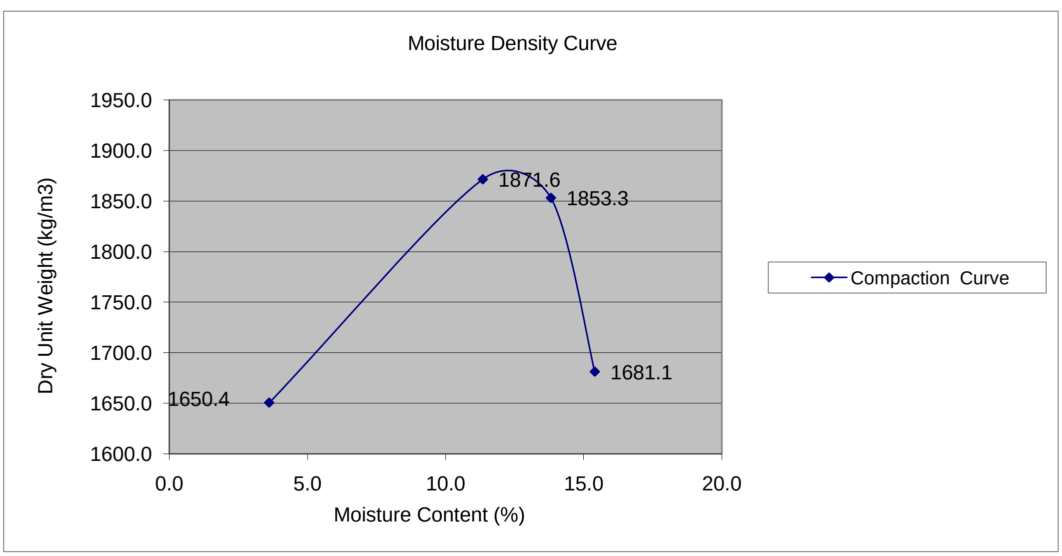
Name of Project: University Culvert

Project #	***10-070***	***Date:***	*8/3/2011*
Boring # unknown	***B1***	***Depth:***	
Sample #	***B1 BLK***	***Tested by: pfp***	
Method Used: Method B		***Checked by:***	*VKG*
Volume of the Mold (m^3) =	*0.000952*		

Description	*Trial#1*	*Trial#2*	*Trial#3*	*Trial#4*	*Trial#5*	*Trial#6*	*Trial#7*
Mold#	2	2	2	2			
Weight of Mold	3644.0	3644.0	3644.0	3644.0			
Wt of soil sample passing #10	2396.0	2276.0	2414.0	2688.0			
Wt of Mold+Compacted soil	5272.0	5628.0	5622.0	5584.0			
Wt of Compacted soil	1628.0	1984.0	2008.0	1940.0			
Amount of water added (ml)	100	180	180	250.0			
Can #	3	3	3	3.0			
Weight of Empty Can	364.0	364.0	364	364.0			
Weight of Can+wet soil	536.0	678	570	604.0			
Weight of Can+dry soil	**530.0**	**646**	**545**	572.0			
Wt of wet soil removed	172	198	198	196.0			
Weight of water	6	32	25	32.0			
Weight of dry soil	166	282	181	208.0			
Moisture Content (%)	3.6	11.3	13.8	15.4			
Wet Density (kg/m^3)	1710.1	2084.0	2109.2	2043.2			
Dry Density(kg/m^3)	1650.4	1871.6	1853.3	1681.1			

Optimum moisture content: **12.50%**

Maximum Dry Density : **1880** **Kg/m3**
117.4 **pcf**



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STANDARD PROCTOR TEST

Name of Project: University Culvert

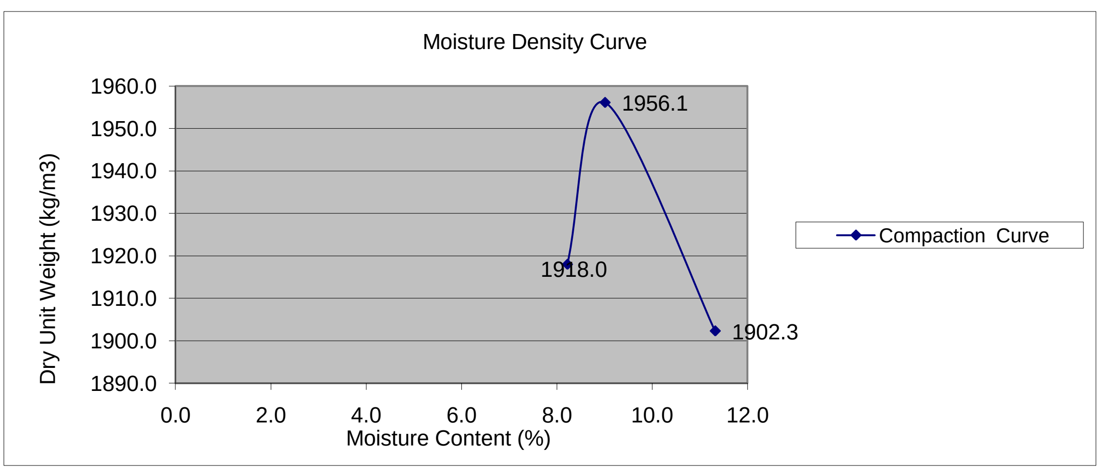
Project #	**10-070**	**Date:**	8/3/2011
Boring #	**B3A**	**Depth:**	
Sample #	**B3A BLK**	**Tested by:**	**pfp**
Method Used: Method B		**Checked by:**	VKG

Volume of the Mold (m^3) = 0.000952

Description	Trial#1	Trial#2	Trial#3	Trial#4	Trial#5	Trial#6	Trial#7
Mold#	2	2	2				
Weight of Mold	3644.0	3644.0	3644.0				
Wt of soil sample passing #10	2082.0	2414.0	2688.0				
Wt of Mold+Compacted soil	5620.0	5674.0	5660.0				
Wt of Compacted soil	1976.0	2030.0	2016.0				
Amount of water added (ml)	100	180	250				
Can #	3	3	3				
Weight of Empty Can	364.0	364.0	364				
Weight of Can+wet soil	522.0	606	600				
Weight of Can+dry soil	**510.0**	**586**	**576**				
Wt of wet soil removed	158	198	196				
Weight of water	12	20	24				
Weight of dry soil	146	222	212				
Moisture Content (%)	8.2	9.0	11.3				
Wet Density (kg/m^3)	2075.6	2132.4	2117.6				
Dry Density(kg/m^3)	1918.0	1956.1	1902.3				

