H. C. NUTTING COMPANY
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS
SINCE 1921

CORPORATE CENTER
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
FAX (513) 321-0294

Keith Tom
Matt John G.
Charlie Cheryl
Jim G. Dan F
Brian
Tracy

April 21, 2005
(Revised April 19, 2005)

W.O. # 50043.009 skh

RECEIVED
APR 22 2005
BLA

Mr. Jim Gulick, P.E.
c/o Mr. John Greaney, P.E.
Bernardin, Lochmueller & Associates, Inc.
6200 Vogel Road
Evansville, IN 47715-4006

RE: **Report of Geotechnical Study**
Proposed 3-Span Bridge Replacement & Realignment
Along CR 375 W over Lick Creek
INDOT PROJECT NO. BRO-9959 ( )
DES. NO. 9982490
STR: ORANGE NO. 34
BLA Project No. 199-0047-0BD
Orangeville Township, Orange County, Indiana

Dear Mr. Greaney:

We are pleased to submit our report of the geotechnical study performed for the proposed 3-span bridge replacement carrying CR 375 W over Lick Creek as well as Log Creek Road (CR 375 W) realignment to straighten the roadway near the bridge location in Orangeville Township, Orange County, Indiana. This geotechnical study was performed in general accordance with our proposal dated February 20, 2004, a field check letter (revised proposal), dated March 5, 2004, and INDOT Materials & Tests Division's authorization on March 30, 2004. The scope of our service included review of available topographic information, engineering site reconnaissance, test boring layout, subsurface exploration and coordination, laboratory examination and testing of subsurface samples, engineering analysis and recommendations, preparation of this report, and preparation of geotechnical drawings per INDOT requirements.

We could also submit the geotechnical drawings in digital format through e-mails for designer use. The drawings were prepared in accordance with INDOT subsurface investigation requirements. Included with this report are copies of the geotechnical drawings plotted to half scale. These drawings include the following:

- Sheets 3A, 3B, and 3C: **Test Boring Location Plans** for the project.
- Sheet 4: **Bridge Boring Location Plan** for the project.

- Sheet 5A: **General Subsurface Condition at Bridge Structure** – The drawing provides a graphical profile of the TB test borings drilled for the bridge, along with the results of the laboratory test data performed on representative samples for each test boring.
- Sheet 5B: **General Subsurface Condition at Retaining Walls** – A graphical profile of the RW test borings drilled along the retaining walls, with the results of the laboratory test data performed on representative samples for each test boring.
- Sheets 6, 7, and 8: **General Subsurface Conditions at Station 91+64, 92+50, and 94+70** – The drawings provide the general subsurface profiles at the drainage structure, and the most critical embankment fill and MSE wall cross-sections for this project.

Included with this report are the plotted and tabulated laboratory test results, including the grain-size distribution test reports, Atterberg Limits, and the unconfined compression tests. Included with the laboratory test results is Table I "Classification Test Data", which includes an entire listing and table of the classification tests (including some Atterberg tests) completed on representative samples, Table II "Moisture Content", and Table III "Tabulation of Undisturbed Data". The classification test data are also shown on the individual test boring logs. In addition, Table IV "Summary of Hand Auger Sounding" is also attached.

H. C. Nutting Company appreciates the opportunity of providing our professional geotechnical services to Bernardin, Lochmueller & Associates and Orange County Office for this project. Please contact us if you have any questions concerning this study, or if we may be of further assistance to you as the project develops into the final design and construction phases.

Thank you for your consideration.

Respectfully submitted,
H. C. NUTTING COMPANY

Peter Chou (pld)

Peter W. Chou, P.E.
Geotechnical Engineer

Swaminathan Srinivasan

Swaminathan Srinivasan, P.E.
Chief Geotechnical Engineer

Mark Lockwood/ss

Mark Lockwood, P.E.
Senior Consultant

\\CINCY\DATA\PROJECTS\6-Engin\50\50043\009\INDOT111904-Revised.doc

REPORT OF
GEOTECHNICAL STUDY

PROPOSED 3-SPAN BRIDGE REPLACEMENT & REALIGNMENT
ALONG COUNTY ROAD 375 W OVER LICK CREEK
INDOT PROJECT NO. BRO-9959 ( )
DES. NO. 9982490
STR: ORANGE NO. 34
BLA PROJECT NO. 199-0047-0BD
ORANGEVILLE TOWNSHIP, ORANGE COUNTY, INDIANA

FOR

BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.
6200 VOGEL ROAD
EVANSVILLE, INDIANA 47715-4006

AUGUST 2004
(REVISED APRIL 2005)

H. C. NUTTING COMPANY
EMPLOYEE OWNED

TABLE OF CONTENTS

PAGE

APPENDIX

SHEET 1 GENERAL SITE PLAN
SHEET 2 SITE VICINITY MAP (U.S.G.S.)
SHEET 3 TEST BORING LOCATION PLAN
SHEET 4 BRIDGE BORING LOCATION PLAN
SHEET 5 GENERAL SUBSURFACE CONDITIONS AT BRIDGE STRUCTURE
SHEETS 6 TO 8 GENERAL SUBSURFACE CONDITIONS
SHEET 9 TYPICAL CONFIGURATION OF MICROPILE AT BRIDGE ABUTMENT
BORING TERMINOLOGY
INDOT SOIL CLASSIFICATION
TEST BORING LOGS (11)
CLASSIFICATION TEST DATA
NATURAL MOISTURE CONTENT DETERMINATION
TABULATION OF UNDISTURBED DATA
SUMMARY OF SOUNDINGS
LABORATORY TEST DATA
ENGINEERING ANALYSES
SPECIAL PROVISION OF MICROPILE FOUNDATION

SUMMARY OF GEOTECHNICAL RECOMMENDATIONS

Soil and Rock Conditions

In general, the encountered overburden alluvial soils near the creek were either soft or loose and extended below a depth of about 12 feet (or to the top of bedrock surface). The thickness of soft/loose soils generally increased toward the creek. In addition, the near-surface reddish brown silty clay and clay had relatively high plastic indices (PI) of over 25 (up to 48) percent. These clay soils (with higher PI) are highly plastic and have moderate to high swell and shrinkage potential if exposed to water (moisture).

With the relatively high water level near the creek and the presence of soft and loose alluvial (permeable) soils, the evidence (sinkholes) of karst geology was observed within the project vicinity. Even though we did not encounter any underground cavity, the recovered rock core samples were solutionable, calcareous, fine-grained limestone and its dissolution feature has potential to continue developing sinkholes and/or underground cavities within the project area. In addition, based on the review of rock core samples, significant amount of fractured zones were observed within the core samples, but the amount of fractures/joints typically decreased with depth.

Geotechnical Assessment

Limestone bedrock is present at a depth of about 10.5 to 12.5 feet below existing grade, and the rock surface is observed to be relatively flat within the bridge location. Several foundation options including spread footings, drilled shafts and driven piles were considered. Based on our past experience on transportation projects in karst geology and discussion with BLA and INDOT, we anticipate that Micropiles would be the most practical foundation system for this 3-span bridge project.

We also performed global stability analyses for the originally proposed MSE wall under long-term conditions and calculated a factor of safety (FOS) of 1.09, which is below the minimum required FOS of 1.3. Therefore, instead of MSE walls, these abutment walls should be designed as cast-in-place reinforced concrete retaining walls and incorporated with the proposed bridge foundation as shown in Sheet 9. With the advantages of minimum overall undercut depth, readily modified pile diameter and length based on field condition, and high capacity into limestone bedrock, we understand that the option of new bridge abutments, interior piers, and retaining walls supported on Micropiles may be the most economical and practical option.

Bridge Foundation

For design purposes, Micropiles are usually assumed to transfer their load to the ground through grout/ground skin friction (bond), without any contribution from end bearing. The advantage of this assumption is that the pile movement needed to mobilize frictional resistance is significantly less than that needed to mobilize end bearing. For piles bearing into limestone, even through pile movement is relatively small, Micropiles can still mobilize the majority of its calculated skin friction during the loading process.

Based on the review of geological setting, and our split-spoon soil and rock core samples, it is our opinion that the skin friction within the entire overburden soil zone should be neglected. Also, negative skin friction within overburden soils discussed in Section 4.2.3 should be considered in the pile capacity design. We anticipate that the proposed 3-span bridge and its abutment walls can be supported on Micropiles bearing into limestone bedrock, with a minimum bond length (rock embedment length) of 10 feet.

Based on the preliminary configuration of the proposed abutment (cast-in-place reinforced concrete walls), we anticipate that Micropiles would be installed prior to any new fill placement and retaining wall construction. Therefore, the Micropile design capacity should also include potential downdrag loads developed within overburden soils due to new fill placement.

Abutment Retaining Walls

Two sets of cast-in-place reinforced concrete retaining walls, which will be supported by Micropiles, are considered as part of the bridge replacement project. The new abutment walls will retain the proposed embankment fill, up to 15 feet above existing grade. The resulting lateral earth pressure will be a function of wall type, traffic, backfill material type, and type of restraint at the top. The new abutment wall will also be supported on drilled Micropiles. The drilled Micropiles can be designed to resist sliding and overturning as outlined in the Sections 4.2.2 and 4.2.3 of this report.

Embankment Construction Considerations

Up to 17 feet of permanent embankment fill is planned on the left and right sides of the existing embankment at station 91+64, Line "B". However, very soft or loose silty loam soils (compressible layer) were encountered to a depth of about 12.5 feet (Elevation 507.5) below existing grade at Boring TB-5. The upper silty soils were easily disturbed by water, and therefore, we recommend that 3 feet of partial undercut below the structure (# 12) invert level, or to the top of friable limestone bedrock, and replacement with granular engineered backfill. With a 3 feet partial undercut option within the new embankment fill subgrade, we anticipate that the consolidation settlement would reduce to about 1 inch. We recommend settlement plates and stakes be installed at 100 feet center to center intervals, and monitored along the proposed embankment between these stations in accordance with Section 204.03(a) of the INDOT Standard Specifications. We recommend that a minimum period of time of 4 weeks be established after the completion of embankment fill prior to any new pavement construction.

With the construction of the proposed 2H to 3H: 1V slope between Station 91+00 and 92+50, Line "B", the minimum FOS against global stability was calculated as 1.09 and 1.22 under the loading conditions without and with undercut, respectively. The factor of safety of the long-term proposed condition falls below the acceptable global stability evaluation. Therefore, based on the review of the analyses, we recommend that the east side (Rt.) of the proposed embankment slope between Station 91+00 to 92+50, Line "B" be reinforced with geogrid from Elevation 510 to 516, with a 2 feet vertical spacing (approximate 4 layers of geogrid required).

The geogrid should extend approximately 25 to 30 behind the slope surface and beyond the potential failure surface. It is critical that the geogrid (such as BX1100) is placed in strict accordance with Tensar's guidelines, especially orienting the geogrid in the strong direction. The geogrid should be placed edge to edge (no overlap) with single, continuous pieces used (no splicing in the long direction). In addition, careful monitoring of the geogrid should be made during construction to confirm the grid is not cut or damaged by the granular embankment fill or heavy equipment. General geogrid construction considerations are summarized in the report text.

Pavement Design Considerations

The majority of the project area will be a fill condition. Based on the preliminary Bridge Plans prepared by BLA, the Average Annual Daily Traffic (AADT) on CR 375W was about 600 Vehicles Per Day (VPD) in 1996, and projected to be about 660 VPD by Year 2020. Based on Subgrade Treatment Recommendation Options prepared by INDOT, Materials and Tests Division, dated August 10, 2004, the subject roadway has a traffic volume projection of (AADT) less than 3,000 VPD, but more than 500 VPD. Therefore, we recommend the subgrade treatment "Type II" be used for pavement design for this realignment project. An estimated California Bearing Ratio (CBR) value of 2.5 (Subgrade Resilient Modulus, M_R = 3, 000 psi) at 95% compaction could be used for the pavement design in the fill and at-grade areas.

Earthwork

We recommend that any soil used as engineered fill should have a Plasticity Index of 25 or less and contain rock fragments less than 4 inch in maximum dimension. All material types which will be used as engineered fill must be tested in the laboratory to determine its project suitability and its compaction characteristics. We anticipate moderately expansive clays (CH), such as encountered at RB-1, with a Plastic Index of over 30 may be encountered during exploration and may swell during soaking. Water is the catalyst that causes the shrinkage and swelling. The on-site clays, which had PI ranging between 26 to 48 and were classified as A-7-6 (38), should not be re-used as engineered fill for roadway embankment due to its sensitivity to moisture and difficulty in achieving the design compaction.

1.0 INTRODUCTION

1.1 Project Identification

Orange County (Indiana) plans to demolish the existing two-span, encased steel beam bridge (No. 34) on County Road 375W (Log Creek Road). The existing bridge carries CR 375W over Lick Creek between US 150/SR 56 and CR 25S, located approximately 2 miles west of Paoli, Orange County, Indiana. The proposed project consists of the construction of the new bridge and realigned roadway embankment. It was decided during our latest conversation that the new 3-span bridge may have cast-in-place concrete wingwall abutments on the both sides. The project begins approximately 1,500 feet south of US 150, travels north crossing Lick Creek, and ends at US 150/SR 56. This project extends from Station 85+50 to Station 99+87, Line "B", consequently with a total length of about 1,437 feet.

The roadway project is identified as INDOT Project No. BRO-9959 ( ), and Designation No. 9982490. The extent and location of the proposed construction is shown on the General Site Plan (Figure 1) and Site Vicinity Map (Figure 2) included in the Appendix.

1.2 Project Description

Orange County plans to replace the existing approximately 80 foot long, 14 foot wide bridge with a 3-span, prestressed concrete I-beam bridge, located approximately 100 feet south of the existing bridge along the new alignment at Station 93+69, Line "B". We also understand that Orange County plans to straighten and realign the existing Log Creek Road at the existing bridge location toward the east (Line "B"). The proposed prestressed I-beam bridge will have spans of 60 feet, 80 feet, and 60 feet, yielding a 200 feet long bridge.

Based on the preliminary Bridge Plans, the proposed structure was originally designed with MSE wall abutments at both end bents, however, cast-in-place concrete retaining walls are currently proposed at the bridge abutment based on the encountered soil/rock conditions. The existing creek channel slopes near the proposed interior piers (#2 and #3) should be protected. We understand that the protected slopes at the existing creek bank (near new interior piers) are proposed to be 2H: 1V.

Based on our past experience in the project vicinity (near Paoli, Indiana), relatively shallow limestone bedrock will likely be encountered. Reportedly, solutioned limestone voids and sinkholes were occasionally encountered near the Town of Paoli, Indiana to the west. Based on our field observations, we anticipate limestone bedrock may be encountered as shallow as 5 to 15 feet below the ground surface at this site (exposed at the creekbed). In addition, **during our recent field check, sinkholes (due to karst geology) with a maximum drop depth of 4 to 5 feet were observed near the project vicinity (south of Abutment #4).**

Proposed Bridge and Abutment Walls

It is anticipated that the new bridge will be supported on deep foundations bearing in limestone bedrock. Based on our discussion with Bernardin, Lochmueller & Associates (BLA) and INDOT, we anticipate that the proposed bridge deep foundations may also support both abutment walls which will retain new embankment fills. The proposed height of abutment walls are up to 15 feet, with a total wall length of about 126 and 108 feet at abutments #1 and #4, respectively.

Proposed Roadway Embankment and Drainage Structure

Significant new fills, generally ranging between 10 to 17 feet, are planned near the proposed bridge approaches and within the new roadway embankment sections. The realignment section generally starts from Station 90+50, Line "B", and ends at the intersection of US 150/SR 56. In addition, approximately 17 feet of new embankment fill is proposed above the proposed 4' x 4' x 120' box culvert (Structure #12) at Station 91+64, Line "B".

Hydraulic Information

According to the preliminary Bridge Plans prepared by BLA, the design 100 year flood discharge of 11,000 cfs will result in a 100 Year Flood Elevation of 523.5 feet, and the design velocity through the proposed structure is 8.74 ft/sec. Based on information on the preliminary bridge plans, the design 500 Year Low Scour Elevation at this structure is 492.72 feet. However, based on our conversation with INDOT, we understand that the provided scour elevations were estimated and subjected to change according to the soil/rock encountered at the bridge location. We anticipate that the 500 year Low Scour Elevation should not be lower than top of competent limestone bedrock elevation.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

Our test boring program was approved by INDOT on March 30, 2004. A total of eleven (11) borings were drilled following INDOT guidelines for soil subgrade investigations "Exhibit C" and "General Instruction for Bridge Structure Investigations" between April 9 and 12, 2004. We drilled four (4) bridge borings, TB-1 through TB-4, at the end bents and interior piers of the proposed bridge (Orange County No. 34) to a depth of about 22 to 30 feet below existing grade. Due to the presence of shallow limestone and siltstone, a minimum of 10 to 15 feet of rock core was obtained at each bridge boring.

A total of three (3) retaining wall borings (RW) were also drilled along the proposed retaining wall alignment to a minimum depth of 1.5 times the proposed wall height or the depth of competent unweathered bedrock, approximately 14 to 25 feet below existing grade. Approximately 5 to 11.1 feet of rock cores were obtained at each RW boring. The spacing between "RW" borings was generally less than 100 feet per INDOT requirements.

Three (3) roadway borings (RB) were drilled to a depth of 7.5 to 10 feet below existing grade to evaluate the suitability of the placement of new roadway embankment. One relatively deep boring (TB-5) was also performed at the new drainage structure location to a depth of 19 feet (including a 5 foot rock core). This boring would also be used to evaluate the placement of 17-foot-high embankment fill at the drainage structure.

Five (5) hand auger soundings along the proposed new alignment (Line "B") were performed to evaluate the thickness of unsuitable soils at the proposed embankment fill and drainage structure locations. The hand auger soundings were performed to a depth of 1.5 to 3.5 feet below existing grade.

The test boring locations were selected by HCN, in consultation with INDOT and BLA, during the preparation of the proposal and after our site reconnaissance (field check). The revised boring program was approved by INDOT on March 30, 2004. The borings and soundings were located in the field by HCN using a standard measuring tape by station and offset in reference to the roadway centerline per the Preliminary Bridge Plans. All test boring locations are shown on the drawings (Sheet 3A through 3C) in the Appendix. The bridge boring locations are shown on Sheet 4. Ground surface elevations at the boring locations were surveyed by standard level methods using one of BLA's project reference points. The elevations are also shown on the boring logs in the Appendix.

2.1 Split-Spoon Sampling

The test borings were performed with an ATV-mounted drill rig and advanced with hollow-stem augers between April 9 and 12, 2005. The test borings were all extended through existing embankment fill, topsoil, and terminated in natural overburden soils or bedrock. The subsurface exploration of the overburden soils consisted of split-spoon sampling. The drilling operations within soils were performed in general conformance with "Penetration Test and Split-Barrel Sampling of Soils", AASHTO T 206-87. Representative soil samples were obtained by means of the split-barrel sampling procedures at 2.5 feet intervals within the entire overburden soil zone. The number of blows required to drive the sampler 12 inches with a 140 pound hammer falling 30 inches, after an initial seating 6 inches, is termed the Standard Penetration Test (SPT) N-values. The SPT N-values are shown on the boring logs.

Rock coring was also performed in general conformance with AASHTO T-206 practices. Rock coring was performed with an NX core barrel, which yields a nominal 2-inch diameter rock core. The rock core was classified using generally accepted engineering geology methods, and the rock core recovery and RQD were measured. The total length of rock core, divided by the length of the run, is referred to as rock core recovery, and is expressed as a percentage. The Rock Quality Designation (RQD) is a measure of the rock mass quality, and is defined as the total length of intact rock core pieces 4 inches or more in length, divided by the length of the rock core run, also expressed as a percentage.

Five (5) hand auger soundings were performed using a 1-inch diameter hand-operated auger to evaluate the thickness of unsuitable materials for removal. The soundings were performed through the surficial unsuitable soils to depths of about 1.5 to 3.5 feet. All the soils at sounding locations were visually classified in the field. No soil samples were recovered for lab classification.

Water level observations were made by the drill foreman in the boreholes during and upon completion of the drilling operations. Some relatively shallow borings, such as RB-1 and RB-3, were backfilled upon completion of drilling. Seven (7) 24-hour groundwater readings after completion of drilling were measured at the deeper boreholes in order to provide long-term groundwater information to aid design recommendations. However, due to the presence of circulating water used during rock coring, at-completion and 24-hour water reading at the borehole may not be reliable, if rock coring was performed. The water level observations are shown on the boring logs in the Appendix.

After drilling operations were complete, and the necessary groundwater information had been obtained, boreholes were bentonite grouted or backfilled with natural soil cuttings or sands, and grout plug at the top of the holes, following INDOT Aquifer Protection Guideline, revised October 30, 1996.

2.2 Laboratory Testing

Upon completion of drilling, all samples obtained in the borings were returned to our Soil Mechanics Laboratory. Each sample was first visually classified by the Project Geotechnical Engineer, in accordance with AASHTO classification system and INDOT Exhibit "C", Requirements for Geotechnical Investigations and Pavement Investigation, revised January 1997. Laboratory tests were performed on selected representative soil and rock samples to provide specific data to aid in evaluating strength, moisture and density characteristics, and to aid in classifying and characterizing the recovered soils and rock. The laboratory testing program consisted of natural moisture content determinations (including both oven-dried and microwave-dried samples), grain size analyses with hydrometer readings, Atterberg Limits determinations, pH, and unconfined compression tests on soil and rock core samples.

The compressive strength test on the rock core specimen was performed in general accordance with ASTM D 2938, Unconfined Compressive Strength of Intact Rock Core Specimens. The results of the laboratory tests are shown on the logs and the tables in the Appendix.

A detailed log of each test boring was prepared by the Project Geotechnical Engineer based on the laboratory examination, laboratory test results, and the drill foreman's field notes. The test boring logs were prepared in INDOT format and are presented in the Appendix.

3.0 EXISTING SUBSURFACE CONDITIONS

The subsurface profiles at the proposed bridge (TB) and retaining wall (RW) locations have been graphically presented on Figures 5A and 5B, respectively, and attached to this report. The graphic subsurface profile at the bridge location illustrates the depth of existing embankment fill, natural overburden soils, and bedrock, Standard Penetration Test (N-values) results, moisture content, rock core recovery and RQD, and observed groundwater level. The specific descriptions of the embankment fill and overburden soils/bedrock are provided on the test boring logs and the following sections.

3.1 Geology of the Site

The regional geologic map for the Vincennes sheet, dated 1970, maps the project area as mainly Mississippian age fine-grained limestone of the Blue River Group. The thickness of the unconsolidated deposit was minimal, ranging from 0 to 50 feet within the project area. The unconsolidated deposits are mapped as part of recent alluvium consisting of mostly silt, sand, and gravel of the Martinsville Formation in Indiana. This formation also includes some colluvial and paludal deposits. The Soil Survey of Orange County Indiana, published by Soil Conservation Service, indicates that overburden soils in the project vicinity may have low to moderate shrink-swell potential. Based on our past experience within the project vicinity in Orange County, relatively shallow, solutionable, vuggy limestone is typically the first member of bedrock encountered.

Based on the maps of Southwestern Indiana Showing Areas Underground Mined for Coal, dated 1981 and 1985, underground or surface coal mines were not indicated within the project area.

3.2 Investigational Findings and Observations

At the existing pavement area (RB-1), approximately 6 inches of asphaltic concrete pavement was placed over about a 2 to 3-inch thick crushed aggregate base. At the remaining boring locations, about 6 to 30 inches of brown and dark brown clayey topsoil were encountered at the surface. Below the existing pavement at Boring RB-1, medium stiff silty clay embankment fill was encountered to a depth of about 2.5 feet below existing grade. Beneath the surficial pavement, topsoil, and/or fill materials, natural alluvial sandy to silty soils, such as silt, silty loam, and sandy loam, were first encountered to depths of about 5 to 12.5 feet below existing grade near existing creek.

The encountered alluvial soils were generally soft (cohesive) or wet and loose (granular) consistency. The thickness of the alluvial soils increased significantly toward the creek. The lower overburden strata consisted of cohesive silty clay, clay, and silty clay loam and loam soils (with occasional limestone fragments), underlain by weathered (friable) limestone bedrock at a depth of about 8.7 to 13.5 feet below existing grade.

The upper 0.3 to 4.5 feet of very highly weathered limestone was friable and contained a significant amount of clay-filled joints or fractures. Below the 1 to 4.5 foot thick weathered limestone, gray limestone was encountered to the explored depth of 14 to 30.1 feet at the test borings. At two (2) of the eleven test borings (TB-1 and TB-2), brown and gray siltstone was encountered below a depth of about 20 to 24 feet to the explored depth of 22 to 27 feet. In general, the encountered bedrock consists of upper brown and gray friable and fractured limestone, lower gray, hard, fine-grained limestone, and occasional underlying siltstone layers. Bedrock was not encountered at RB borings due to the relatively shallow explored depth.

The soils were classified in the laboratory per INDOT Classification System. The following is a description of the pertinent physical characteristics of each major stratum encountered in this exploration in order of increasing depth below existing grade.

3.2.1 Existing Pavement and Embankment Fill

At Boring RB-1, approximately 6 inches of asphaltic concrete pavement was encountered immediately above a 2-inch thick crushed limestone base. Below the pavement materials, brown and reddish brown silty clay fill was encountered to a depth of about 2.5 feet below existing grade. The recovered cohesive fill sample was moist and of very soft to medium stiff consistency. The tested sample had a moisture content of 37 percent, with pocket penetrometer readings (indicative of the approximate unconfined compressive strength) of 1.75 tsf. An Atterberg limit test on the A-7-6 silty clay fill sample indicated a liquid limit (LL) of 77, a plastic limit (PL) of 29, and a plastic index (PI) of 48.

3.2.2 Upper Cohesive Alluvial Soils

Underlying the pavement, embankment fill, or organic topsoil, natural alluvial soils, consisting of silty clay, clay, silty clay loam, silt (cohesive), silty and sandy loam, and loam, were encountered to a depth of about 2.5 to 12.5 feet below existing grade at the borings. The thickness of the encountered alluvial soils increased, up to 12 feet, toward the existing creek. In the areas "relatively" farther from the creek, an approximate 1.7 to 5.2 foot thick alluvial soil stratum was revealed by the exploration.

The cohesive alluvial soils were generally very soft (near the creek) to medium stiff, with pocket penetrometer readings that ranged from 0.25 to 2.75 tsf. The SPT N-values ranged from less than hammer weight (less than 1) to 9 blows per foot. The lower alluvial soils at borings RW-2 and RW-3 had N-values of in excess of 50 blows per foot due to the occasional presence of rock fragments/limestone floaters. Oven-dried moisture contents within the tested samples ranged from 18 to 31 percent. Microwave-dried moisture content tests were also performed on several samples and indicated a moisture content range between 18 and 26 percent.

Three Atterberg limit tests on A-6 and A-7-6 soils indicated liquid limits (LL) of 29 to 30 and 56, plastic limits (PL) of 16 to 18 and 22, and plastic indices (PI) of 12 to 13 and 34, respectively. Several Atterberg limit tests on A-4 soils indicated liquid limits (LL) of 19 to 28, plastic limits (PL) of 17 to 20, and plastic indices (PI) of 2 to 10.

Five unconfined compressive strength tests were performed within the upper alluvial strata. The majority of unconfined compressive strengths (ranged between 0.28 and 1.88 tsf) of test samples were below 1 tsf, with dry densities of about 99 to 115 pcf.

The tested A-6 and A-7-6 soil samples had about 41 (A-6) to 98 (A-7-6) percent of fines passing the No. 200 sieve size based on grain size analysis. In addition, the tested silt, silt loam, and loam samples (A-4) had about 79 to 97 percent of fines passing No. 200 sieve size.

3.2.3 Cohesionless Alluvial Soils

Below the cohesive alluvium, natural cohesionless alluvial soils, generally 2 to 3 foot thick silty and sandy loam, were encountered at depths of about 7.5 to 12.5 feet at test borings TB-1 and RW-3. The cohesionless silty loam was generally very moist to wet, and had a very loose compactness, with SPT N-values generally less than 3 bpf. Moisture content of a tested sample was 24 percent. Two Atterberg limit tests on silty loam samples indicated liquid limits (LL) of 21 to 22, plastic limits (PL) of 15 to 18, and plastic indices (PI) of 4 to 6.

This stratum contained frequent silt and sand seams, was very moist to a depth of about 10 feet below existing grade, and became wet and saturated below that depth at the borings.

One sieve analysis was performed on the selected cohesionless silty loam and sandy loam samples at boring RW-3 and TB-1, respectively. The amount of sand was about 37 percent, and it had 63 percent of fines passing the No. 200 sieve in the silty loam sample. The sandy loam was 59 percent sand and 41 percent fines passing the No. 200 sieve.

3.2.4 Lower Overburden Soils above Bedrock

"Relatively" high strength overburden soils were only encountered at the test borings farther from the existing creek, such as silty clay and clay encountered at RB borings and loam encountered at RW-1. These lower, over-consolidated clay soils were encountered at depths of about 5 to 10 feet below existing grade at RB borings and RW-1. The over-consolidated clay soils were generally medium stiff to very stiff, with pocket penetrometer readings that ranged from 1 to 2.5 tsf (SPT N-values ranged from 9 to 26 blows per foot). Occasional rock fragments were commonly observed within this lower clay stratum, which may result in SPT N-values in excess of 50 blows. Moisture contents within the tested samples ranged from 19 to 31 percent. Microwave dried moisture content tests indicated a moisture content between 26 and 31 percent. Atterberg limit tests on an A-7-6 clay and an A-4 Loam indicated liquid limits (LL) of 49 and 25, plastic limits (PL) of 23 and 15, and plastic indices (PI) of 26 and 10.

An unconfined compressive strength test was performed on Sample 3/SS recovered from Boring RB-2. The tested clay sample had an unconfined compressive strength of 1.78 tsf, with a dry density of 101 pcf. In addition, a tested loam sample had about 60 percent of fines passing the No. 200 sieve size based on laboratory grain size analysis.

3.2.5 Bedrock

A total of nineteen (19) 5-foot rock cores were obtained at depths of about 9 to 30.1 feet in TB and RW borings. The recovered bedrock core samples consisted of fine-grained, calcareous limestone and interbedded siltstone. The upper 1 to 4 feet of encountered bedrock was brown and gray friable and fractured limestone, and occurred in 1 to 6 inch pieces. The degree of fracturing of limestone bedrock decreases while the rock competency increases with depth. Below the fractured zones, gray, calcareous, stylolitic, very hard limestone containing occasional open and high angle joints, fractured zones, and interbedded siltstone seams (see attached test boring logs) was encountered to the explored depths of 14 to 30.1 feet. In addition, gray, calcareous, hard siltstone was encountered at deeper depths of about 20 to 27 feet in borings TB-1 and TB-2. The limestone comprised approximately 90 percent of the total rock matrix.

The majority of the rock cores obtained deeper in the profile was well cemented and well distributed in 6 to 30 inch pieces. The rock cores had rock quality designations (RQD) ranged from 0 (friable bedrock sample) to 88 percent, with a recovery of 94 to 100 percent. The calculated RQD generally increased with depth.

The elevation of the encountered fractured (or friable) bedrock, and top of competent limestone and siltstone are summarized in order of increasing station number (from west to east) in the table below:

TOP OF ENCOUNTERED BEDROCK ELEVATION

TEST BORING NO.	EXISTING SURFACE ELEVATION (FT.)	TOP OF FRACTURED ROCK ELEVATION (FT.)	TOP OF COMPETENT LIMESTONE ELEVATION (FT.)
TB-5	520.0	507.5	505.0
RW-1	518.1	508.1	504.6
RW-2	523.1	514.4	514.1
TB-1	517.9	507.4	505.9
TB-2	517.3	506.3	505.3
TB-3	518.2	505.7	501.2
RW-3	519.8	506.3	504.3
TB-4	518.2	505.7	502.7

In general, the bedrock surface mimics the existing ground surface contour and slopes toward the existing creekbed. At the location of the proposed bridge, competent bedrock was generally encountered at elevations from 501 to 506. Based on the recovery and RQD per INDOT Exhibit "C", the encountered bedrock appeared to have poor to good quality (except the friable and fractured zones). In terms of bedrock hardness, the limestone and siltstone were rated as hard to very hard. Also note that circulating water of rock core was lost at about 13 feet below existing grade in Boring RW-2 during rock coring, which suggests the possible presence of underlying open joints or cavities at that depth.

Sixteen (16) uniaxial compressive strengths were performed on the recovered rock core samples. Uniaxial compressive strengths on tested rock samples ranged between 202 and 921 tsf, and generally increased with depth.

3.2.6 Groundwater Conditions

Water level observations were made during and at the completion of the test borings. Because of the relatively shallow explored depths and the short duration (24 hours or shorter) that the boreholes were open at RB borings, no water (dry conditions) was observed during drilling and at-completion of drilling. In addition, during drilling, the borehole at most of the TB and RW borings was reported as in a dry condition. An exception to this was at Boring TB-4 where seepage water was observed at a depth of about 12.5 feet (Elevation 505.7) below existing grade. A "dry" condition is reported when no water is observed on the drilling tools and no water accumulation is observed in the open boreholes.

However, at completion of TB and RW borings, water was observed at a depth of about 8 to 13 feet below existing grade in seven (7) of the eight borings. A 24-hour groundwater reading was performed at seven test boring locations. The 24-hour groundwater levels were observed at depths of 8 to 10.5 feet, approximate elevations 507.3 to 512. Note that circulating water was used during the rock coring operations, and therefore, the at-completion and 24-hour groundwater readings may not be reliable.

In fine-grained, relatively impermeable soils such as the encountered silty clay and clay (A-6 and A-7-6) at this site, the water levels in the boreholes often are not representative of the actual groundwater level because the boreholes remain open only for a relatively short time. To obtain long-term measurements, it is necessary to install water level observation wells or piezometers. Based on our past experience, seepage and perched water was commonly observed at the soil/rock interfaces.

Based on the water level observations, we anticipate the groundwater level at the bridge location at the time of our borings was generally at or below a depth of about 8 feet, or Elevation 512. Local areas of perched or trapped water could be present at shallower depths. The long-term groundwater level should be expected to fluctuate with time, water level in Lick Creek, amount of precipitation, runoff characteristics, degree of evaporation, and other related hydrogeological factors.

3.3 Site Specific Geotechnical (Soil/Rock) Condition

In general, the encountered overburden alluvial soils near the creek were either soft or loose and extended below a depth of about 12 feet (or to the top of bedrock surface). The thickness of soft/loose soils generally increased toward the creek. In addition, the near-surface reddish brown silty clay and clay had relatively high plastic indices (PI) of over 25 (up to 48) percent. These clay soils (with higher PI) are highly plastic and have moderate to high swell and shrinkage potential if exposed to water (moisture).

With the relatively high water level near the creek and the presence of soft and loose alluvial (permeable) soils, evidence (sinkholes) of karst geology was also observed within the project vicinity. Even though we did not encounter any underground cavity, the recovered rock core samples were solutionable, calcareous, fine-grained limestone and its dissolution feature has potential to continue developing sinkholes and/or underground cavities within the project area. In addition, based on the review of rock core samples a significant amount of fractured zones were observed within the core samples, but the amount of fractures/joints decreased with depth.

4.0 CONCLUSIONS AND RECOMMENDATIONS

We understand the existing two-span encased steel beam bridge will be completely removed and the new bridge structure over Lick Creek will be a 3-span prestressed I-beam bridge located approximately 100 feet south of the existing bridge. Due to the design of a relatively tall embankment (fill condition) and the limited space for a flatter abutment slope, MSE wall abutments were originally proposed at this bridge location. The existing creek channel at the downslope side of the new bridge will have spill through slopes, which will be protected by riprap over geotextile or other alternatives, as specified in INDOT Construction and Material Specifications. Based on the proposed construction and the soil/rock encountered at the site, geotechnical issues regarding the bridge and abutment walls are summarized below.

4.1 Geotechnical Assessment

(1) Bridge Foundation Assessment:

Limestone bedrock was present at a depth of about 10.5 to 12.5 feet below existing grade, and the rock surface was determined to be relatively flat within the bridge location. Both shallow footings supported on competent limestone bedrock and deep foundations (H-piles, drilled shafts, or Micropiles) socketed or driven into competent limestone bedrock layer can provide sufficient vertical and lateral compressive capacity for the proposed bridge structure. However, based on the laboratory review of rock core samples, we understand the upper 2 to 4 feet of the encountered limestone bedrock is friable and fractured, and therefore, not suitable for foundation support.

If shallow (spread) foundations are chosen, we anticipate that a minimum of 12 to 15.5 feet of foundation excavation would be required to extend the foundation bearing surface to competent rock at both end bents and interior piers. Based on the presence of very soft overburden alluvial soils and relatively high water level at the bridge location, a shallow foundation system bearing on competent limestone may not be cost-effective. A relatively flat (flatter than 1.5 to 2H to 1V) temporary excavation slope or a temporary retention system with a proper dewatering system would be required, and a significant amount (depending on the size of a footing) of upper limestone bedrock excavation may make it difficult and time consuming to extend the shallow foundation bearing surface to the competent limestone.

Secondly, we understand that H-piles are preferably used for bridge foundations supported on sound shallow bedrock in Indiana. Our boring results indicated that the H-piles may have to extend approximately 5 feet into limestone bedrock in order to bear on competent rock. Therefore, predrilling and rock coring to the desired pile tip elevation should be required prior to each H-pile driving operation. However, predrilling and driven H-piles are considered as an end bearing foundation element. In the potential karst developing area, any open joints or fractures may possibly develop into underground cavities or sinkholes, and therefore, we do not recommend that H-piles be used for bridge foundation support for this project.

Drilled shafts can be designed as a friction dependent element. Due to the presence of ruggy limestone and the potential for development of sinkholes or cavities in the future, use of end bearing drilled shafts is considered to have a higher element of risk. The shafts would be socketed a certain depth into competent limestone in order to develop enough skin friction resistance to achieve the design loads. However, considering the typical size of drilled shafts (a minimum of 2 to 4 feet) for bridge construction, the rock coring sometimes may be difficult in limestone bedrock. In addition, if drilled shafts are selected to support the new bridge, a percussion test hole should be drilled at each drilled shaft to a depth of at least 10 feet below the proposed bearing elevation to confirm the soundness of the rock and to identify any voids, open joints, or underground cavities that may affect foundation support. We anticipate that this process would be time consuming and costly. In addition, it may also increase the difficulty of the new bridge end bent design, if integral abutment is considered. For integral abutment, the estimated stresses in the drilled shafts due to shrinkage and temperature movements should be checked and not exceed the allowable stress.

Based on our past experience on transportation projects in karst geology and discussion with BLA and INDOT, we anticipate that Micropiles would be the most practical foundation system for this 3-span bridge project. Micropiles can be used at both abutments and interior piers. Typical Micropile diameter ranges between 7 and 12 inches (relatively easy to install in limestone), and can be considered solely as a friction resistance element (skin friction area is significantly larger than end bearing area). Based on the size of the project with possibility of more than 50 pile foundations required, it is our opinion that a Micropile foundation system is an attractive and viable alternative from design, constructability and time saving standpoints. Also, relative risks due to potential sinkholes or cavities in bedrock is greatly minimized by use of Micropiles. All bridge foundations should be designed to resist vertical, uplift, and lateral forces and overturning moments, if any. If it is necessary, temporary (or permanent) steel casing used during installation of Micropiles can be left in place to resist lateral forces and overturning moments.

(2) Abutment Wall Assessment:

We have also performed preliminary engineering analyses (see attached Analysis 1) for the primary geotechnical issues pertaining to the originally proposed MSE walls at both abutments. The results have been submitted and discussed with BLA on June 4, 2004. The MSE wall section chosen for preliminary analysis is at Station 92+50, with a maximum exposed height of approximately 14 feet above finished grade. For MSE walls nears waterways, a No. 8 aggregate should be used up to the Q_{500} high water elevation instead of "B" borrow.

A minimum reinforcement length of 11 feet for a maximum total wall height of 15.5 feet (including embedment depth) has been chosen for preliminary evaluation, based on Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines, Publication No. FHWA-SA-96-071. The length of reinforcements must be extended beyond the zone of Rankine failure. Due to the presence of very soft/loose silt/silty loam (see borings RW-1, RW-3, TB-1, TB-2, and, TB-3) adjacent to the existing creek, ground modification is required to satisfy MSE reinforcing zone bearing capacity and global stability requirements.

Based on our analyses, in order to provide sufficient bearing resistance a 5 foot depth of undercut with two layers of geogrid reinforcement (12 inches spacing) in the upper 2 feet is required. With a minimum 5 foot partial undercut and replacement with granular material below the bottom of the proposed leveling pad (assuming 6.5 feet below grade), the calculated factors of safety (FS) against sliding, bearing failure, and overturning failure satisfies the minimum requirement of FS.

However, we also performed global stability analyses for the referenced wall section under long-term conditions and calculated a factor of safety (FOS) of 1.09, which is below the minimum required FOS of 1.3 (see attached PCSTABL5M sheet). This result indicated that the required depth of undercut needs to be significantly deeper than 5 feet. Further ground modification will be required to satisfy the global stability requirement.

The following options can be considered for improvement of the foundation soils at the proposed abutment walls.

1) Instead of MSE walls, these abutment walls would be designed as cast-in-place reinforced concrete retaining walls and incorporated with the proposed bridge

foundation as shown in Sheet 9 (FHWA manual), which illustrates a typical configuration of bridge and retaining wall foundations supported on a couple rows of Micropiles. The advantage of this option is that only limited undercut is required to take care of embankment settlement issues behind the cast-in-place walls, which could also be minimized with staged construction methods.

2) A row of drilled piers (soldier beams with 5 to 6 feet on-center spacing and a minimum 8 feet embedment into limestone bedrock) in front of the proposed MSE walls could be considered to resist the global failure. This option could significantly improve the global stability, but however, does not preclude the risk of MSE bearing capacity failure. Therefore, a minimum 5-foot undercut (with a 2 foot zone of geogrid) below the MSE soil reinforcing zone would still be required. We do not anticipate this option to be cost-effective.

3) Ground improvement such as stone columns and Geopiers® are also widely used at this kind of soft ground condition prior to the new MSE wall and embankment construction. Minimum undercut will be required after ground improvement is performed. However, the mobilization cost may or may not be cost effective for this size of project. It is envisioned that 30-inch diameter stone columns (about 3 rows) below the MSE wall-reinforcing zone would be needed as a minimum.

4) A complete removal of 5 to 12.5 foot thickness of unsuitable soils to the top of bedrock, or at least stiff soils, and replacement with granular engineered fill. However, this option may not be economical and practical due to the significant volume of excavation, presence of groundwater, and potential need for temporary shoring to perform the planned undercut.

With the advantages of minimum overall undercut depth, readily modified pile diameter and length based on field conditions, and high capacity into limestone bedrock, we understand that the option of new bridge abutments, interior piers, and retaining walls supported on Micropiles may be the most economical and practical option, based on our phone discussion with BLA on June 8, 2004.

Site specific recommendations for site preparation, design and construction of bridge and retaining wall foundations, drainage structure, embankment and pavement construction, temporary excavation, and temporary and permanent slopes, and undercut and structural fill placement are detailed in the following paragraphs.

4.2 Foundation Design and Construction

Due to the presence of relatively shallow competent limestone bedrock in karst geology, we recommend that the Micropile foundation option be used for this project. Based on the encountered soil/rock conditions at the test borings, it is our opinion that Micropiles would be the most practical solution to support both the bridge and concrete retaining wall abutment for this project.

4.2.1 Embankment Settlement at Bridge Foundation

Based on the planned configuration (Sheet 9), we anticipate that Micropiles be constructed prior to retaining wall and embankment fill placement. We anticipate that approximately 15 feet of fill will be placed above the existing grade at the bridge abutment to match the proposed bridge approach. Due to the placement of about 15 feet of embankment fill and construction of retaining walls, settlements associated with displacement of the abutment foundation soils will occur as the fill is being built (immediate settlement) and for some time after the completion of the abutment walls (consolidation settlement).

A settlement analysis (Analysis 4) was performed near Abutment #4 at Station 94+70. The soil parameters for the settlement analysis were based on laboratory testing results, correlation with published data, and our experience with similar soil profiles. Due to the complex soil deposition history and associated preconsolidation pressures (past soil stress history), some variation in the predicted settlement should be expected. We calculated the consolidation settlement due to the proposed fills (up to 15 feet) to be about 1¾ to 2 inches at Abutment #4. Stress distributions were determined using the Bousinessq Theory. We also estimated the immediate (elastic) settlement to be about 1 to 2 inches, occurring during fill placement.

Due to the presence of sand partings, seams, and layers encountered in the profile within the test borings, two-way drainage has been conservatively assumed. The presence of additional thin sand seams within the soil profile could considerably reduce the length of the actual drainage paths. Therefore, consolidation could occur at a much faster rate than predicted using two-way drainage. We anticipate the majority of settlement will occur within 4 to 6 weeks after the completion of embankment construction. We anticipate that less than ½ inch of additional consolidation settlement will be observed after 4 to 6 weeks.

The time rate of settlement is influenced by several poorly quantified factors: including the construction schedule, i.e., rate of load application; variation in the location and distribution of pervious layers, i.e., paths of internal drainage; soil moisture conditions during fill placement, and change in the index properties of the various materials comprising the embankment. Hence, some variation in time rate of settlement estimates should be anticipated.

4.2.2 Bridge Foundation

For design purposes, Micropiles are usually assumed to transfer their load to the ground through grout/ground skin friction (bond), without any contribution from end bearing. The advantage of this assumption is that the pile movement needed to mobilize frictional resistance is significantly less than that needed to mobilize end bearing. For piles bearing into limestone, even though pile movement is relatively small, Micropiles can still mobilize the majority of its calculated skin friction during loading process.

Based on the reviews of geological settings, and our split-spoon soil and rock core samples, it is our opinion that the skin friction within the entire overburden soil zone should be neglected. Also, negative skin friction within overburden soils discussed in Section 4.2.3 should be considered in the pile capacity design. We anticipate that the proposed 3-span bridge and its abutment walls can be supported on Micropiles bearing into limestone bedrock, with a minimum bond length (rock embedment length) of 10 feet.

We anticipate that an approximate 1 to 4 foot thick fractured limestone zone would be encountered immediately below the limestone surface at both end bents and interior pier locations. Below the friable and fractured zone, competent limestone and/or siltstone bedrock were then encountered. A minimum 10 feet bond length into bedrock is recommended, and therefore, based on the test borings, the Micropiles should have a minimum length ranging between 21 and 23 feet. Two different allowable grout-to-ground bond strengths are recommended for the encountered bedrock for this project.

GEOTECHNICAL DESIGN PROPERTIES FOR MICROPILES (A FOS OF 2.5 APPLIED)

Depth (ft)	Soil/Rock Type	Total * Unit Weight (pcf)	Allowable Skin ** Friction (Bond Strength) Type A (psi)	Allowable Skin *** Friction (Bond Strength) Types B or D (psi)
0 to 12	Overburden	120	--	--
12 to 16	Limestone	140	25	30
16 +	LimeStone	140	60	75
16+	Siltstone	140	50	60

Notes:
1) * Submerged unit weight should be used below a depth of 10 feet.
2) * To obtain the submerged unit weight, the total unit weight should be reduced by the unit weight of water (62.4 pcf)
3) ** Type A gravity grouted piles are usually used to bear into bedrock.
4) *** Specialty contractor can be consulted if pressure grouted (types B or D) piles can be used in bedrock or karst geology (loss of grout within fractured zone).
5) Micropiles should extend deeper than at least 21 to 23 feet below existing grade at the bridge location. No Micropile should terminate in the upper fractured zone.

In addition, structural capacity of the Micropiles should also be evaluated to confirm if design capacity exceeds the steel strength. If allowable pile capacity calculated from bond strength is higher than structural capacity, structural capacity should be used for design. For allowable tension load of Micropiles, the following equation should be used:

$$\text{Tension}_{\text{allowable}} = 0.55\ Fy_{\text{steel}} \times (\text{Area}_{\text{bar}} + \text{Area}_{\text{casing}}),$$
$$\text{where } Fy_{\text{steel}} = \text{the minimum of } Fy_{\text{bar}} \text{ or } Fy_{\text{casing}}$$

For compression and allowable load of Micropiles, the following equation should be used:

$$\text{Compression}_{\text{allowable}} = Fy_{\text{steel}} / \text{FOS},$$
$$\text{where FOS} = 2.12$$

$$\text{Allowable Load} = 0.40\ f'c_{\text{grout}} \times \text{Area}_{\text{grout}} + 0.47\ Fy_{\text{steel}} \times (\text{Area}_{\text{bar}} + \text{Area}_{\text{casing}})$$

The group effect for pressure grouted (types B or D) Micropiles is much less significant. For pressure grouted Micropiles with a typical grouted diameter of about 8 to 12 inches, it is unnecessary to consider a group reduction effect. However, Type A gravity grouted piles are more than likely to be used at this site. In general, we recommend that the Micropiles should not be spaced within a distance less than 3 times the drilled pile/grout diameter. This spacing is to eliminate group effect for axially loaded piles. For gravity grouted (Type A) piles, if pile spacing is less than 3 times the drilled pile/grout diameter, a similar group reduction factor for drilled shafts should be applied or a Micropile group should be treated as one pile, with the perimeter and base area of the group establishing the pile dimension.

The Micropile reinforcement, consisting of a central reinforcing bar and a upper steel casing (if left-in-place), should be designed by the project structural engineer per FHWA Micropile Design and Construction Guideline (FHWA-SA-97-070), published in June, 2000, and INDOT general project requirements. It is recommended that the following criteria be used in the design and construction of the Micropile foundation system:

1. Micropiles be used for structural support typically include grouting Type A (gravity grouted), Type B (pressure grouted), or Type D (postgrouted) method, due to the required high individual capacity. For Micropiles bearing into bedrock, Type A is commonly used. A specialty contractor should be consulted to determine the appropriate type of grouting method in karst geology.

2. It is recommended that the design bottom elevation for each Micropile be shown on the plans. The design bottom elevation should include the minimum bond length of 10 feet in limestone bedrock. The actual installed pile tip elevation should be determined in the field, based on the actual top of bedrock surface encountered. The specifications should be clear that the bottom of pile elevations shown on the plans are for estimating purposes only. The design consultant should include a table in the plans showing micropile tip elevation, pile length, design load and factor of safety used in estimating the ultimate loads.

3. We recommend that the contractor have appropriate equipment on site to facilitate drilling/excavation through these hard materials. The drill rig should have adequate torque and downpressure to facilitate drilling or coring through competent limestone.

4. The installation of Micropiles requires the use of overburden and rock drilling techniques to drill through the weak soil zone, and penetrate and derive support into competent limestone bedrock. A temporary or permanent casing is required to provide full length side support. Depending on the project requirement, steel casing may be left in place within the overburden soil zone.

5. Water or polymer drilling mud (for flushing the hole during drilling) can be used to provide temporary side support and facilitate rock coring during pile installation. Bentonite slurry may impair grout/ground bond capacity and therefore, it is not recommended.

6. No significant loss of grout from any part of the pile should be observed. This can be achieved by grouting to refusal during pile formation, i.e. continue grouting until no more grout take occurs. A filler for plugging the permeable layer or regrouting after set of the initial grout may be necessary to reduce grout loss in grouting through the fractured rock zone.

7. Steel reinforcing bars that have a continuous full-length thread, such as high strength, Grade 150, Dywidag bars, can be used as reinforcement. Reinforcement can be placed either prior to grouting, or placed into the grout-filled borehole before the temporary support is withdrawn (or permanent casing left-in-place). The drill casings can also be left in place from the pile cap down to the top of the bond length of the pile. The steel casing can provide high shear and bending capacity to resist lateral loads. Suitable centralizers should be firmly fixed to maintain the specified grout cover. We recommend that the drilling, installation of the reinforcement, and grouting of a particular pile be completed in short and continuous processes.

8. Close attention must be paid to the control and quality of the grout. A grout quality control plan, at the minimum, should include cube compression testing and grout density (water/cement ratio) testing.

9. The pile load test program and proof tests during installation should also be conducted as described in the project specifications. Testing procedures and results should be inspected and reviewed by DOT representative, and are subject to DOT approval. In general, the compression load test should be designed in accordance with ASTM D1143 and tension load test in accordance with ASTM D3689.

In addition, based on the preliminary configuration of the proposed abutment (cast-in-place reinforced concrete walls), we anticipate that Micropiles would be installed prior to any new fill placement and retaining wall construction. Therefore, the Micropile design capacity should also include potential downdrag loads (see Section 4.2.3) developed within overburden soils due to new fill placement. The recommendations are presented in the following section.

4.2.3 Downdrag Considerations On Micropiles at Abutments

In general, the amount of soil settlement relative to the pile movement that is necessary to mobilize negative skin friction (downdrag) should be greater than about ½ inch. Based on our settlement analysis, the neutral point, defined as the point at which the relative settlement between the pile and soil is greater than 0.5 inches, may be about 10 feet below existing grade. Based on our settlement calculations for Abutment #4, the neutral point is at about Elevation 510, with approximately (8 x d) tons of ultimate downdrag load (where "d" is the Micropile diameter in feet) along the upper 10-foot soil zones between Elevations 510 to 520. At the interior pier locations, we do not anticipate that significant embankment fill will be placed, and therefore, no downdrag loads are estimated at the interior piers.

Micropiles would need to be designed for the estimated downdrag loads (approximate 7 x d tons). Typically, it is recommended that the downdrag load be considered as an additional load added to the design load. The pile design would need to consider both the geotechnical capacity (bond resistance) and structural capacity (steel Fy) due to the added downdrag loads.

4.2.4 Retaining Walls

MSE wall abutments were originally planned for this project. However, due to the presence of relatively thick unsuitable overburden soils and the steep downslope creek bank, complete undercut and replacement as discussed earlier in this report becomes impractical and time consuming. Therefore, two sets of cast-in-place reinforcing concrete retaining walls which will be supported by Micropiles are considered as part of the bridge replacement project. The new abutment walls will retain the proposed embankment fill, up to 15 feet above existing grade. The design consultant should include a table in the plans showing the micropile length, tip elevation, design load and safety factor. The following paragraphs provide our geotechnical design recommendations for the retaining walls.

Based on the anticipated amount of fill placement, we understand that the proposed abutment walls will be backfilled with imported INDOT approved soil/material to achieve the proposed roadway approach slab grade.

Depending on the wall type, the proposed abutment wall may be restrained from movement at the top by the bridge reinforced concrete beam. Thus, the proposed abutment wall may be designed based on "at-rest" earth pressure conditions with a rectangular earth pressure distribution. We recommend that, at a minimum, 3 foot INDOT B Borrow granular fill should be used immediately behind the retaining walls. Since the rigid abutment wall will be backfilled with granular materials, the wall can be designed based on a rectangular distribution of 30H psf, where "H" is the height of the wall in feet. The wall will receive surcharge loading from pavement traffic and potentially from cranes during the bridge construction. Therefore, 50% of any surface surcharge loading adjacent to the wall should be included in the abutment wall design. No hydrostatic pressures have been included in our recommended design earth pressures; hence, drainage provisions should be provided.

If the proposed abutment wall is a cantilever wall, free to rotate at the top, the cantilever retaining wall could be designed based on active earth pressure conditions. An equivalent fluid weight of 45 pcf could be utilized for cantilever retaining wall design. The equivalent fluid weight acts in a triangular distribution from the top of the wall to the bottom of the wall footing. The surcharge loading should be included in addition to the active earth pressure. One-third of the surcharge pressure should be applied uniformly to the wall in addition to the active earth pressure.

Note that these lateral earth pressure values are all based upon the assumption that the wall is backfilled with a minimum 3 foot zone of free-draining granular material. Additionally, the granular material should be properly drained by foundation drains or weepholes. No hydrostatic pressure is included in our equivalent fluid weight. If free-draining backfill and drainage are not provided, the lateral earth pressures could be substantially higher than recommended above.

The new abutment wall will also be supported on drilled Micropiles. The drilled Micropiles can be designed to resist sliding and overturning, as outlined in the Sections 4.2.2 and 4.2.3 of this report.

Wall Backfill

The wall backfill will also be utilized to support the roadway pavement. Settlement of the roadway pavement can occur if the wall backfill is not compacted properly. The 3 foot thick free-draining zone immediately behind the retaining wall should consist of a relatively well-graded, free-draining granular material, having no more than 7% passing the No. 200 sieve. A geocomposite in conjunction with a footing drain could be utilized in lieu of the traditional granular backfill.

The backfill should be placed in loose lifts, having a maximum 6" thickness. Each lift should be compacted to at least 95 percent of the maximum dry density as determined by AASHTO T-99, as outlined in INDOT Construction and Material Specifications. To avoid overstressing the wall, hand compaction equipment should be utilized within 5 feet of the wall face. Use of heavy compaction equipment should be avoided near the wall. If a granular backfill is utilized, it may be advantageous to install surface settlement stakes and monitor them to ensure that the majority of the settlement has occurred before paving.

4.3 Embankment Settlement and Slope Stability

Based on the preliminary Bridge Plans provided, the proposed profile indicates a significant amount of embankment fill, up to 17 feet, will be required within the project limits. We understand that the proposed embankment slope will be near 2 H: 1V to 3 H: 1V. Three (3) critical sections were evaluated and the recommendations of ground modification below the proposed embankment are described below.

We recommend that any unsuitable materials revealed during subgrade preparation be undercut and replaced with INDOT structural backfill. Due to the consolidation settlements (over an inch) based on settlement analyses (Analyses 2, 4, and 6), partial undercuts with or without geogrid reinforcement are proposed within the following stations. The recommended partial undercut areas and anticipated depths are solely based on widely spaced test borings, and therefore, field verification of subgrade soils will be required during roadway construction:

SUMMARY OF THE SETTLEMENT ANALYSES PRESENTED IN THE APPENDIX

STATION NO.	ANTICIPATED DEPTH OF UNDERCUT	SETTLEMENT (INCH)		TIME RATE OF SETTLEMENT (WEEK)	
		30' OFFSET	CL	50%	90%
91+50 to 92+25 (Analysis 2)	Without undercut	2	1½	2	6
	3' below new embankment fill area and Str. No. 12 area*	1	¾		
94+70 (Analysis 4)	Without undercut	1½	2	2	6
96+50 to 97+50 (Analysis 6)	Without undercut	1½	1½	2	8

Note: * Undercut areas proposed in this section are all partial undercut option only (to control excessive settlement and provide a minimum of 3-foot thick stable subgrade for new fill and box culvert placement). Due to the occasional presence of sand seams and layers, time rate of settlement for no undercut and partial undercut options may vary.

For the partial undercut option, in order to provide a stable base for new engineered fill placement on the revealed soft soils, we recommend that a minimum 12" to 18" thick INDOT No. 8 Stone be placed at the partial undercut bottom. Following dumping of INDOT No. 8 stone in the soft and wet soils, the contractor should punch stone fills into the soft soils by heavy equipment or roller compactor to create a stable working base for new fill placement. A non-woven geotextile per INDOT Specification 913.18 could be placed on this stone/rock surface before the embankment with new fill is constructed. The geotextile should have an Equivalent Opening Size (EOS) suitable to act as a filter to prevent loss of material from the new INDOT B Borrow fill into the underlying No. 8 Stone.

Six (6) global slope stability analyses have also been performed at the two (2) most critical cross sections along the entire alignment and are presented in the following paragraphs.

4.3.1 Station 91+50 to 92+25, Line "B"

Up to 17 feet of permanent embankment fill is planned on the left and right sides of the existing embankment at this location. However, very soft or loose silty loam soils (compressible soils) were encountered to a depth of about 12.5 feet (Elevation 507.5) below existing grade at Boring TB-5. The soil sample recovered at about 10 to 11.5 feet had an unconfined compressive strength of 0.34 tsf. We anticipate the water level may be at or below Structure No. 12 invert elevation of about Elevation 510 feet. The upper silty soils were easily disturbed by water, and therefore, we recommend 3 feet of partial undercut below structure (# 12) invert level, or to the top of friable limestone bedrock, and replacement with granular engineered backfill.

Based on our settlement analysis (Analysis 2), we estimate primary post-construction settlement may be between 1½ to 2 inches settlement at the proposed pavement section without the partial undercut option. With the 3 feet partial undercut option within the new embankment fill subgrade, we estimate that the consolidation settlement would reduce to about 1 inch. We recommend settlement plates and stakes be installed at 100 feet center to center intervals, and monitored along the proposed embankment between Stations 91+25 and 92+50 in accordance with Section 204.03(a) of the INDOT Standard Specifications. Settlement should be monitored for the proposed duration and should be less than 0.01 ft. for four consecutive weeks. We anticipate that a time period of 4 to 6 weeks could be needed for embankment settlement to occur after the completion of embankment fill but prior to any new pavement construction.

Moreover, four cases of global slope stability analyses were performed at Station 91+64, Line "B". The external stability against rotational slip-surface failure was performed by using PCSTABL5M and PCSTABL6H developed at Purdue University. Both existing and proposed conditions (with and without undercut and/or geogrid) are considered in the analyses. The factor of safety was determined using the modified Bishop method for circular shaped failure surfaces. The factor of safety (FOS) calculated are also summarized below:

CASE	GLOBAL SLOPE STABILITY WITHOUT SEISMIC	LONG TERM CONDITION MINIMUM FOS	SHORT TERM CONDITION MINIMUM FOS
1	Sections without Drainage Structure (without any undercut) *	1.09	-
2	Sections at Drainage Structure + 3' undercut below the invert (or to the top of bedrock) **	1.22	-
3	Sections with 3' undercut + 4 layers of geogrid between Elevation 510 and 516	1.45	1.51
Minimum Required FOS		1.30	1.30

Notes:
* The existing embankment, approximate 5 to 8 feet in height, will remain the same and receive new fill.
** At Sta. 91+64, due to the construction of new box culvert, the existing embankment will be completely removed in the culvert vicinity, plus 3 feet undercut below the invert.

With the construction of the proposed 2H to 3H: 1V slope, the calculated most critical FOS against global stability was 1.09 and 1.22 under the loading conditions without and with 3-ft. undercut, respectively. The factor of safety of the long-term proposed condition falls below the acceptable global stability minimum required FOS. In addition, the provided cross-section drawings also indicated a relatively steep slope located at the existing creek channel (creek bank) where new embankment will be placed. Therefore, based on the review of the analyses, we recommend that the east side (Rt.) of the proposed embankment slope between Station 91+00 to 92+50, Line "B" be reinforced with geogrid from Elevation 510 to 516, with a 2 foot vertical spacing (approximate 4 layers of geogrid required).

The geogrid should extend approximately 25 to 30 feet behind the slope surface and beyond the potential failure surface. It is critical that the geogrid (such as Type I geogrid per INDOT Specifications 913.21) is placed in strict accordance with manufacturers guidelines, especially orienting the geogrid in the strong direction. The geogrid should be placed edge to edge (no overlap) with single, continuous pieces used (no splicing in the long direction). In addition, careful monitoring of the geogrid should be made during installation and construction to confirm the grid is not cut or damaged by the granular embankment fill or heavy equipment. General geogrid construction considerations are summarized below:

Geogrid Placement

a. Place and incorporate the Geogrid within the structural backfill. It is important that the contractor understands that the strength of the primary reinforcement is directional. It is critical that the roll direction, as described on the drawing and manufacturers specifications be followed.

b. Primary Geogrid reinforcement – This reinforcement shall be placed perpendicular to the slope contours and shall be butt jointed on the edges. No overlap is allowed for reinforcement. Each longitudinal piece shall be continuous without any splicing.

c. Geogrid Placement – The geogrid reinforcement can be placed directly on the prepared fill surface. It is expected that a self-propelled sheepsfoot roller shall be used to compact the fill soil. There is no need for special surface treatment, such as leveling and smoothing the sheepsfoot imprint surface prior to grid placement.

d. The grid reinforcement shall be tensioned by hand until it is taut and free of wrinkles and laying flat. Fill soil shall then be placed directly on the grid. Care shall be taken to prevent wrinkle development and/or slippage of reinforcement during fill placement and spreading. When practical, fill shall be placed transverse to the direction in which the reinforcement is laying. Rubber tired equipment can pass on the bare reinforcement at slow speeds without sudden braking. However, tracked equipment shall not be allowed on the bare reinforcement. A minimum of 6" of fill shall be on top of the grid reinforcement before tracked equipment can operate, to avoid damage to the reinforcement.

Compacted Fill

a. INDOT approved B Borrow materials shall be placed in horizontal layers (which are benched into the existing embankment) and compacted in a controlled fashion. Embankment fill shall be uniformly compacted per INDOT Standard Specifications.

b. Care shall be taken to avoid damaging the geogrid by the larger limestone pieces.

Erosion Protection

The east side of the proposed embankment slope along Lick Creek should be protected from erosion. The erosion of the toe of the slope may possibly trigger a local landslide in this area after the embankment is constructed. Therefore, we strongly recommend that an erosion control mat (or revetment riprap on geotextiles) be required from the toe of slope to Elevation 516 on the east (Rt.) side of CR 375W from Station 91+50 to 92+70, Line "B" or beyond.

Alternately, a cast-in-place concrete wingwall (or retaining wall), sheeting piles, or soldier piles with lagging retention system can be considered to retain the proposed embankment fill parallel to the existing creek channel. However, the concrete retaining walls will have to extend to the top of limestone bedrock and may not be economical and practical due to relatively high water level along Lick Creek. If sheeting piles or a solider beam with lagging system is desired by the designer, a detailed analysis of the selected type of retention system(s) can be provided.

4.3.2 Stations 94+70, Line "B"

In the vicinity of test borings TB-4 and RW-3, about 12.5 to 13.5 feet of soft to medium stiff, moderately to highly compressive, alluvial soils were encountered below existing grade. Based on our settlement analysis (Analysis 4), we estimate primary post-construction settlement of the 15 foot high embankment may be up to 2 inches at the pavement section. We anticipate 50 percent of this settlement (1 inch) may occur within 2 weeks following completion of embankment construction. We recommend a minimum 4-week waiting period following completion of embankment construction at both bridge abutments prior to constructing new pavement. We estimate that less than ½ inch of additional consolidation settlement will occur after the 4 week waiting period. We recommend settlement plates and stakes be installed at 25 feet center to center intervals at Station 94+70, Line "B", and monitored along the proposed embankment near the bridge abutments.

In addition, two slope stability analyses (long term and short term condition) were evaluated at Station 94+70, Line "B". Soil parameters used in the global stability analyses were based upon laboratory test results and our past experience with similar soils. The long-term shear strength parameters selected are likely between residual and peak values. Soil parameters used in the analyses are depicted on the attached calculations. In addition, a 250 psf traffic surcharge was also included in the long-term and short-term analyses to model either traffic load or construction traffic. Graphical results of the global stability analyses are included in the Appendix of this report. The minimum factor of safety (FOS) is presented in the following tables:

GLOBAL SLOPE STABILITY WITHOUT SEISMIC	LONG TERM CONDITION	SHORT TERM CONDITION
Minimum FOS	1.48	2.00
Minimum Required FOS	1.30	1.30

Based on the slope stability analyses without seismic considerations, the factor of safety in short term and long term conditions exceeds the minimum required factor of safety of 1.3.

If the less plastic and sandier soils are used for the embankment construction, some sloughing failures may occur. Oftentimes, the shallow sloughing may progressively increase with time. If shallow sloughing failures occur, then remedial maintenance will be required, which generally entails removal of the sloughed soils and replacement with coarse aggregate crushed stone. Vegetation should be established on the embankment side slopes as soon as possible after construction to help reduce the potential of shallow sloughing failures.

4.3.3 Stations 96+50 to 97+50, Line "B"

Approximately 10 feet of embankment fill is planned at Station 97+00, Line "B". Moderately plastic A-7-6 clays (compressible soils) were encountered to the explored depth of about 10 feet below existing grade at borings RB-2 and RB-3. The encountered A-7-6 clays had a PL of 23 and a moisture content of 28 to 30 percent. Based on the settlement analysis (Analysis 6), we estimate post-construction settlements up to about 1½ inches could develop along the embankment section.

Based on the analyses of the rate of settlement, we recommend a minimum 2 week waiting period (50% of consolidation settlement) following completion of embankment construction prior to constructing pavement between Station 96+50 and 97+50. Also, we estimate the majority (90%) of consolidation settlement would take place within about 2 to 9 weeks. We recommend that settlement plates and stakes be installed at 50 to 100 feet center to center intervals along the proposed embankment between these stations. Also, the settlement must be less than 0.01 ft. per week for four consecutive weeks.

4.3.4 Lateral Squeeze Potential

Lateral squeeze potential was also evaluated at the above mentioned cross sections. The analysis used the criteria that lateral squeeze potential exists if the applied embankment overburden pressure (height of fill x fill density) is greater than three times the undrained shear strength of the embankment foundation soils. Based on this criteria, we determined that lateral squeeze potential is not a significant geotechnical issue due to the granular nature of the encountered soils near Lick Creek.

4.4 Creek Channel Slope Protection and Riprap Placement

Currently, we understand creek channel slopes are proposed to be 2H:1V near the bridge. Based on the soil conditions encountered in borings TB-2 and TB-3 at the proposed interior piers, we anticipate the proposed end slopes should be properly protected by either erosion control mats or revetment riprap. Localized wet, soft clay soils, if encountered, should be undercut, properly benched, and replaced with granular engineered fill (INDOT B Borrow) prior to new fill placement. We anticipate that the height of the slopes will be less than 12 feet from the flow line of the existing creek.

Erosion control could include a minimum 24 inch thick layer of revetment riprap placed directly over a layer of geotextile (filter fabric) on the proposed creek channel slopes near the bridge and embankment locations. An acceptable filter fabric meeting the requirements of the current INDOT Standard Specifications (Section 616.10, 1999 edition) should be used in conjunction with the riprap. The purpose of the geotextile fabric is to reduce the migration and loss of fines from below the riprap. In addition, a minimum 3 foot deep riprap key should be provided at the toe of the riprap and should be encased with an acceptable filter fabric.

4.5 Major Drainage Structure Installations

We understand that Orange County plans to completely remove and replace the existing drainage structure with a new 4'x 4' box culvert at Station 91+64, Line "B". The existing box culvert beneath the existing pavement had been extended once and the original section of the culvert appeared to be in a poor condition. Based on the provided Preliminary Bridge Plans, the existing culvert length is about 30 feet, with invert elevations ranging between elevations 514 and 515.5. The anticipated flowline at the upstream and downstream of the proposed culvert will be at elevations of 512.25 and 517.92, respectively. The major drainage structure, location, invert elevation, and anticipated depth of unsuitable soil below invert elevation based on the boring information are included in the following table.

ANTICIPATED SUBGRADE CONDITIONS AT MAJOR DRAINAGE STRUCTURE

STATION NO. (STATION NO., LINE "B")	STRUCTURE SIZE & TYPE	PROPOSED INVERT ELEVATION, FEET	ANTICIPATED SUBGRADE MATERIAL	ANTIC. DEPTH OF UNDERCUT BELOW INVERT ELEV., (FT)	NEAREST BORINGS
Structure No. 12 (91+64)	4' x 4' x 120' Box Culvert	512 to 518	Very Soft to Soft Silt/Silty Loam to a depth of about 10 feet	3 feet below invert or to the top of bedrock, whichever is shallower *	TB-5

* The section is within recommended embankment partial undercut stations (3' undercut), and therefore, additional undercut may or may not be required.

Due to the presence of relatively deep, very soft to soft silty soils, we anticipate partial or complete undercut may not be feasible for the proposed drainage structure. If any soft soil is encountered at structure bearing elevation elsewhere, we recommend soft soils be undercut a minimum of 3 feet and replaced with INDOT granular B Borrow to provide adequate stable support for the proposed drainage structure. Geogrid reinforcement can also be used below the culvert, if very soft condition is still exposed at the bottom of undercut. The undercut sections should also be incorporated with partial undercut areas presented in the roadway embankment and settlement analysis section.

4.6 Earthwork

4.6.1 Bridge Structure

The new bridge and its abutments will be supported on Micropiles. The existing bridge and bridge abutments will be completely removed. New structural fills, up to 15 feet high embankments, will be required at the new bridge abutment locations. Due to the presence of very soft silty soils at TB borings, we recommend that the localized wet, soft soils be undercut during site preparation prior to new bridge construction. Proper erosion control methods should be implemented to prevent erosion of the existing creek banks at interior piers.

4.6.2 Rock Excavation

Even though not anticipated, if the excavation of the upper limestone bedrock is required during construction, the rock excavation will likely require the use of large heavy-duty construction equipment. A ripping tooth or other appropriate piece of equipment may be needed to loosen the rock prior to excavation. Appropriate drilling/coring equipment should be used during Micropile installation into bedrock.

4.6.3 Roadway Embankment

Significant embankment fills, up to 15 feet high, will be required between Station 91+50 and 97+50, Line "B", over Lick Creek. In addition, approximately 10 to 17 feet of fill is also planned above and near the proposed box culvert (Structure No. 12 at Station 91+64, Line "B"). In general, the proposed embankment fills will be placed over the natural very soft to medium stiff silty soils (alluvium). We recommend that all surficial soft clayey topsoil (up to 2.5 feet near the creek) be completely stripped, and the existing slope be properly benched prior to engineered fill placement.

Special care should be exercised along the existing creek channel where a significant amount of fill new embankment will be placed on the top of the existing relatively steep creek bank. At a minimum, we anticipate some ground modification consisting of a 3 foot undercut and replacement with compacted engineered fill and geogrid required along the east side slope of the proposed embankment between Station 91+00 to 92+50.

In addition, five (5) hand auger soundings were performed in the existing drainage ditch on both sides of the existing culvert and along the proposed embankment, where the proposed engineered fill (up to 10 feet in height) will be placed. The result of soundings S-1 through S-5 showed a layer of grass covered topsoil, about 6 inches in thickness, was present at the existing surface (see Summary of Soundings in the Appendix). Below the surficial topsoil, unsuitable soft silty clay to clay was generally encountered to a depth of approximately 2 to 3 feet below existing grade. An exception to this was at S-3 (downstream side of the existing culvert) and S-5, where wet, loose sandy soils were encountered to a depth of about 1.5 to 2.5 feet below existing grade.

Based on our site visits, it is our opinion that the proposed site is relatively wet (ponding water) and the clay soils (A-7-6) in the area appeared to have poor drainage characteristics. We recommend that any soft, wet, or loose soils be undercut and replaced with compacted INDOT B Borrow in accordance with INDOT Standard Specification. At the location of the new box culvert (Structure #12), we anticipate that 3 foot of additional undercut and replacement, incorporating layers of geogrid, will be needed to remove the wet, soft and/or loose soils prior to the new box culvert placement. We estimate that new box culvert invert elevation will be about 5 feet above the bedrock surface. If the existing box culvert, when removed, was placed on top of bedrock, the new box culvert should also be found on top of bedrock. Depending on the weather during the proposed construction, we anticipate that dewatering will likely be required during undercutting and fill placement operations near Lick Creek.

Based on the encountered very soft silty alluvial soils, with occasional water seepage at the borings (Type C soil), a 1.5H:1V or flatter temporary cut slope can be considered for excavations up to 20 feet, per OSHA regulations. Temporary cut slopes should be flattened or a temporary retention system considered if sloughing of the temporary slopes is observed. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

The proposed embankment slope within this project area has a grade of about 2H:1V and will terminate within Lick Creek at the toe of the slope. Based on the results of the test borings, we believe that soft silty loam and/or silt will be encountered near the proposed toe of slope (edge of the creek). The flow of water within Lick Creek will erode the banks of the creek and the toe of new embankment which will reduce passive resistance at the toe. We recommend that at least a 3 foot undercut be performed in the areas where new embankment fill is planned, and then replace the soft soils with compacted engineered fill to create a stronger keyway at the toe of embankment slope. The erosion of the toe of the slope may possibly trigger a local landslide in this area after the embankment is constructed. Therefore, we strongly recommend that an erosion control mat (or revetment riprap on geotextile) be required for the toe of slope on the east side of CR 375W from Station 91+00 to 92+50, Line "B" and on the both sides of the creek bank at the new bridge location.

The majority of the proposed subgrade will be developed by placement of new engineered fill for this project. Upon the removal of surficial asphaltic concrete pavement or topsoil, we recommend that the proposed embankment areas and bridge approaches should be proofrolled under the observation of a qualified geotechnical engineer prior to the fill placements. Proofrolling of the existing subgrade should be performed in accordance with INDOT Standard Specification. All soft, loose or yielding subgrade revealed during the proofrolling operations should be undercut and replaced with engineered fill. Soft and unsuitable soils revealed by proofrolling should be re-compacted or undercut and replaced with compacted B Borrow in accordance with INDOT Standard Specification, Subsection 203.23 to 203.26, latest edition.

Fill and backfilling may be accomplished in accordance with Section 203.09, INDOT Specifications. Special care should be exercised while proofrolling the on-site moderately plastic clays (A-7-6). Any "wet" clay exposed should be undercut prior to any new fill placement. These materials are sensitive to moisture and disturbance. If subgrade soils via localized areas become unstable due to the presence of ponded water, seepage water, or disturbance by construction trafficking, we anticipate that it may be stabilized through undercutting a minimum depth of 12 inches and replacing the undercut soils with compacted INDOT B Borrow or No. 53 crushed aggregate, in combination with a geogrid, if necessary. We recommend that a field engineer assigned by INDOT observe proofrolling and subgrade preparation, so that INDOT or HCN may evaluate the suitability of subgrade soils and adequacy of any undercut which may be required.

4.7 Engineered Fill Placement and Compaction

In general, granular fill should not be used in the drainage side ditches (except for draining backfill), or within 12 inches of the required finished surfaces of embankment slopes, to reduce the risk of erosion.

The embankment material should be a non-erodable, environmentally clean soil free from lumps, wood, topsoil, clods, debris, organic matter, and stones. Due to the proposed relatively small volume of cut, we anticipate that this project will require a significant amount of imported fill. Any on-site clays planned to be reused for engineered fill should be evaluated at the site by the project geotechnical engineer, based on its plasticity and moisture content. The engineered fill should be placed in lifts not exceeding 8 inches in loose thickness and be compacted to the required density as specified in the latest INDOT Standard Specifications.

We recommend that any soil used as engineered fill should meet INDOT Specifications. All material types which will be used as engineered fill must be tested in the laboratory to determine its project suitability and its compaction characteristics. We anticipate moderately expansive clays (CH), such as encountered at RB-1, with a Plastic Index over 30 may be encountered during excavation and may swell during soaking. Water is the catalyst that causes the shrinkage and swelling. The on-site clays, which had PI's ranging between 26 to 48 and were classified as A-7-6 (38), should not be re-used as engineered fill for roadway embankment due to its sensitivity to the moisture and difficulty in achieving the required degree of compaction.

The moisture content of the near surface cohesive soils at the boring locations were considered higher than the optimum moisture content of the encountered soils at the time of drilling. Additional moisture conditioning may be required if on-site soils are to be reused. Depending on seasonal conditions and recent rainfall events during the time of construction, some drying and manipulating of the on-site soil (determined suitable) through reworking (discing) may be required before the soil can be properly compacted. It may also be necessary to add moisture during extended periods of hot, dry weather should the soil become too dry to achieve compaction. Since imported materials will be needed for this project, we recommend that imported INDOT B Borrow or similar materials be used as engineered fill for roadway embankments. ? most probably miss spelling

Structure backfill surrounding all pipe structures in the excavated trenches should be compacted to 95 percent of the maximum dry density as determined by AASHTO T-99 (Standard Proctor). The soil in the bottom of the excavations, any bedding material and the structure backfill at the drainage structure, should also be tested to determine that tested locations comply with this density criterion.

Prior to new engineered fill placement on natural soil slopes or existing fill slopes for 4:1 or flatter, the existing ground surfaces should be plowed or deeply scarified or, if the nature of the ground indicates greater precautions should be taken for integrating the proposed fill materials with the existing slopes, benches should be done as directed by the geotechnical technician. Prior to new engineered fill placement on natural soil slopes or existing fill slopes steeper than 4:1, benches a minimum of 10 ft. wide, unless otherwise specified, should be cut into the slopes.

When the level of the fill reaches the top of the drainage structure, two lifts, about 6 inches each, should be carefully spread and hand compacted over the structure without the use of heavy equipment. The backfill should be compacted to at least 95 percent of the maximum dry density as determined by AASHTO T-99 except the first 2 lifts above the structure. Mechanical compaction over the drainage structures may commence after the second lift is placed and compacted.

4.8 Groundwater Control

We anticipate that groundwater and water from Lick Creek will be encountered during excavation and construction of the new bridge and box culvert, and may be encountered in the undercut area within the new embankment subgrade. Cofferdams and sandbags will be required to provide a relatively dry construction environment for the construction of the proposed bridge foundation and embankment undercut. Due to groundwater encountered at a depth of about 10 feet in the borings, we recommended that the contractor be required to perform dewatering by conventional sump and pump system, if groundwater is encountered during excavation. An appropriate dewatering operation would also facilitate the undercutting and fill placement activities during construction.

The contractor should also plan appropriate site drainage prior to commencing any excavation. Proper site drainage of surface runoff water will help to alleviate unwanted intrusion into the excavation during the excavation and construction operations.

4.9 Pavement Design Considerations

A new bituminous pavement is planned between Station 85+50 to 92+47 and between Station 94+90 to 99+87, Line "B" within the project limits. The near-surface soils encountered along the proposed new alignment at the borings were very soft to medium stiff silty loam (A-4) and clay (A-7-6). Proofrolling would identify areas of weak subgrade soils. If any soft or yielding subgrade is revealed during the proofrolling operation, further undercut and replacement with compacted INDOT B Borrow would be required. Closely monitoring the proofrolling operation is important for this project if A-7-6 clay is exposed at the pavement or embankment subgrade. The plastic A-7-6 clay, if too wet or soft, may swell or shrink with changes in soil moisture content after the pavement is constructed.

The majority of the project will be developed by fill placement. Based on the preliminary Bridge Plans prepared by BLA, the Average Annual Daily Traffic (AADT) on CR 375W was about 600 Vehicle Per Day (VPD) in 1996, and projected to be about 660 VPD by Year 2020. Based on Subgrade Treatment Recommendation Options prepared by INDOT, Materials and Tests Division, dated August 10, 2004, the subject roadway has a traffic volume projection of (AADT) less than 3,000 VPD, but more than 500 VPD. Therefore, we recommend the subgrade treatment "Type II" be used for pavement design for this realignment project. We recommend that a subgrade Resilient Modulus (M_R) of 3, 000 psi be used for the pavement design. The recommendation is summarized below:

LINE	TRAFFIC	RECOMMENDATIONS		REMARKS
B	< 3,000 VPD > 500 VPD	Subgrade Type II	M_R = 3, 000 psi	Reconstruction and Realignment

The silt content of the encountered cohesive soils were up to about 87 percent (silt and silty loam). However, the majority of the pavement will be supported on engineered fill. Based on INDOT general practices, underdrains could be used for this project only if the adjacent pavement has underdrains. If pavement underdrains are used, subsurface drains should be perforated corrugated plastic pipe meeting the requirements of Section 718 of the INDOT Standard Specifications, Underdrains. The Apparent Opening Size (AOS) should be compatible with the openings in the drain tile and the grain size of the surrounding soils, to reduce the risk of loss of fines into the drainage system and clogging. Outlets should be provided at regular intervals to convey the water collected in the subsurface drains. The subsurface drain outlets should be covered with screen. If subsurface drains are used, filter fabric will be required for this project.

The pavement section should be graded to prevent ponding of surface water. Side ditches should be provided and the pavement base course should be daylighted through the ditch sideslope to facilitate drainage of the base course. The base stone should be protected from water inflow along drainage paths. Additionally, the base stone should extend beyond the edges of the pavement in low areas to allow water that enters the base stone a path for exit. Subgrade slopes should also follow surface slopes. We recommend a minimum slope of 2 percent, where feasible, to facilitate drainage.

4.10 Potential Sinkhole Treatment

The Blue River Group Limestone is well known for its dissolution feature, and the site resides in an area with high potential for karst development. If sinkholes are exposed during site grading or undercutting within the proposed embankment area, it is imperative that the sinkhole that are not used for drainage purposes should be filled and/or capped with lean concrete. Sinkholes to be used for drainage purposes should be constructed with "granular" embankment. Special care should be exercised during construction to prevent siltation of the sinkholes. The sinkholes exposed should be excavated to expose the throat. Soil domes or cavities exposed during grading must be investigated. Where a sinkhole or soil dome is below the proposed embankment area, the risk is greater, and therefore we recommend that a concrete plug be constructed at the apparent throat of the sinkhole, if it is not an active drainage path. The concrete plug can be a plain concrete plug if the throat is identifiable and narrow. A concrete plug does have the disadvantage of redirecting the infiltration of surface water from the existing sinkhole and potentially contributing to the formation of new sinkholes. However, in HCN's opinion this is a lesser risk than the risk to the proposed roadway posed by incomplete plugging of a sinkhole.

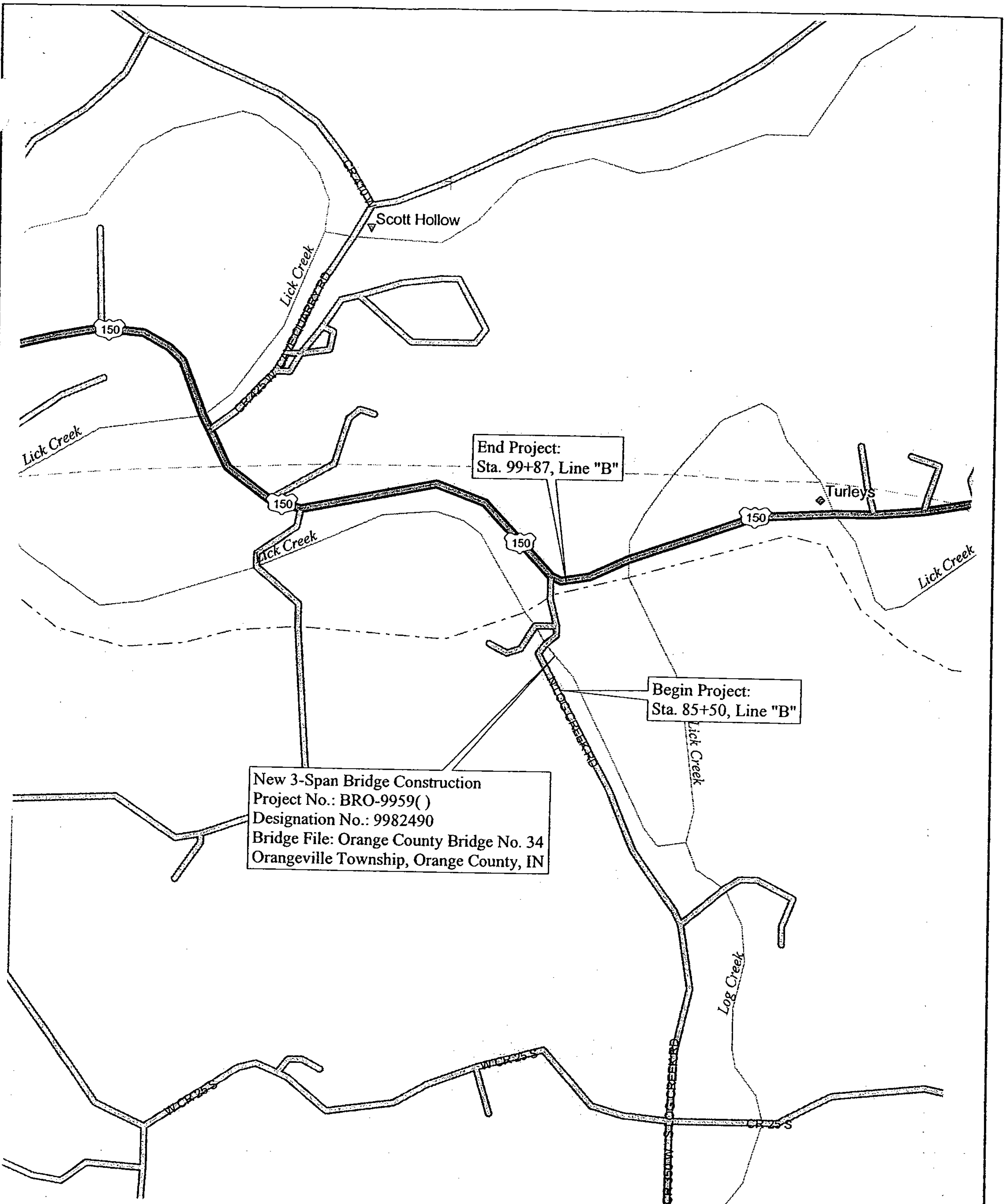
The risk of undetected soil domes which could develop into sinkholes could be reduced, though not eliminated, by geophysical exploration. Methods of geophysical exploration which could be considered at this site would include electrical resistivity and crosshole seismic methods.