Optimum moisture content: **9.00%**

Maximum Dry Density : **1956 Kg/m3**
122.1 pcf



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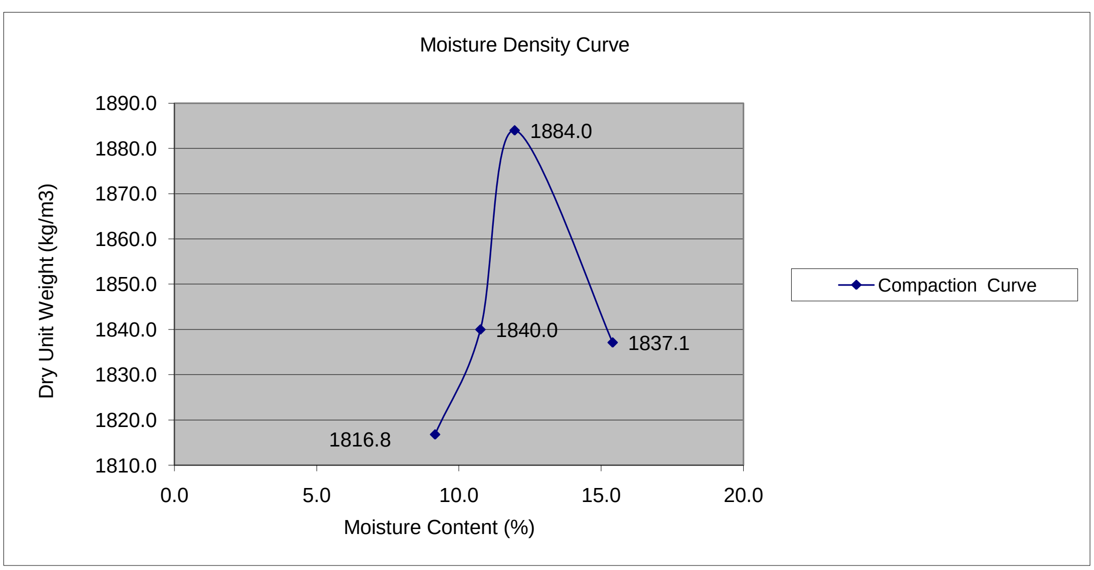
STANDARD PROCTOR TEST

Name of Project: University Culvert

Project #	***10-070***	***Date:***	*8/3/2011*
Boring #	***B4***	***Depth:***	
Sample #	***B4 BLK2***	***Tested by:***	***pfp***
Method Used: Method B		***Checked by:***	*VKG*
Volume of the Mold (m^3) =	*0.000952*		

Description	***Trial#1***	***Trial#2***	***Trial#3***	***Trial#4***	***Trial#5***	***Trial#6***	***Trial#7***
Mold#	2	2	2	2			
Weight of Mold	3644.0	3644.0	3644.0	3644.0			
Wt of soil sample passing #10	2396.0	2414.0	2414.0	2688.0			
Wt of Mold+Compacted soil	5532.0	5584.0	5652.0	5662.0			
Wt of Compacted soil	1888.0	1940.0	2008.0	2018.0			
Amount of water added (ml)	100	180	180	250.0			
Can #	3	3	3	3.0			
Weight of Empty Can	364.0	364.0	364	364.0			
Weight of Can+wet soil	650.0	570	570	604.0			
Weight of Can+dry soil	**626.0**	**550**	**548**	572.0			
Wt of wet soil removed	286	198	198	196.0			
Weight of water	24	20	22	32.0			
Weight of dry soil	262	186	184	208.0			
Moisture Content (%)	9.2	10.8	12.0	15.4			
Wet Density (kg/m^3)	1983.2	2037.8	2109.2	2119.7			
Dry Density(kg/m^3)	1816.8	1840.0	1884.0	1837.1			

Optimum moisture content:	**12.00%**	
Maximum Dry Density :	**1884**	**Kg/m3**
	117.6	**pcf**



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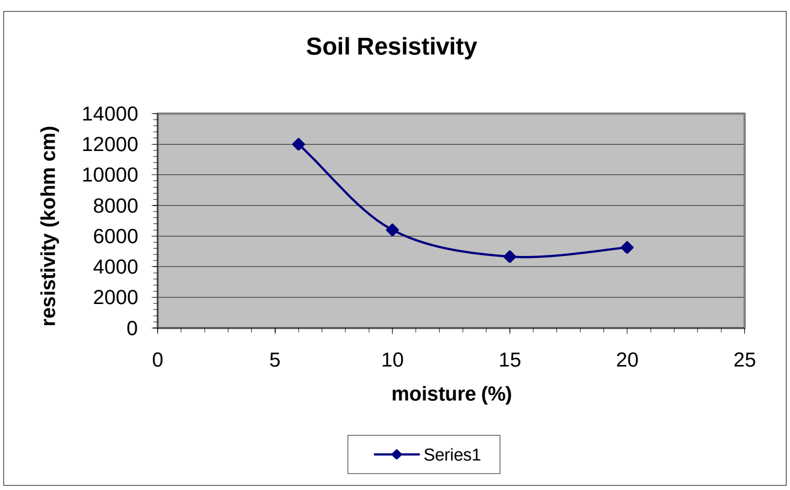
DETERMINATION OF SOIL RESISTIVITY

Project #	10-070	*Date*	*8/2/2011*
Boring #	B1	*Depth*	*10-15'*
Total Dry Sample Wt.(g)	1500	*Tested by*	*HWB*
		Checked by	*VKG*

#	*Water Quantity(ml)*	*Temp °C*	*Can #*	*Wt of Can (g)*	*Wt of can+ Wet soil(g)*	*Wt of can+ Dry soil(g)*	*Wt of Water(g)*	*Wt of Soil (g)*	*Moisture Content (%)*	*Resistivity k ohm .cm*
1	*75*	*28*	*29*	*29.8*	*157.3*	*117.2*	*40.1*	*87.4*	*6*	*12000*
2	*50*	*28*	*8*	*30.9*	*146.9*	*139.1*	*7.8*	*108.2*	*10*	*6400*
3	*50*	*28*	*25*	*29.9*	*155.9*	*111.3*	*44.6*	*81.4*	*15*	*4650*
4	*50*	*28*	*44*	*30.4*	*174.4*	*134.9*	*39.5*	*104.5*	*20*	*5250*
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										

B1



Minimun Soil Resistivity	**4650**
Maximum Soil Moisture	**16%**

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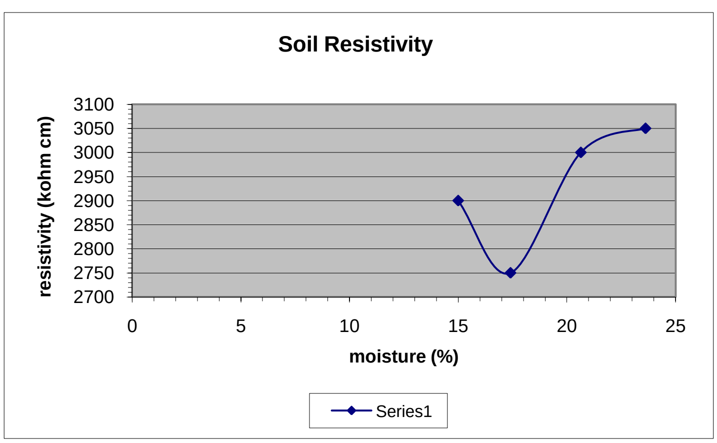
DETERMINATION OF SOIL RESISTIVITY