5.0 CONSTRUCTION TESTING, MONITORING, AND INSPECTION

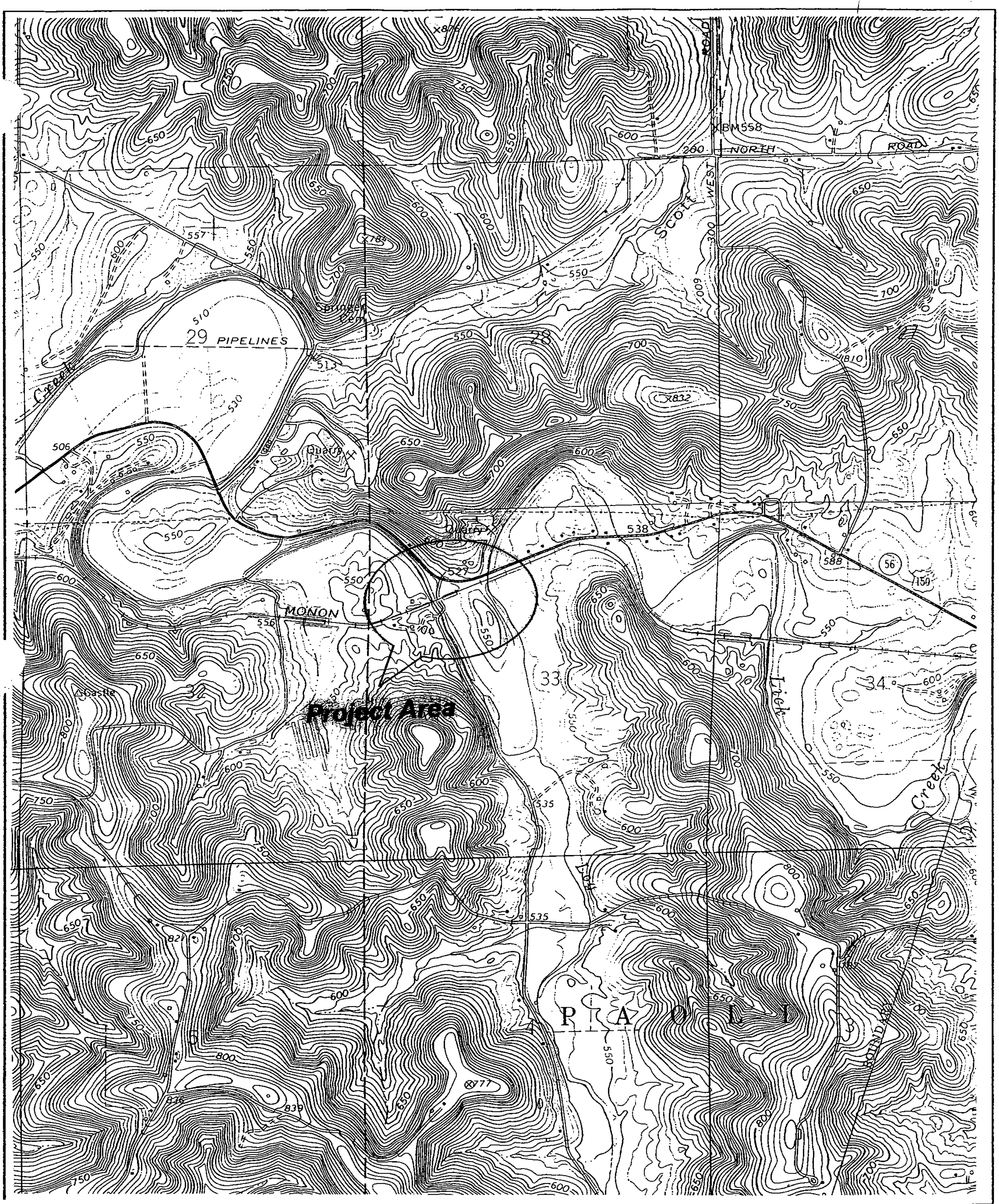
All excavations and Micropile installations, structural fill placement and compaction operations should be monitored and inspected by a qualified geotechnical technician. The technician will perform under the supervision of the Project Geotechnical Engineer assigned by INDOT or BLA. It will be the technician's responsibility to approve all items in this regard. It is recommended that all foundation and retaining wall excavations be inspected prior to concrete placement to ensure that adequate bearing materials or elevations have been exposed. Concrete/grout testing and inspection should also be provided.

The H. C. Nutting Company respectfully requests continued involvement in this project by providing testing and monitoring services throughout the construction phase. Applicable scope of work and related fees for these services can be provided upon request.

APPENDIX

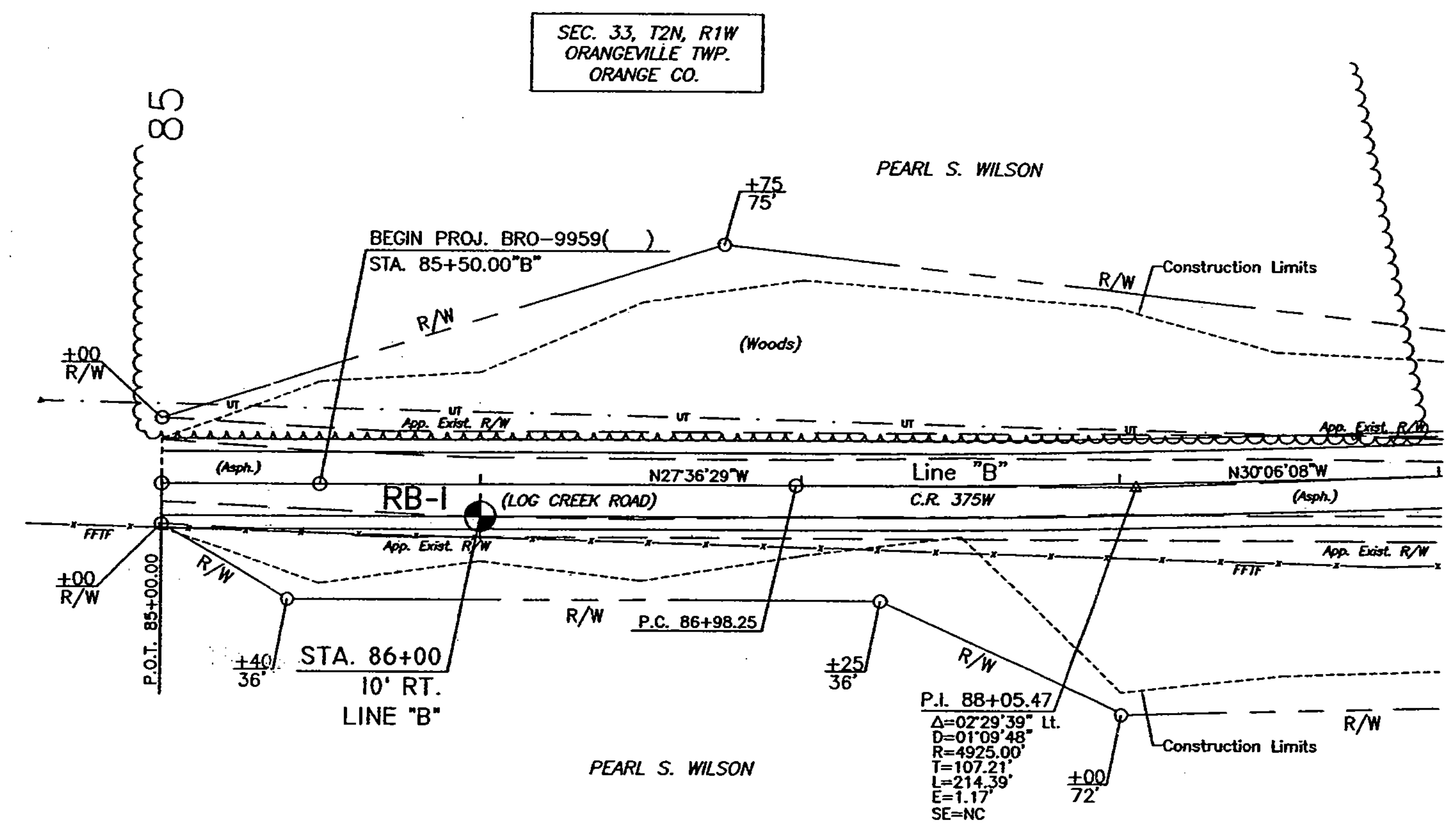


BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.	HORIZONTAL SCALE	BRIDGE FILE
	NO SCALE	ORANGE CO. BR. 34
	VERTICAL SCALE	DESIGNATION
	NO SCALE	9982490
GENERAL SITE PLAN	SURVEY BOOK	SHEETS
		1 of 8
	CONTRACT	PROJECT
		BRO-9959( )



BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.	HORIZONTAL SCALE 1"=2000'	BRIDGE FILE ORANGE CO. BR. 34
	VERTICAL SCALE 1"=2000'	DESIGNATION 9982490
SITE VICINITY MAP (U.S.G.S.)	SURVEY BOOK	SHEETS 2 of 8
	CONTRACT	PROJECT BRO-9959( )

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



Mag Nail SE. Side Of 8" Poplar, 12" Up; Mag Nail NE. Side Of 15" Poplar, 12" Up; Mag Nail NW. Side Of 15" Sycamore, 2' Up; 22.39'; 31.23'; 19.28'; Line "B"; R.R. Spk. Set	Mag Nail NE. Side Of 15" Oak, 12" Up; Mag Nail NE. Side Of 6" Tulip, 12" Up; Mag Nail SW. Side Of 8" Oak, 6" Up; 15.78'; 21.40'; 33.37'; Line "B"; R.R. Spk. Set	Mag Nail SE. Side Of 8" Oak, 6" Up; Mag Nail NE. Side Of 12" Poplar, 12" Up; Mag Nail S. Side Of 8" Elm, 4' Up; 23.70'; 23.66'; 23.33'; Line "B"; R.R. Spk. Set
P.O.T. 85+00 Line "B"	P.C. 86+98.25 Line "B"	P.I. 88+05.47 Line "B"

RECOMMENDED FOR APPROVAL ____ DESIGN ENGINEER ____ DATE

DESIGNED: B.R.L.	DRAWN: T.A.M.
CHECKED: C.R.B.	CHECKED: C.R.B.

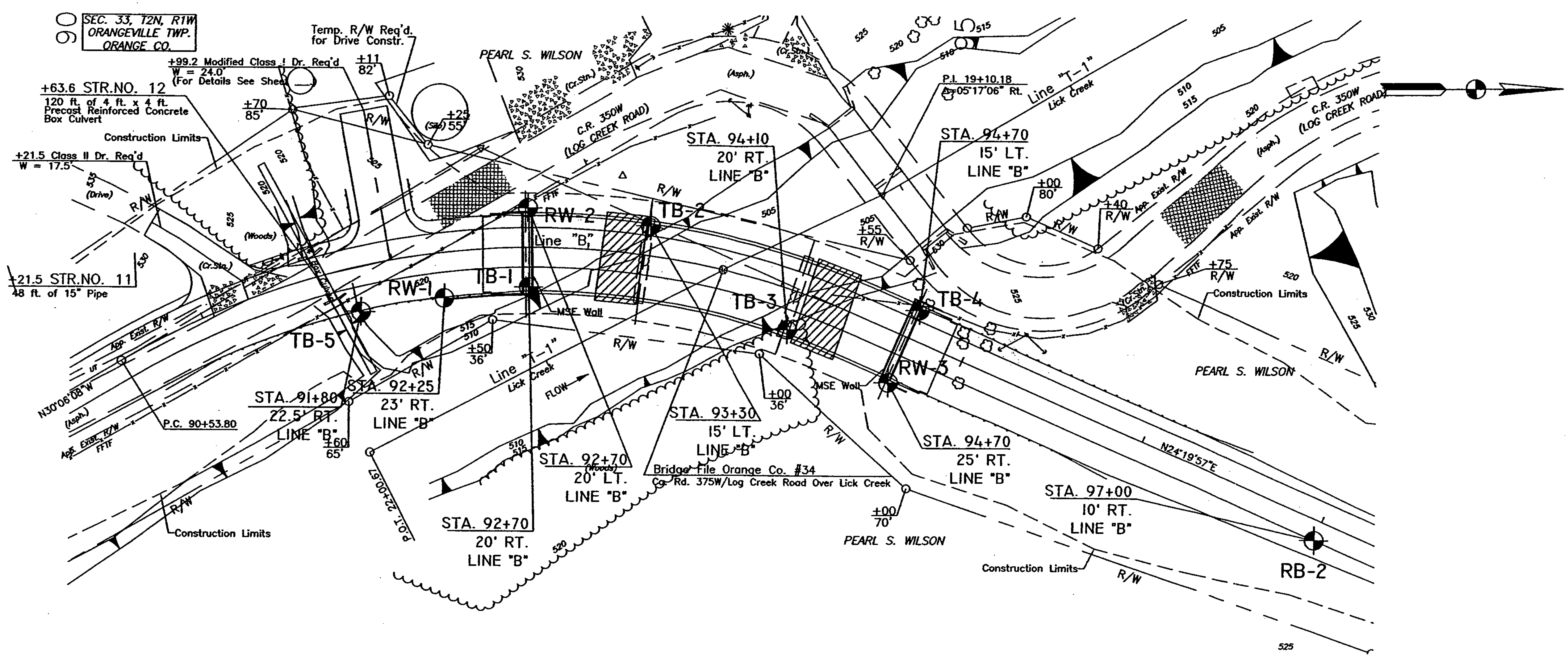
BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

TEST BORING
LOCATION PLAN

HORIZONTAL SCALE	BRIDGE FILE
1"=30'	Orange Co. Br. 34
VERTICAL SCALE	DESIGNATION
1"=10'	9982490
SURVEY BOOK	SHEETS
	3A of 8
CONTRACT	PROJECT
	BRO-9959( )

S:\Projects\199-0047\EP00\RCp01001.dwg 03/09/04 14:36 John

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



RECOMMENDED FOR APPROVAL	DESIGN ENGINEER	DATE
DESIGNED: B.R.L.	DRAWN: T.A.M.	
CHECKED: C.R.B.	CHECKED: C.R.B.	

BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

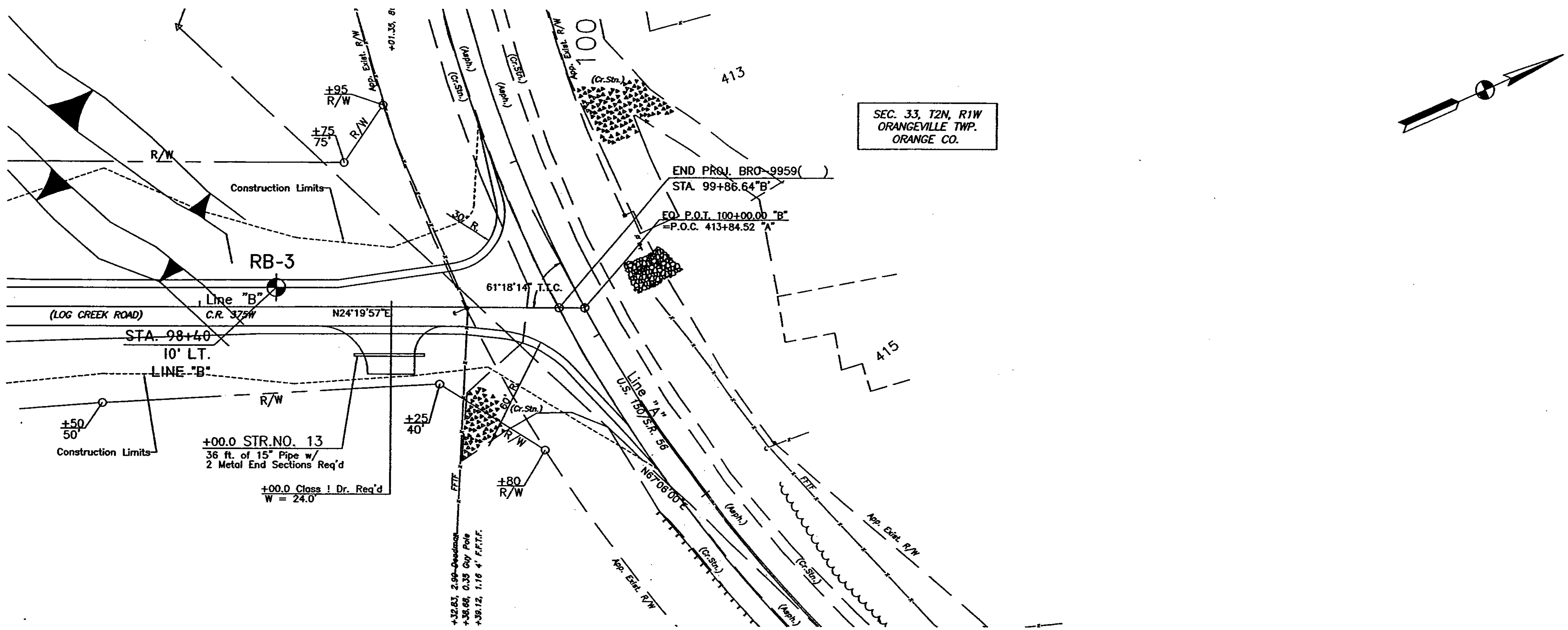
TEST BORING LOCATION PLAN

HORIZONTAL SCALE 1"=30'	BRIDGE FILE Orange Co. Br. 34
VERTICAL SCALE 1"=10'	DESIGNATION 9982490
SURVEY BOOK	SHEETS 3B of 8
CONTRACT	PROJECT BRO-9959( )

S:\Projects\199-0047\Bridge\Layout.dwg 03/25/04 09:36 vaughn

S:\Projects\199-0047\EP00\RCp03001.dwg 01/13/04 10:09 Vaughn

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



Mag Nail W. Side Dead Man Pole 12" Up; Mag Nail W. Side TFP #9/108 @ Base; 61.00'; 45.69'; 37.67'; Line "B"; R.R. Spk. Set; Line "A"; Mag Nail W. Side Fence Post 12" Up

P.O.T. 100+00.00 Line "B"

Mag Nail SW. Side Of 30" Ash; Mag Nail E. Side 36" Ash; Mag Nail N. Side Of 12" Box Elder; 44.64'; 37.00'; 48.36'; Line "A"; Brass Plug in Box Fnd.

P.C. 407+67.48 Line "A"

Mag Nail NE. Side Of TFP; Mag Nail NE. Side Of TFP; 142.98'; 120.83'; 96.5'; Line "A"; 3/4" RBR w/ Ref. Cap Flush; 5/8" Rebar w/ Firm Cap Set

P.I.412+49.65 Line "A"

Notch in S. Edge Sign Post (Curve Ahead); Brass Plug in Monument Box Fnd.; 37.00'; 29.15'; 50.10'; Line "A"; Mag Nail E. Side Of TFP; Mag Nail N. Side Of Cedar Fence Post

P.T. 416+16.23 Line "A"

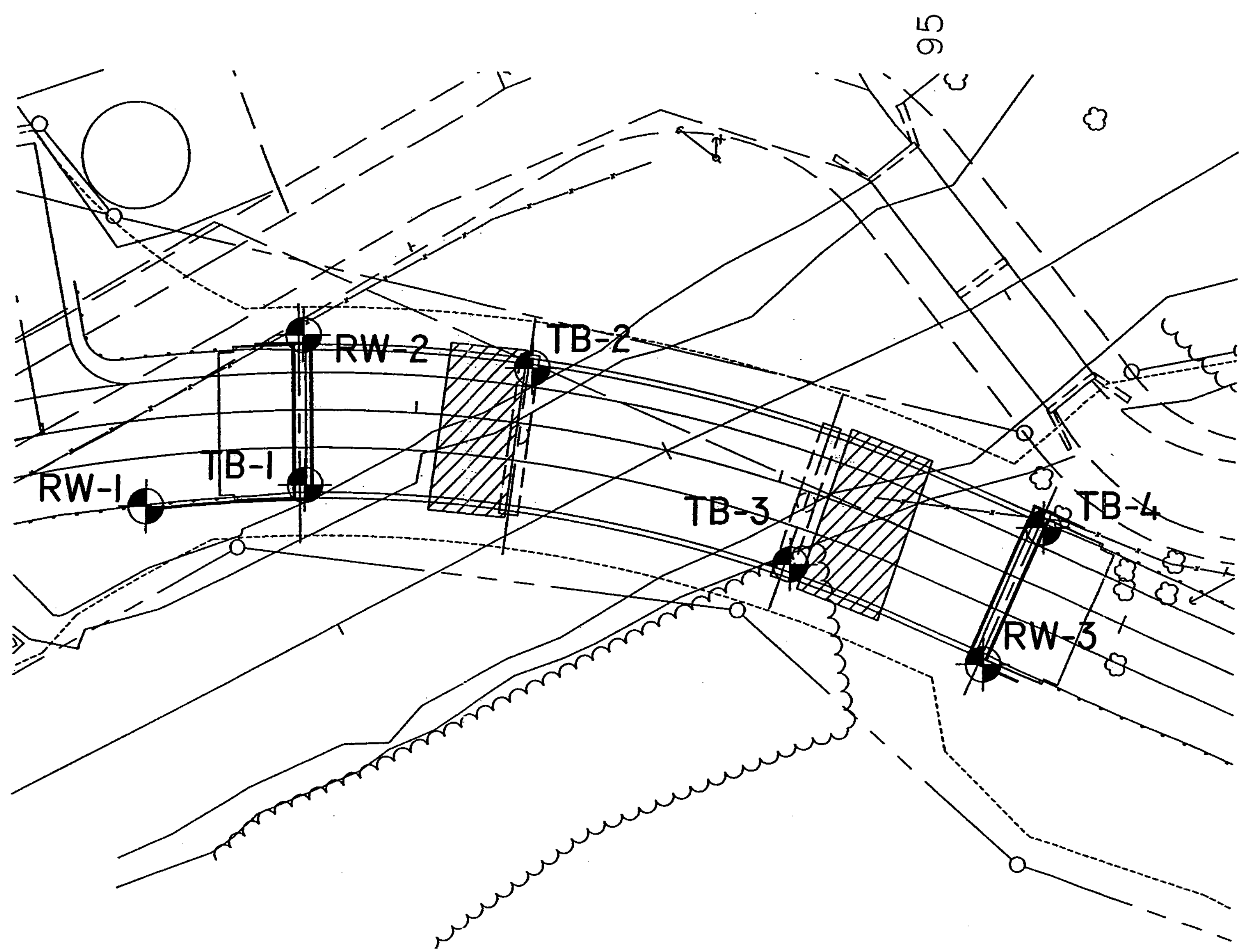
RECOMMENDED FOR APPROVAL		DESIGN ENGINEER	DATE
DESIGNED: B.R.L.		DRAWN: T.A.M.	
CHECKED: C.R.B.		CHECKED: C.R.B.	

BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

TEST BORING
LOCATION PLAN

HORIZONTAL SCALE 1"=30'	BRIDGE FILE Orange Co. Br. 34
VERTICAL SCALE 1"=10'	DESIGNATION 9982490
SURVEY BOOK	SHEETS 3C of 8
CONTRACT	PROJECT BRO-9959( )

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



P.T. 89+12.65 Line "B"	P.C. 90+53.80 Line "B"	P.I. 92+64.67 Line "B"	P.T. 94+43.33 Line "B"

RECOMMENDED FOR APPROVAL ______ DESIGN ENGINEER ______ DATE

DESIGNED: B.R.L. DRAWN: T.A.M.

CHECKED: C.R.B. CHECKED: C.R.B.

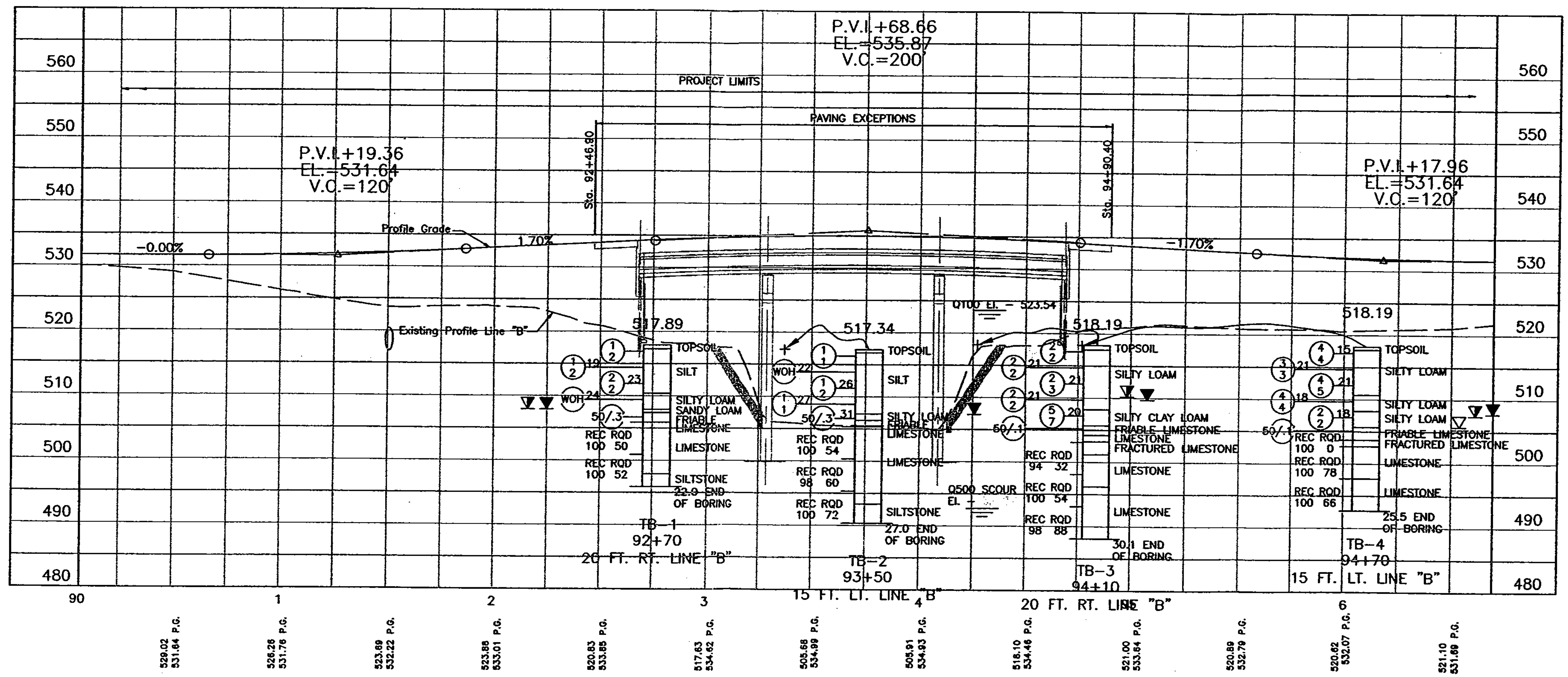
BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

BRIDGE BORING LOCATION PLAN

HORIZONTAL SCALE 1"=20'	BRIDGE FILE Orange Co. Br. 34
VERTICAL SCALE 1"=20'	DESIGNATION 9982490
SURVEY BOOK	SHEETS 4 of 8
CONTRACT	PROJECT BRO-9959( )

S:\Projects\199-0047\EP00\ROp02001.dwg 03/09/04 16:07 John

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



S:\Projects\199-0047\Bridge\Layout.dwg 03/25/04 09:36 vaughn

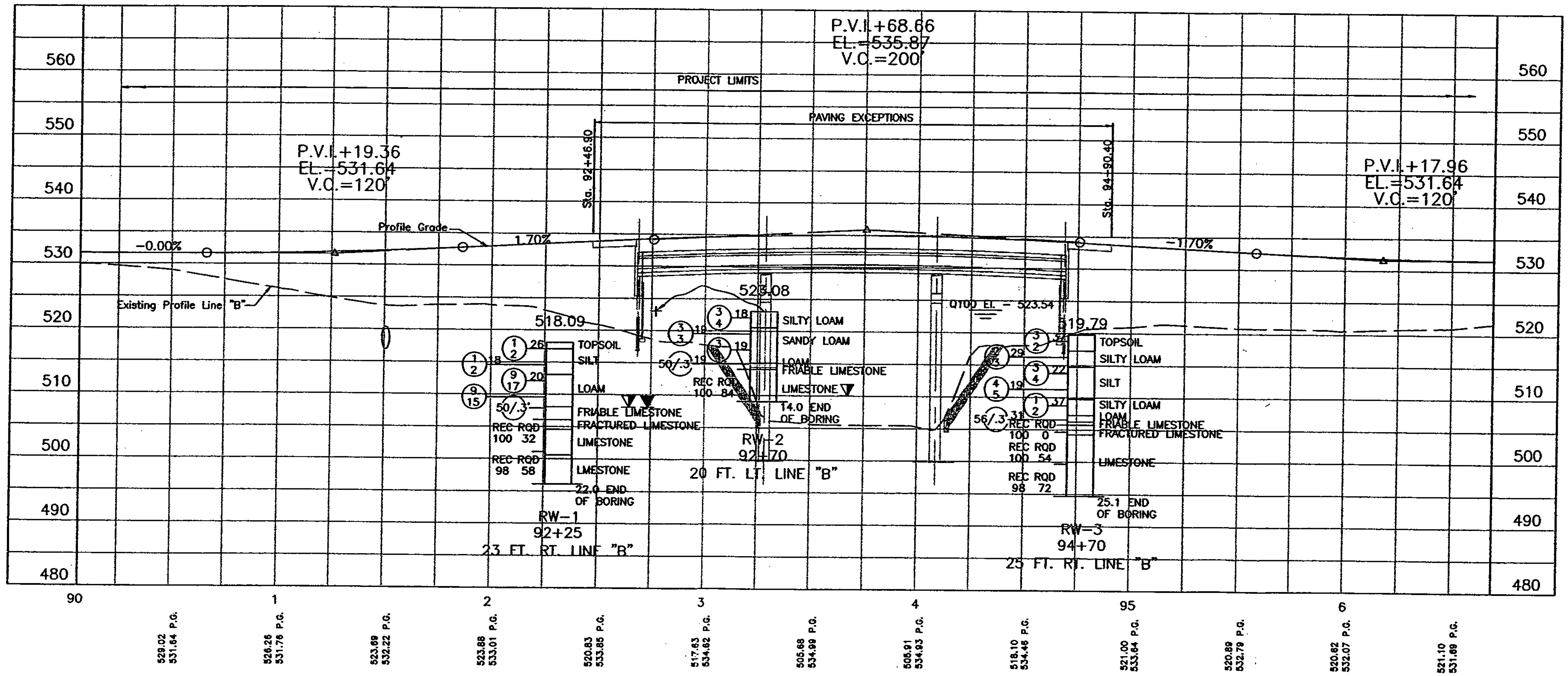
RECOMMENDED FOR APPROVAL	DESIGN ENGINEER	DATE
DESIGNED: B.R.L.	DRAWN: T.A.M.	
CHECKED: C.R.B.	CHECKED: C.R.B.	

BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

GENERAL SUBSURFACE CONDITIONS AT THE BRIDGE STRUCTURE

HORIZONTAL SCALE 1"=30"	BRIDGE FILE Orange Co. Br. 34
VERTICAL SCALE 1"=10"	DESIGNATION 9982490
SURVEY BOOK	SHEETS 5A of 8
CONTRACT	PROJECT BRO-9959( )

LOCAL ROAD
DESIGN SPEED 35 m.p.h.



S:\Projects\199-0047\Bridge\Layout.dwg 03/25/04 09:36 vaughn

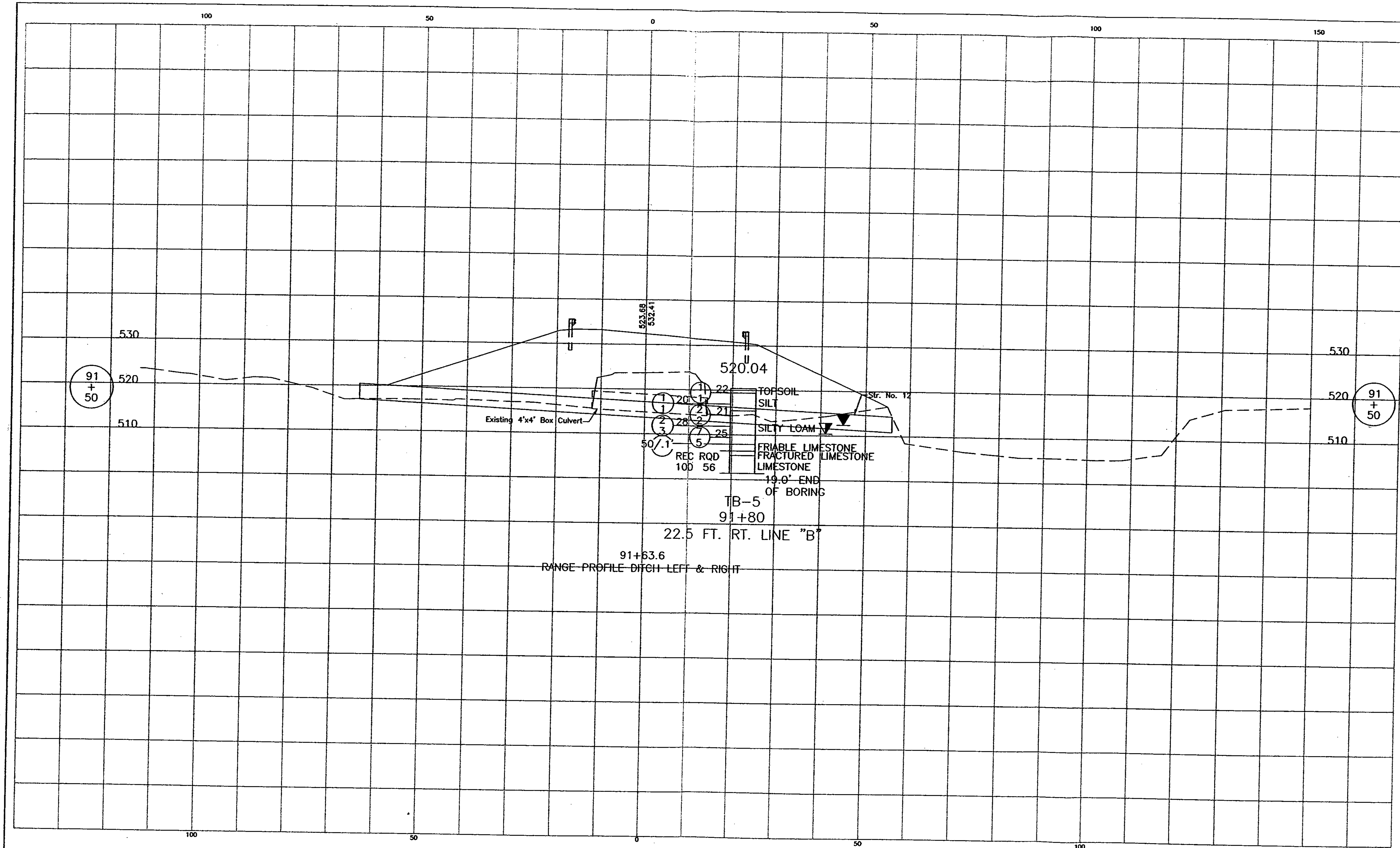
RECOMMENDED FOR APPROVAL ____ DESIGN ENGINEER ____ DATE

DESIGNED: B.R.L. DRAWN: T.A.M.
CHECKED: C.R.B. CHECKED: C.R.B.

BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

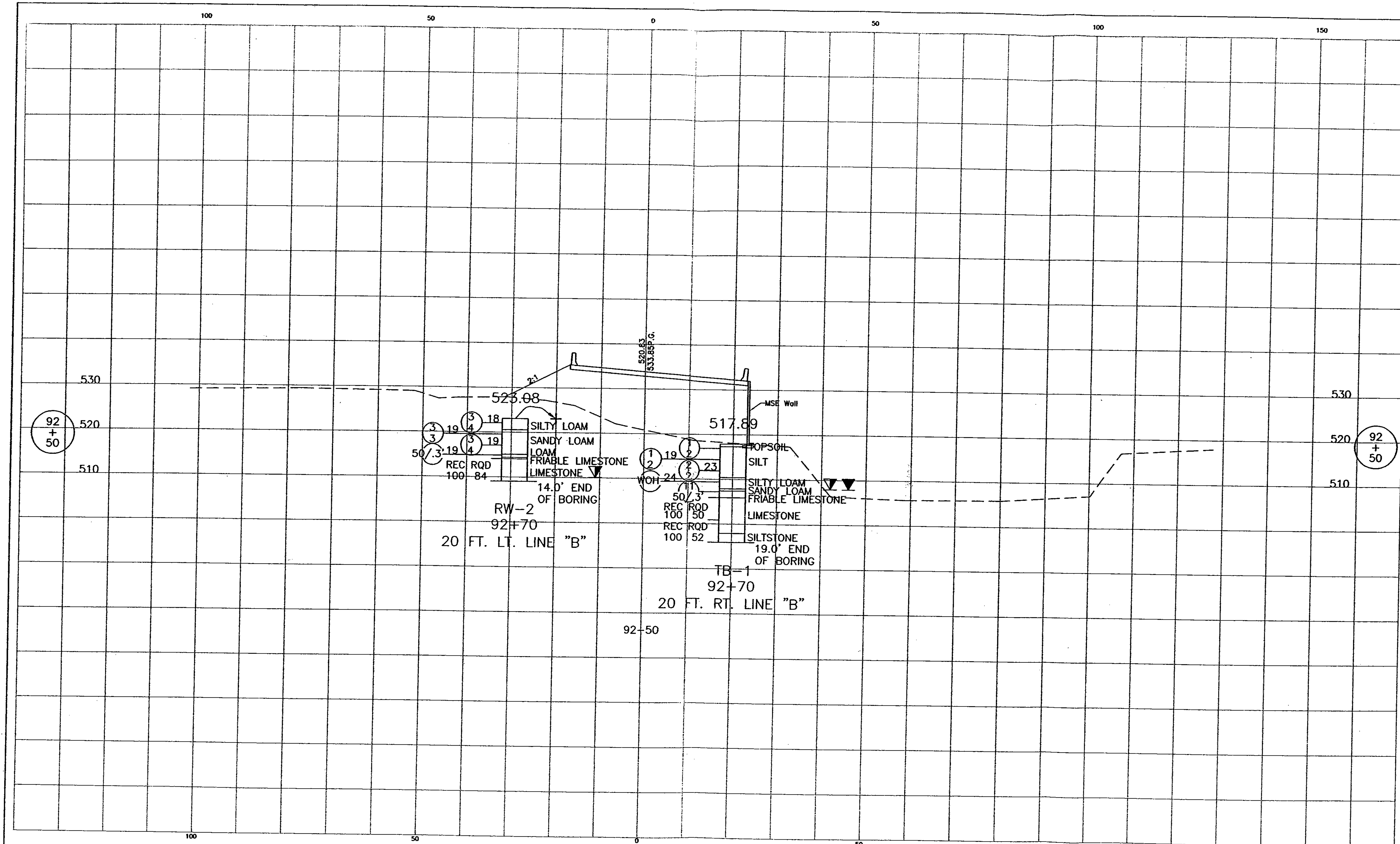
GENERAL SUBSURFACE CONDITIONS AT THE RETAINING STRUCTURE

HORIZONTAL SCALE 1"=30"	BRIDGE FILE Orange Co. Br. 34
VERTICAL SCALE 1"=10"	DESIGNATION 9982490
SURVEY BOOK	SHEETS 5B of 8
CONTRACT	PROJECT BRO-9959( )



DESIGNED: J.J.G.	DRAWN: T.A.M.	**BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.**	HORIZONTAL SCALE 1"=10' / VERTICAL SCALE 1"=10'
CHECKED: B.R.L.	CHECKED: B.R.L.	GENERAL SUBSURFACE CONDITIONS AT STA. 91+63.5	SURVEY BOOK / CONTRACT

BRIDGE FILE: ORANGE CO. BR. 34
DESIGNATION: 9982490
SHEETS 6 of 8
PROJECT

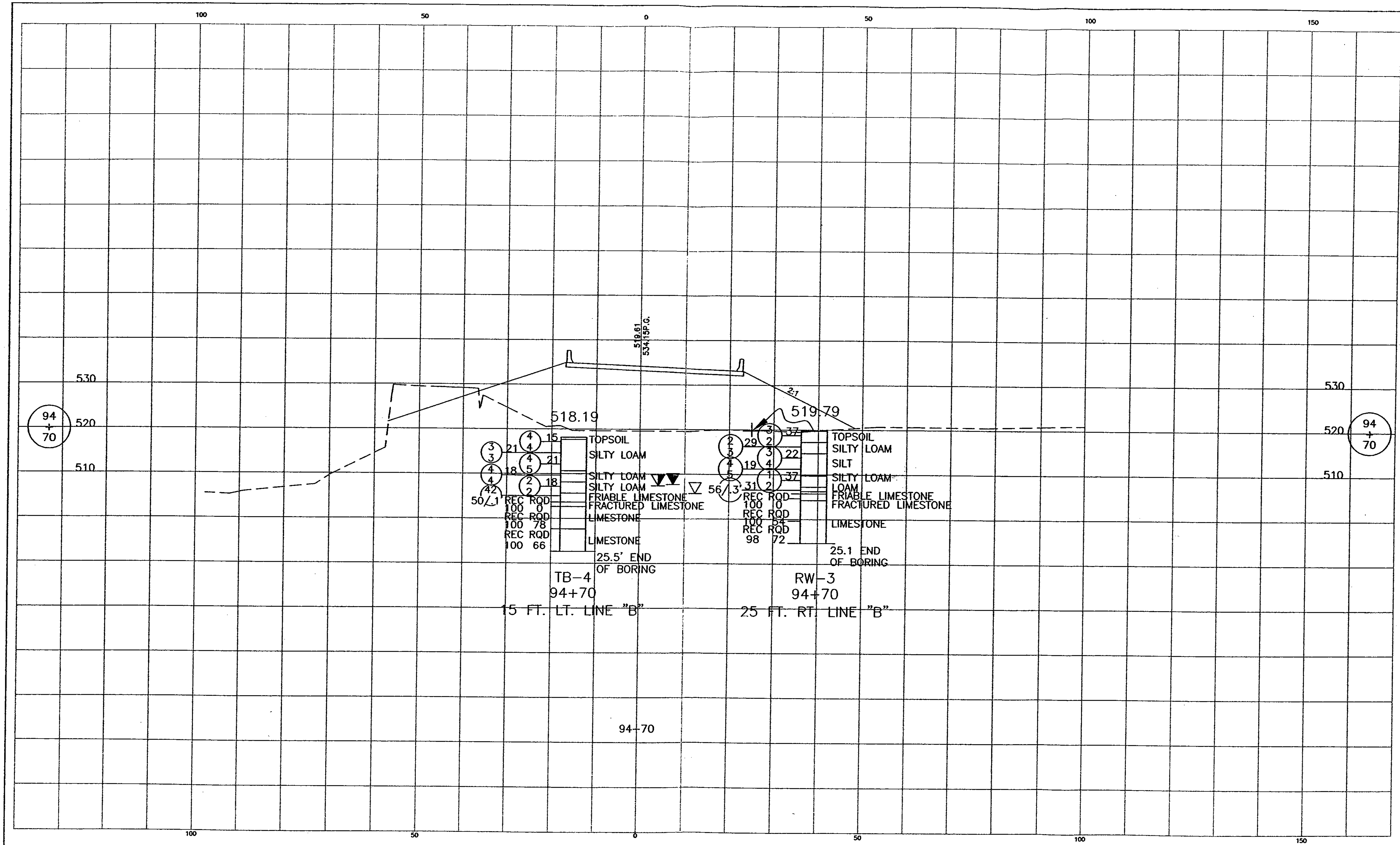


DESIGNED: J.J.G.	DRAWN: T.A.M.
CHECKED: B.R.L.	CHECKED: B.R.L.

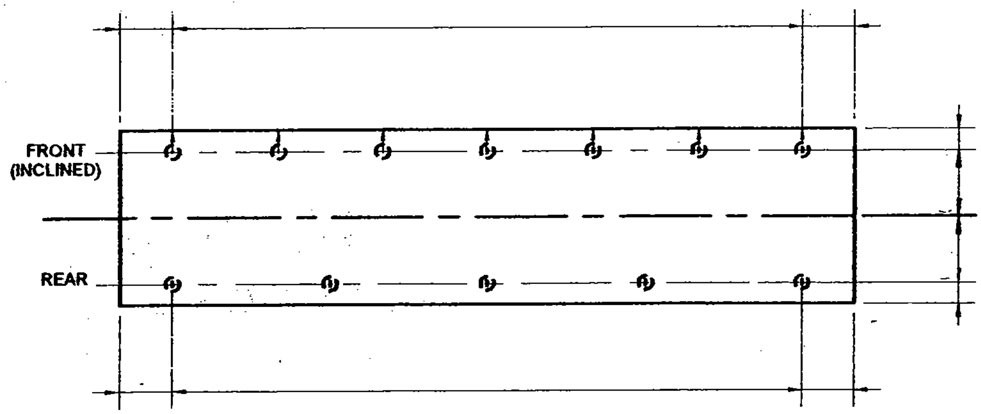
BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.

GENERAL SUBSURFACE CONDITIONS
AT STA. 92+50

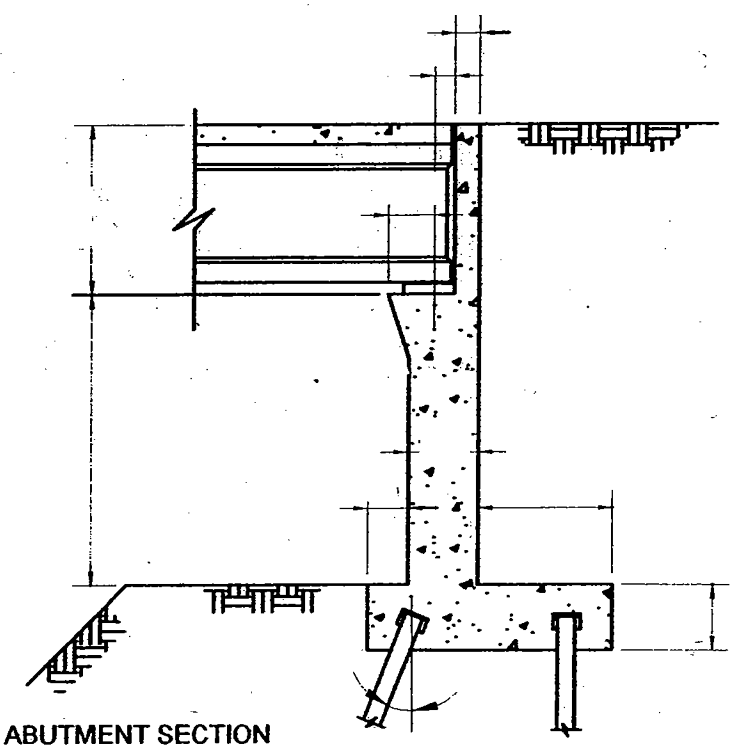
HORIZONTAL SCALE	BRIDGE FILE
1"=10'	ORANGE CO. BR. 34
VERTICAL SCALE	DESIGNATION
1"=10'	9982490
SURVEY BOOK	SHEETS
	7 of 8
CONTRACT	PROJECT



DESIGNED: J.J.G.	DRAWN: T.A.M.	**BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.**	HORIZONTAL SCALE 1"=10' / VERTICAL SCALE 1"=10'
CHECKED: B.R.L.	CHECKED: B.R.L.	GENERAL SUBSURFACE CONDITIONS AT STA. 94+70	BRIDGE FILE ORANGE CO. BR. 34 / DESIGNATION 9982490
			SURVEY BOOK / CONTRACT
			SHEETS 8 of 8 / PROJECT BRO-9959( )



ABUTMENT FOOTING – PLAN VIEW
NOT TO SCALE



ABUTMENT SECTION
NOT TO SCALE

H.C. NUTTING COMPANY
CORPORATE OFFICE – 611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

TYPICAL CONFIGURATION OF MICROPILES
SUPPORTING BRIDGE ABUTMENT
AND CAST-IN-PLACE RETAINING WALL

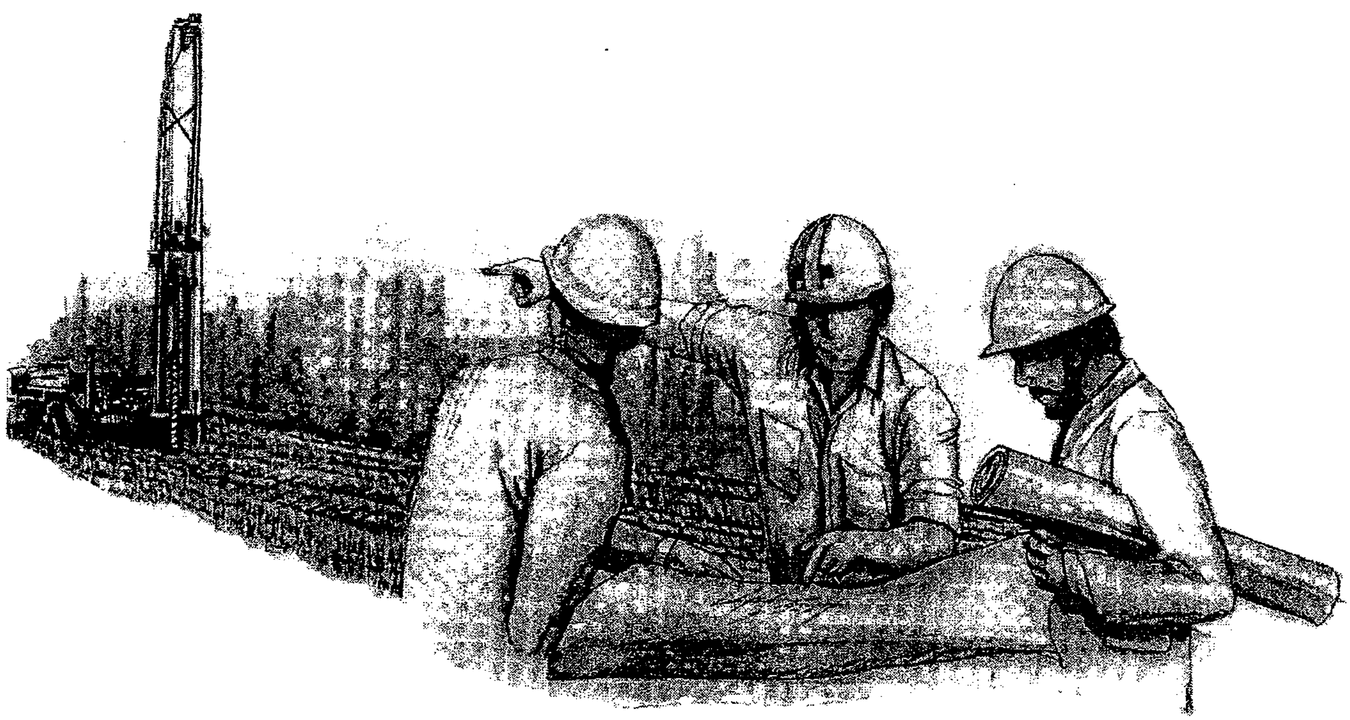
BRIDGE REPLACEMENT AND REALIGNMENT
CARRYING CR 375W OVER LICK CREEK
ORANGE COUNTY, INDIANA

SHEET 9

SCALE: NTS	08-27-04	DWG NO: 50043.009 DWG ID NO. TITLEBLK009

A description of terminology and symbols used in the logs of test borings, and a copy of ASTM D 2487, "Classification of Soils for Engineering Purposes", are included in the following two pages.

Readers of this report who wish an in-depth discussion on the basis for geotechnics, including procedures used in subsurface exploration, laboratory testing, and geotechnical analyses are referred to The H. C. Nutting Geotechnical and Test Engineering Manual. Those readers not having a copy of this manual may obtain one at nominal cost by contacting The H. C. Nutting Company at (513) 321-5816.



STANDARD PENETRATION TEST (INDOT)

THE PENETRATION RESISTANCE OR N-VALUE AS IT IS COMMONLY REFERRED TO IS THE SUMMATION OF THE NUMBER OF BLOWS REQUIRED TO DRIVE TWO SUCCESSIVE S'PENETRATIONS OF THE 2'OD SPLIT BARREL SAMPLER. THE SAMPLER IS DRIVEN WITH A 140 LB. WEIGHT FALLING 30' AND IS SEATED TO A DEPTH OF 6'BEFORE COMMENCING THE STANDARD PENETRANON TEST.

THE STANDARD PENETRATION TEST IS PERFORMED IN COMPLIANCE WITH PROCEDURES AS SET FORTH IN AASHTO T 206-87

TERMINOLOGY

GRAIN SIZE (PER INDOT EXHIBIT "C")

SOIL FRACTION		PARTICLE SIZE	U.S. STANDARD SIEVE SIZE
BOULDERS		LARGER THAN 10" (254mm)	LARGER THAN 10"
COBBLES		3' (75 mm) TO 10' (254 mm)	3" TO 10"
GRAVEL:	COARSE	¾" (19 mm) TO 3' (75 mm)	¾" TO 3"
	FINE	2.00 mm TO 19mm	#10 TO ¾"
SAND:	COARSE	8.425 mm TO 2.00 mm	#40 TO #10
	FINE	0.075 mm TO 0.425 mm	#200 TO #40
	(SILTS & CLAYS)	SMALLER THAN 0.075 mm	SMALLER THAN #200
FINES:	SILT	0.002 MM TO 0.075 MM	
	CLAY	SMALLER THAN 0.002 MM	

PLASTICITY CHARACTERISTICS DIFFERENTIATE BETWEEN SILTS AND CLAYS

RELATIVE DENSITY OF GRANULAR SOILS

TERM*	N VALUE
VERY LOOSE	0 – 5
LOOSE	6 – 10
MEDIUM DENSE	11 – 30
DENSE	31 – 50
VERY DENSE	OVER 50

*THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE AND SOIL DENSITY DATA.

RELATIVE PROPORTIONS OF COHESIONLESS SOILS

(Per INDOT EXHIBIT "C")

PROPORTIONAL TERM	DEFINING RANGE BY PERCENTAGE OF WEIGHT
SOME	20 TO 35%
AND	36 TO 50%

FOR RELATIVE PERCENTAGE OF GRAVELS, SAND AND FINES.

CONSISTENCY OF COHESIVE SOILS

TERM	N VALUE*	STRENGTH (QU, TSF)	IDENTIFICATION PROCEDURE
VERY SOFT	0 – 3	0 – 0.25	EASILY PENETRATED SEVERAL INCHES BY FIST.
SOFT	4 – 5	0.25 – 0.5	EASILY PENETRATED SEVERAL INCHES BY
MEDIUM STIFF	6 – 10	0.5 – 1.0	PENETRATED SEVERAL INCHES BY THUMB WITH MODERATE EFFORT.
STIFF	11 – 15	1.0 – 2.0	READILY INDENTED BY THUMB, BUT PENETRATED WITH GREAT EFFORT
VERY STIFF	16 – 30	2.0 – 4.0	READILY INDENTED BY THUMBNAIL
HARD	OVER 30	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

*N-value correction is approximate and typically only used in absence of actual filled or laboratory strength data.

RELATIVE PROPORTIONS OF COHESIONLESS SOILS

(INDOT EXHIBIT "C")

DRY	ABESENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
SLIGHTLY MOIST	
MOIST	DAMP BUT NO VISIBLE WATER
VERY MOIST	
WET	VISIBLE FREE WATER, USUALLY SOIL IS BELOW WATER TABLE

SYMBOLS

DRILLING AND SAMPLING

- RC – ROCK CORING: SIZE NW, NX = 2-1/8" diameter
- RQD – ROCK QUALITY DESIGNATION
- FT – FISH TAIL
- DC – DRIVE CASING
- C – CASING SIZE NW, 4", HW, 6"
- CW – CLEAR WATER
- DM – DRILLING MUD
- HAS – HOLLOW STEM AUGER
- FA – FLIGHT AUGER
- HA – HAND AUGER
- COA – CLEAN-OUT AUGER
- SS – 2" DIAMETER SPLIT BARREL SAMPLE
- ST – 3" DIAMETER THIN-WALLED TUBE SAMPLE
- PT – 3" DIAMETER PISTON TUBESAMPLE
- AS – AUGER SAMPLE
- WS – WASH SAMPLE
- PTS – PEAT SAMPLE
- PS – PITCHER SAMPLE
- NR – NO RECOVERY
- S – SOUNDING
- PMT – BOREHOLE PRESSUREMETER TEST
- VS – VANE SHEAR TEST
- WPT – WATER PRESSURE TEST
- ATV – ALL TERRAIN VEHICLE
- R – REFUSAL CONDITION

LABORATORY TESTS

- PP – PENETROMETER READING, TONS/SQ. FT.
- QU – UNCONFINED STRENGTH, TONS/SQ. FT.
- W – MOISTURE CONTENT, %
- LL – LIQUID LIMIT, %
- PL – PLASTIC LIMIT, %
- SL – SHRINKAGE LIMIT, %
- LOI – LOSS ON IGNITION, %
- D – DRY UNIT WEIGHT, LBS./CU. FT.
- PH – MEASURE OF SOIL ALKALINITY OR ACIDITY

WATER LEVER MEASUREMENT

- NW – NO WATER ENCOUNTERED
- WD – WHILE DRILLING
- BCR – BEFORE CASING REMOVAL
- ACR – AFTER CASING REMOVAL
- CM – CAVED AND MOIST
- BF – BACKFILLED UPON COMPLETION

NOTE: WATER LEVEL MEASUREMENTS SHOWN ON THE BORING LOGS REPRESENT CONDITIONS AT THE TIME INDICATED AND MAY NOT REFLECT STATIC LEVELS, ESPECIALLY IN COHESIVE SOILS

STANDARD PENETRATION TEST

THE PENETRATION RESISTANCE OR N-VALUE AS IT IS COMMONLY REFERRED TO IS THE SUMMATION OF THE NUMBER OF BLOWS REQUIRED TO DRIVE TWO SUCCESSIVE 6" PENETRATIONS OF THE 2" O.D. SPLIT BARREL SAMPLER. THE SAMPLER IS DRIVEN WITH A 140 LB. WEIGHT FALLING 30" AND IS SEATED TO A DEPTH OF 6' BEFORE COMMENCING THE STANDARD PENETRATION TEST.

THE STANDARD PENETRATION TEST IS PERFORMED IN COMPLIANCE WITH PROCEDURES AS SET FORTH IN ASTM D-1586.

TERMINOLOGY

GRAIN SIZE (PER ASTM D-2487)

SOIL FRACTION		PARTICLE SIZE	U.S. STANDARD SIEVE SIZE
BOULDERS		LARGER THAN 12" (300mm)	LARGER THAN 12"
COBBLES		3" (75 mm) TO 12" (300 mm)	3" TO 12"
GRAVEL:	COARSE	¾" (19 mm) TO 3" (75 mm)	¾" TO 3"
	FINE	4.75 mm TO 19mm	#4 TO ¾"
SAND:	COARSE	2.00 mm TO 4.75 mm	#10 TO #4
	MEDIUM	0.425 mm TO 2.00 mm	#40 TO #10
	FINE	0.075 mm TO 0.425 mm	#200 TO #40
FINES:	(SILTS & CLAYS)	SMALLER THAN 0.075 mm	SMALLER THAN #200

PLASTICITY CHARACTERISTICS DIFFERENTIATE BETWEEN SILTS AND CLAYS

RELATIVE DENSITY OF GRANULAR SOILS

TERM*	N VALUE
VERY LOOSE	0 – 4
LOOSE	5 – 10
MEDIUM DENSE	11 – 29
DENSE	30 – 50
VERY DENSE	OVER 50

*THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE AND SOIL DENSITY DATA.

RELATIVE PROPORTIONS OF COHESIONLESS SOILS

(Per ASTM D2488)

PROPORTIONAL TERM	DEFINING RANGE BY PERCENTAGE OF WEIGHT
TRACE	<5%
FEW	5 TO 10%
LITTLE	15 TO 25%
SOME	30 TO 45%

FOR RELATIVE PERCENTAGE OF GRAVELS, SAND AND FINES.

CONSISTENCY OF COHESIVE SOILS

TERM	N VALUE*	STRENGTH (QU, TSF)	IDENTIFICATION PROCEDURE
VERY SOFT	0 – 2	0 – 0.25	EASILY PENETRATED SEVERAL INCHES BY FIST.
SOFT	3 – 4	0.25 – 0.5	EASILY PENETRATED SEVERAL INCHES BY THUMB
MEDIUM STIFF	5 – 8	0.5 – 1.0	PENETRATED SEVERAL INCHES BY THUMB WITH MODERATE EFFORT.
STIFF	9 – 15	1.0 – 2.0	READILY INDENTED BY THUMB, BUT PENETRATED WITH GREAT EFFORT
VERY STIFF	16 – 30	2.0 – 4.0	READILY INDENTED BY THUMBNAIL
HARD	OVER 30	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

*N-value correction is approximate and typically only used in absence of actual field or laboratory strength data.

RELATIVE PROPORTIONS OF COHESIONLESS SOILS

(Per ASTM D2488)

DRY	ABESENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL IS BELOW WATER TABLE

SYMBOLS

DRILLING AND SAMPLING

- RC – ROCK CORING: SIZE NW, NX = 2-1/8" diameter
- RQD – ROCK QUALITY DESIGNATION
- FT – FISH TAIL
- DC – DRIVE CASING
- C – CASING SIZE NW, 4", HW, 6"
- CW – CLEAR WATER
- DM – DRILLING MUD
- HAS – HOLLOW STEM AUGER
- FA – FLIGHT AUGER
- HA – HAND AUGER
- COA – CLEAN-OUT AUGER
- SS – 2" DIAMETER SPLIT BARREL SAMPLE
- ST – 3" DIAMETER THIN-WALLED TUBE SAMPLE
- PT – 3" DIAMETER PISTON TUBE SAMPLE
- AS – AUGER SAMPLE
- WS – WASH SAMPLE
- PTS – PEAT SAMPLE
- PS – PITCHER SAMPLE
- NR – NO RECOVERY
- S – SOUNDING
- PMT – BOREHOLE PRESSUREMETER TEST
- VS – VANE SHEAR TEST
- WPT – WATER PRESSURE TEST
- ATV – ALL TERRAIN VEHICLE
- R – REFUSAL CONDITION

LABORATORY TESTS

- PP – PENETROMETER READING, TONS/SQ. FT.
- QU – UNCONFINED STRENGTH, TONS/SQ. FT.
- W – MOISTURE CONTENT, %
- LL – LIQUID LIMIT, %
- PL – PLASTIC LIMIT, %
- SL – SHRINKAGE LIMIT, %
- LOI – LOSS ON IGNITION, %
- D – DRY UNIT WEIGHT, LBS./CU. FT.
- PH – MEASURE OF SOIL ALKALINITY OR ACIDITY

WATER LEVER MEASUREMENT

- NW – NO WATER ENCOUNTERED
- WD – WHILE DRILLING
- BCR – BEFORE CASING REMOVAL
- ACR – AFTER CASING REMOVAL
- CM – CAVED AND MOIST
- BF – BACKFILLED UPON COMPLETION

NOTE: WATER LEVEL MEASUREMENTS SHOWN ON THE BORING LOGS REPRESENT CONDITIONS AT THE TIME INDICATED AND MAY NOT REFLECT STATIC LEVELS, ESPECIALLY IN COHESIVE SOILS

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
790 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT., STE B-8
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	**RB-1**
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/12/2004
Boring Location	Station 86+00, 10' Rt., Line "B" Orange County, IN	Date Completed	4/12/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)
529.00	0.0		
528.50	0.5	0.5	Asphaltic Concrete Pavement (Visual)
528.30	0.7	0.2	
526.50	2.5	1.8	Brown and trace gray crushed limestone fragments (BASE), moist, loose (Visual)
522.50	6.5	4.0	Reddish brown silty clay (FILL), moist, medium stiff, - with occasional limestone fragments and silt seams A-7-6, Lab No. 2848
521.50	7.5	1.0	Brown SILTY CLAY, moist, medium stiff to very stiff, A-7-6 (38), Lab No. 2848
			Reddish brown SILTY CLAY, moist, very stiff, - with occasional limestone fragments/floaters A-7-6, Lab No. 2848
			BORING COMPLETED @ 7.5'

SAMPLE

NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
1	SS	0.5-2.0 1.0-2.0	4-4-3 (7)	60		37	77	48		1.75
2	SS	2.5-4.0	3-4-4 (8)	100		26	56	34		2.75
3	SS	6.0-7.5 6.5-7.5	18-6-12 (18)	100		31				2.5 2.5

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 7' following removal of augers

Water Level Observations

Immediate	DRY ft.
At Completion	DRY ft.
After	0 Hrs. BF ft.
Water used in drilling	NONE ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

LOG OF TEST BORING

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
790 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT., STE B-8
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	RB-2
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 97+00, 10' Rt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.		DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	SAMPLE NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
522.79	0.0													
521.99	0.8	0.8	Dark brown silty clay (TOPSOIL), moist, medium stiff, - with frequent silt seams (Visual)	1	SS	0.0-1.5 1.0-1.5	4-4-4 (8)	100		22				
520.29	2.5	1.7	Brown SILTY LOAM, moist, medium stiff, A-6, Lab No. 2884	2	SS	2.5-4.0	2-2-2 (4)	100		26	28	8		1.75
517.79	5.0	2.5	Brown SILT, moist, soft, A-4 (7), Lab No. 2852											
			Reddish brown CLAY, moist, medium stiff to stiff, - with occasional limestone fragments at 6.5' A-7-6, Lab No. 2848	3	SS	5.0-6.5	3-4-5 (9)	100		22			1.78	2.25
512.79	10.0	5.0		4	SS	8.5-10.0	3-7-8 (15)	33		28	49	26		2.25
			BORING COMPLETED @ 10.0'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 9' following removal of augers

Water Level Observations

Immediate	DRY ft.
At Completion	DRY ft.
After	24 Hrs. DRY ft.
Water used in drilling	NONE ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	RB-3
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 98+40, 10' Lt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	SAMPLE NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
531.64	0.0												
530.84	0.8	0.8 Dark brown silty clay (TOPSOIL), moist, medium stiff (Visual)	1	SS	0.0-1.5 1.0-1.5	3-3-4 (7)	100		22				2.0
529.14	2.5	1.7 Brown SILTY LOAM, moist, medium stiff, A-6, Lab No. 2884											
		Reddish brown CLAY, moist, medium stiff, A-7-6, Lab No. 2848	2	SS	2.5-4.0	2-3-4 (7)	100		24				2.25
525.64	6.0	3.5											
524.14	7.5	1.5 Brown CLAY, moist, very stiff, - with occasional limestone fragments/floaters A-7-6, Lab No. 2848	3	SS	6.0-7.5	7-9-13 (22)	100		30				2.5
		BORING COMPLETED @ 7.5'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 7' following removal of augers

Water Level Observations

Immediate	DRY	ft.
At Completion	DRY	ft.
After	0 Hrs. BF	ft.
Water used in drilling	NONE	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
790 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT., STE B-8
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	RW-1
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/11/2004
Boring Location	Station 92+25, 23' Rt., Line "B" Orange County, IN	Date Completed	4/11/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	SAMPLE NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
518.09	0.0												
517.09	1.0	1.0 Dark brown silty clay (TOPSOIL), moist, very soft (Visual)	1	SS	0.0-1.5 1.0-1.5	2-1-2 (3)	100		26				0.75
513.09	5.0	4.0 Brown SILT, moist, very soft, - with frequent silt seams/layers; with occasional fine roots A-4, Lab No. 2852	2	SS	2.5-4.0	1-1-2 (3)	80		18				1.25
		5.0 Brown LOAM, moist, very stiff, - with occasional limestone fragments/floaters at 6'~9' A-4 (3), Lab No. 2860	3	SS	5.0-6.5	5-9-17 (26)	100		20	25	10		1.0
508.09 ▼	10.0		4	SS	7.5-9.0	6-9-15 (24)	47						
506.09	12.0	2.0 Brown and gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints	5	SS	10.0-10.7	45-50/.3'	43						
504.59	13.5	1.5 Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely to closely spaced, rough, open, and moderately to steeply dipping (high angle) joints; occurs in 1" to 4" pieces	1	RC	12.0-17.0		100	32				792	
500.59	17.5	4.0 Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close spaced, rough, tight, and moderately dipping joints; occurs in 6"~12"; occasional open joints and fractured rock zone (high angle joints) at 17', occasional steeply dipping joints at 14.5' and 16.5', trace to some fractured stylolitic joints were observed in the core runs.											
496.09	22.0	4.5 Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close to widely spaced, rough, tight, and moderately dipping joints; occurs well distributed in 6" to 12" pieces; occasional vertically dipping joints at 20'~22'	2	RC	17.0-22.0		98	58				823	
		BORING COMPLETED @ 22.0'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 20' following removal of augers

Water Level Observations

Immediate	DRY ft.
At Completion	10.0 ft. ▽
After 24 Hrs.	10.0 ft. ▼
Water used in drilling	12.0 ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	RW-2
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/12/2004
Boring Location	Station 92+70, 20' Lt., Line "B" Orange County, IN	Date Completed	4/12/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	SAMPLE NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
523.08	0.0													
520.58	2.5	2.5	Brown SILTY LOAM, moist, medium stiff, A-4, Lab No. 2852	1	SS	0.0-1.5	1-3-4 (7)	33		18				1.5
515.08	8.0	5.5	Reddish brown and brown SANDY LOAM, moist, medium stiff, A-6 (2), Lab No. 2863	2	SS	2.5-4.0	4-3-3 (6)	33		19				2.0
				3	SS	5.0-6.5	2-3-4 (7)	53		19	29	13		1.75
514.38	8.7	0.7	Brown and reddish brown LOAM, moist, medium stiff to hard, - with occasional limestone fragments A-4, Lab No. 2860	4	SS	7.5-8.7	2-3-50/.3'	0		19	27	12		1.5
514.08	9.0	0.3	Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints											
509.08	14.0	5.0	Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close to widely spaced, rough, tight, and moderately dipping joints; occurs well distributed in 18" to 30" layers; occasional open, high angle joints and fractured rock fragments at 13'~14',, but the remaining core run are well cemented.	1	RC	9.0-14.0		100	84				837	

BORING COMPLETED @ 14.0'

General Notes		Remarks	Water Level Observations	
Driller	J. Gilbert	Project No. BRO-9959(), Designation No. 9982490	Immediate	DRY ft.
Rig No.	550	(1) Cave-in at a depth of 13' following removal of augers, (2) Circulating	At Completion	13 ft. ▼
Rig Type	ATV	water lost at 13'	After	0 Hrs. BF ft.
Method	RC/SS		Water used in drilling	9.0 ft.
Inspector			BF = BACKFILLED NW = NO WATER (Measured from ground surface)	

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
790 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-8 CONWAY CT., STE B-8
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	RW-3
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2003
Boring Location	Station 94+70, 25' Rt., Line "B" Orange County, IN	Date Completed	4/10/2003
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
519.79	0.0													
517.29	2.5	2.5	Dark brown silty clay (TOPSOIL), moist, soft, with noted tree stems (Visual)	1	SS	0.0-1.5	4-3-2 (5)	100						
514.79	5.0	2.5	Brown SILTY LOAM, very moist, soft, - with occasional silt seams/layers A-6, Lab No. 2884	2	SS	2.5-4.0	3-2-3 (5)	67		29				0.75
509.79	10.0	5.0	Brown SILT, moist, medium stiff, - with occasional wet silt seams A-4, Lab No. 2852	3	SS	5.0-6.5	2-3-4 (7)	100		22				1.25
				4	SS	7.5-9.0	3-4-5 (9)	100		19			0.96	2.25
507.29	12.5	2.5	Brown and trace gray SILTY LOAM, wet, very loose, A-4 (1), Lab No. 2868	5	SS	10.0-11.5	2-1-2 (3)	100			21	6		
506.29	13.5	1.0	Brown LOAM, very moist to wet, soft to hard, - with occasional limestone fragments and clay seams A-4, Lab No. 2860	6	SS	12.5-13.7	2-2-56/.3'	100		31	27	10		1.75
505.79	14.0	0.5												
504.29	15.5	1.5	Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained	1	RC	14.0-15.1		100	0					
			Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely to closely spaced, rough, open, and steeply deeping (high angle) joints; occurs in 1" to 6" pieces.	2	RC	15.1-20.1		100	54				202 to 817	
494.69	25.1	9.6	Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close to widely spaced, rough, tight, and moderately dipping joints; occasional steeply dipping (high angle) joints at 17'~19' and 23'~24'; occurs well distributed in 12" to 18" layers;, occasional fractured limestone zone at 18', but generally well cemented.	3	RC	20.1-25.1		98	72					
			BORING COMPLETED @ 25.1'											

General Notes	
Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

(1) Cave-in at a depth of 21' following removal of augers

Water Level Observations		
Immediate	DRY	ft.
At Completion	DRY	ft.
After	0 Hrs. BF	ft.
Water used in drilling	14.0	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-1
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/11/2004
Boring Location	Station 92+70, 20' Rt., Line "B" Orange County, IN	Date Completed	4/11/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
517.89	0.0													
517.29	0.6	0.6	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)	1	SS	0.0-1.5 / 1.0-1.5	1-1-2 (3)	100						0.75
		6.9	Brown SILT, moist, very soft to soft, - with frequent silt seams/layers; with noted tree stems A-4, Lab No. 2852	2	SS	2.5-4.0	1-1-2 (3)	100		19				0.75
510.39	7.5			3	SS	5.0-6.5	1-2-2 (4)	100		23				0.5
507.89 ▼	10.0	2.5	Brown SILTY LOAM, very moist to wet, very loose, - with frequent wet silt and sand seams A-4, Lab No. 2868	4	SS	7.5-9.0	WOH	100		24	22	4		
507.39	10.5	0.5	Brown SANDY LOAM, wet, medium dense, - with frequent wet silt seams and limestone fragments A-4 (0), Lab No. 2873	5	SS	10.0-11.2	1-11-50/.3'	100						
505.89	12.0	1.5	Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained	1	RC	12.0-17.0		100	50				355 to 689	
497.89	20.0	8.0	Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close spaced, rough, tight, and moderately dipping joints; occasional steeply dipping joints at 13.5' and 15' to 16', occurs well distributed in 4" to 18" layers;, occasional open joints and fractured rock fragments at 12', 17.5', and 19.5'	2	RC	17.0-22.0		100	52				678	
495.89	22.0	2.0	Brown and gray SILTSTONE, hard, fine-grained, trace calcareous, moderately close spaced, smooth, tight, and moderately dipping joints; occasional high angle joints at 21'											
			BORING COMPLETED @ 22.0'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 20' following removal of augers
(2) WOH= Weight of Hammer

Water Level Observations

Immediate	DRY ft.
At Completion	10.0 ft. ▽
After 24 Hrs.	10.0 ft. ▼
Water used in drilling	12.0 ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION 912 MORRIS STREET CHARLESTON, WV 25031 (304) 344-0821 FAX (304) 342-4711
CENTRAL OHIO REGION 790 MORRISON ROAD COLUMBUS, OH 43230 (614) 863-3113 FAX (614) 863-0475
INDIANA REGION 349 WALNUT STREET, STE 8 LAWRENCEBURG, IN 47025 (812) 539-4300 FAX (812) 539-4301
BLUEGRASS REGION 470-B CONWAY CT., STE B-8 LEXINGTON, KY 40511 (859) 455-8530 FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-2
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/11/2004
Boring Location	Station 93+30, 15' Lt., Line "B" Orange County, IN	Date Completed	4/11/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
517.34	0.0													
516.84	0.5	0.5	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)	1	SS	0.0-1.5	1-1-1 (2)	67						
		9.5	Brown SILT, moist, very soft, - with occasional silt seams/layers A-4, Lab No. 2852	2	SS	2.5-4.0	WOH-1	80		22				0.5
				3	SS	5.0-6.5	1-1-2 (3)	27		26				0.5
				4	SS	7.5-9.0	1-1-1 (2)	33		27				0.75
507.34	10.0													
506.34	11.0	1.0	Brown and trace gray SILTY LOAM, very moist, very soft, - with occasional wet silt seams A-4 (6), Lab No. 2877	5	SS	10.0-11.2	WOH-50/.3'	83		31	28	10		0.25
505.34	12.0	1.0	Gray and trace brown FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints	1	RC	12.0-17.0		100	54				921	
		12.0	Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close to widely spaced, rough, tight, and moderately dipping joints, occasional steeply dipping joints at 15'~16', 21'~22', and 23'~24', occurs well distributed in 12" to 24" layers,, occasional fractured rock zones (high angle joints) at 15' and 19', but generally well cemented	2	RC	17.0-22.0		98	60				851	
493.34	24.0													
490.34	27.0	3.0	Brown and gray SILTSTONE, hard, fine-grained, trace calcareous, moderately close spaced, smooth, tight, and moderately dipping joints; occasional high angle joints and fractured rock zone at 25'	3	RC	22.0-27.0		100	72					
			BORING COMPLETED @ 27.0'											

General Notes		Remarks	Water Level Observations	
Driller	J. Gilbert	Project No. BRO-9959(), Designation No. 9982490	Immediate	DRY ft.
Rig No.	550	WOH: Weight of Hammer	At Completion	10.0 ft. ▽
Rig Type	ATV		After 24 Hrs.	10.0 ft. ▼
Method	RC/SS		Water used in drilling	12.0 ft.
Inspector			BF = BACKFILLED NW = NO WATER (Measured from ground surface)	

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	**TB-3**
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/9/2004
Boring Location	Station 94+110, 20' Rt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

DESCRIPTION OF MATERIALS

color, material description, moisture, stiffness/density/hardness

ELEV. ft.	DEPTH ft.	Thickness	Description
518.19	0.0		
517.49	0.7	0.7	Dark brown silty clay (TOPSOIL), moist, soft (Visual)
508.19	10.0	9.3	Brown SILTY LOAM, moist, soft, - with occasional silt seams/layers; became very moist to wet at 7.5' A-4, Lab No. 2868
505.69	12.5	2.5	Brown SILTY CLAY LOAM, very moist, stiff, - with occasional wet silt seams and soft zones A-4, Lab No. 2877
504.19	14.0	1.5	Brown and gray FRIABLE LIMESTONE AND CLAY SEAMS/LAYERS, - Limestone, moderately hard and fine-grained - Clay, very stiff and moist
503.19	15.0	1.0	
501.19	17.0	2.0	
496.19	22.0	5.0	Gray LIMESTONE, hard to very hard, fine-grained, moderately spaced, rough, tight, and moderately dipping joints Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and moderately dipping joints; occurs well distributed in 1" to 3" pieces.
488.09	30.1	8.1	Gray LIMESTONE, very hard, calcareous, fine-grained, moderately close to widely spaced, rough, and moderately dipping joints; occasional steeply dipping (high angle) joints at 21'-22'; occurs well distributed in 6" to 24" layers Gray LIMESTONE, very hard, calcareous, slightly crystalline, widely spaced, rough, and moderately dipping joints; well distributed in 12" to 30" layers; interbedded with occasional siltstone seams/layers
			BORING COMPLETED @ 30.1'

SAMPLE

NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
1	SS	0.0-1.5 1.0-1.5	2-2-2 (4)	100						0.75
2	SS	2.5-4.0	2-2-2 (4)	100		21				0.5
3	SS	5.0-6.5	2-2-3 (5)	67		21				0.75
4	SS	7.5-9.0	2-2-2 (4)	53		21				0.5
5	SS	10.0-11.5	5-5-7 (12)	100		20			0.28	0.25
6	SS	12.5-13.5	27-39-50/.1'	80						
1	RC	14.0-15.1		100	45				784	
2	RC	15.1-20.1		94	32				560	
3	RC	20.1-25.1		100	54					
4	RC	25.1-30.1		98	88				876	

General Notes		Remarks
Driller	J. Gilbert	Project No. BRO-9959(), Designation No. 9982490
Rig No.	550	
Rig Type	ATV	
Method	RC/SS	
Inspector		

Water Level Observations

Immediate	DRY	ft.
At Completion	8.0	ft. ▽
After 24 Hrs.	8.5	ft. ▼
Water used in drilling	14.0	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-4
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 94+70, 15' Lt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
518.19	0.0													
517.69	0.5	0.5	Dark brown silty clay (TOPSOIL), moist, medium stiff (Visual)	1	SS	0.0-1.5 1.0-1.5	4-4-4 (8)	100		15				2.5
		7.0	Brown SILTY LOAM, moist, medium stiff, - with occasional silt seams/layers A-6 (9), Lab No. 2884	2	SS	2.5-4.0	2-3-3 (6)	100		21	30	12		2.75
510.69	7.5			3	SS	5.0-6.5	2-4-5 (9)	100		21			1.88	1.5
508.19	10.0	2.5	Brown and trace gray SILTY LOAM, moist, medium stiff, A-6, Lab No. 2884	4	SS	7.5-9.0	3-4-4 (8)	100		18			1.26	1.5
505.69	12.5	2.5	Brown and gray SILTY LOAM, moist to very moist, soft, - with occasional wet silt and sand seams A-4, Lab No. 2868	5	SS	10.0-11.5	2-2-2 (4)	100		18	19	2		1.0
503.69	14.5	2.0	Gray FRIABLE LIMESTONE AND SILTY LOAM SEAMS/LAYERS, - Limestone, moderately hard and fine-grained - Silty Loam, medium dense and wet	6	SS	12.5-13.6	24-42-50/.1'	91						
502.69	15.5	1.0		1	RC	14.5-15.5		100	0					
497.69	20.5	5.0	Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and moderately to steeply dipping (high angle) joints; occurs well distributed in 1" to 3" pieces	2	RC	15.5-20.5		100	78				742	
492.69	25.5	5.0	Gray LIMESTONE, very hard, calcareous, fine-grained, moderately to widely spaced, rough, and moderately dipping joints; occasional steeply dipping (high angle) joints and fractured rock zones at 19'~20'; occurs distributed in 12"-18" layers	3	RC	20.5-25.5		100	66				776	
			Gray LIMESTONE, very hard, calcareous, crystalline, moderately to widely spaced, rough, and moderately dipping joints; interbedded with occasional siltstone seams/layers											
			BORING COMPLETED @ 25.5'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

Water Level Observations

Immediate	12.5	ft. ▽
At Completion	10.8	ft.
After 24 Hrs.	10.5	ft. ▼
Water used in drilling	14.5	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
790 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT., STE B-8
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-5
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 91+80, 22.5' Rt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.		DESCRIPTION OF MATERIALS color, material description, moisture, stiffness/density/hardness	SAMPLE NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
520.04	0.0													
519.04	1.0	1.0	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)	1	SS	0.0-1.5 1.0-1.5	1-1-1 (2)	100		22				0.5
515.04	5.0	4.0	Brown SILT, moist, very soft, A-4, Lab No. 2852	2	SS	2.5-4.0	1-1-1 (2)	20		20				
507.54	12.5	7.5	Brown SILTY LOAM, moist, soft to stiff, - with occasional silt seams; with occasional limestone fragments/floaters at 10' A-4, Lab No. 2877	3	SS	5.0-6.5	2-2-2 (4)	80		21	27	9		1.0
				4	SS	7.5-9.0	3-2-3 (5)	80		28				0.75
				5	SS	10.0-11.5	32-7-5 (12)	100		25			0.34	0.5
506.04	14.0	1.5	Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints	6	SS	12.5-12.6	50/1	100						
505.04	15.0	1.0	Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and steeply dipping (high angle) joints; occurs distributed in 1" to 3" pieces	1	RC	14.0-19.0		100	56				758	
501.04	19.0	4.0	Gray LIMESTONE, very hard, calcareous, crystalline, fine-grained, moderately close spaced, rough, and moderately dipping joints; occurs well distributed in 6" to 12" layers with occasional fractured, steeply dipping (high angle) joints at 18'-19'											
			BORING COMPLETED @ 19.0'											

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

Water Level Observations

Immediate	DRY ft.
At Completion	10.0 ft. ▽
After 24 Hrs.	8.0 ft. ▼
Water used in drilling	14.0 ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

The H.C. N. ıg Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

ɔernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over Lick Creek
Project:BRO-9959 ( ), DES No. 9982490
Orange County Bridge File No. 34
Orange County, IN
W.O. # 50043.009

TABLE IA: CLASSIFICATION TEST DATA AND ATTERBERG LIMITS

TEST NO.	BORING NO.	SAMPLE NO.	DEPTH		DESCRIPTION	AASHTO	GRL %	CS %	FS %	SILT %	CLAY %	pH +	L.L. %	P.L. %	P.I. %
			Feet	Meters											
STA. 86+00, 10'RT, LINE B															
2848	RB-1	2/SS	2.5 - 4	0.75 - 1.4	Silty Clay	A-7-6(38)	0	0	2	54	44	4.14	56	22	34
STA. 97+00, 10.0'RT, LINE B															
2852	RB-2	2/SS	2.5- - 4	0.75 - 1.4	Silt	A-4(7)	0	0	3	87	10	6.40	28	20	8
STA. 92+25, 23.0'RT, LINE B															
2860	RW-1	3/SS	5 - 6.5	1.5 - 2.0	Loam	A-4(3)	3	0	37	47	13	6.96	25	15	10
STA 92+70, 20.0'LT, LINE B															
2863	RW-2	3/SS	5 - 6.5	1.5 - 2.0	Sandy Loam	A-6(2)	1	3	55	27	14	4.63	29	16	13
STA. 94+70, 25.0'RT, LINE B															
2868	RW-3	5/SS	10 - 11.5	3.0 - 3.5	Silty Loam	A-4(1)	0	0	37	52	11	5.94	21	15	6
STA. 92+70, 20.0'RT, LINE B															
2873	TB-1	5/SS	10 - 11.2	2.5 - 3.4	Sandy Loam	A-4(0)	0	1	58	36	5	7.36	NP	NP	NP
STA. 93+30, 15.0'LT, LINE B															
2877	TB-2	5/SS	10 - 11.2	2.5 - 3.0	Silty Loam	A-4(6)	2	1	18	62	17	7.04	28	18	10
STA. 94+70, 15.0'LT, LINE B															
2884	TB-4	2/SS	2.5 - 4	0.7 - 1.2	Silty Loam	A-6(9)	0	1	16	67	16	4.03	30	18	12

The H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over Lick Creek
Project:BRO-9959 ( ), DES No. 9982490
Orange County Bridge File No. 34
Orange County, IN
W.O. # 50043.009

TABLE IB: ADDITIONAL ATTERBERG LIMITS

TEST NO.	BORING NO.	SAMPLE NO.	DEPTH		DESCRIPTION	AASHTO	GRL %	CS %	FS %	SILT %	CLAY %	pH +	L.L. %	P.L. %	P.I. %
			Feet	Meters											
STA. 86+00, 10'RT, LINE B															
2847	RB-1	1/SS	1 - 1.5	0.3 - 0.4									77	29	48
STA. 97+00, 10.0'RT, LINE B															
2854	RB-2	4/SS	8.5 - 10	2.5 - 3.0									49	23	26
STA 92+70, 20.0'LT, LINE B															
2864	RW-2	4/SS	7.5 - 8.7	2.3 - 2.6									27	15	12
STA. 94+70, 25.0'RT, LINE B															
2869	RW-3	6/SS	12.5 - 13.7	3.8 - 4.2									27	17	10
STA. 92+70, 20.0'RT, LINE B															
2871	TB-1	4/SS	7.5 - 9	2.3 - 2.7									22	18	4
STA. 94+70, 15.0'LT, LINE B															
2887	TB-4	5/SS	10 - 11.5	2.5 - 3.5									19	17	2
STA. 91+80, 22.5'RT, LINE B															
2890	TB-5	3/SS	5 - 6.5	1.5 - 2.0									27	18	9

H. C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over
Lick Creek, Project:BRO-9959 ( ), Des. No. 9982490
Orange Co. Bridge File No. 34, Orange Co., IN
W.O. #50043.009