Project #	10-070	*Date*	*8/2/2011*
Boring #	B3A	*Depth*	*10-15'*
Total Dry Sample Wt.(g)	1500	*Tested by*	*HWB*
		Checked by	*VKG*

#	Water Quantity(ml)	Temp °C	Can #	Wt of Can (g)	Wt of can+ Wet soil(g)	Wt of can+ Dry soil(g)	Wt of Water(g)	Wt of Soil (g)	Moisture Content (%)	Resistivity k ohm .cm
1	50	28	3	31.4	129.9	117.2	12.7	85.8	15	2900
2	75	28	7	31.1	157.9	139.1	18.8	108.0	17	2750
3	100	28	47	30.4	128.0	111.3	16.7	80.9	21	3000
4	50	28	26	29.9	159.7	134.9	24.8	105.0	24	3050
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										

B3A



Minimun Soil Resistivity	**2750**
Maximum Soil Moisture	**17%**

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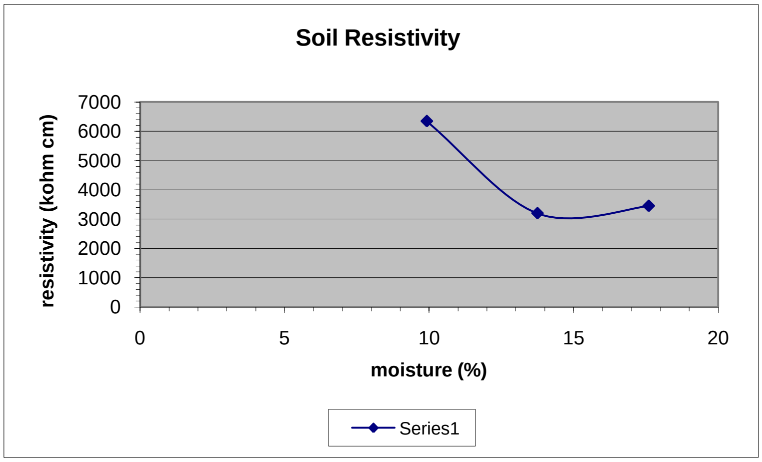
DETERMINATION OF SOIL RESISTIVITY

Project #	10-070	*Date*	*8/3/2011*
Boring #	B4	*Depth*	*10-15'*
Total Dry Sample Wt.(g)	1500	*Tested by*	*HWB*
		Checked by	*VKG*

#	Water Quantity(ml)	Temp °C	Can #	Wt of Can (g)	Wt of can+ Wet soil(g)	Wt of can+ Dry soil(g)	Wt of Water(g)	Wt of Soil (g)	Moisture Content (%)	Resistivity k ohm .cm
1	100	28	41	30.6	135.9	126.4	9.5	95.8	10	6350
2	50	28	45	30.3	147.8	133.6	14.2	103.3	14	3200
3	50	28	18	31.1	144.7	127.7	17.0	96.6	18	3450
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										

B4



Minimun Soil Resistivity	**3000**
Maximum Soil Moisture	**15%**

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CALIFORNIA BEARING RATIO

BEARING RATIO DATA

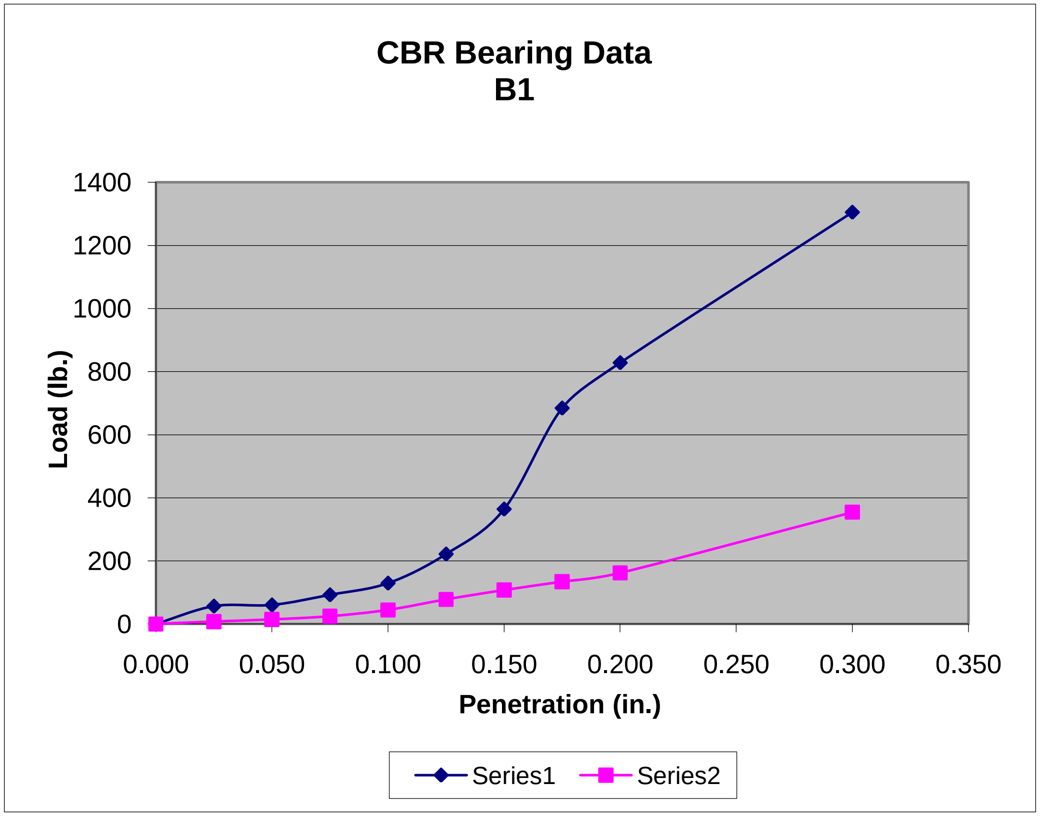
Bulk Sample B1	Weight of surcharge (lbs)	**10**	**Sample preparation by**
	Area of Piston (in^2)	**3.14159**	**ASTM D968**