TABLE II: NATURAL MOISTURE CONTENT DETERMINATION

Test No.	Boring No.	Sample No.	Depth (Feet)	Depth (Meters)	Moisture Content (%)	**Microwave** Moisture Content (%)
STA. 86+00, 10.0'RT, LINE B						
2847	RB-1	1/SS	1-1.5	0.3-0.4	37.4	
2848		2/SS	2.5-4	0.75-1.4	25.8	
2849		3/SS	6-6.5	1.8-2.0	31.4	26.4
STA. 87+00, 10.0'RT, LINE B						
2851	RB-2	1/SS	1-1.5	0.3-0.5	22.4	
2852		2/SS	2.5-4	0.8-1.2	25.6	
2853		3/SS	5-6.5	1.5-2.0	22.2	
2854		4/SS	8.5-10	2.6-3.0	27.9	26.0
STA. 98+50, 10.0'LT, LINE B						
2855	RB-3	1/SS	1-1.5	0.3-0.5	21.8	
2856		2/SS	2.5-4	0.8-1.2	24.3	25.0
2857		3/SS	6-7.5	1.8-2.3	29.8	30.5
STA. 92+25, 23.0'RT, LINE B						
2858	RW-1	1/SS	1-1.5	0.3-0.5	26.2	
2859		2/SS	2.5-4	0.8-1.2	18.3	19.0
2860		3/SS	5-6.5	1.5-2.0	19.9	
STA. 92+70, 20.0'LT, LINE B						
2861	RW-2	1/SS	0-1.5	0-0.5	17.6	
2862		2/SS	2.5-4	0.8-1.2	19.3	18.4
2863		3/SS	5-6.5	1.5-2.0	19.0	
2864		4/SS	7.5-8.7	2.3-2.6	19.3	21.2
STA. 94+70, 25.0'RT, LINE B						
2865	RW-3	2/SS	2.5-4	0.8-1.2	29.3	
2866		3/SS	5-6.5	1.5-2.0	21.6	20.1
2867		4/SS	7.5-9	2.3-2.7	18.5	
2869		6/SS	12.5-13.7	3.8-4.2	31.1	
STA. 92+70, 20.0'RT, LINE B						
2870	TB-1	2/SS	2.5-4	0.8-1.2	19.4	18.7
2871		3/SS	5-6.5	1.5-2.0	23.2	
2872		4/SS	7.5-9	2.3-2.7	24.4	24.0

H. C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over
Lick Creek, Project:BRO-9959 ( ), Des. No. 9982490
Orange Co. Bridge File No. 34, Orange Co., IN
W.O. #50043.009

TABLE II: NATURAL MOISTURE CONTENT DETERMINATION

Test No.	Boring No.	Sample No.	Depth (Feet)	Depth (Meters)	Moisture Content (%)	**Microwave** Moisture Content (%)
STA. 93+25, 30.0'LT, LINE B						
2874	TB-2	2/SS	2.5-4	0.8-1.2	22.0	22.2
2875		3/SS	5-6.5	1.5-2.0	25.6	
2876		4/SS	7.5-9	2.3-2.7	26.7	26.1
2877		5/SS	10-11.2	3.0-3.4	30.9	
STA. 94+10, 20.0'RT, LINE B						
2878	TB-3	1/SS	1-1.5	0.3-0.5	20.6	
2879		2/SS	2.5-4	0.8-1.2	20.7	19.4
2880		3/SS	5-6.5	1.5-2.0	20.7	
2881		4/SS	7.5-9	2.3-2.7	21.1	23.4
2882		5/SS	10-11.5	3-3.4	20.2	
STA. 94+70, 15.0'LT, LINE B						
2883	TB-4	1/SS	1-1.5	0.3-0.5	15.3	
2884		2/SS	2.5-4	0.8-1.2	21.4	
2885		3/SS	5-6.5	1.5-2.0	20.8	22.6
2886		4/SS	7.5-9	2.3-2.7	18.3	18.3
2887		5/SS	10-11.5	3.0-3.4	18.3	
STA. 91+80, 22.5'RT, LINE B						
2888	TB-5	1/SS	1-1.5	0.3-0.5	22.4	
2889		2/SS	2.5-4	0.8-1.2	20.3	18.0
2890		3/SS	5-6.5	1.5-2.0	20.5	20.6
2891		4/SS	7.5-9	2.3-2.7	27.6	
2892		5/SS	10-11.5	3-3.5	24.5	

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carryng CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.O. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 87+00, 10.0'RT, LINE B									
2853	RB-2	3/SS	5-6.5	1.5-2.0	1.78	0	9.0	101.0	22.2
STA. 92+25, 23.0'RT, LINE B									
3158	RW-1	1/RC	13.5	4.1	792	0	2.4	165.9	0.2
3159		2/RC	17.5	5.3	823	0	0.8	165.0	0.2
STA. 92+70, 20.0'LT, LINE B									
3160	RW-2	1/RC	9.5	2.9	837	0	0.6	156.4	0.3
STA. 94+00, 25.0'RT, LINE B									
2867	RW-3	4/SS	7.5-9	2.3-2.7	0.96	0	5.4	111.1	18.5
3161		2/RC	16.0	4.9	817	0	0.8	165.0	0.2
3162		2/RC	20.0	6.1	201.8	0	0.5	160.7	0.4

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carrying CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.0. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 92+70, 20.0'RT, LINE B									
3150	TB-1	1/RC	12.5	3.8	689	0	0.8	161.5	0.6
3151		1RC	16.5	5.0	355	0	0.8	158.4	0.7
3152		2/RC	20.5	6.2	678	0	1.0	150.3	4.1
STA. 93+30, 15.0'LT, LINE B									
3153	TB-2	1/RC	13.5	4.1	921	0	1.0	164.4	0.2
3154		2/RC	16.0	4.9	851	0	1.0	162.6	0.7
STA. 94+10, 20.0'RT, LINE B									
2882	TB-3	5/SS	10-11.5	3-3.4	0.28	0	15.0	110.5	20.2
3147		1/RC	14.0	4.3	784	0	0.8	165.4	0.1
3148		2/RC	18.0	5.5	560	0	1.0	162.3	0.1
3149		4/RC	29.0	8.8	876	0	2.1	154.2	0.9

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carrying CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.O. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 94+70, 15.0'LT, LINE B									
2885	TB-4	3/SS	5-6.5	1.5-2.0	1.88	0	7.1	104.6	20.8
2886		4/SS	7.5-9	2.3-2.7	1.26	0	10.7	114.9	18.3
3155		2/RC	15.5	4.7	742	0	1.3	157.3	0.7
3156		3/RC	22.0	6.7	776	0	1.4	168.4	0.2
STA. 91+70, 22.5'RT, LINE B									
2892	TB-5	5/SS	10-11.5	3-3.5	0.34	0	15.6	99.1	24.5
3157		1/RC	15.5	4.7	758	0	0.7	167.1	0.2

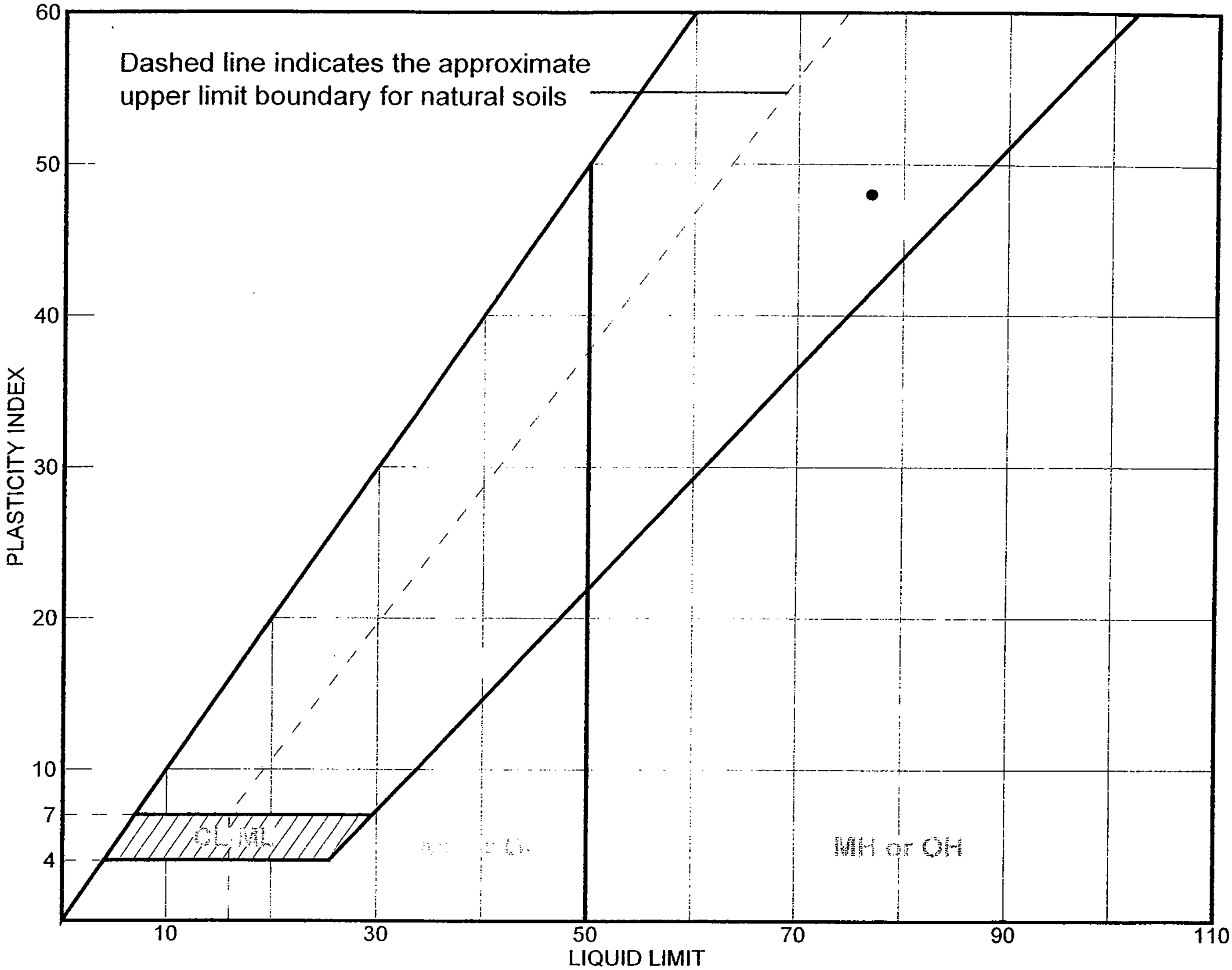
TABLE IV: SUMMARY OF HAND AUGER SOUNDINGS

CLIENT: BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.
PROJECT: ORANGE COUNTY BRIDGE #34 RECONSTRUCTION
LOCATION: ORANGEVILLE TOWNSHIP, ORANGE COUNTY, INDIANA
DATE: March 8, 2004

PROJECT NO.: BRO-9959-( )
DESIGNATION NO.: 9982490
HCN W.O.: 50043.009
BLA P.N.: 199-0047-0BD

SOUNDING NO.	STATION	OFFSET LINE "A"	APPROX. ELEVATION (Ft.)	SOUNDING DEPTH (Ft.)	FIELD OBSERVATION (TOP TO BOTTOM)	NOTE
S-1	91+00	20' Rt.	526.0	3.5	0.5' Topsoil, 2.5' Br. SOFT Silty Clay, 0.5' soft to medium stiff Br. Clay	4' Proposed Fill along existing pavement
S-2	91+64	20' Lt.	516.5	2.0	2.0' Br. SOFT Silty Clay on Cobbles	17' Proposed Fill along existing drainage ditch
S-3	91+65	20' Rt.	514.0	1.5	1.5' Br. wet Silty Sand on Cobbles	
S-4	96+00	10' Lt.	520.5	2.5	1.5' to 2' Br. and Gr. SOFT Silty Clay, 0.5' to 1' medium stiff to stiff Silty Clay	12' Proposed Fill
S-5	98+00	10' Rt.	527.5	3.5	0.5' Topsoil, 2.5' Br. wet Silty Sand, 0.5' soft to medium stiff Br. Clay	5' Proposed Fill

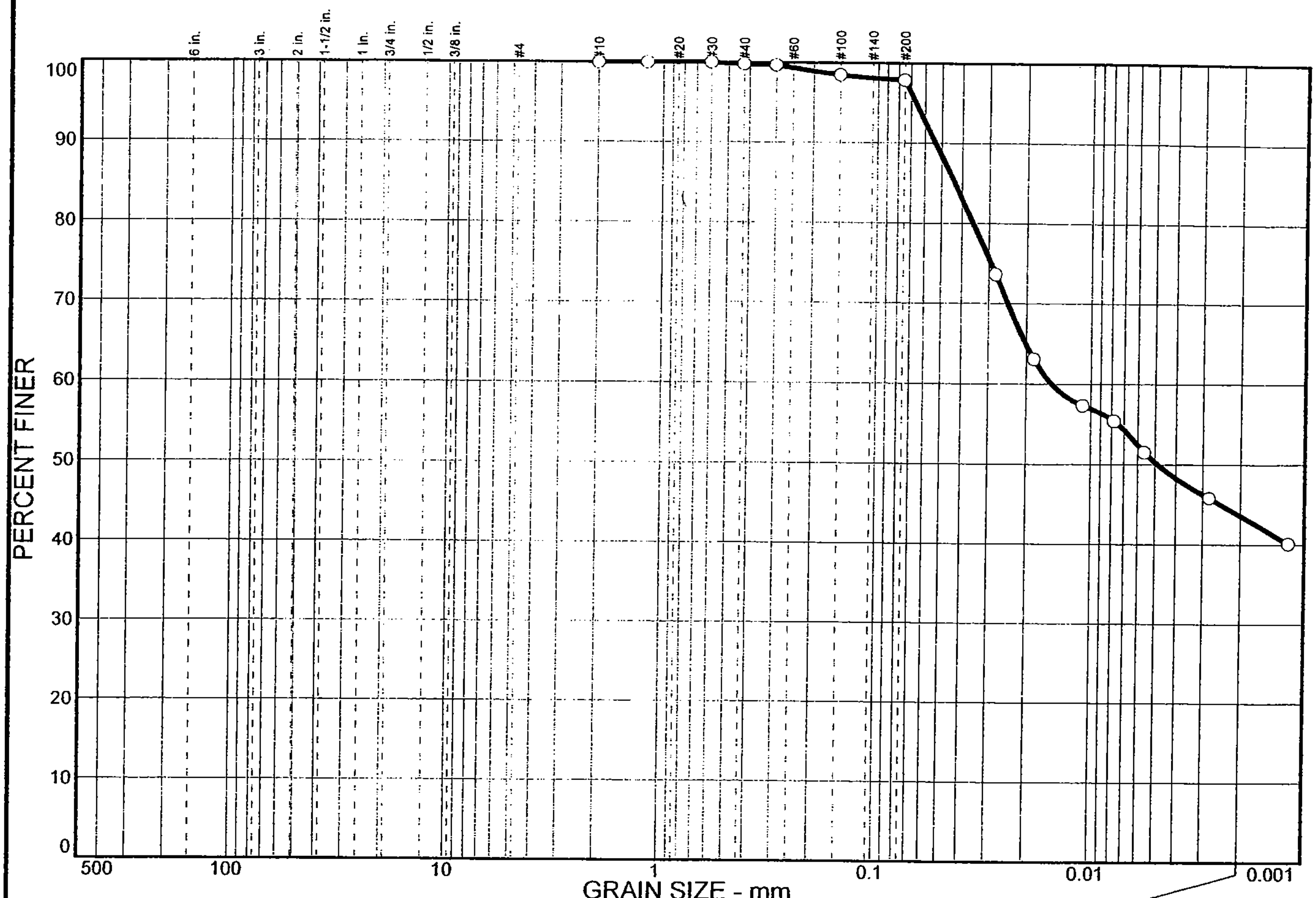
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RB-1	1/SS	1-1.5'	37.4	29	77	48	

LIQUID AND PLASTIC LIMITS TEST REPORT	Client: BERNARDIN LOCHMUELLER & ASSOC.
H. C. NUTTING COMPANY	Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
	Project No.: 50043.009 Plate 2847

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	2.1	54.3	43.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#40	99.8		
#50	99.7		
#100	98.5		
#200	97.9		

* (no specification provided)

Soil Description

SILTY CLAY

Atterberg Limits

PL= 22 LL= 56 PI= 34

Coefficients

D_{85}= 0.0433 D_{60}= 0.0152 D_{50}= 0.0049
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-7-6(38)

Remarks

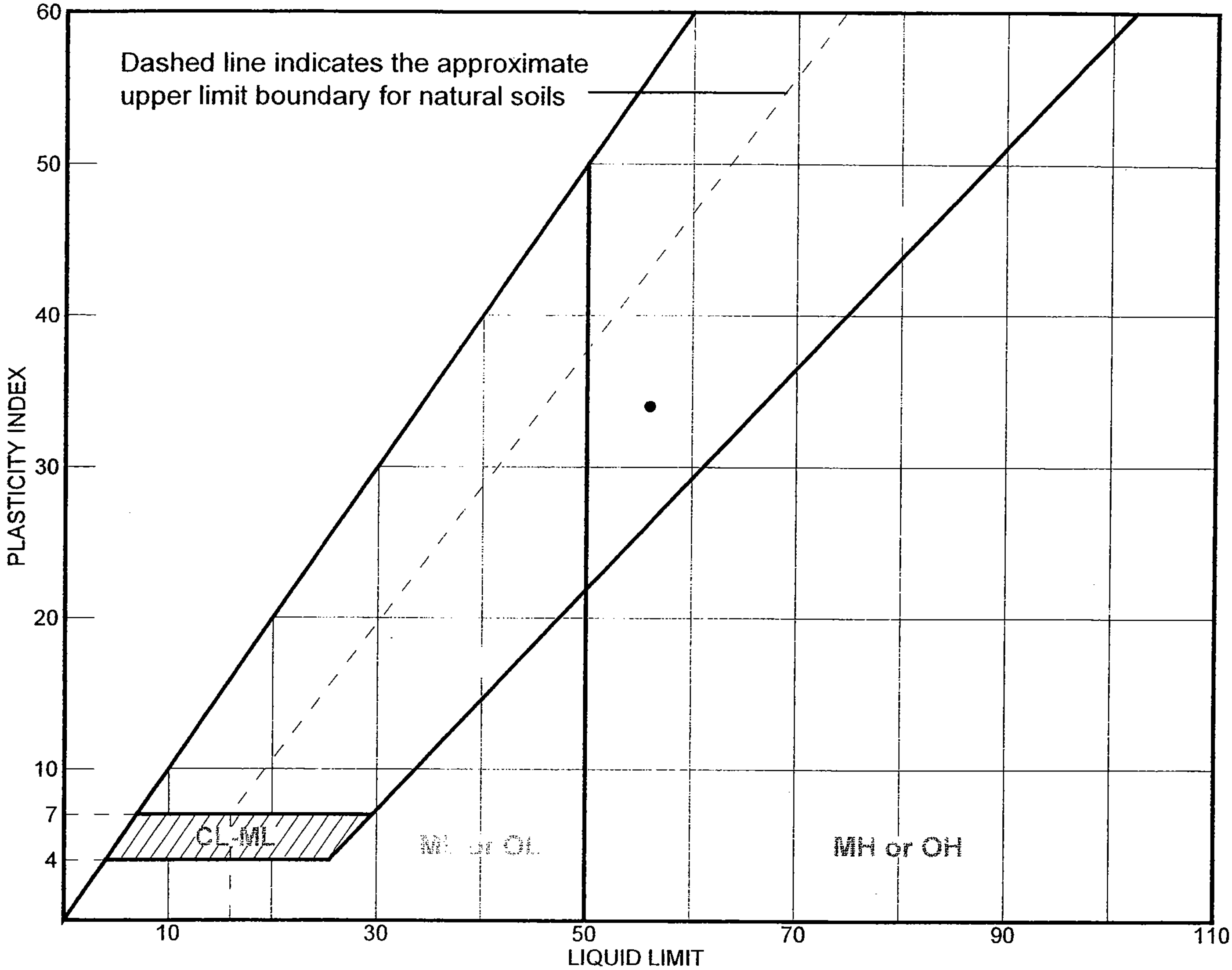
+pH=4.14 WC=25.8%

Sample No.: 2/SS **Source of Sample:** RB-1 **Date:** 5/18/04
Location: STA.86+00,10'RT,LINE B **Elev./Depth:** 2.5-4

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2848

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RB-1	2/SS	2.5-4	25.8	22	56	34	

LIQUID AND PLASTIC LIMITS TEST REPORT

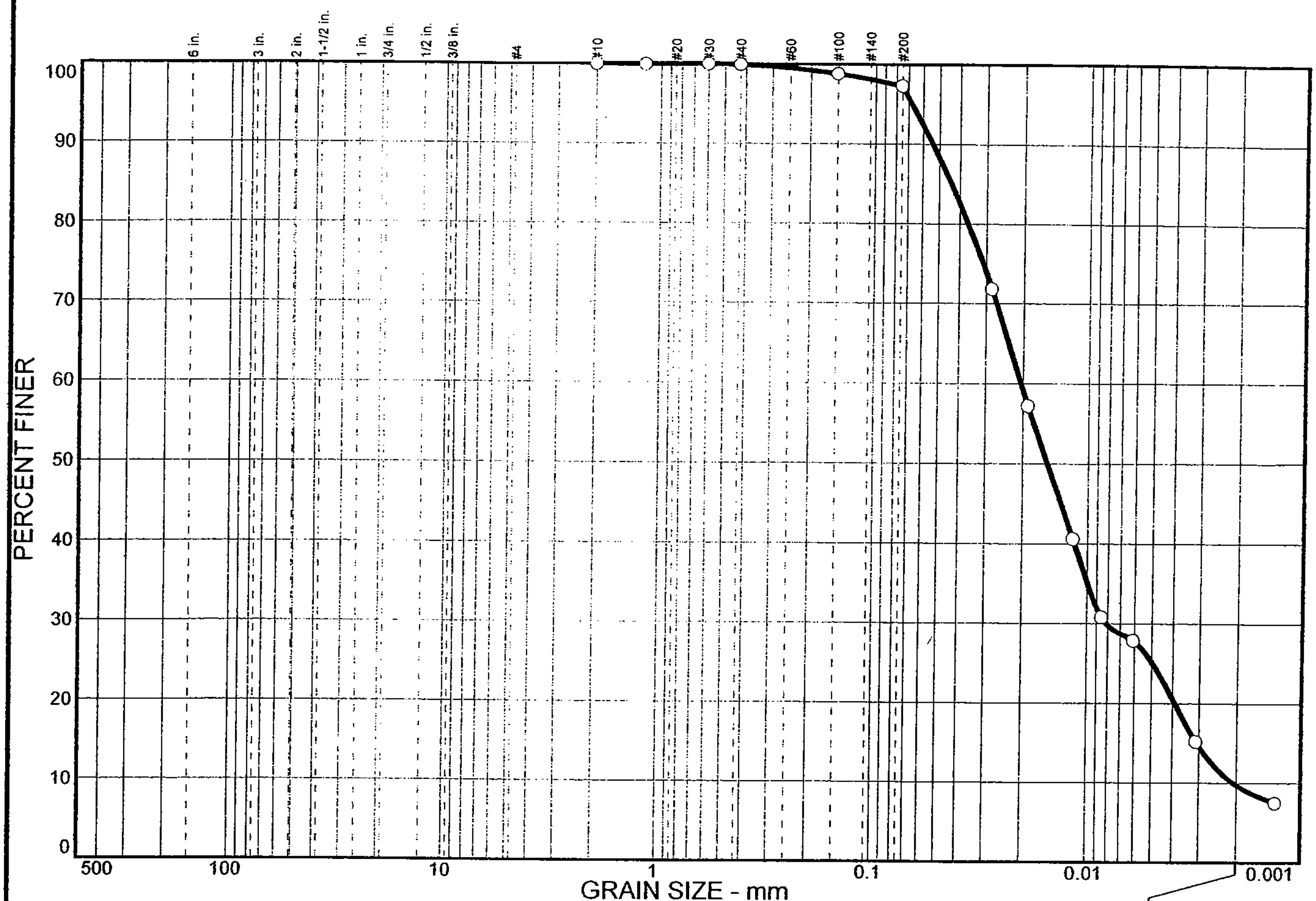
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2848

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	2.7	87.5	9.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#40	99.9		
#100	98.8		
#200	97.3		

* (no specification provided)

Soil Description

SILT

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 0.0438 D_{60}= 0.0205 D_{50}= 0.0154
D_{30}= 0.0081 D_{15}= 0.0031 D_{10}= 0.0021
C_u= 9.92 C_c= 1.56

Classification

USCS= AASHTO= A-4(7)

Remarks

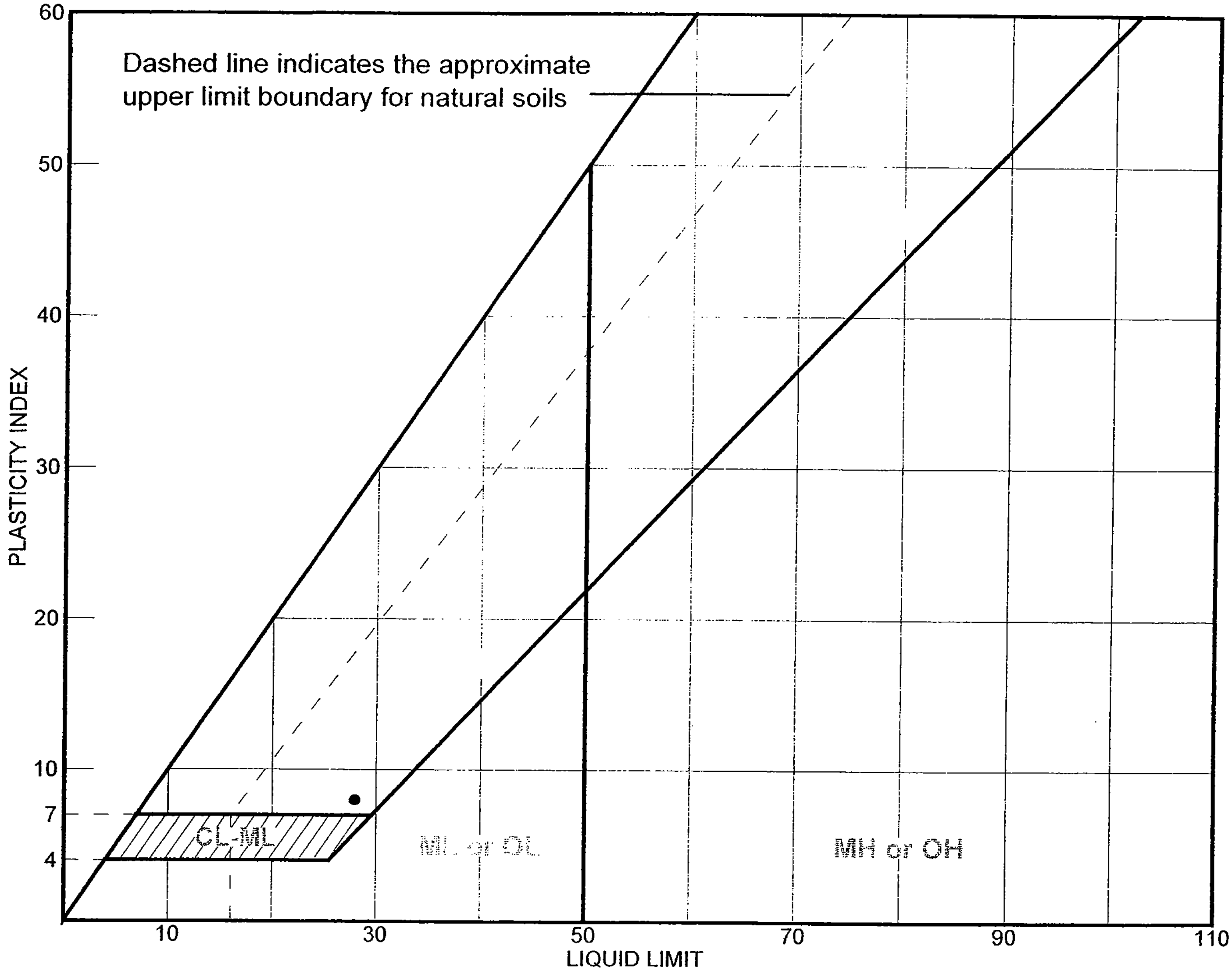
+pH=6.40 WC=25.6%

Sample No.: 2/SS **Source of Sample:** RB-2 **Date:** 5/18/04
Location: STA.87+00,10.0'RT,LINE B **Elev./Depth:** 2.5-4'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2852

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RB-2	2/SS	2.5-4'	25.6	20	28	8	

LIQUID AND PLASTIC LIMITS TEST REPORT

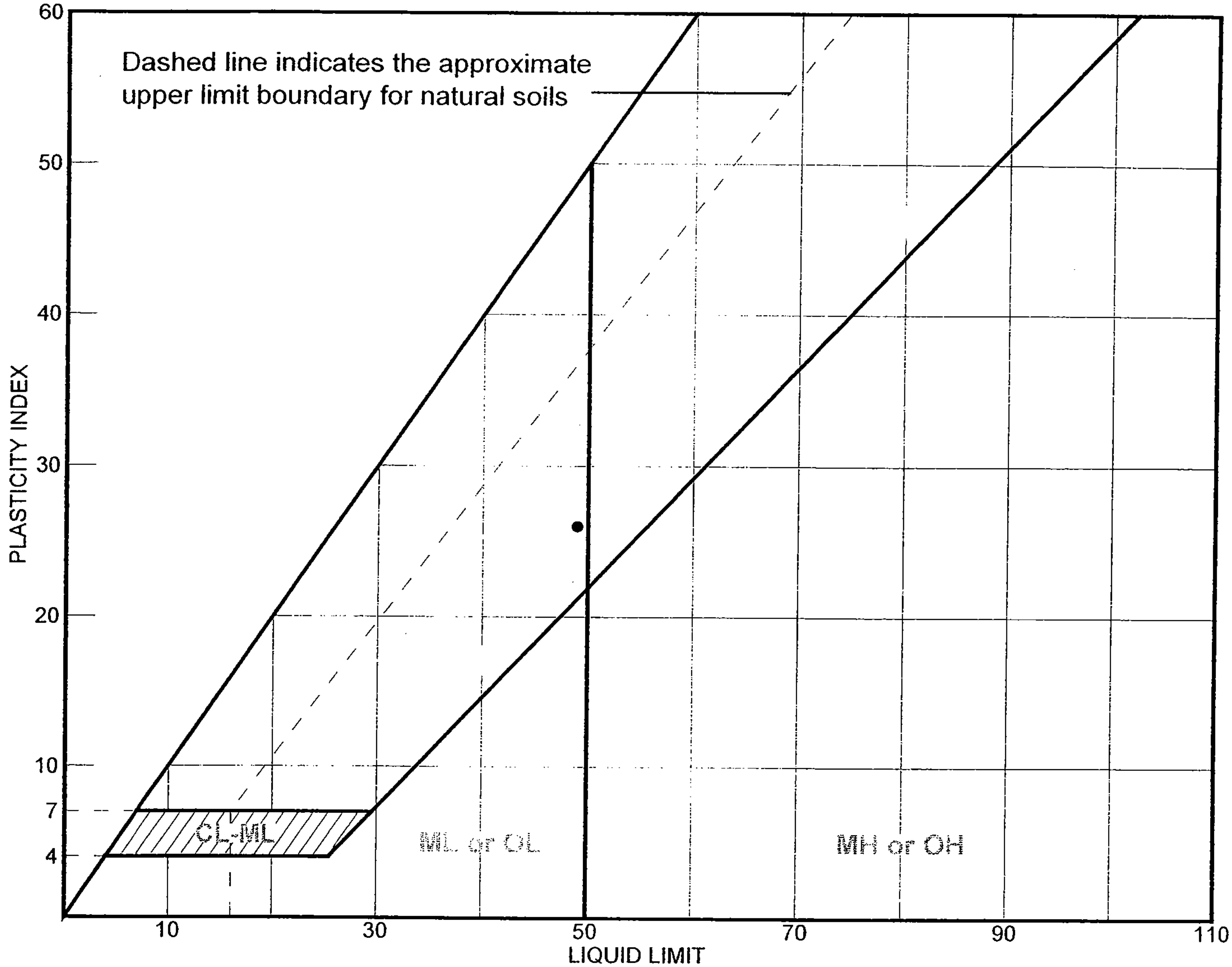
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2852

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RB-2	4/SS	8.5-10'	27.9	23	49	26	

LIQUID AND PLASTIC LIMITS TEST REPORT

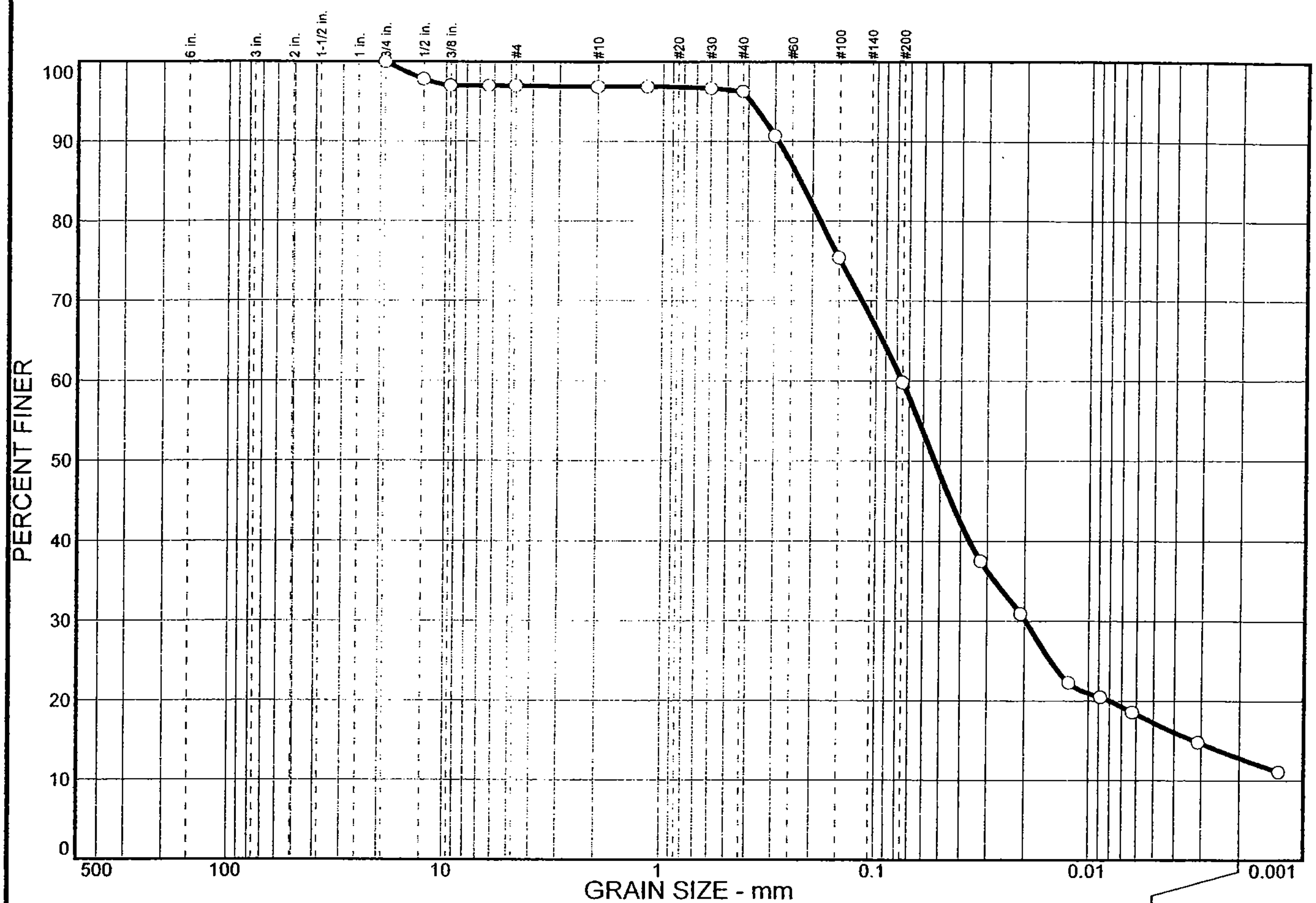
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2854

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.1	37.1	47.0	12.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75 in.	100.0		
.5 in.	97.8		
.375 in.	97.0		
.25 in.	97.0		
#4	97.0		
#10	96.9		
#16	96.9		
#30	96.7		
#40	96.3		
#50	90.7		
#100	75.4		
#200	59.8		

* (no specification provided)

Soil Description

LOAM

Atterberg Limits

PL= 15 LL= 25 PI= 10

Coefficients

D_{85}= 0.228 D_{60}= 0.0756 D_{50}= 0.0529
D_{30}= 0.0197 D_{15}= 0.0032 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(3)

Remarks

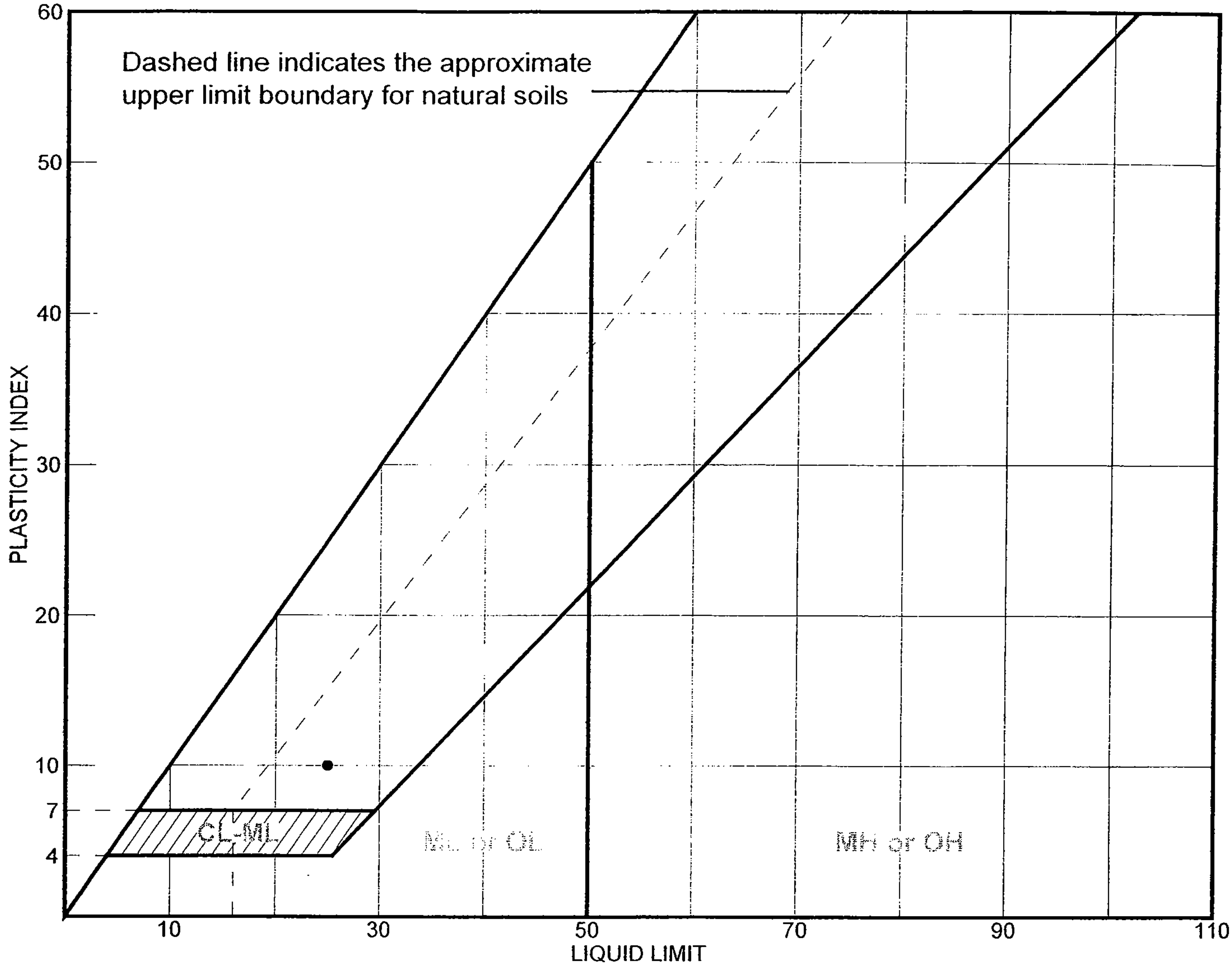
+pH=6.96

Sample No.: 3/SS **Source of Sample:** RW-1 **Date:** 5/18/04
Location: STA.82+25,23.0'RT,LINE B **Elev./Depth:** 5-6.5'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2860

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RW-1	3/SS	5-6.5'	19.9	15	25	10	

LIQUID AND PLASTIC LIMITS TEST REPORT

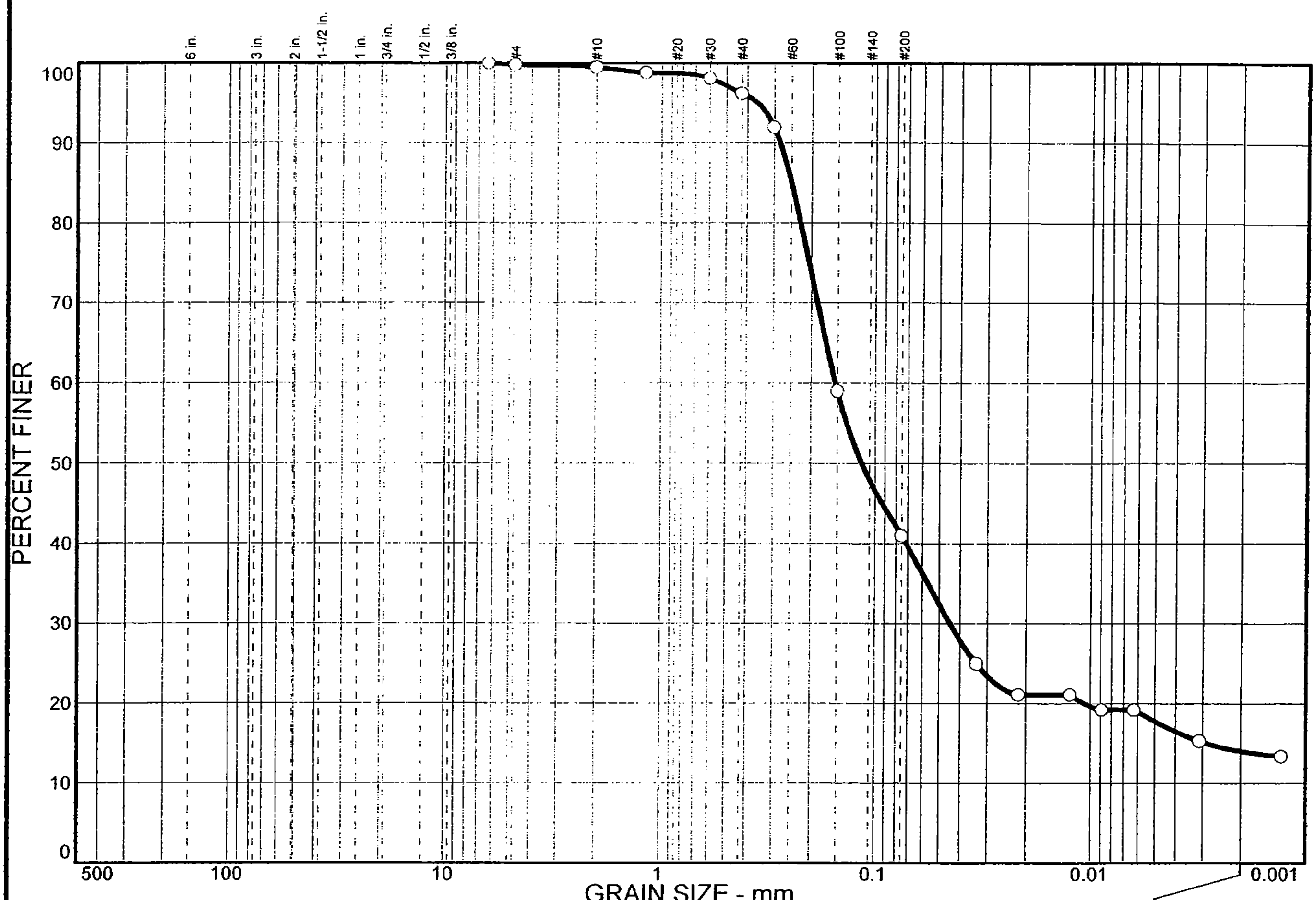
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2860