#	Piston Penetration (in.)	UNSOAKED PSI	SOAKED PSI	UNSOAKED Piston Load (lbs) Actual	SOAKED Piston Load (lbs) Actual
1	0.000	**0**	**0**	**0**	**0**
2	0.025	**56**	**8**	**177**	**24**
3	0.05	**60**	**14**	**189**	**45**
4	0.075	**92**	**25**	**290**	**77**
5	0.100	**129**	**45**	**406**	**140**
6	0.125	**222**	**78**	**696**	**245**
7	0.15	**364**	**108**	**1144**	**338**
8	0.175	**684**	**134**	**2150**	**422**
9	0.200	**828**	**162**	**2602**	**509**
10	0.300	**1305**	**355**	**4101**	**1114**
11	0.400	**0**	**640**		**2011**
12	0.500	**0**	**979**		**3076**

	UNSOAKED	SOAKED
CBR VALUE @ 0.100 in.(%)	**13%**	**4%**
CBR VALUE @ 0.200 in.(%)	**55%**	**11%**

SWELL DATA

	DESCRIPTION	QUANTITY
1	Surcharge Weight	**10.0lbs**
2	Date at the beginning of soaking period	**7/30/2011**
3	Date at the end of soaking period	**8/4/2011**
4	Initial height of specimen (in.)	**4.600**
5	Initial Dial Gauge Reading	**0.3594**
6	Final Dial Gauge Reading	**0.4278**
7	Change in Height	**0.0684**
	Swell Index (7/4)*100	**1.49%**



CBR Bearing Data
B1
Load (lb.)
1400
1200
1000
800
600
400
200
0
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
Penetration (in.)
Series1
Series2

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CALIFORNIA BEARING RATIO

BEARING RATIO DATA

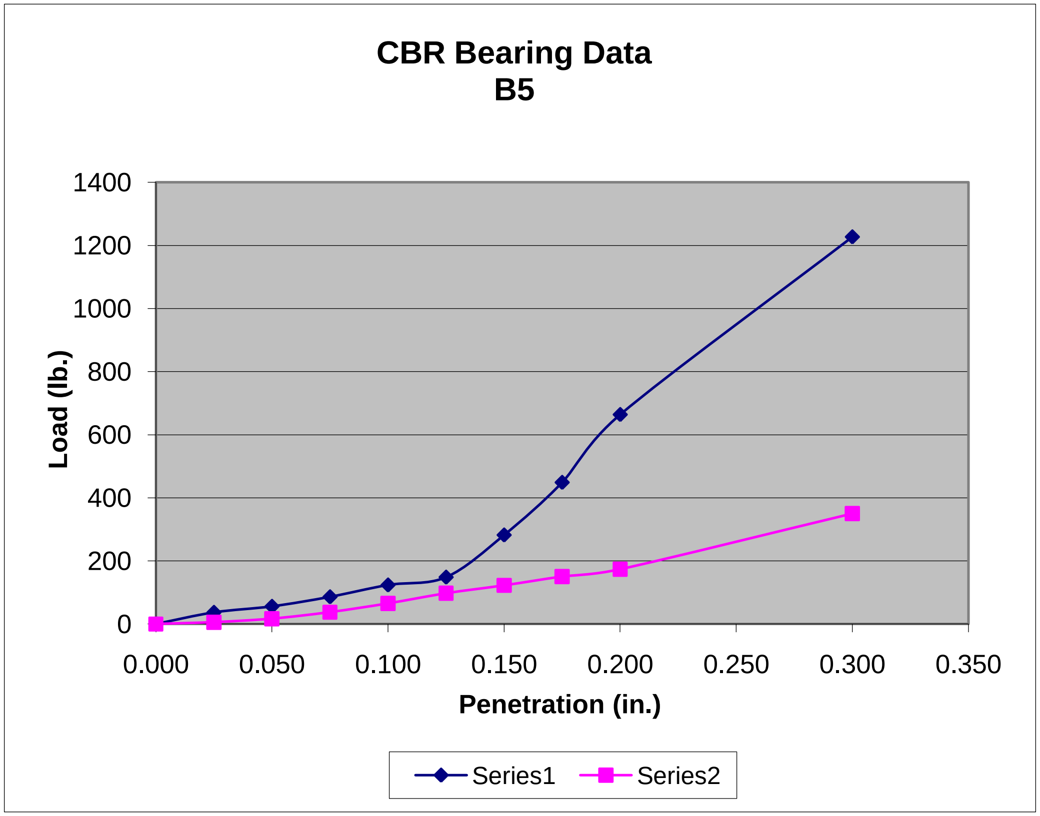
Bulk Sample B6	Weight of surcharge (lbs)	**10**	**Sample preparation by**
	Area of Piston (in^2)	**3.14159**	**ASTM D968**

#	Piston Penetration (in.)	UNSOAKED PSI	SOAKED PSI	UNSOAKED Piston Load (lbs) Actual	SOAKED Piston Load (lbs) Actual
1	0.000	**0**	**0**	**0**	**0**
2	0.025	**37**	**6**	**115**	**18**
3	0.05	**56**	**17**	**175**	**53**
4	0.075	**86**	**38**	**270**	**118**
5	0.100	**124**	**65**	**388**	**205**
6	0.125	**148**	**97**	**466**	**306**
7	0.15	**282**	**123**	**886**	**385**
8	0.175	**449**	**151**	**1410**	**473**
9	0.200	**664**	**174**	**2086**	**546**
10	0.300	**1227**	**350**	**3856**	**1101**
11	0.400	**0**	**0**		
12	0.500	**0**	**0**		

	UNSOAKED	SOAKED
CBR VALUE @ 0.100 in.(%)	**12%**	**7%**
CBR VALUE @ 0.200 in.(%)	**44%**	**12%**

SWELL DATA

	DESCRIPTION	QUANTITY
1	Surcharge Weight	**10.0lbs**
2	Date at the beginning of soaking period	**7/30/2011**
3	Date at the end of soaking period	**8/4/2011**
4	Initial height of specimen (in.)	**4.600**
5	Initial Dial Gauge Reading	**0.103**
6	Final Dial Gauge Reading	**0.112**
7	Change in Height	**0.009**
	Swell Index (7/4)*100	**0.20%**


CBR Bearing Data
B5
Load (lb.)
1400
1200
1000
800
600
400
200
0
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
Penetration (in.)
Series1
Series2

SOIL CLASSIFICATION CHART

	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	SM	SILTY SAND, SAND - SILT MIXTURES
	SC	CLAYEY SANDS, SAND - CLAY MIXTURES
	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEANS CLAYS
	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
	CH	INORGANIC CLAYS OF HIGH PLASTICITY

	MLS	MIXTURE OF SILT AND SAND
	CLS	MIXTURE OF CLAY AND SAND
	ML-CL	MIXTURE OF SILT AND CLAY

PWR	PWR	PARTIALLY WEATHERED ROCK
	Rock	ROCK

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A1 Consulting Group, Inc.
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BORING LOG

117 International Drive
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Project Name: **Durham University Drive Culvert**
Boring Location: **Grass beside road, near culvert outlet**
Date Started: **July 21, 2011**
Date Completed: **July 21, 2011**
Drill Rig Type: **CME 75**
Hammer Type: **63.5 kg hammer**
Driller's Name: **Mario**
Drilling Method: **3.75 inch (Hollow Stem Auger)**

Boring Number: B1
Collar elevation: ~ 303 feet
Total Depth of Boring (ft): 15.5
Depth to Groundwater (ft) : ______.
Depth to Groundwater (ft) -24 hr.____.

A1 Geologist/Engineer: **B. Boyd .**

Elevation (ft)	Depth (ft)	Blow Counts 6 in.	6 in.	6 in.	N	Sample #	Moisture	Recovery %	Log	Description
303	0.0									2" grass, topsoil
302.5	0.5	3	2	3	5	1	Moist	100		
	1.0									
	1.5									Brown/Gray mottled mixed fill with mica; samples as silty sand
	2.0									
	2.5									
	3.0									
299.5	3.5	2	3	2	5	2	Wet	100		
	4.0									
	4.5									Same as above
	5.0									
	5.5									
297	6.0	2	1	2	3	3	S	100		
	6.5									
	7.0									
	7.5									Gray silty medium sand with mica
	8.0									
	8.5									
	9.0									
293.5	9.5	2	2	6	8	4	Wet	100		
	10.0									
	10.5									
	11.0									Same as above; (all fill to 10')
	11.5									
	12.0									
	12.5									
290	13.0	9	50/2"		100+					
	13.5									
	14.0									Brown/Orange mottle PWR sampling as silty fine sand; Very hard drilling @ 15'; refusal 15.5'
	14.5								PWR	
	15.0									
	15.5									
287	16.0									
	16.5									
	17.0									
	17.5									
	18.0									
	18.5									
	19.0									Rock
	19.5									
	20.0									
	20.5									
	21.0									
	21.5									

Drilling tereminated at: Auger Refusal / Predetermined depth: 15.5 feet

A1 Consulting Group, Inc.
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BORING LOG

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Project Name: Durham University Drive Culvert
Boring Location: Road
Date Started: July 20, 2011
Date Completed: July 20, 2011
Drill Rig Type: CME 75
Hammer Type: 63.5 kg hammer
Driller's Name: Mario
Drilling Method: 3.75 inch (Hollow Stem Auger)

Boring Number: B2
Collar elevation: ~ 306 feet
Total Depth of Boring (ft): 20
Depth to Groundwater (ft) : ______.
Depth to Groundwater (ft) -24 hr.____.

A1 Geologist/Engineer: B. Boyd .

Elevation (ft)	Depth (ft)	Blow Counts 6 in.	Blow Counts 6 in.	Blow Counts 6 in.	N	Sample #	Moisture	Recovery %	Log	Description
306	0.0									4" asphalt
305.5	0.5	2	4	16	20	1	Moist	100		Orange / black mottled (fill) sandy silt
	1.0									
	1.5									
	2.0									
	2.5									
	3.0									
302.5	3.5	1	1	0	1	2	Wet	50		Gray/ Brown silt (fill)
	4.0									
	4.5									
	5.0									
	5.5									
3	6.0	WH				3	Wet	30		Same as above
	6.5									
	7.0									
	7.5									
	8.0									
297.5	8.5	4	3	2	5	4	Wet	100		Same as above
	9.0									
	9.5									
	10.0									
	10.5									
	11.0									
	11.5									
	12.0									
	12.5									
	13.0									
292.5	13.5	3	3	10	13	5	Moist	100		3" saturated sand; 15" brown / black mottled sandy silt
	14.0									
	14.5									
	15.0									
	15.5									
	16.0									
	16.5									
	17.0									
	17.5									
	18.0									
287.5	18.5	50/4"				6	Dry	100		6" gray dry fine sand
	19.0									
	19.5									
	20.0									

Drilling tereminated at: Auger Refusal / Predetermined depth: 20 feet

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BORING LOG

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Project Name: **Durham University Drive Culvert**
Boring Location: **Front Parking Lot**
Date Started: **July 20, 2011**
Date Completed: **July 20, 2011**
Drill Rig Type: **CME 75**
Hammer Type: **63.5 kg hammer**
Driller's Name: **Mario**
Drilling Method: **3.75 inch (Hollow Stem Auger)**

Boring Number: B3
Collar elevation: ~ 308 feet
Total Depth of Boring (ft): 25
Depth to Groundwater (ft) : _______.
Depth to Groundwater (ft) -24 hr.____.

A1 Geologist/Engineer: **B. Boyd .**

Elevation (ft)	Depth (ft)	Blow Counts 6 in.	6 in.	6 in.	N	Sample #	Moisture	Recovery %	Log	Description
308	0.0									1" asphalt; 3" concrete
307.5	0.5	2	2	2	4	1	Moist	100		
	1.0									
	1.5									Brown silty sand with mica
	2.0									
	2.5									
	3.0									
304.5	3.5	2	1	3	4	2	Wet	100		
	4.0									
	4.5									Same as above
	5.0									
	5.5									
302	6.0	1	1	1	2	3	Wet	100		
	6.5									
	7.0									Brown clay sand with mica; plastic
	7.5									
	8.0									
299.5	8.5	2	1	2	3	4	Wet	100		
	9.0									
	9.5									
	10.0									
	10.5									Gray sandy clay plastic; minor fine mica
	11.0									
	11.5									
	12.0									
	12.5									
	13.0									
294.5	13.5	2	2	2	4	5	Moist	100		
	14.0									
	14.5									
	15.0									
	15.5									Gray silty sand, non plastic; saturated mud to 17'; harder drilling
	16.0									
	16.5									
	17.0									
	17.5									
	18.0									
289.5	18.5	50/6"				6	Moist	100		
	19.0									
	19.5									
	20.0									Gray silty sand with mica
	20.5									
	21.0									
	22.0									
	22.5									
285	23.0	50/3"								
	23.5									
	24.0									Same as above
	24.5									
	25.0									

Drilling tereminated at: Auger Refusal / Predetermined depth: 25 feet

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BORING LOG

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Project Name: Durham University Drive Culvert
Boring Location: Side Parking Lot
Date Started: July 20, 2011
Date Completed: July 20, 2011
Drill Rig Type: CME 75
Hammer Type: 63.5 kg hammer
Driller's Name: Mario
Drilling Method: 3.75 inch (Hollow Stem Auger)

Boring Number: B4
Collar elevation: ~ 310 feet
Total Depth of Boring (ft): 25
Depth to Groundwater (ft) : dry______.
Depth to Groundwater (ft) -24 hr. 8.4'

A1 Geologist/Engineer: B. Boyd .