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.5	58.5	27.0	14.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.25 in.	100.0		
#4	99.8		
#10	99.5		
#16	98.8		
#30	98.1		
#40	96.2		
#50	91.9		
#100	59.0		
#200	41.0		

* (no specification provided)

Soil Description

SANDY LOAM

Atterberg Limits

PL= 16 LL= 29 PI= 13

Coefficients

D_{85}= 0.249 D_{60}= 0.153 D_{50}= 0.115
D_{30}= 0.0445 D_{15}= 0.0029 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-6(2)

Remarks

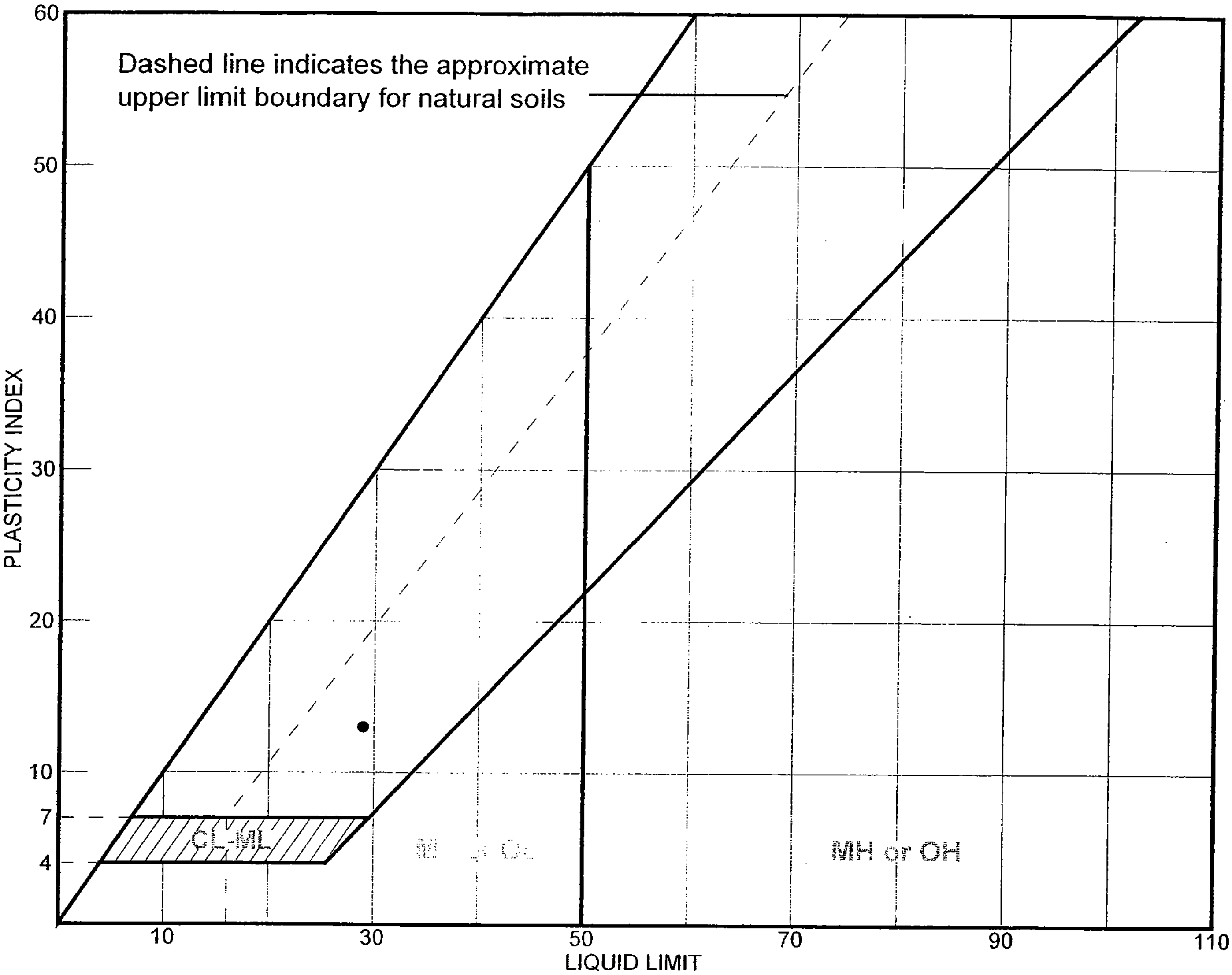
+pH=4.63 WC=19.0%

Sample No.: 3/SS **Source of Sample:** RW-2 **Date:** 5/18/04
Location: STA.92+70,20.0'LT,LINE B **Elev./Depth:** 5-6.5'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2863

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RW-2	3/SS	5-6.5'	19.0	16	29	13	

LIQUID AND PLASTIC LIMITS TEST REPORT

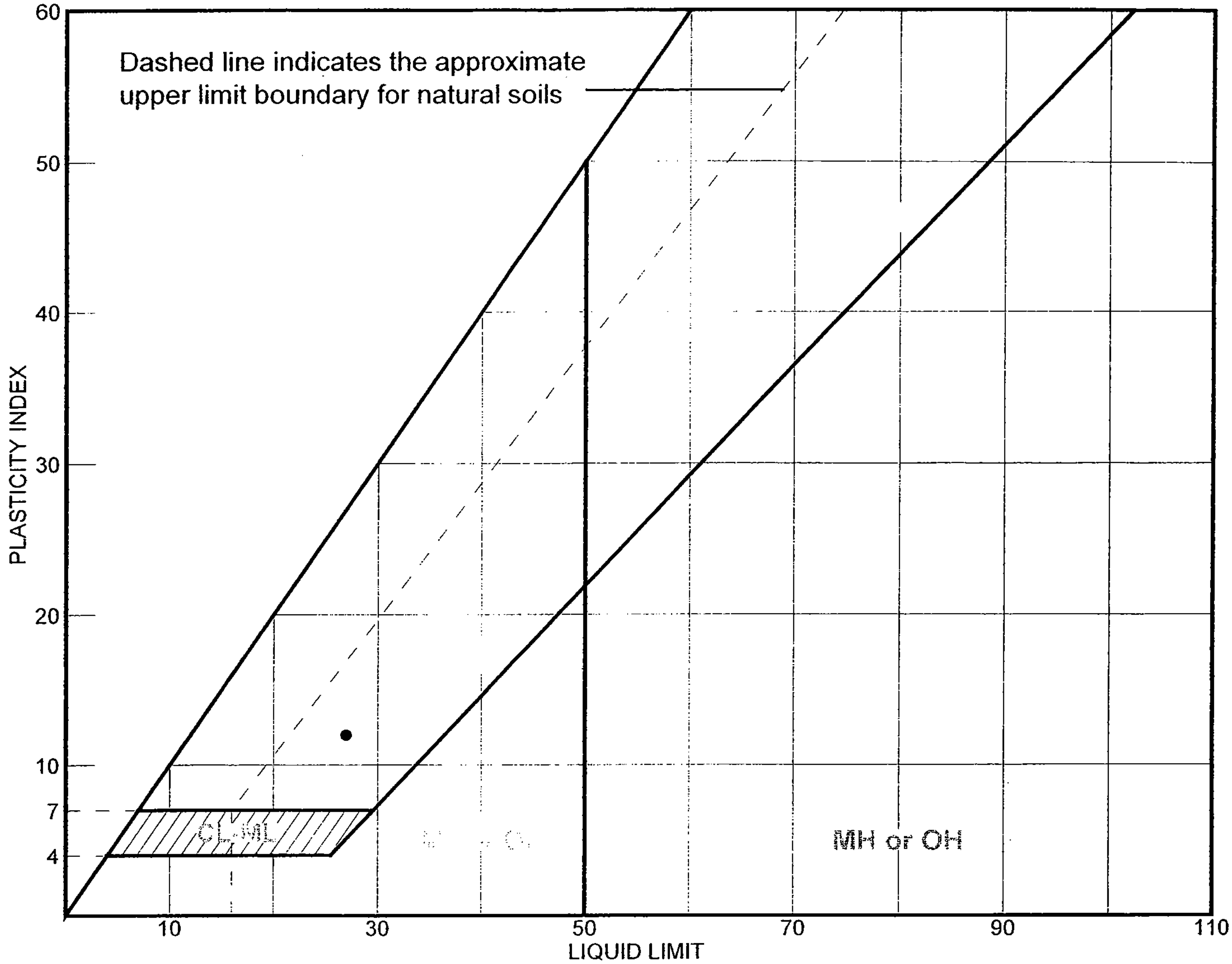
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 28.63

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RW-2	4/SS	7.5-8.7'	19.3	15	27	12	

LIQUID AND PLASTIC LIMITS TEST REPORT

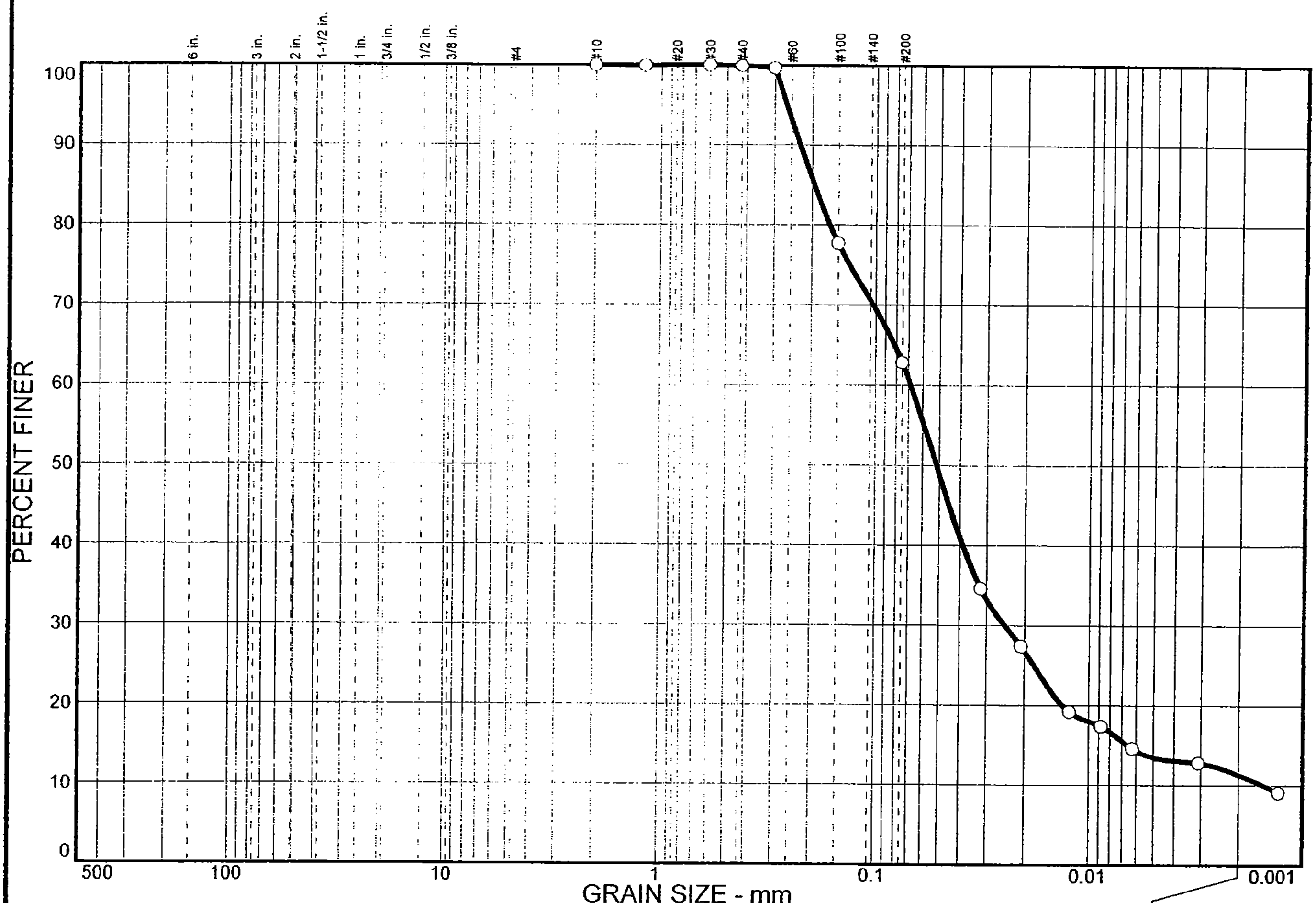
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2864

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	37.2	51.4	11.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#40	99.9		
#50	99.7		
#100	77.7		
#200	62.8		

* (no specification provided)

Soil Description

SILTY LOAM

Atterberg Limits

PL= 15 LL= 21 PI= 6

Coefficients

D_{85}= 0.195 D_{60}= 0.0683 D_{50}= 0.0515
D_{30}= 0.0247 D_{15}= 0.0066 D_{10}= 0.0015
C_u= 44.62 C_c= 5.84

Classification

USCS= AASHTO= A-4(1)

Remarks

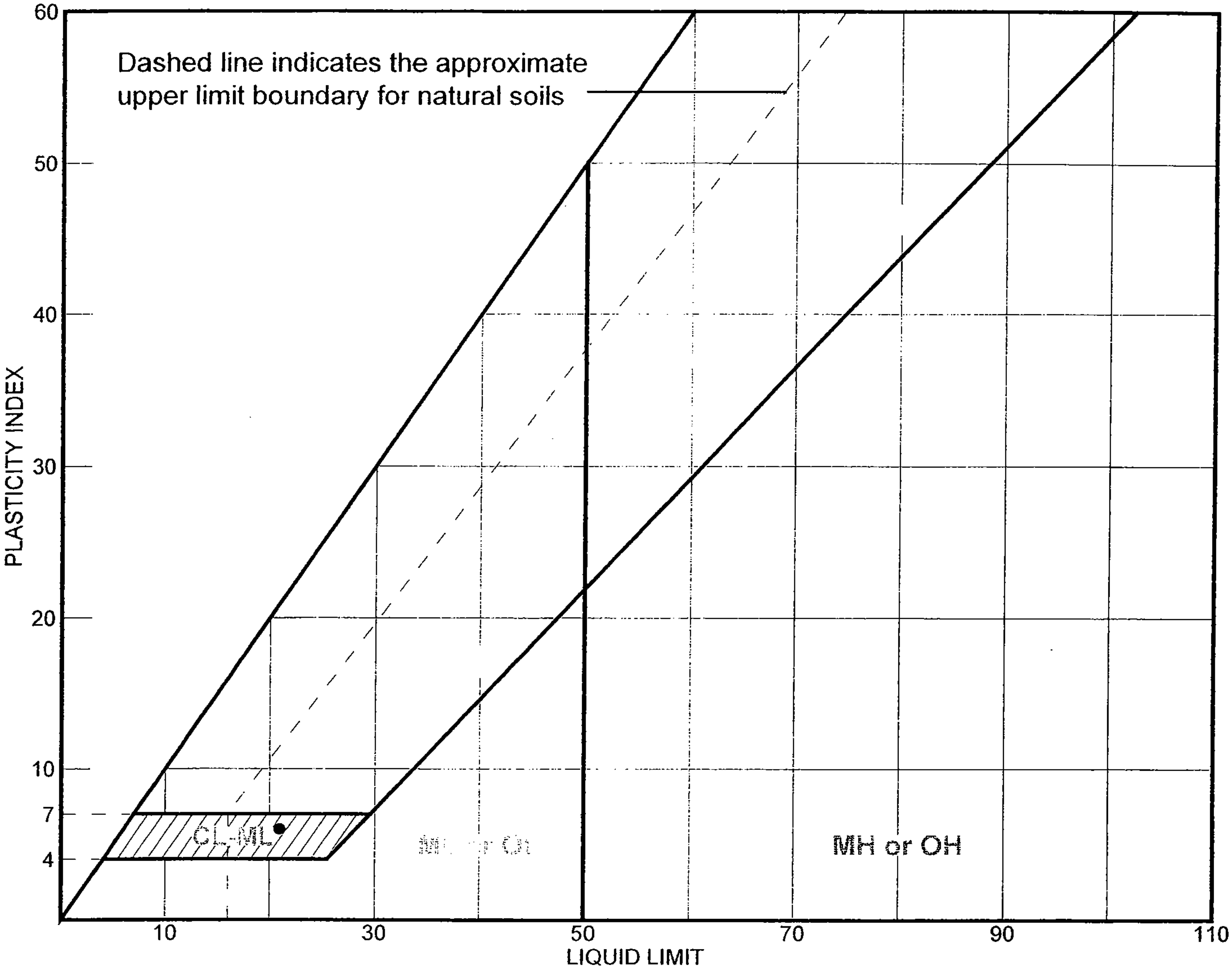
+pH=5.94

Sample No.: 5/SS **Source of Sample:** RW-3 **Date:** 5/18/04
Location: STA.94+70,25.0'RT,LINE B **Elev./Depth:** 10-11.5'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2868

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RW-3	5/SS	10-11.5'		15	21	6	

LIQUID AND PLASTIC LIMITS TEST REPORT

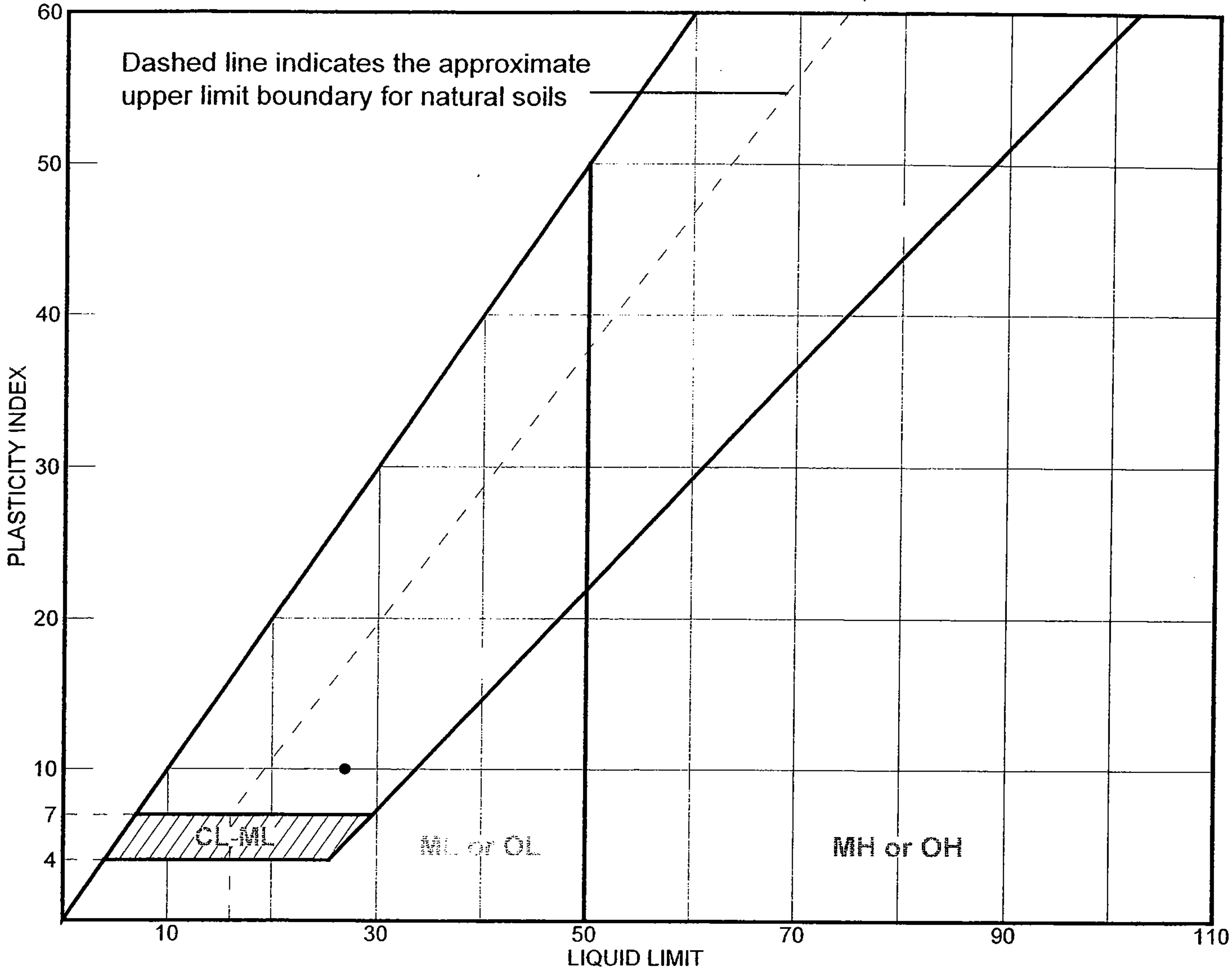
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2868

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	RW-3	6 /SS	12.5-13.7'	31.1	17	27	10	

LIQUID AND PLASTIC LIMITS TEST REPORT

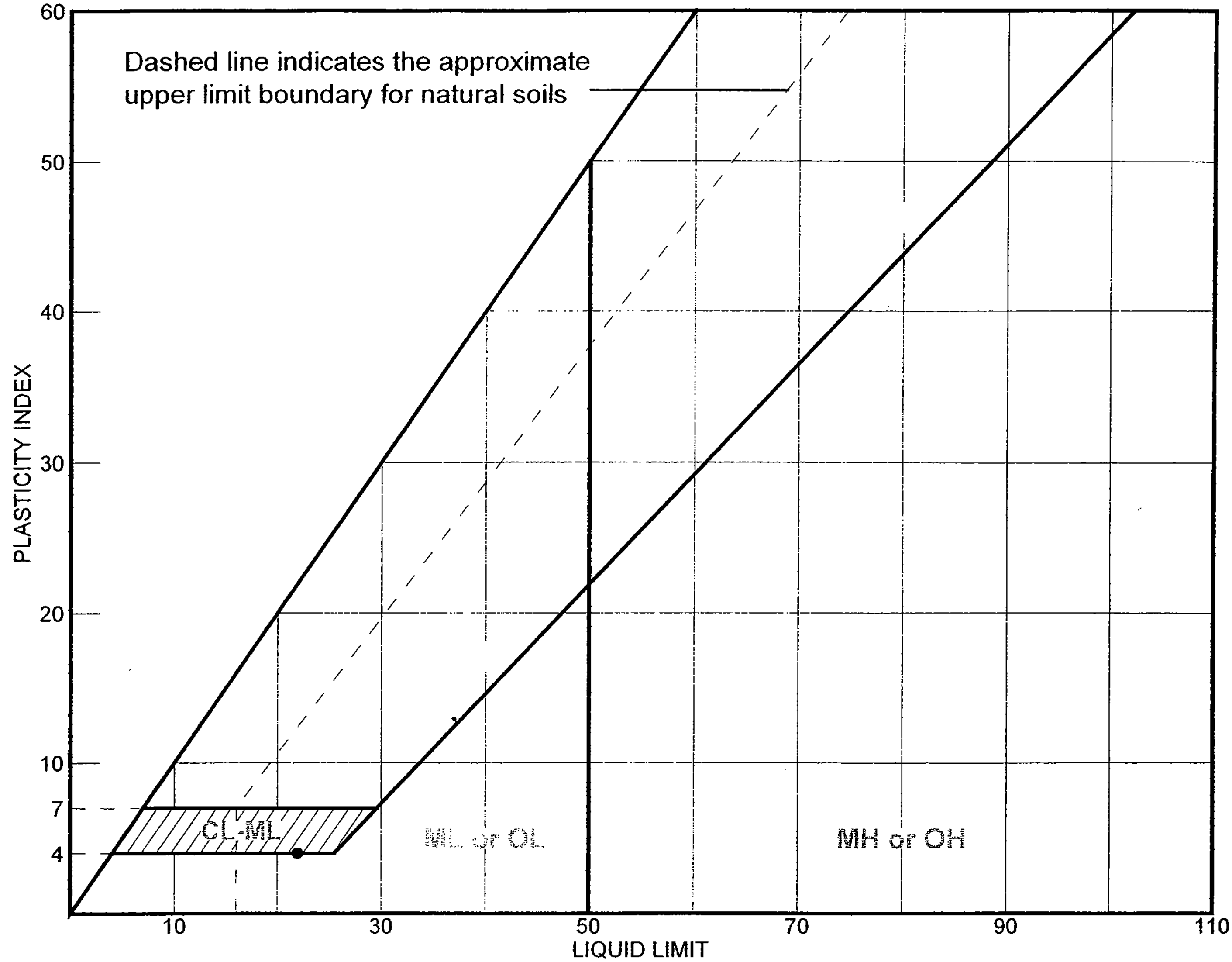
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2869

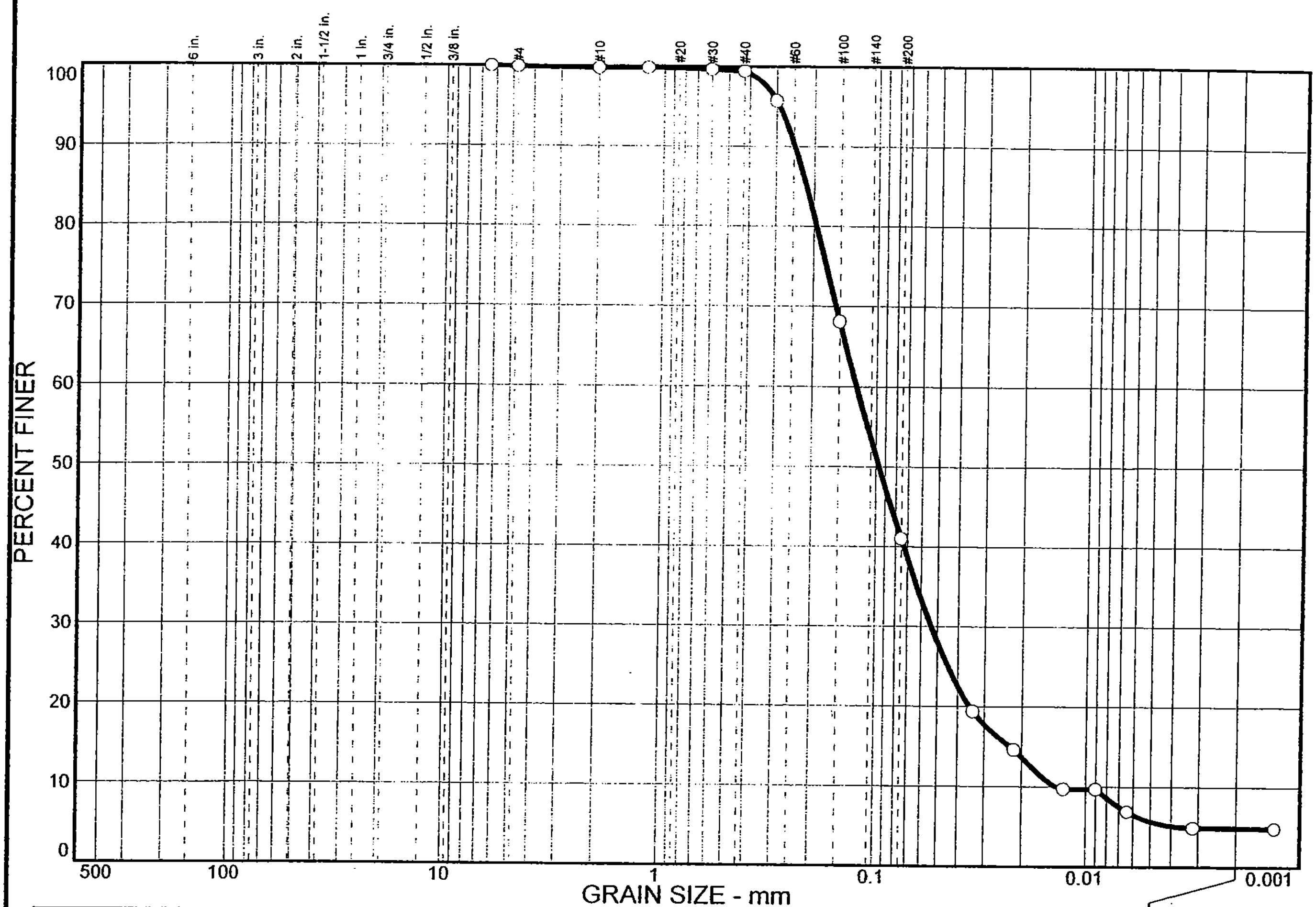
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TB-1	4/SS	7.5'-9'	23.2	18	22	4	

LIQUID AND PLASTIC LIMITS TEST REPORT **H. C. NUTTING COMPANY**	**Client:** BERNARDIN LOCHMUELLER & ASSOC. **Project:** BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK **Project No.:** 50043.009 **Plate** 28.71

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	58.8	36.2	4.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.25 in.	100.0		
#4	99.9		
#10	99.8		
#16	99.8		
#30	99.6		
#40	99.4		
#50	95.7		
#100	68.1		
#200	41.0		

* (no specification provided)

Soil Description

SANDY LOAM

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 0.217 D_{60}= 0.124 D_{50}= 0.0964
D_{30}= 0.0534 D_{15}= 0.0226 D_{10}= 0.0137
C_u= 9.06 C_c= 1.67

Classification

USCS=
AASHTO= A-4(0)

Remarks

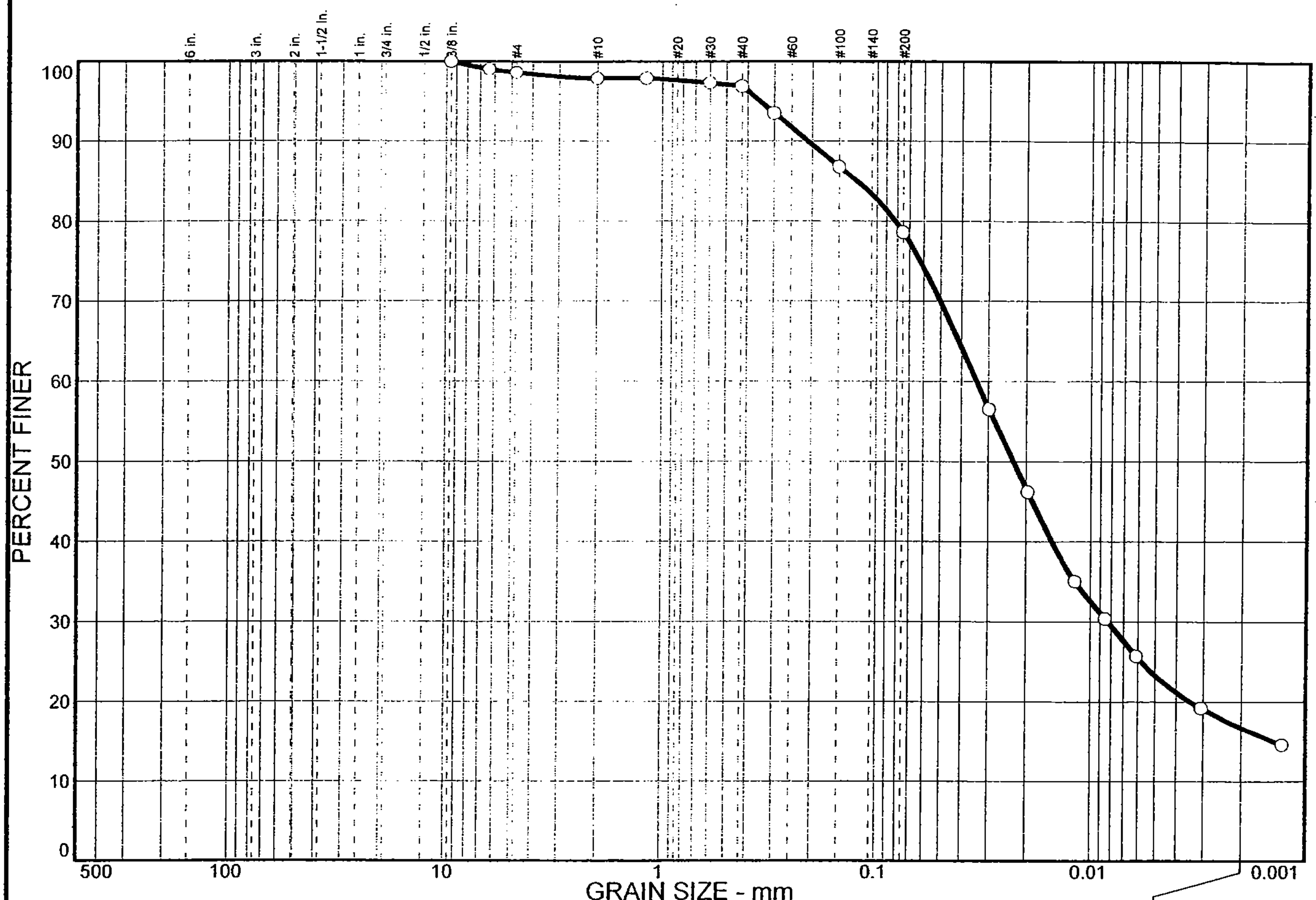
+pH=7.36

Sample No.: 5/SS **Source of Sample:** TB-1 **Date:** 5/18/04
Location: STA.92+70,20.0'RT,LINE B **Elev./Depth:** 10-11.2'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2873

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.1	19.3	61.9	16.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
.25 in.	99.0		
#4	98.6		
#10	97.9		
#16	97.9		
#30	97.3		
#40	96.9		
#50	93.5		
#100	86.8		
#200	78.6		

* (no specification provided)

Soil Description

SILTY LOAM

Atterberg Limits

PL= 18 LL= 28 PI= 10

Coefficients

D_{85}= 0.124 D_{60}= 0.0340 D_{50}= 0.0229
D_{30}= 0.0083 D_{15}= 0.0014 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(6)

Remarks

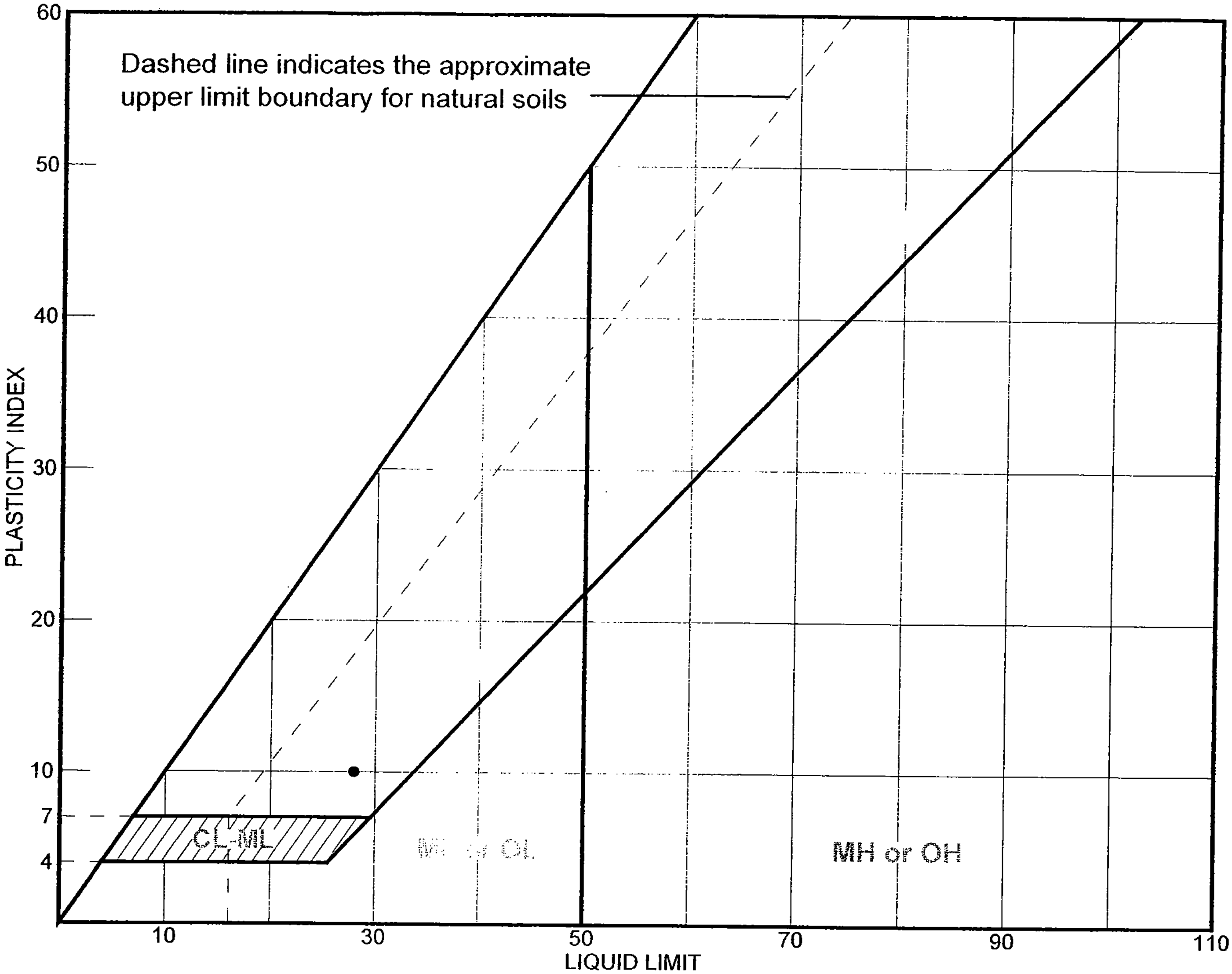
+pH=7.04 WC=30.9%

Sample No.: 5/SS **Source of Sample:** TB-2 **Date:** 5/18/04
Location: STA.93+25,30.0'LT,LINE B **Elev./Depth:** 10-11.2'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2877

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TB-2	5/SS	10-11.2'	30.9	18	28	10	

LIQUID AND PLASTIC LIMITS TEST REPORT

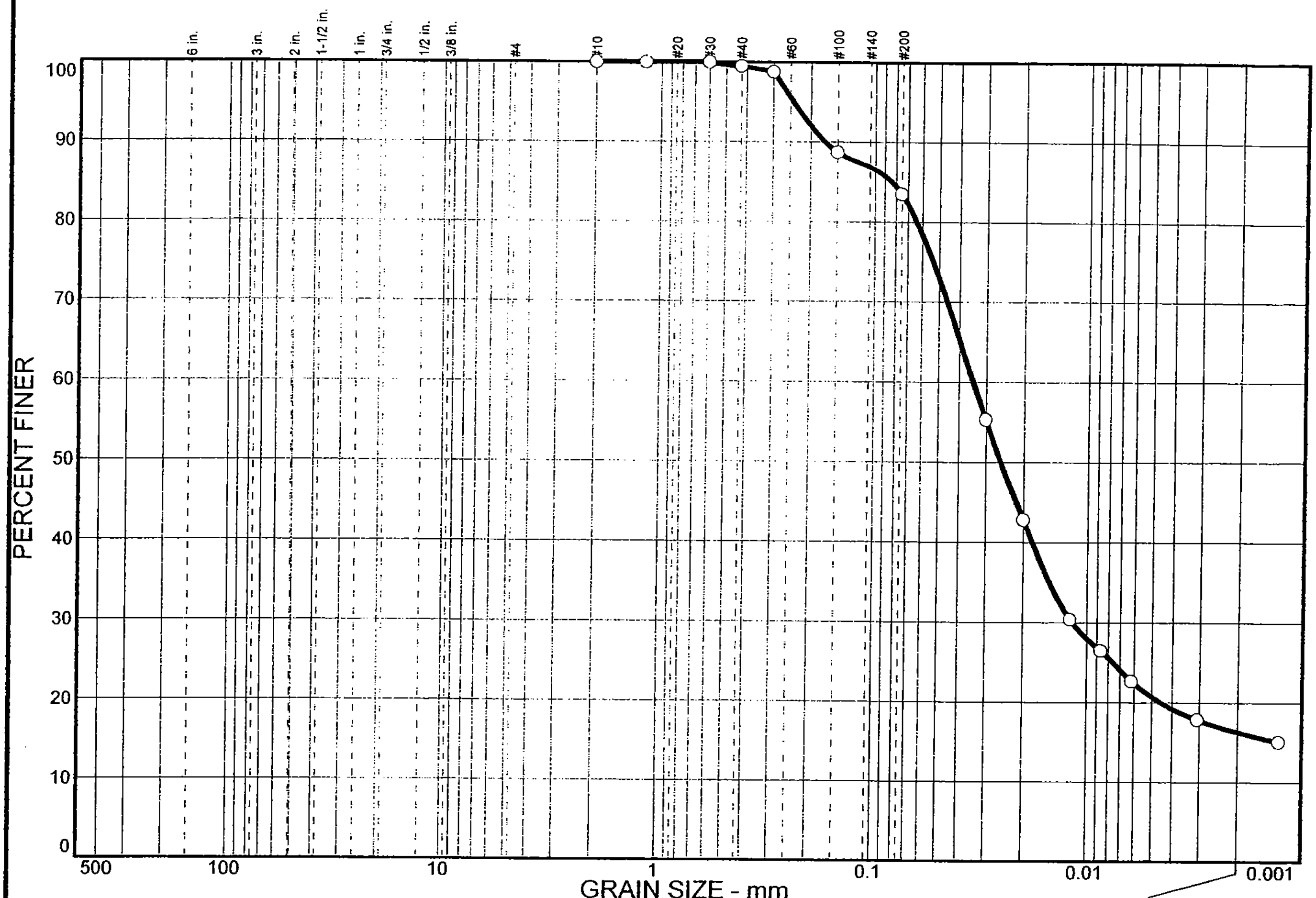
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2877

Grain Size Distribution Test Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	16.5	67.2	16.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#40	99.5		
#50	98.8		
#100	88.7		
#200	83.5		

* (no specification provided)

Soil Description

SILTY LOAM

Atterberg Limits

PL= 18 LL= 30 PI= 12

Coefficients

D_{85}= 0.0833 D_{60}= 0.0345 D_{50}= 0.0255
D_{30}= 0.0117 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-6(9)

Remarks

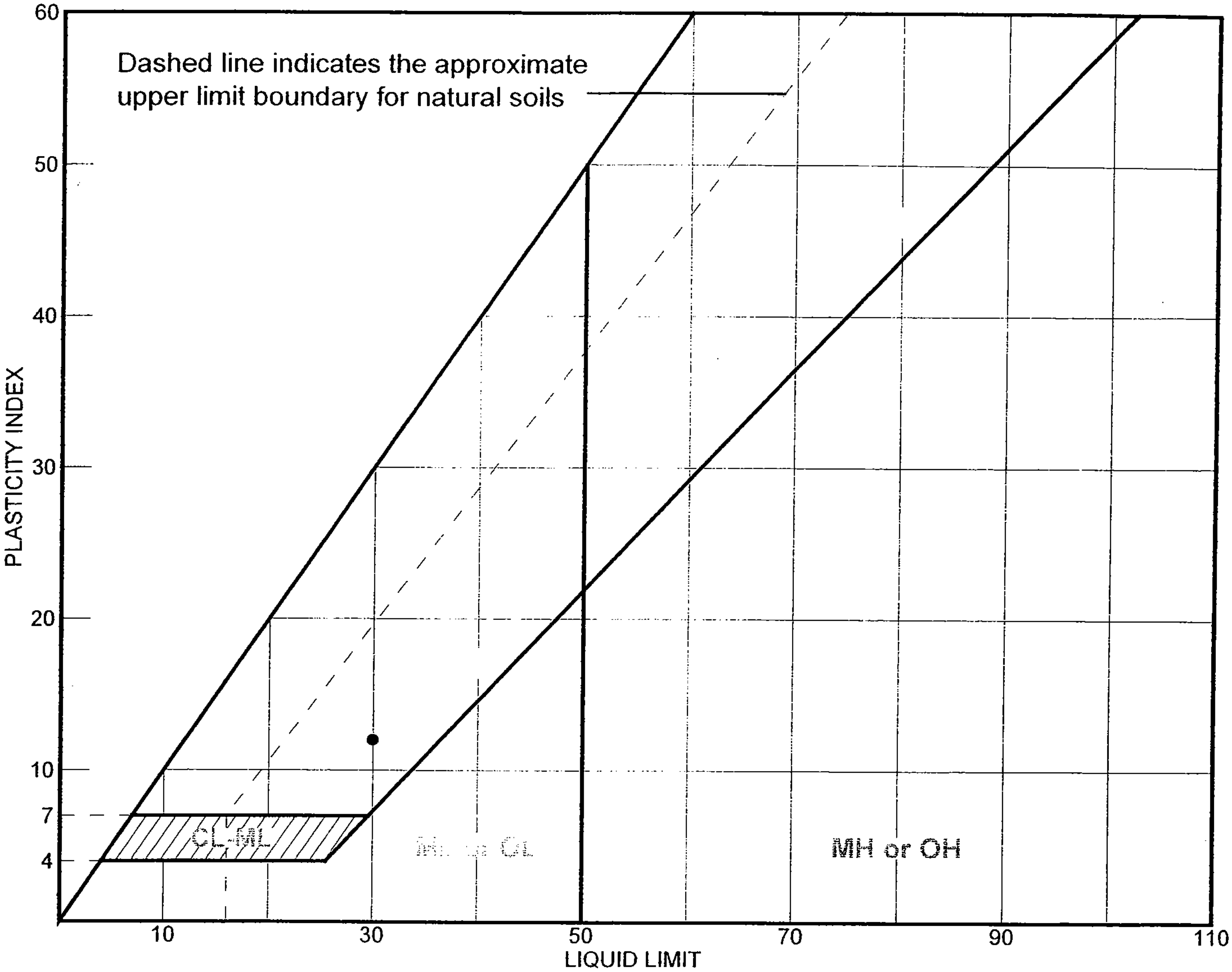
+pH=4.03 WC=21.4%

Sample No.: 2/SS **Source of Sample:** TB-4 **Date:** 5/18/04
Location: STA.94+70,15.0'LT, LINE B **Elev./Depth:** 2.5-4'