Elevation (ft)	Depth (ft)	Blow Counts 6 in.	6 in.	6 in.	N	Sample #	Moisture	Recovery %	Log	Description
310	0.0									6" concrete; 2" stone
	0.5									
309	1.0	3	4	4	8	1	Moist	100		Brown silty sand - fill
	1.5									
	2.0									
	2.5									
	3.0									
306.5	3.5	3	2	3	5	2	Moist	100		Brown gray silty sand - fill
	4.0									
	4.5									
	5.0									
	5.5									
304	6.0	1	2	2	4	3	Moist	100		Dry sand with mica
	6.5									
	7.0									
	7.5									
	8.0									
301.5	8.5	2	2	2	4	4	Moist	100		Silty clay sand with mica 13'; harder drilling
	9.0									
	9.5									
	10.0									
	10.5									
	11.0									
	11.5									
	12.0									
	12.5									
	13.0									
296.5	13.5	50/4"			100+	5	Dry	100		Gray silty sand with mica - friable
	14.0									
	14.5									
	15.0									
	15.5									
	16.0									
	16.5									
	17.0									
	17.5									
	18.0									
291.5	18.5	50/3"				6	Dry	100		Gray / Black silty sand with mica friable; very hard drilling
	19.0									
	19.5									
	20.0									
	20.5									
	21.0									
	22.0									
	22.5									
287	23.0	50/2"				7	Dry	100		Same as above; Refusal broke machine in attempt to advance
	23.5									
	24.0									
	24.5									
	25.0									

Drilling tereminated at: Auger Refusal / Predetermined depth: 25 feet

A1 Consulting Group, Inc.
Consulting Engineers

BORING LOG

117 International Drive
Morrisville, NC 27560
(919) 469-4800

Project Name: Durham University Drive Culvert
Boring Location: Parking Lot Near Dumpsters
Date Started: July 20, 2011
Date Completed: July 20, 2011
Drill Rig Type: CME 75
Hammer Type: 63.5 kg hammer
Driller's Name: Lee
Drilling Method: 3.75 inch (Hollow Stem Auger)

Boring Number: B5
Collar elevation: ~ 314 feet
Total Depth of Boring (ft): 20
Depth to Groundwater (ft) : dry .
Depth to Groundwater (ft) -24 hr. 12'

A1 Geologist/Engineer: B. Boyd .

Elevation (ft)	Depth (ft)	Blow Counts 6 in.	6 in.	6 in.	N	Sample #	Moisture	Recovery %	Log	Description
314	0.0									8" concrete
	0.5									
313	1.0	7	7	6	13	1	Dry	100		Brown fill - silty sand with mica
	1.5									
	2.0									
	2.5									
	3.0									
310.5	3.5	6	8	6	14	2	Dry	100		Same as above
	4.0									
	4.5									
	5.0									
	5.5									
308	6.0	5	8	9	17	3	Dry	100		Brown / Orange silty sand with mica
	6.5									
	7.0									
	7.5									
	8.0									
305.5	8.5	17	30	50	80	4	Dry	100		Hard friable gray silty sand w mica
	9.0									
	9.5									
	10.0									
	10.5									
	11.0									
	11.5									
	12.0									
	12.5									
	13.0									
300.5	13.5	50/4"				5	Dry	100		Same as above; very hard, friable
	14.0									
	14.5									
	15.0									
	15.5									
	16.0									
	16.5									
	17.0									
	17.5									
	18.0									
295.5	18.5	50/3"				6	Dry	100	PWR	Partially weathered rock sampling as silty fine sand
	19.0									
	19.5									
	20.0									

Drilling tereminated at: Auger Refusal / Predetermined depth: 20 feet

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Consulting Engineers

BORING LOG

117 International Drive
Morrisville, NC 27560
(919) 469-4800

Project Name: **Durham University Drive Culvert**
Boring Location: **Rear Parking Lot @ Culvert**
Date Started: **July 21, 2011**
Date Completed: **July 21, 2011**
Drill Rig Type: **CME 75**
Hammer Type: **63.5 kg hammer**
Driller's Name: **Lee**
Drilling Method: **3.75 inch (Hollow Stem Auger)**

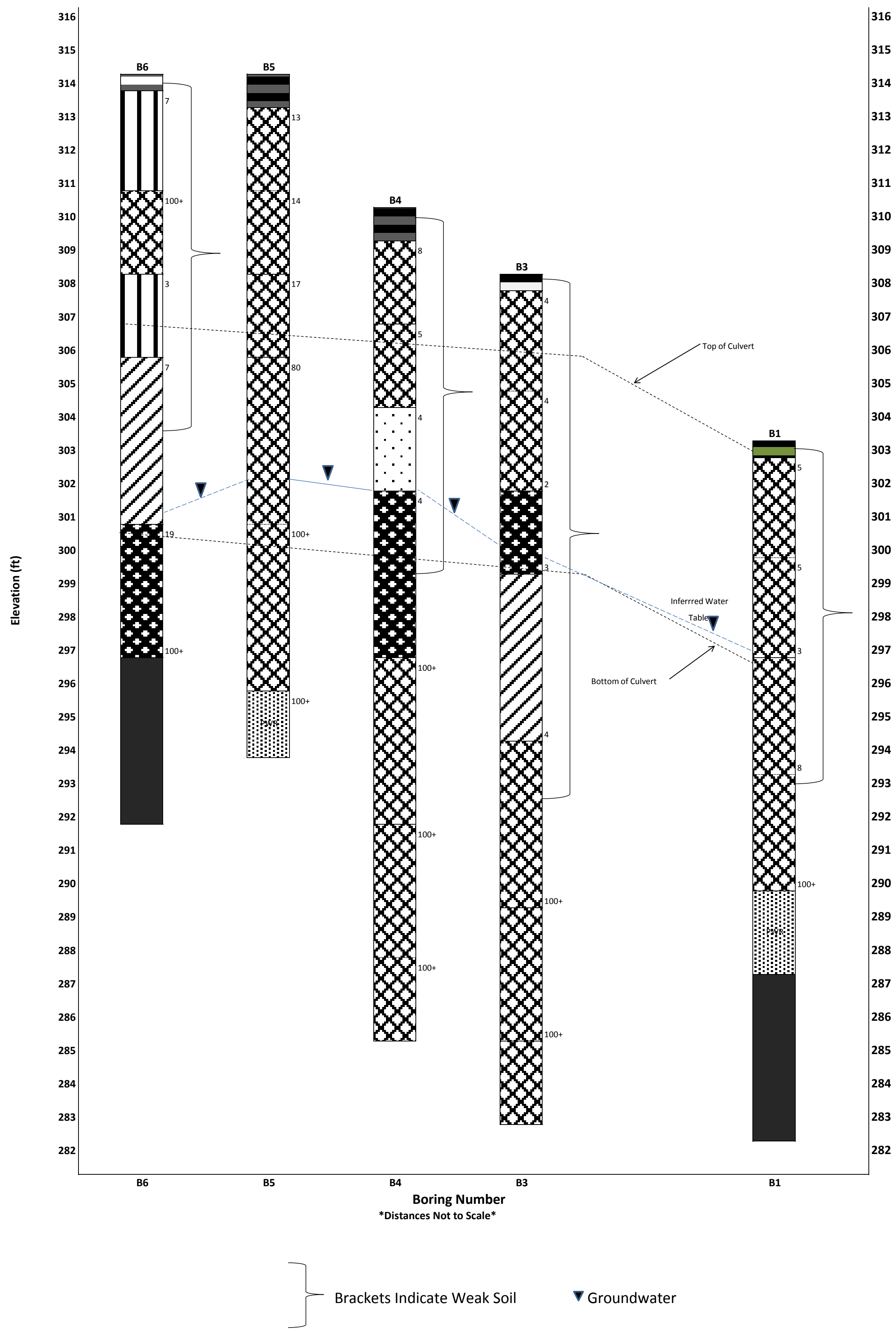
Boring Number: B6
Collar elevation: ~ 314 feet
Total Depth of Boring (ft): 17
Depth to Groundwater (ft) : 13 .
Depth to Groundwater (ft) -24 hr. 12'

A1 Geologist/Engineer: **B. Boyd .**

Elevation (ft)	Depth (ft)	Blow Counts			N	Sample #	Moisture	Recovery %	Log	Description
		6 in.	6 in.	6 in.						
314	0.0									6" concrete
313.5	0.5	4	4	3	7	1	Moist	100		Brown mixed fill- sand silt with mica
	1.0									
	1.5									
	2.0									
	2.5									
	3.0									
310.5	3.5	50/2"					Dry	100		Gray concrete (?), slighty silty sand with stones
	4.0									
	4.5									
	5.0									
	5.5									
308	6.0	2	2	1	3	3	Wet	100		Brown sandy silt plastic minor mica
	6.5									
	7.0									
	7.5									
	8.0									
305.5	8.5	2	3	4	7	4	Wet	100		Brown sandy clay, plastic abundant fine mica
	9.0									
	9.5									
	10.0									
	10.5									
	11.0									
	11.5									
	12.0									
	12.5									
	13.0									
300.5	13.5	6	10	9	19	5	Wet	100		Brown / gray clayey sand mixed cse and fine plastic; 17' hard rock - auger refusal
	14.0									
	14.5									
	15.0									
	15.5									
	16.0									
	16.5									
	17.0									
296.5	17.5									Rock
	18.0									
	18.5									
	19.0									
	19.5									
	20.0									
	20.5									
	21.0									
	21.5									
	22.0									

Drilling tereminated at: Auger Refusal / Predetermined depth: 17 feet

Subsurface Conditions That May Impact Construction



Rock Core B6

Run: 5’ Recovery: 85% RQD: 67.5% (fair rock)

Coring began at a depth of 17’

At the top of the core there is approximately 6” of river jack – rounded stream cobbles. Sue to the nature of the material it the exact length of core containing river jack cannot be determined.

Below the river jack is 35” continuous of gray fine to medium grain sound sandstone. There are no obvious joints or fractures.

After a break there is another 10.5” of the same material.

Rock Core B1

Run: 5’ Recovery: 80% RQD: 63% (fair rock)

Coring began at depth of 15.5’

At the top there is 15” of sound fine/medium grain sandstone with no obvious joints or fractures. Fracture occurs at change in composition.

Below the fracture there is 2” dark gray sandstone, ½” of the same, then 2” sandstone with vugs and non-displaced vertical fractures.

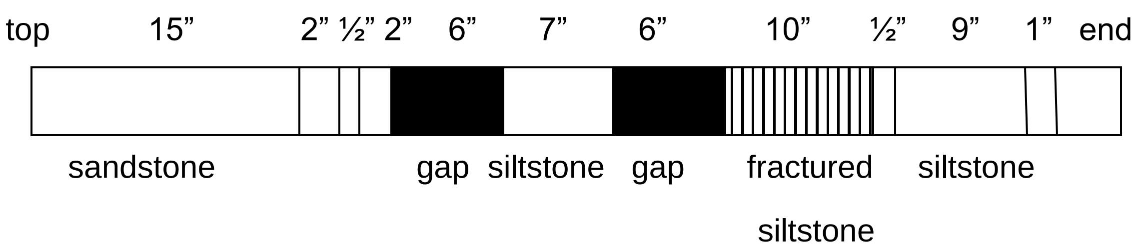
Below this is a 6” gap.

Below the gap begins 7” of brown siltstone with horizontal bedding and evidence of healed horizontal fractures and filled vugs.

Another 6” gap.

Below the second gap is 10” of brown siltstone with low angle fractures, 3 of which have no displacement. At the bottom of this section is a fracture and an approximately ½” section of the same material.

The core ends with 9” of brown siltstone with 1 displaced fracture.



1615.1.1 Site class definitions. The site shall be classified as one of the site classes defined in Table 1615.1.1. Where the soil shear wave velocity, $\bar{v}_s$, is not known, site class shall be determined, as permitted in Table 1615.1.1, from standard penetration resistance, $\bar{N}$, or from soil undrained shear strength, $\bar{s}_u$, calculated per Section 1615.1.5. Where site-specific data are not available to a depth of 100 feet (30 480 mm), appropriate soil properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official determines that Site Class E or F soil is likely to be present at the site.

1615.1.2 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects, shall be determined by Equations 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_s \qquad \textbf{(Equation 16-16)}$$

$$S_{M1} = F_v S_1 \qquad \textbf{(Equation 16-17)}$$

where:

F_a = Site coefficient defined in Table 1615.1.2(1).

F_v = Site coefficient defined in Table 1615.1.2(2).

S_S = The mapped spectral accelerations for short periods as determined in Section 1615.1.

S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1615.1.