H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.
Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK
Project No: 50043.009 **Plate** 2884

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TB-4	2/SS	2.5-4'	21.4	18	30	12	

LIQUID AND PLASTIC LIMITS TEST REPORT

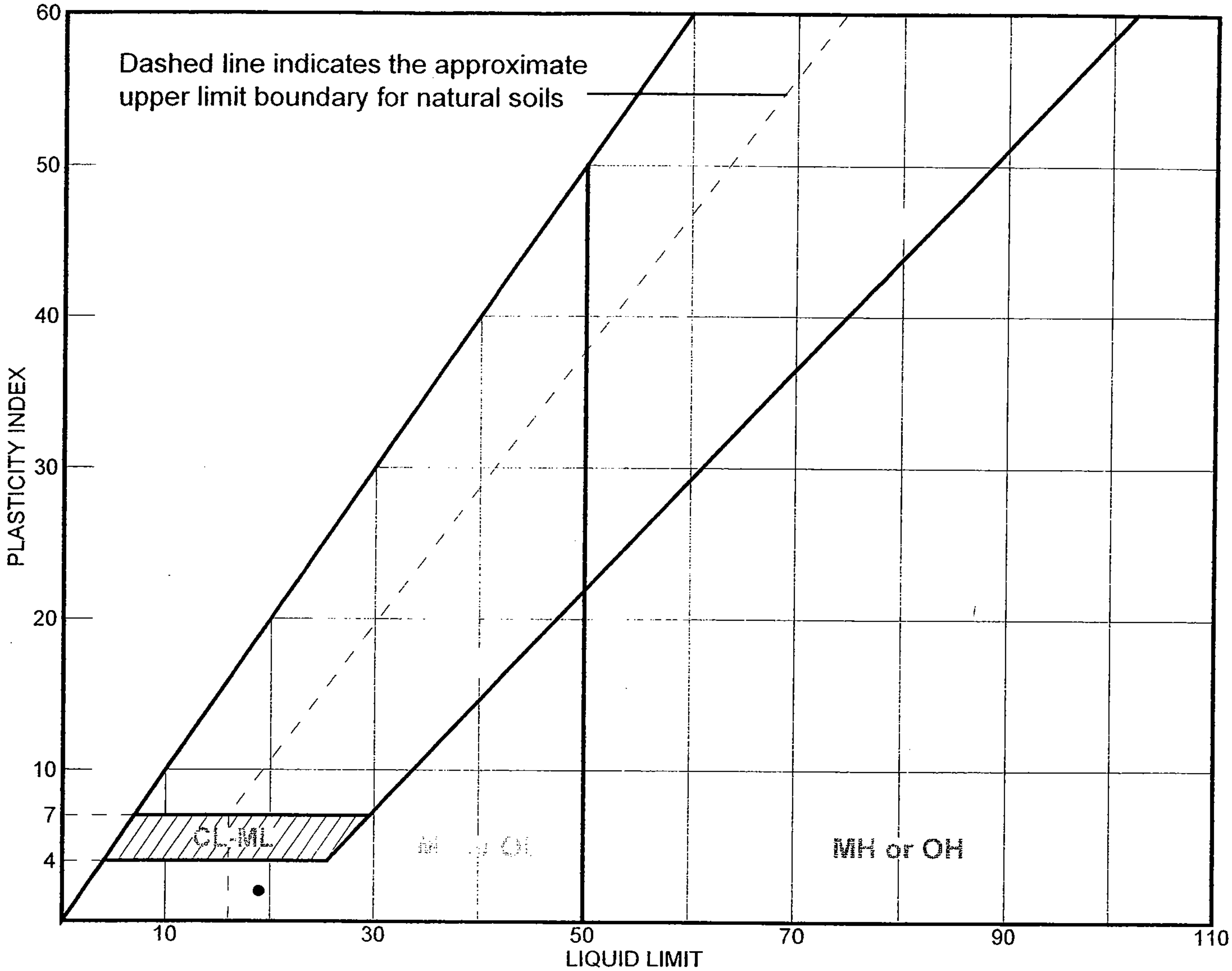
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2884

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TB-4	5/SS	10-11.5'	18.3	17	19	2	

LIQUID AND PLASTIC LIMITS TEST REPORT

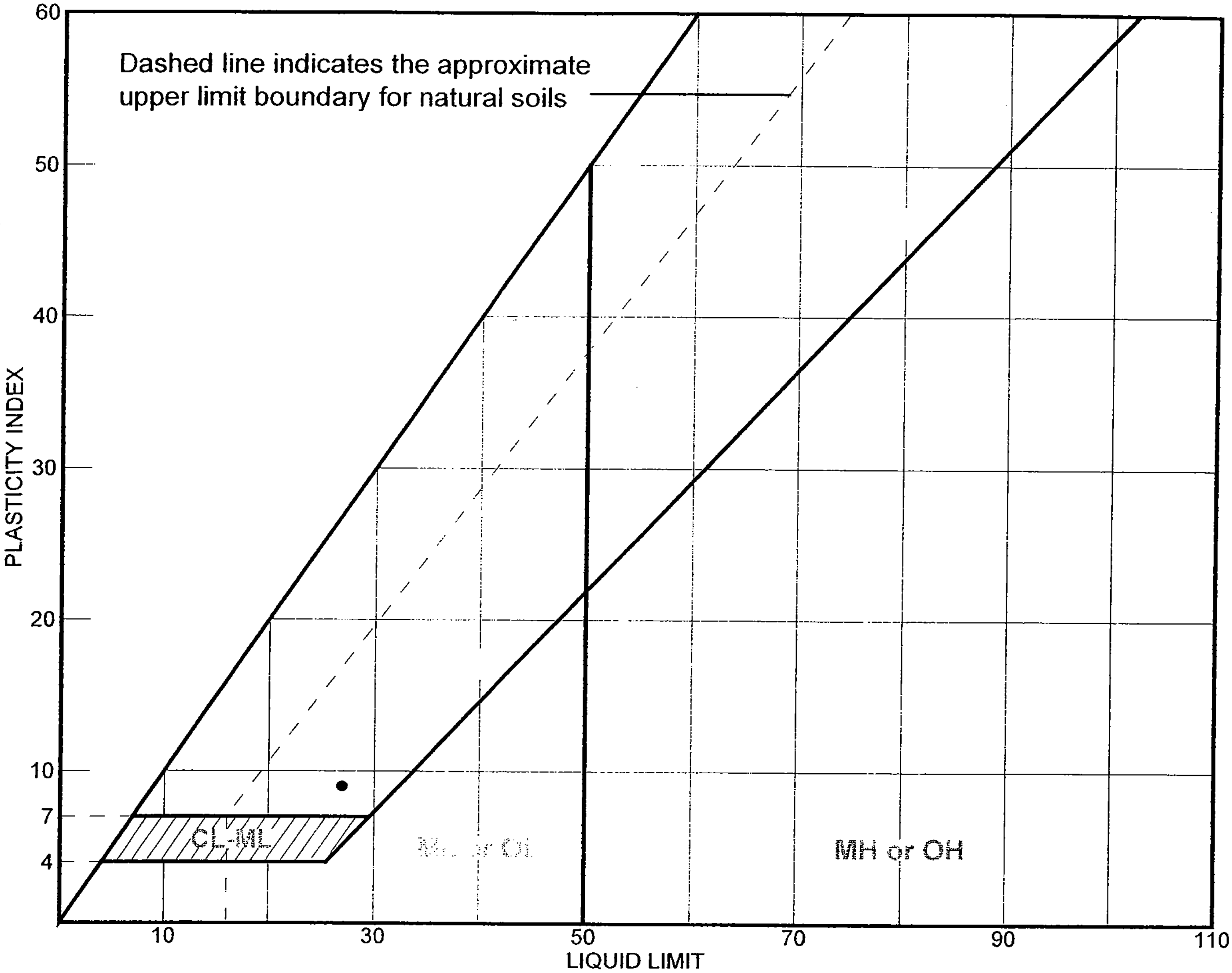
H. C. NUTTING COMPANY

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK

Project No.: 50043.009 **Plate** 2887

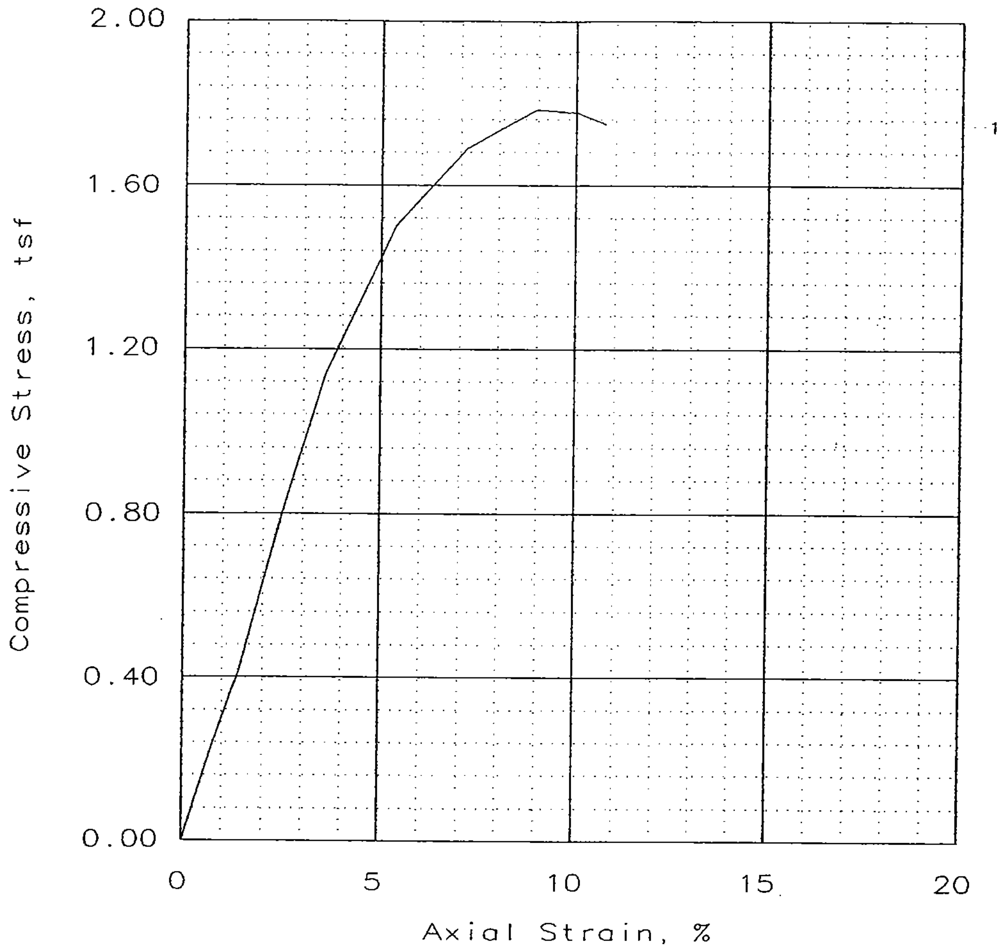
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TB-5	3/SS	5-6.5'	20.5	18	27	9	

LIQUID AND PLASTIC LIMITS TEST REPORT **H. C. NUTTING COMPANY**	**Client:** BERNARDIN LOCHMUELLER & ASSOC. **Project:** BRIDGE REPLACEMENT CARRYING CR 375W OVER LICK CREEK **Project No.:** 50043.009 **Plate** 2890

UNCONFINED COMPRESSION TEST



Sample number:	1			
Unconfined strength, tsf	1.78			
Undrained shear strength, tsf	0.89			
Strain rate, %/min	1.000			
Water content, %	22.2			
Wet density, pcf	123.3			
Dry density, pcf	101.0			
Saturation, %	0.6821			
Void ratio	88.4			
Specimen diameter, in	1.50			
Specimen height, in	2.79			

Description: BR LEAN CLAY, MOIST-STIFF

LL =	PL =	PI =	GS = 2.72	Type:

Project No.: 50043.009

Date: 4/20/04

Remarks:

LAB NO. 2853

Fig No.

Client: BERNARDIN LOCHMUELLER & ASSOC.

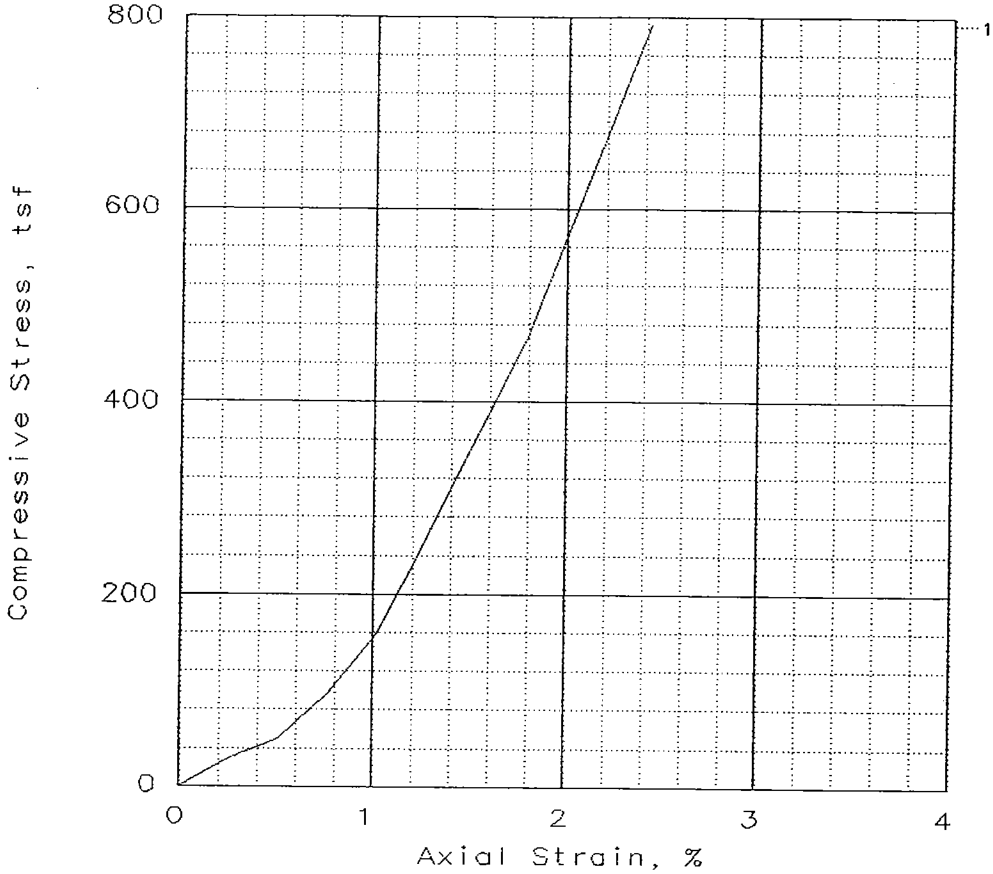
Project: PROP. 3-SPAN BRIDGE VOER LICK CREEK, ORANGE CO., IN

Location: STA.87+00,10.0'RT, LINE B
BORING:RB-2 DEPTH:5-6.5' SAMPLE:3

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	792			
Undrained shear strength, tsf	396			
Failure strain, %	2.4			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	166.1			
Dry density, pcf	165.9			
Saturation, %	12.0			
Void ratio	0.0351			
Specimen diameter, in	1.96			
Specimen height, in	3.91			
Height/diameter ratio	1.99			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3158

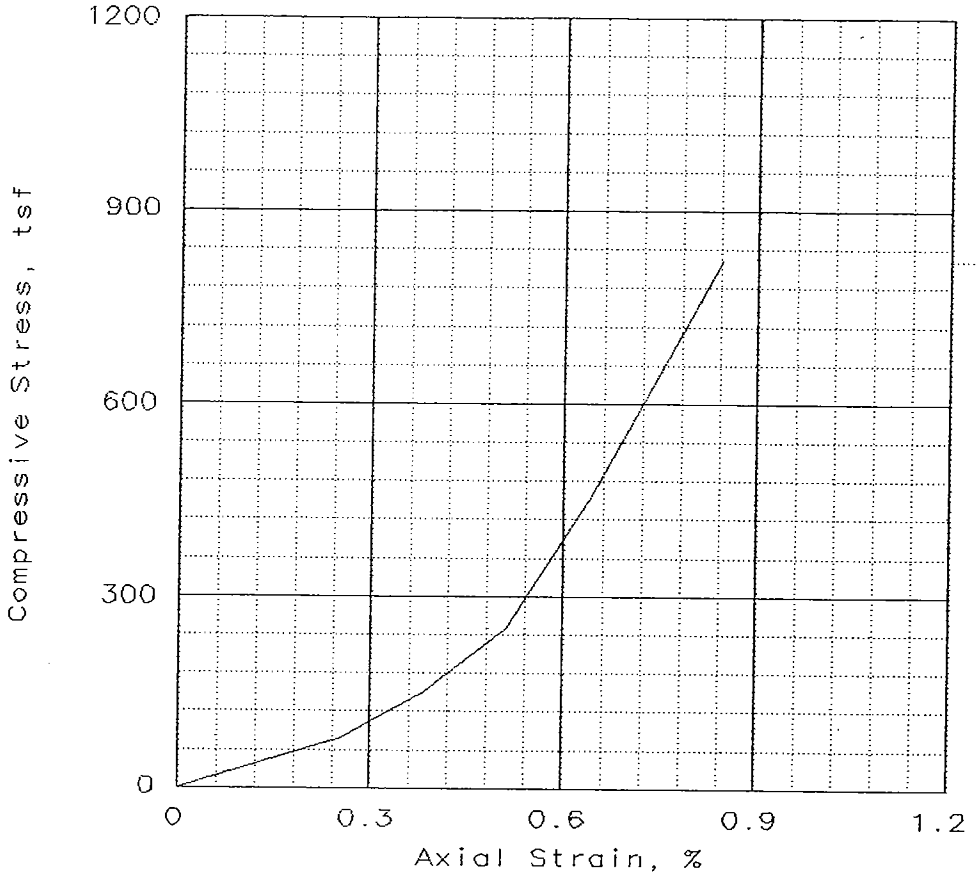
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.92+25,23.0'RT,LINE B
BORNG:RW-1 DEPTH:13.5' SAMPLE:1/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

Fig. No.: ______

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	823			
Undrained shear strength, tsf	412			
Failure strain, %	0.8			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	165.4			
Dry density, pcf	165.0			
Saturation, %	13.9			
Void ratio	0.0403			
Specimen diameter, in	1.96			
Specimen height, in	3.91			
Height/diameter ratio	1.99			

Description: LT GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3159

Fig. No.: ______

Client: BERNARDIN LOCHMUELLER & ASSOC.

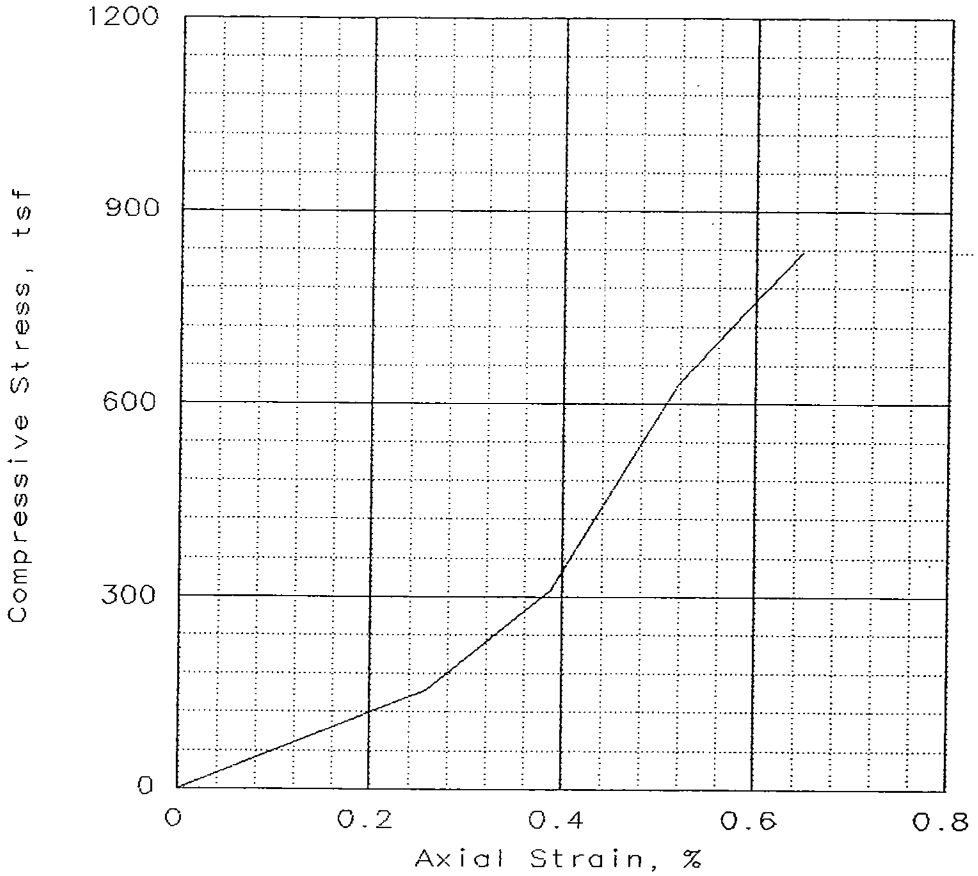
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.92+25,23.0'RT,LINE B
BORNG:RW-1 DEPTH:17.5' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	837			
Undrained shear strength, tsf	418			
Failure strain, %	0.6			
Strain rate, %/min	1.00			
Water content, %	0.3			
Wet density, pcf	156.9			
Dry density, pcf	156.4			
Saturation, %	11.6			
Void ratio	0.0738			
Specimen diameter, in	1.96			
Specimen height, in	3.86			
Height/diameter ratio	1.97			

Description: LT GR SANDSTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3160

Fig. No.: ____

Client: BERNARDIN LOCHMUELLER & ASSOC.

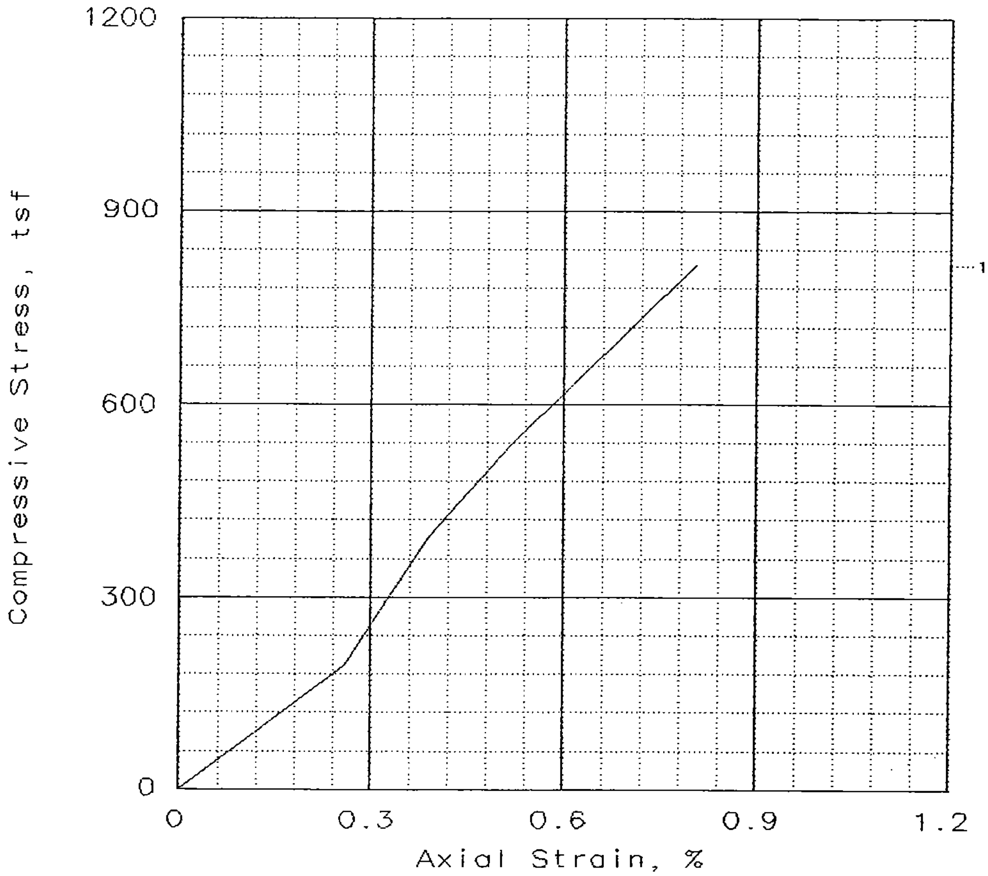
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.92+70,20.0'LT,LINE B
BORNG:RW-2 DEPTH:9.5' SAMPLE:1/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	817			
Undrained shear strength, tsf	408			
Failure strain, %	0.8			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	165.3			
Dry density, pcf	165.0			
Saturation, %	11.7			
Void ratio	0.0402			
Specimen diameter, in	1.96			
Specimen height, in	3.85			
Height/diameter ratio	1.96			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3161

Fig. No.: ____

Client: BERNARDIN LOCHMUELLER & ASSOC.

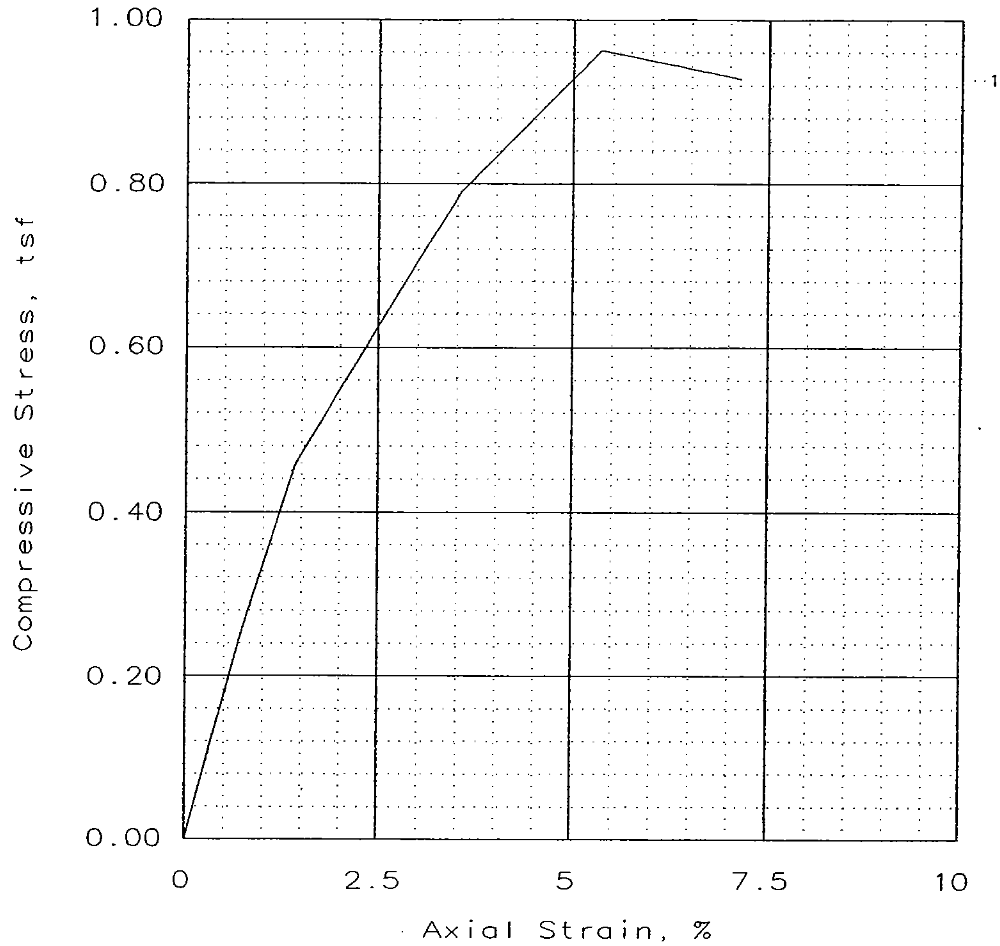
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.94+70,25.0'RT,LINE B
BORNG:RW-3 DEPTH:16.0' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



Sample number:	1			
Unconfined strength, tsf	0.96			
Undrained shear strength, tsf	0.48			
Strain rate, %/min	1.000			
Water content, %	18.5			
Wet density, pcf	131.7			
Dry density, pcf	111.1			
Saturation, %	0.5279			
Void ratio	95.3			
Specimen diameter, in	1.40			
Specimen height, in	2.80			

Description: BR SANDY LEAN CLAY MOIST-STIFF

LL =	PL =	PI =	GS = 2.72	Type:

Project No.: 50043.009
Date: 4/20/04
Remarks:
LAB NO. 2867

Fig No.

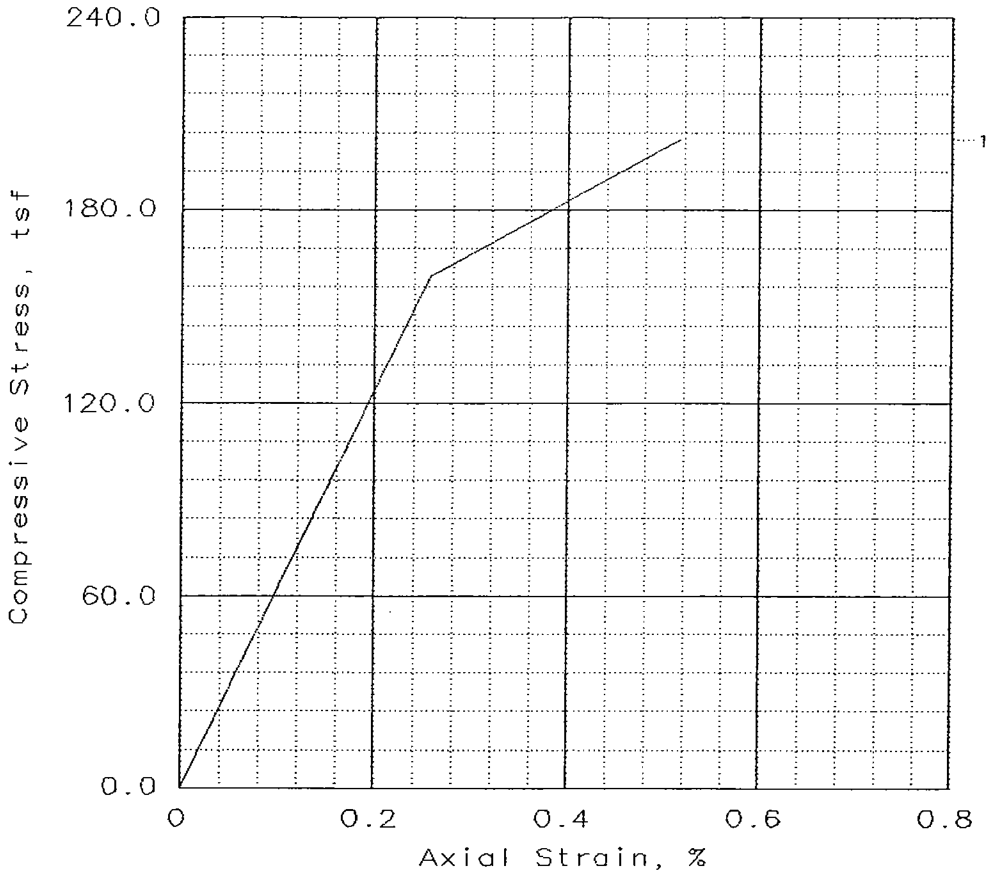
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.94+00,0'RT, LINE B
BORING:RW-3 DEPTH:7.5-9' SAMPLE:4

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	201.8			
Undrained shear strength, tsf	100.9			
Failure strain, %	0.5			
Strain rate, %/min	1.00			
Water content, %	0.4			
Wet density, pcf	161.4			
Dry density, pcf	160.7			
Saturation, %	16.2			
Void ratio	0.0681			
Specimen diameter, in	1.96			
Specimen height, in	3.88			
Height/diameter ratio	1.98			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3162

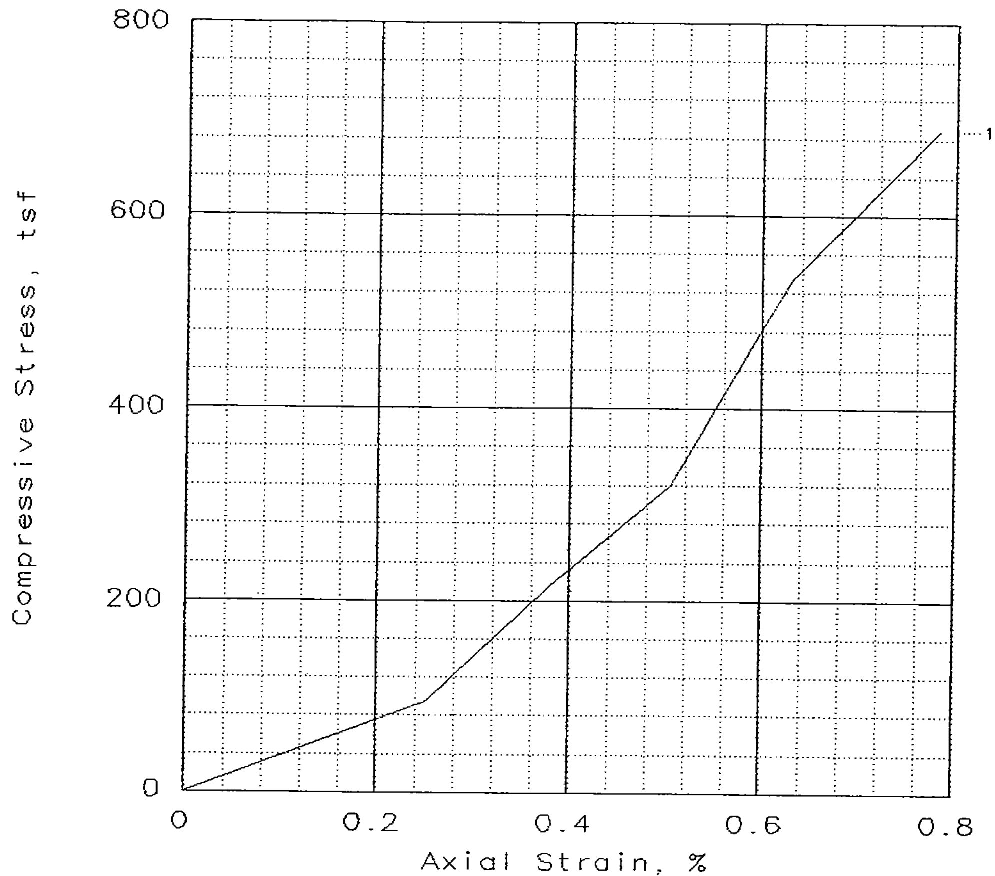
Fig. No.: ———

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.94+70,25.0'RT,LINE B
BORNG:RW-3 DEPTH:20.0' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	689			
Undrained shear strength, tsf	344			
Failure strain, %	0.8			
Strain rate, %/min	1.00			
Water content, %	0.6			
Wet density, pcf	162.5			
Dry density, pcf	161.5			
Saturation, %	26.7			
Void ratio	0.0628			
Specimen diameter, in	1.96			
Specimen height, in	3.96			
Height/diameter ratio	2.02			

Description: LT BR LIMESTONE

GS= 2.75 Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3150

Fig. No.: ——

Client: BERNARDIN LOCHMUELLER & ASSOC.

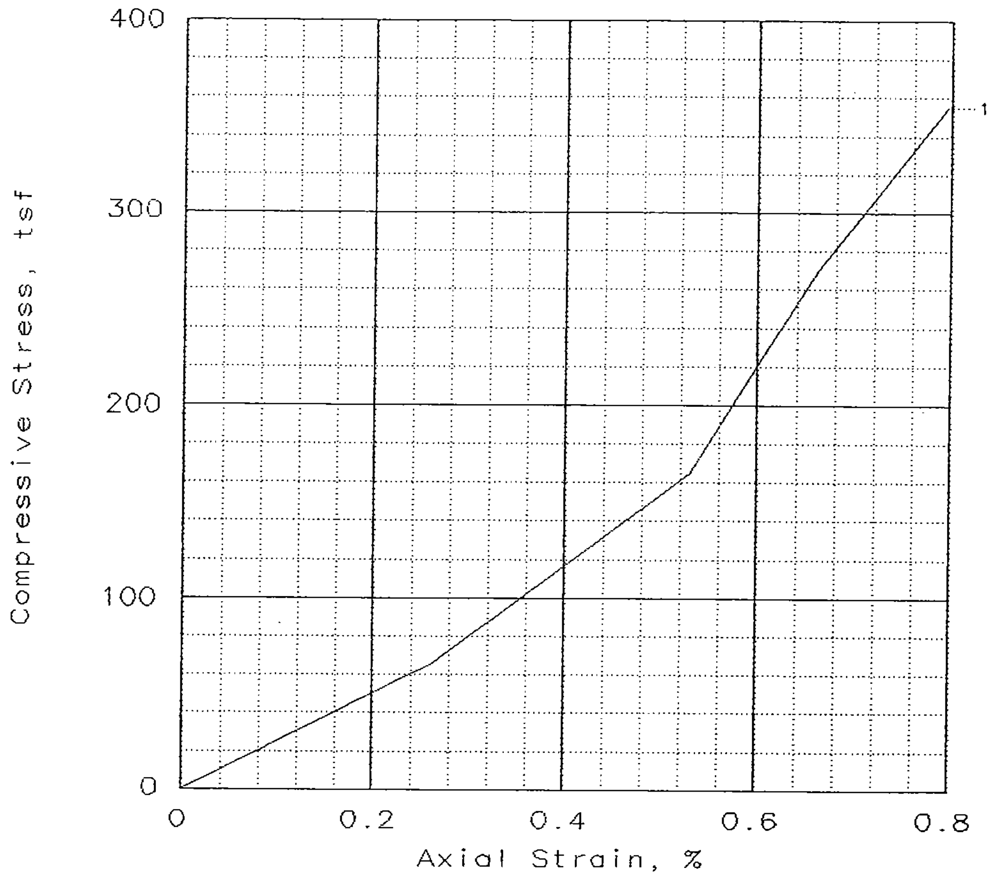
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.92+70,20.0'RT,LINE B
BORNG:TB-1 DEPTH:12.5' SAMPLE:1/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	355			
Undrained shear strength, tsf	178			
Failure strain, %	0.8			
Strain rate, %/min	1.00			
Water content, %	0.7			
Wet density, pcf	159.5			
Dry density, pcf	158.4			
Saturation, %	22.5			
Void ratio	0.0836			
Specimen diameter, in	1.96			
Specimen height, in	3.77			
Height/diameter ratio	1.92			

Description: LT BR LIMESTONE

GS= 2.75 | Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3151

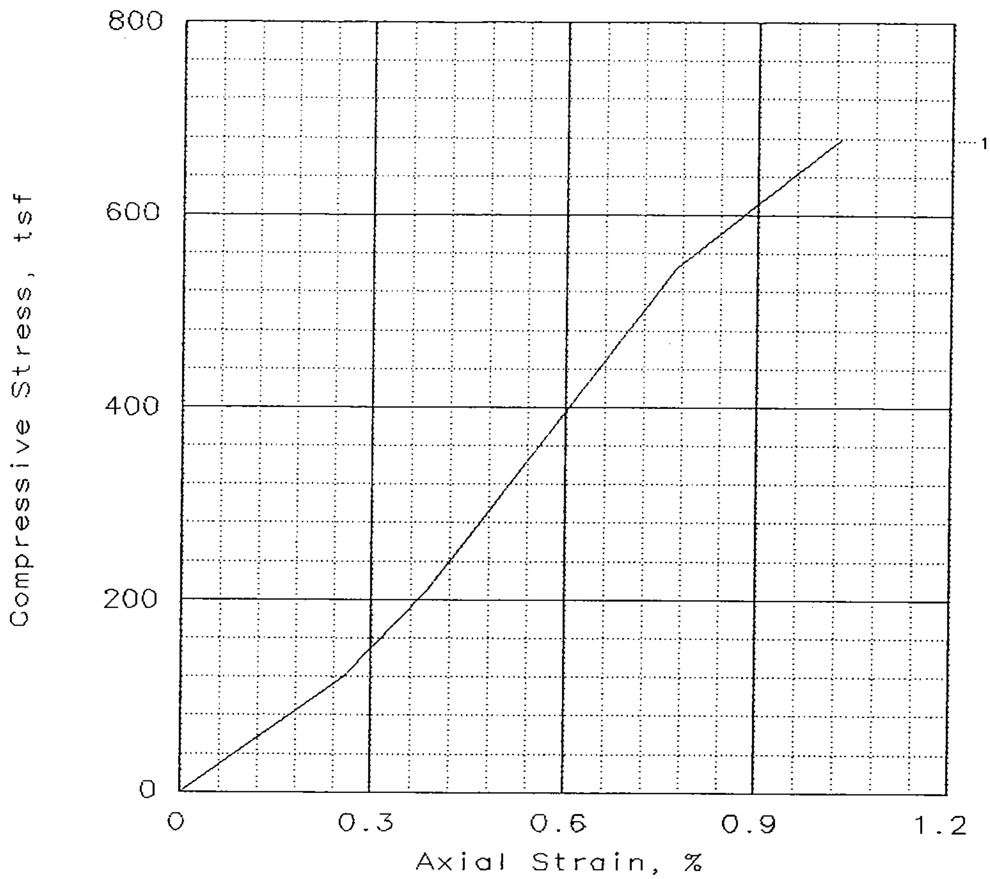
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.92+70,20.0'RT,LINE B
BORNG:TB-1 DEPTH:16.5' SAMPLE:1/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

Fig. No.: ____

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	678			
Undrained shear strength, tsf	339			
Failure strain, %	1.0			
Strain rate, %/min	1.00			
Water content, %	4.1			
Wet density, pcf	156.6			
Dry density, pcf	150.3			
Saturation, %	80.4			
Void ratio	0.1419			
Specimen diameter, in	1.96			
Specimen height, in	3.89			
Height/diameter ratio	1.98			

Description: GR LIMESTONE

			GS=	Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3152

Fig. No.: ____

Client: BERNARDIN LOCHMUELLER & ASSOC.