1615.1.3 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1 second period, S_{D1}, shall be determined from Equations 16-18 and 16-19, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Equation 16-18)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Equation 16-19)}$$

where:

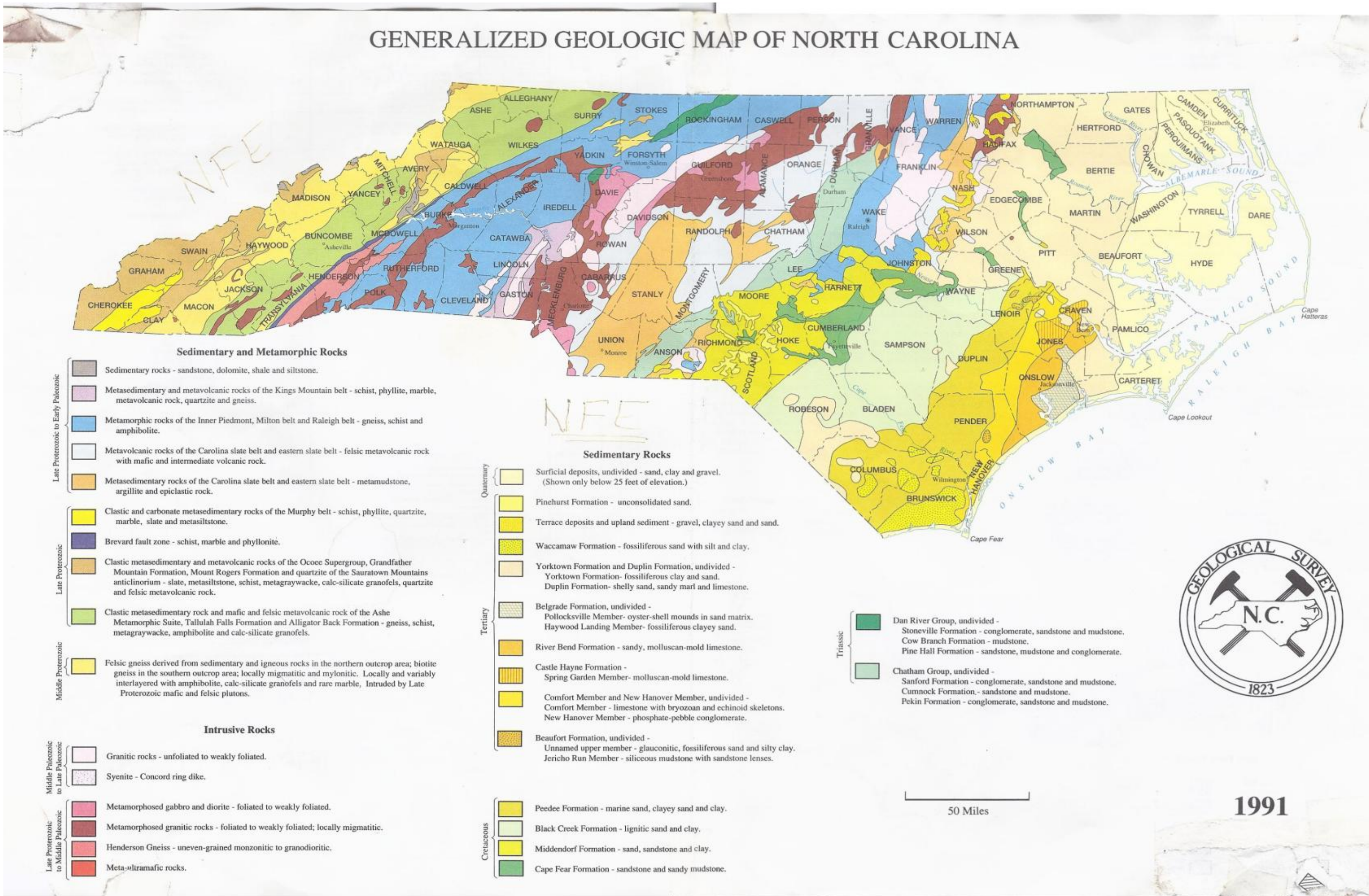
S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1615.1.2.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1 second period as determined in Section 1615.1.2.

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having 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 ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa.



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Morrisville, NC 27560

ACTIVE FORCE ON CULVERT WALL W/EARTHQUAKE FORCES

Mononobe - Okabe equations

$P_{ae} = \frac{1}{2}\gamma H^2(1-k_v)K_a'$ (Active force per unit length of wall)

where

$$K_a' = \frac{\cos^2(\phi' - \theta - \bar{\beta})}{\cos^2\theta \cos\bar{\beta} \cos(\delta + \theta + \bar{\beta}) \left\{ 1 + \left[\frac{\sin(\delta + \phi')\sin(\phi' - \alpha - \bar{\beta})}{\cos(\delta + \theta + \bar{\beta})\cos(\theta - \alpha)} \right]^{1/2} \right\}^2}$$

and

$\bar{\beta} = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$

Horizontal inertial force $= k_h W$
Vertical inertial force $= k_v W$
W = Weight of soil wedge

For vertical wall, $\theta = 0$ and assume $k_v = 0$ for no vertical force,

For horizontal backfill, $\alpha = 0°$ and $\delta = 0°$

For N.C., Zone 1 and 2A are used for Seismic activity and k_h is from 0.05 to 0.10.

Use $k_h = 0.10$ (Horizontal Acceleration Coefficient)

$\phi' = 30°$ (Conservative value for cohesionless soils)

$K_a' = 0.397$

$P_{ae} = \frac{1}{2}(118\text{ pcf})(8')^2(1-0)(0.397) \cong 1500$ lb/ft

$Z \cong 0.67H = 0.67(8') = 5.36'$ from bottom of culvert

A1 Consulting Group, Inc

CALCULATION SHEET

PROJECT University Dr. - Durham, NC

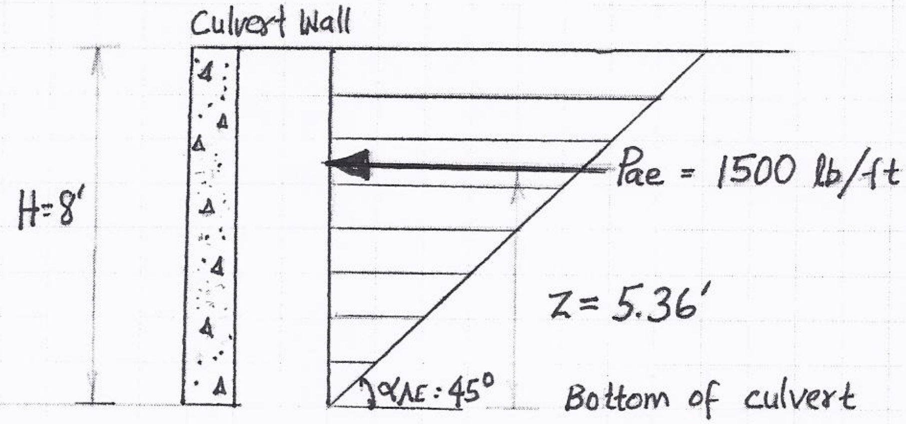
SUBJECT Seismic Active Pressure Diagram

PREPARED BY NS / JC

JOB NO. 10-070

DATE 8/21/11

SHEET NO. 2 OF 2



α_{AE} = Active Failure Plane Angle = 45°