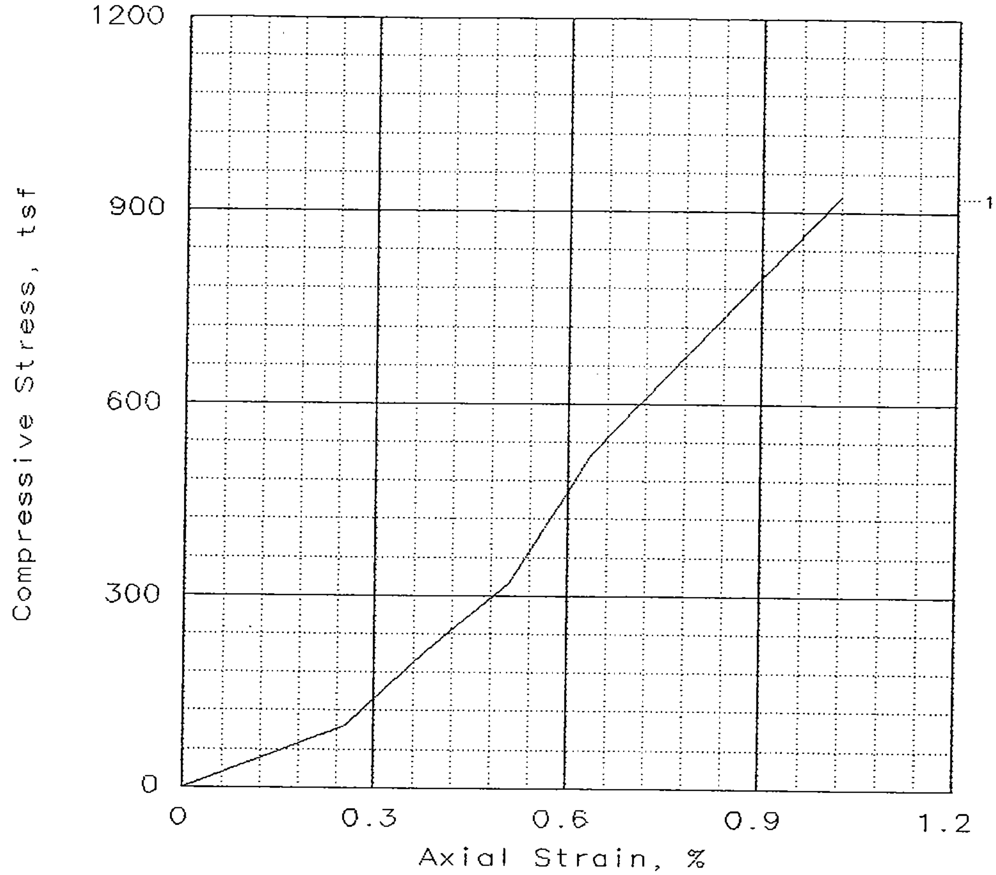
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.92+70,20.0'RT,LINE B
BORNG:TB-1 DEPTH:20.5' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	921			
Undrained shear strength, tsf	461			
Failure strain, %	1.0			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	164.8			
Dry density, pcf	164.4			
Saturation, %	13.0			
Void ratio	0.0442			
Specimen diameter, in	1.96			
Specimen height, in	3.92			
Height/diameter ratio	2.00			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3153

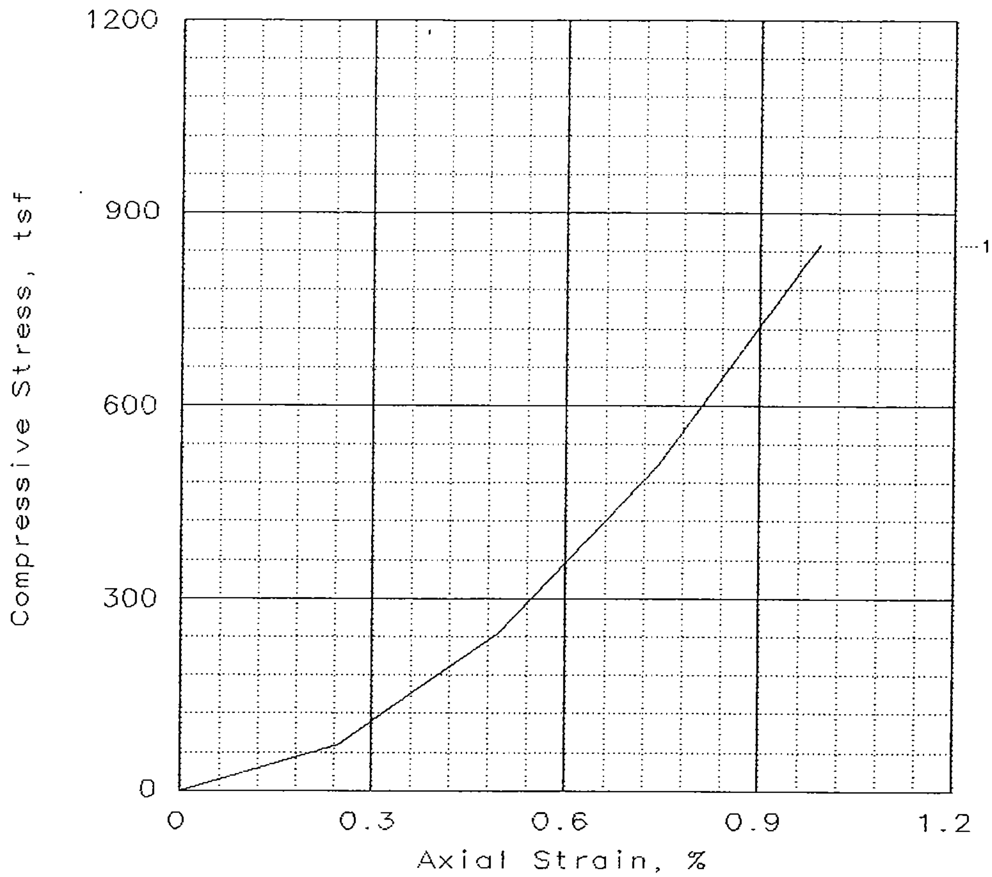
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.93+30,15.0'LT,LINE B
BORNG:TB-2 DEPTH:13.5' SAMPLE:1/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

Fig. No.: ____

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	851			
Undrained shear strength, tsf	425			
Failure strain, %	1.0			
Strain rate, %/min	1.00			
Water content, %	0.7			
Wet density, pcf	163.7			
Dry density, pcf	162.6			
Saturation, %	33.1			
Void ratio	0.0555			
Specimen diameter, in	1.96			
Specimen height, in	4.03			
Height/diameter ratio	2.06			

Description: GR LIMESTONE

			GS=	Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3154

Fig. No.: ____

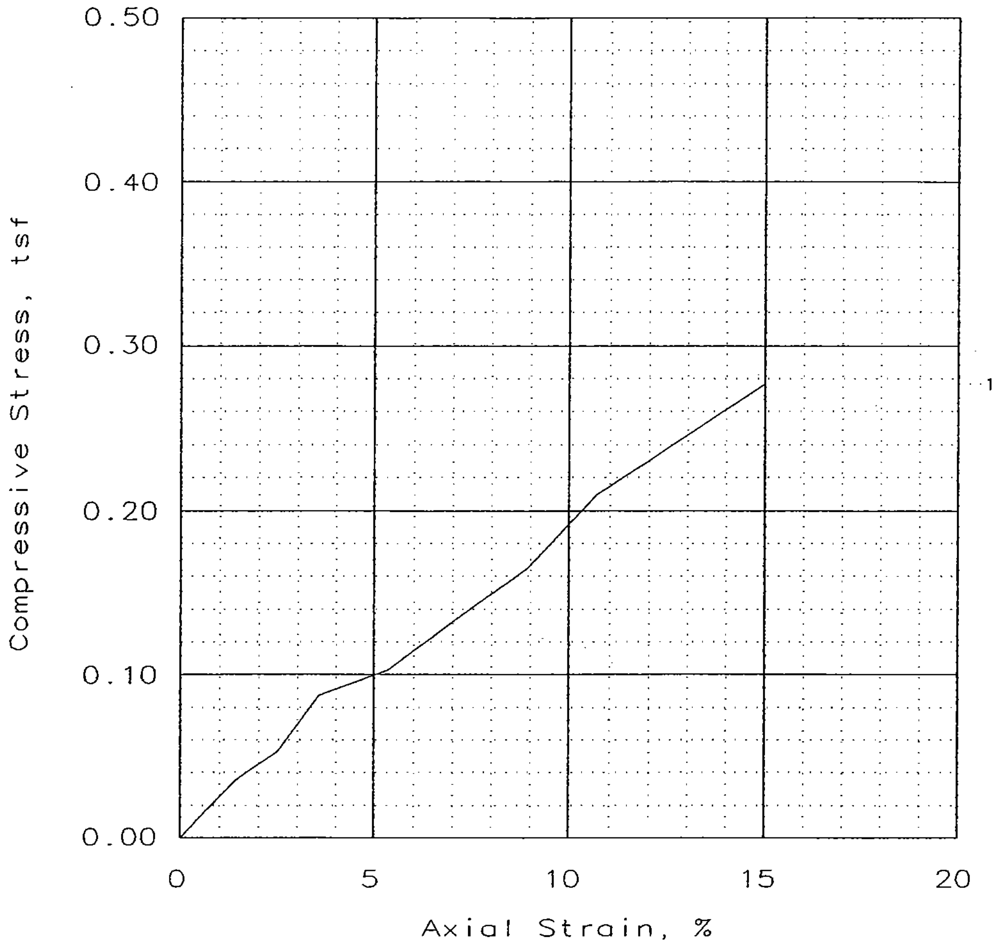
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.93+30,15.0'LT,LINE B
BORNG:TB-2 DEPTH:16.0' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



Sample number:	1			
Unconfined strength, tsf	0.28			
Undrained shear strength, tsf	0.14			
Strain rate, %/min	1.000			
Water content, %	20.2			
Wet density, pcf	132.8			
Dry density, pcf	110.5			
Saturation, %	0.5369			
Void ratio	102.2			
Specimen diameter, in	1.39			
Specimen height, in	2.80			

Description: BR SANDY LEAN CLAY MOIST-SOFT

LL =	PL =	PI =	GS = 2.72	Type:

Project No.: 50043.009
Date: 4/20/04
Remarks:
LAB NO. 2882

Client: BERNARDIN LOCHMUELLER & ASSOC.

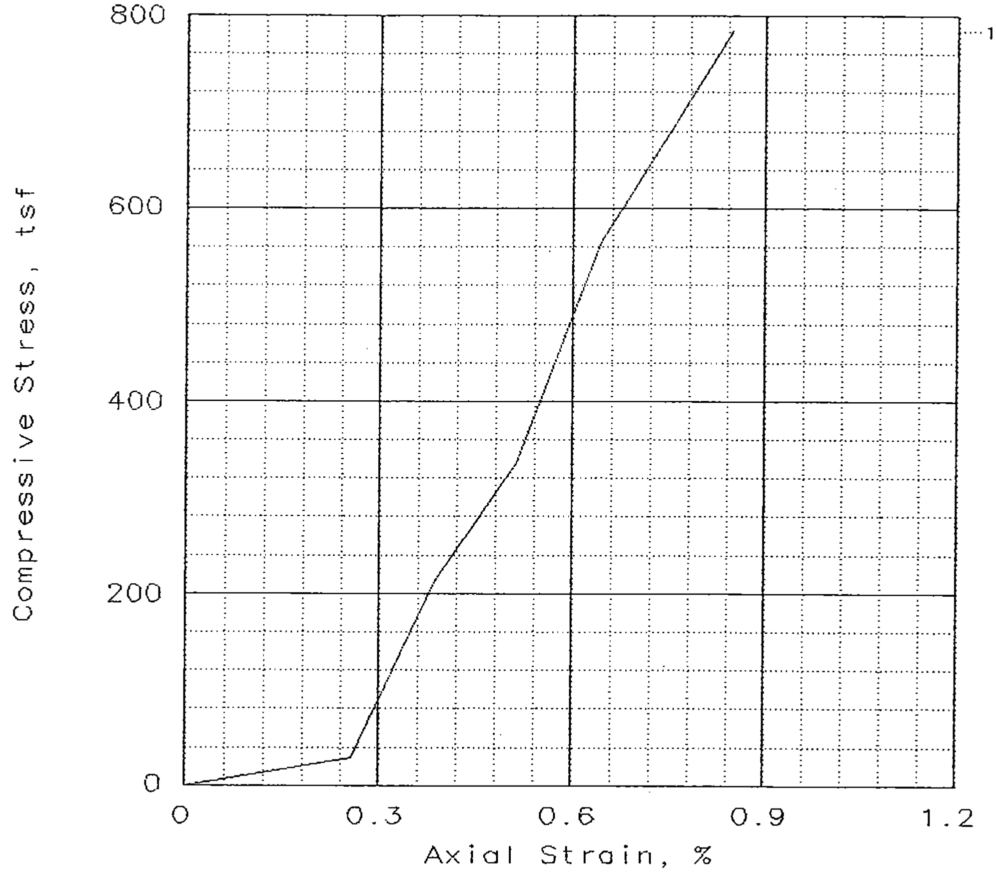
Project: PROP. 3-SPAN BRIDGE VOER LICK CREEK, ORANGE CO., IN
Location: STA.94+10, 20'RT, LINE B
BORING:TB-3 DEPTH:10-11.5' SAMPLE:5A

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Fig No.

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	784			
Undrained shear strength, tsf	392			
Failure strain, %	0.8			
Strain rate, %/min	1.00			
Water content, %	0.1			
Wet density, pcf	165.6			
Dry density, pcf	165.4			
Saturation, %	8.9			
Void ratio	0.0380			
Specimen diameter, in	1.95			
Specimen height, in	3.89			
Height/diameter ratio	1.99			

Description: GR LIMESTONE

GS=

Type:

Project No.: 50043.009
Date: 5/18/04
Remarks:
LAB NO. 3147

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

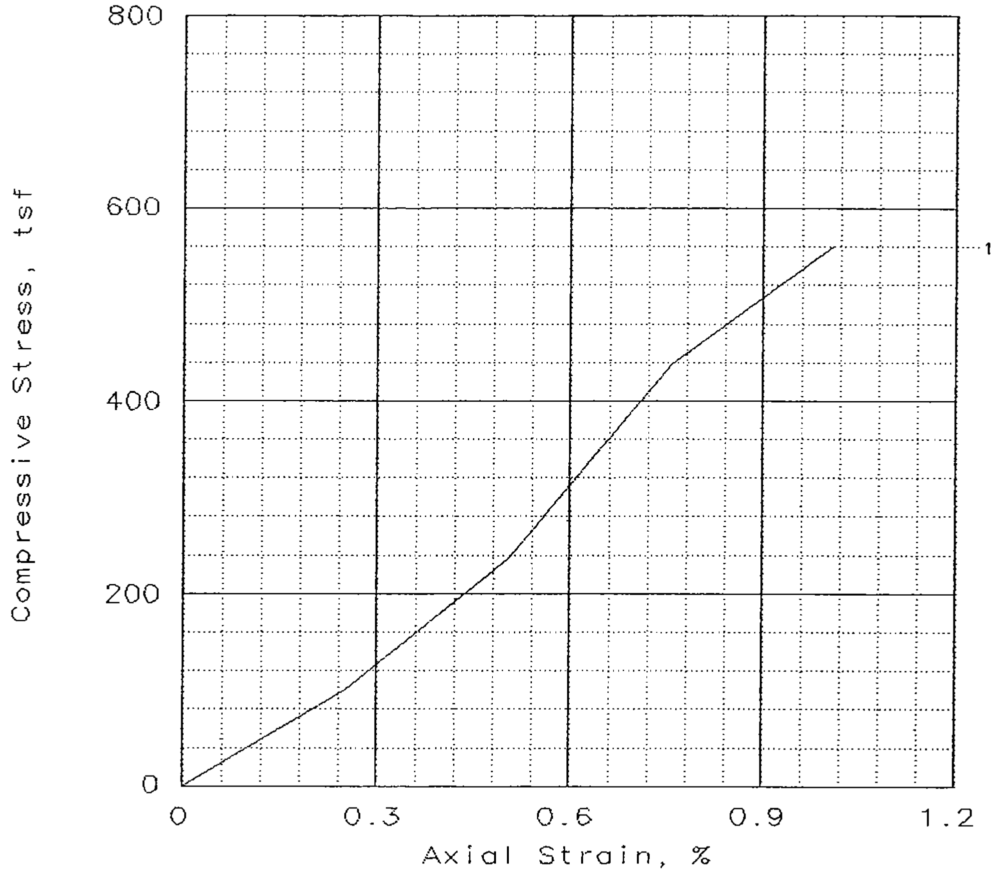
Location: STA.94+10,20.0'45,LINE B
BORING:TB-3 DEPTH:14.0 SAMPLE:1/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Fig. No.: ———

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	560			
Undrained shear strength, tsf	280			
Failure strain, %	1.0			
Strain rate, %/min	1.00			
Water content, %	0.1			
Wet density, pcf	162.6			
Dry density, pcf	162.3			
Saturation, %	7.1			
Void ratio	0.0575			
Specimen diameter, in	1.96			
Specimen height, in	3.96			
Height/diameter ratio	2.02			

Description: GR LIMESTONE

GS= Type:

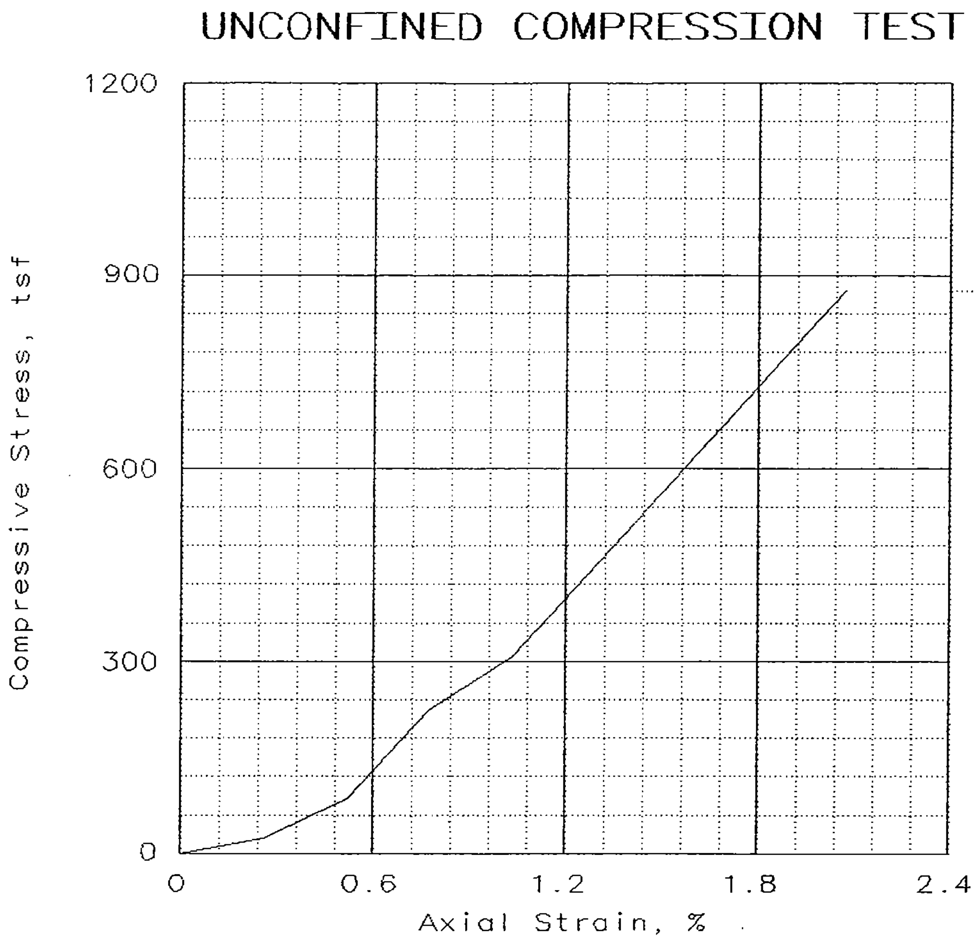
Project No.: 50043.009
Date: 5/18/04
Remarks:
LAB NO. 3148

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.94+10,20.0'45,LINE B
BORING:TB-3 DEPTH:18.0 SAMPLE:2/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

Fig. No.: ——



SAMPLE NO.	1			
Unconfined strength, tsf	876			
Undrained shear strength, tsf	438			
Failure strain, %	2.1			
Strain rate, %/min	1.00			
Water content, %	0.9			
Wet density, pcf	155.5			
Dry density, pcf	154.2			
Saturation, %	20.7			
Void ratio	0.1134			
Specimen diameter, in	1.96			
Specimen height, in	3.86			
Height/diameter ratio	1.97			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009

Date: 5/18/04

Remarks:

LAB NO. 3149

Fig. No.: ____

Client: BERNARDIN LOCHMUELLER & ASSOC.

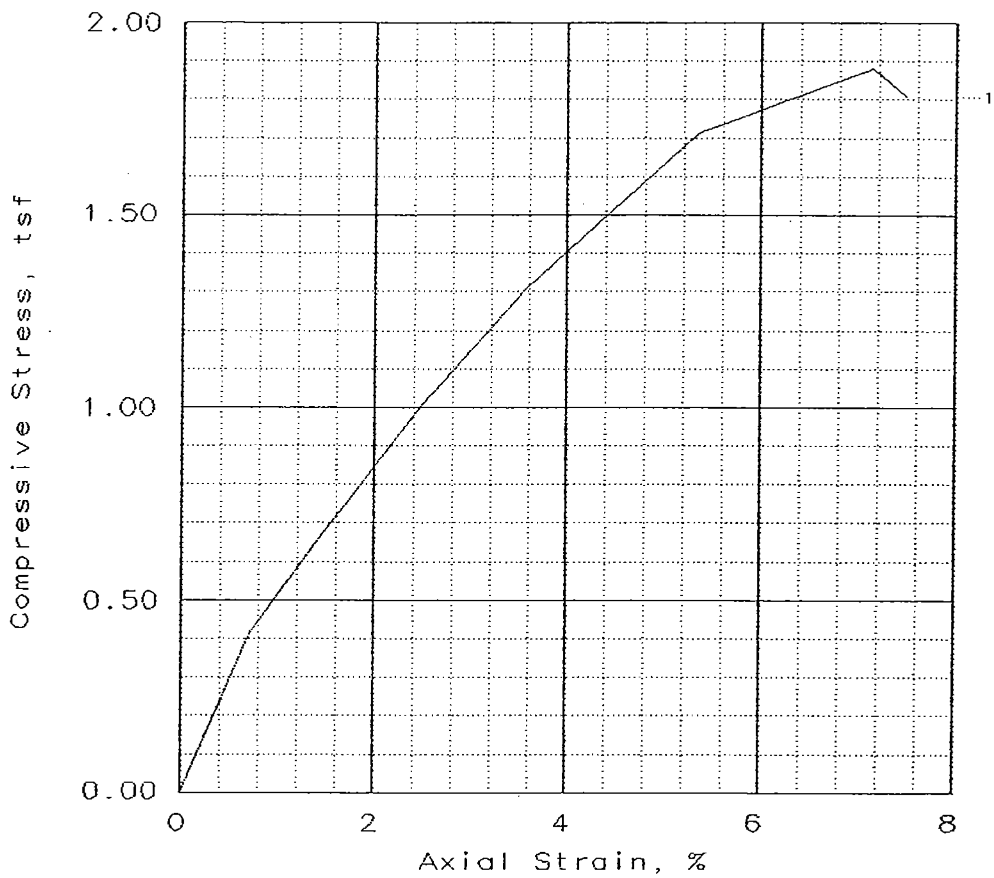
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.94+10,20.0'45,LINE B
BORING:TB-3 DEPTH:29.0 SAMPLE:4/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	1.88			
Undrained shear strength, tsf	0.94			
Failure strain, %	7.1			
Strain rate, %/min	1.00			
Water content, %	20.8			
Wet density, pcf	126.3			
Dry density, pcf	104.6			
Saturation, %	90.8			
Void ratio	0.6239			
Specimen diameter, in	1.39			
Specimen height, in	2.80			
Height/diameter ratio	2.01			

Description: BR & GR LEAN CLAY W/SAND, MOIST-STIFF

			GS=	Type:

Project No.: 50043.009
Date: 4/20/04
Remarks:
LAB NO. 2885

Fig. No.: ______

Client: BERNARDIN LOCHMUELLER & ASSOC.

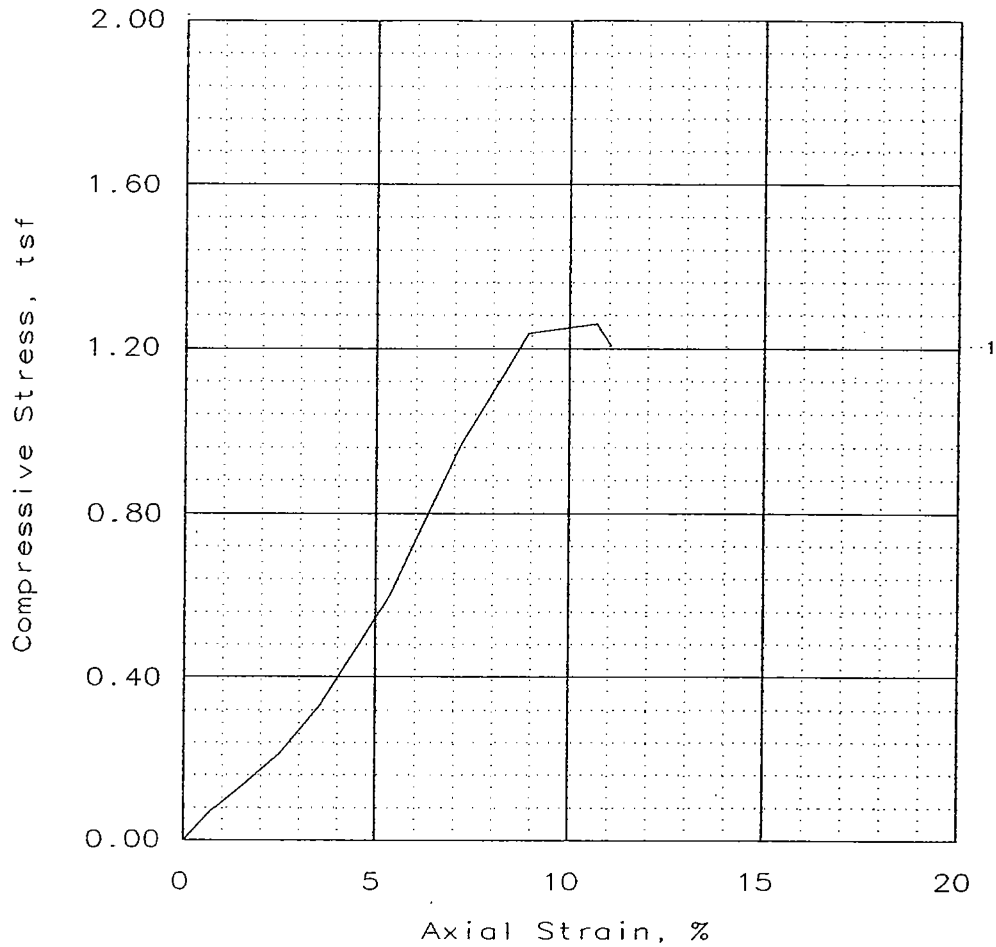
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.94+70, 15.0'LT, LINE B
BORING: TB-4 DEPTH: 5-6.5' SAMPLE: 3A

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



Sample number:	1			
Unconfined strength, tsf	1.26			
Undrained shear strength, tsf	0.63			
Strain rate, %/min	1.000			
Water content, %	18.3			
Wet density, pcf	136.0			
Dry density, pcf	114.9			
Saturation, %	0.4775			
Void ratio	104.5			
Specimen diameter, in	1.39			
Specimen height, in	2.80			

Description: BR & GR LEAN CLAY W/SAND, MOIST-STIFF

LL =	PL =	PI =	GS = 2.72	Type:

Project No.: 50043.009
Date: 4/20/04
Remarks:
LAB NO. 2886

Client: BERNARDINLOCHMUELLER & ASSOC.

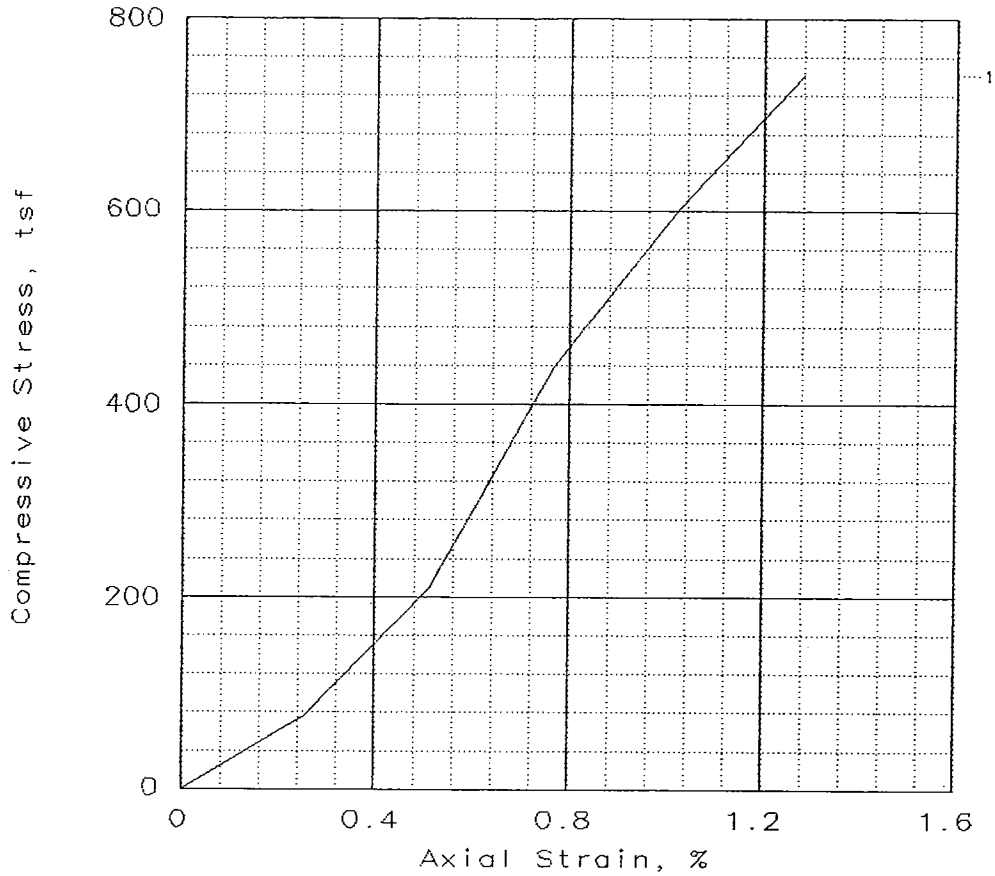
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.94+70,15.0'LT,LINE B
BORING:TB-4 DEPTH:7.5-9' SAMPLE:4A

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Fig No.

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	742			
Undrained shear strength, tsf	371			
Failure strain, %	1.3			
Strain rate, %/min	1.00			
Water content, %	0.7			
Wet density, pcf	158.4			
Dry density, pcf	157.3			
Saturation, %	21.2			
Void ratio	0.0911			
Specimen diameter, in	1.96			
Specimen height, in	3.90			
Height/diameter ratio	1.99			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3155

Fig. No.: ______

Client: BERNARDIN LOCHMUELLER & ASSOC.

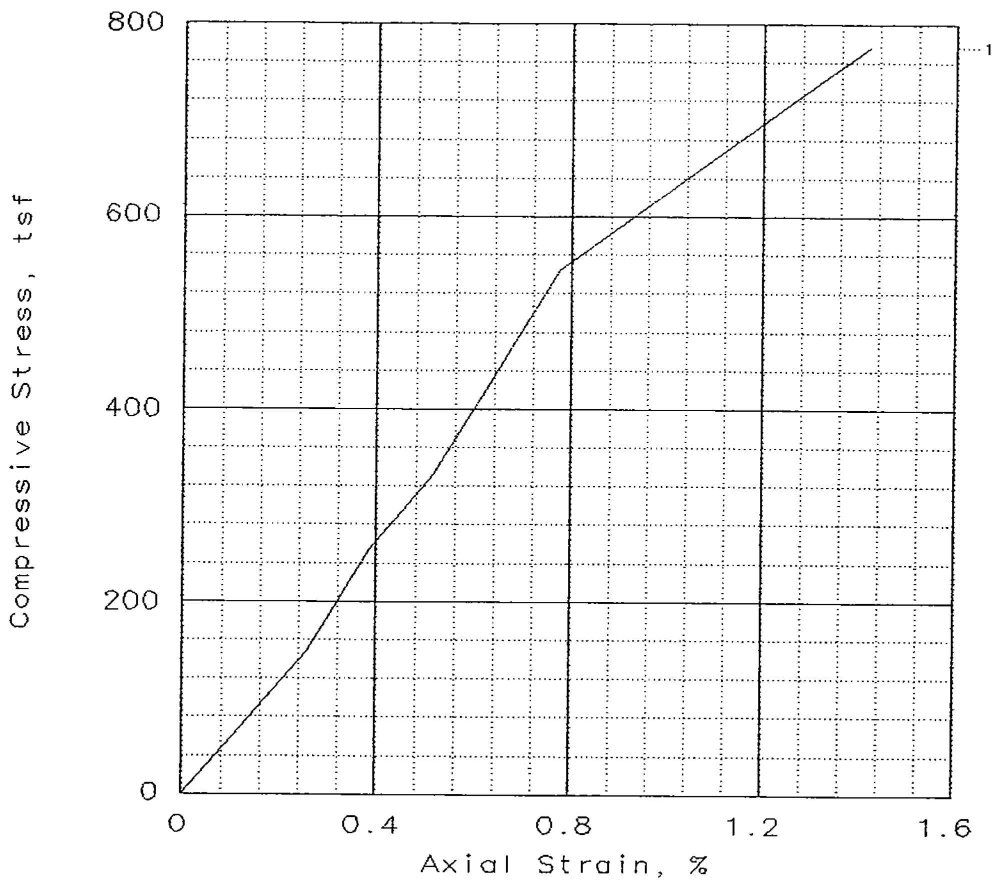
Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN

Location: STA.94+70,15.0'LT,LINE B
BORNG:TB-4 DEPTH:15.5' SAMPLE:2/RC

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	776			
Undrained shear strength, tsf	388			
Failure strain, %	1.4			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	168.7			
Dry density, pcf	168.4			
Saturation, %	29.1			
Void ratio	0.0197			
Specimen diameter, in	1.96			
Specimen height, in	3.87			
Height/diameter ratio	1.97			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3156

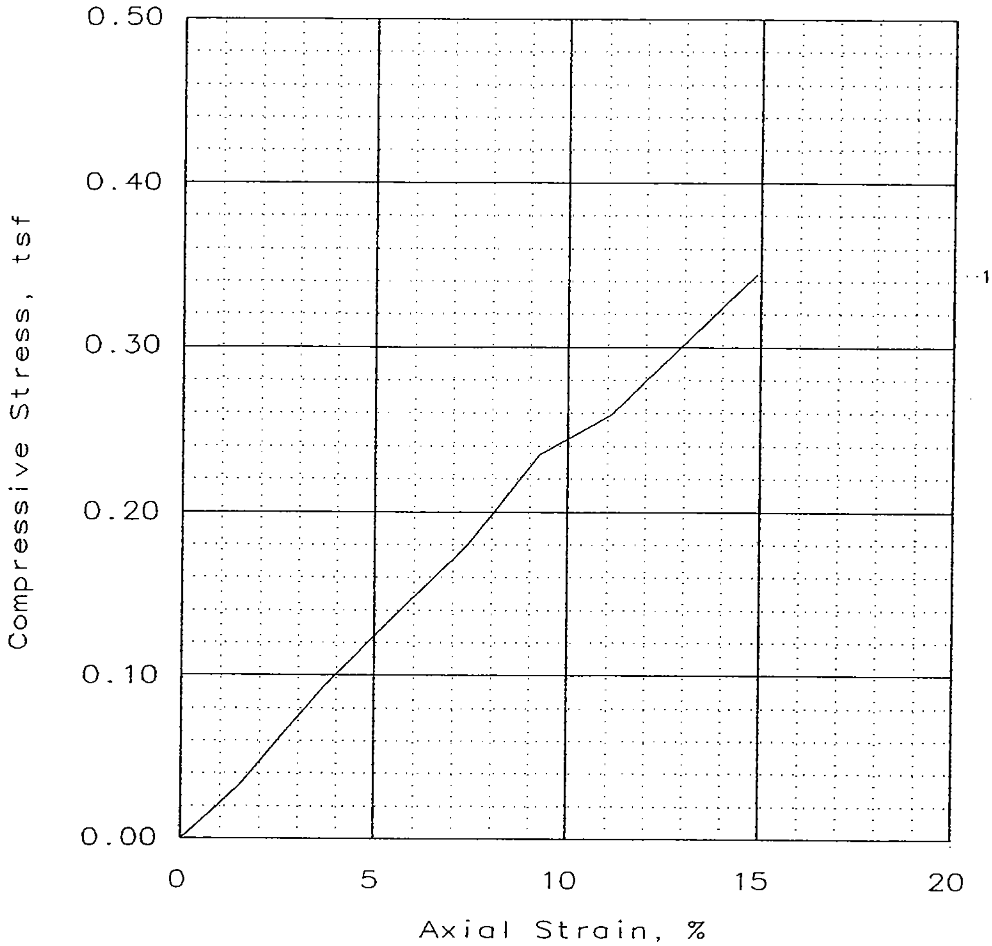
Fig. No.: ______

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.94+70,15.0'LT,LINE B
BORNG:TB-4 DEPTH:22.0' SAMPLE:3/RC

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



Sample number:	1			
Unconfined strength, tsf	0.34			
Undrained shear strength, tsf	0.17			
Strain rate, %/min	1.000			
Water content, %	24.5			
Wet density, pcf	123.4			
Dry density, pcf	99.1			
Saturation, %	0.7140			
Void ratio	93.4			
Specimen diameter, in	1.47			
Specimen height, in	2.70			

Description: BR LEAN CLAY, MOIST-SOFT

LL =	PL =	PI =	GS = 2.72	Type:

Project No.: 50043.009
Date: 4/20/04
Remarks:
LAB NO. 2892

Fig No.

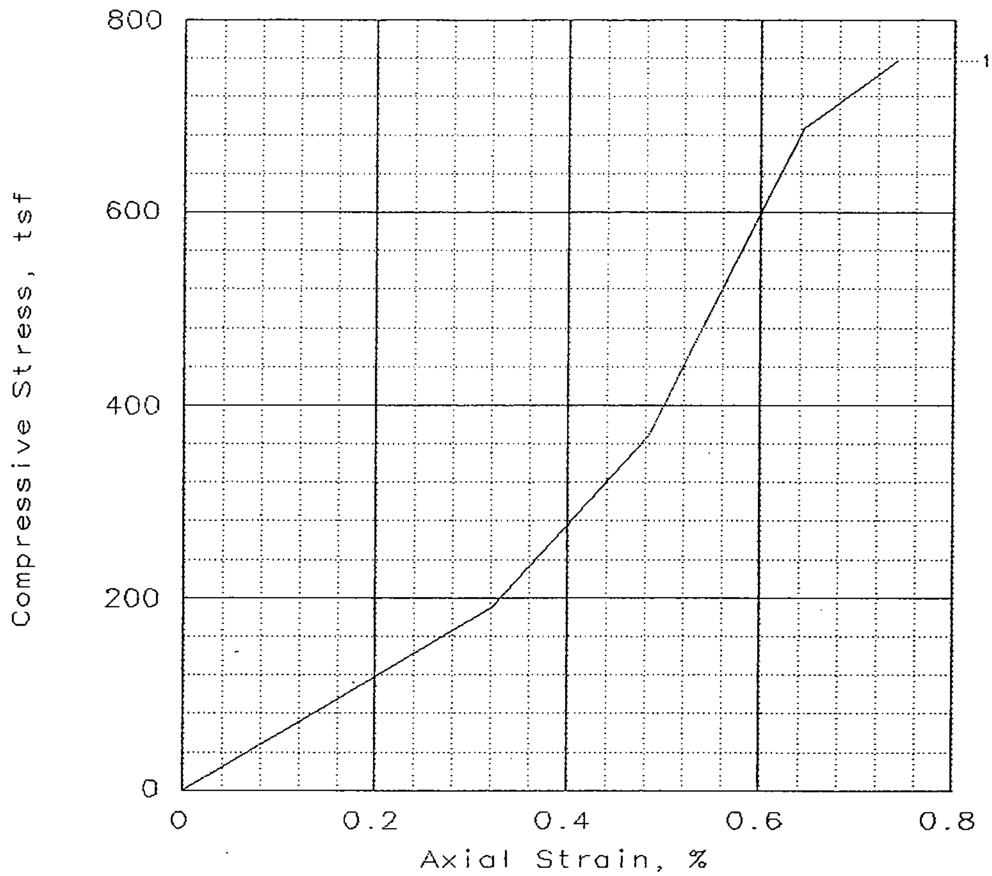
Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN.
Location: STA.91+70,22.5'RT,LINE B
BORING:TB-5 DEPTH:10-11.5' SAMPLE:5A

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

UNCONFINED COMPRESSION TEST



SAMPLE NO.	1			
Unconfined strength, tsf	758			
Undrained shear strength, tsf	379			
Failure strain, %	0.7			
Strain rate, %/min	1.00			
Water content, %	0.2			
Wet density, pcf	167.3			
Dry density, pcf	167.1			
Saturation, %	15.3			
Void ratio	0.0276			
Specimen diameter, in	1.96			
Specimen height, in	3.10			
Height/diameter ratio	1.58			

Description: GR LIMESTONE

GS= Type:

Project No.: 50043.009
Date: 5/20/04
Remarks:
LAB NO. 3157

Fig. No.: ____

Client: BERNARDIN LOCHMUELLER & ASSOC.

Project: PROP. 3-SPAN BRIDGE OVER LICK CREEK, ORANGE CO., IN
Location: STA.91+80,22.5'RT,LINE B
BORNG:TB-5 DEPTH:15.5' SAMPLE:1/RC

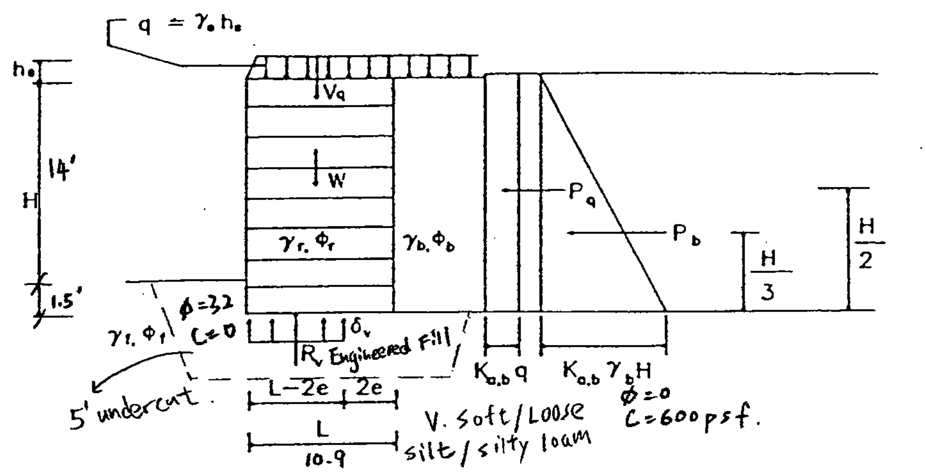
UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

Analysis I.
(Preliminary Evaluation)

JOB: Prop. 3-Span Bridge Replacement BRO-9959C
SHEET NO. 1 OF 6
CALCULATED BY: PC DATE: 6/3/04
CHECKED BY: DATE:
SCALE: Sta. 92+50, Line "B"

MSE WALL Analysis at Abutment # 1 (original Design)



Four potential external failure mechanisms are usually considered for soil structures:

- Sliding on the base (FS ≥ 1.5)
- ~~Overturning (FS ≥ 2.0)~~
- ~~Bearing Capacity Failure (FS ≥ 2.0)~~
- Global Stability (FS ≥ 1.3)

To calculate the bearing capacity:

$V_q = \gamma_s h_s L$

$W = \gamma_r H L$

$P_b = 0.5 K_{a,b} \gamma_b H^2$

$P_q = K_{a,b} \gamma_s h_s H$

$K_{a,b} = \tan^2(45 - \overset{30^\circ}{\phi_b}/2) = 0.33$

q_a = allowable bearing capacity of the soil.

$V_q = 250\,psf \times 10.9 = 2725$

$W = 125\,psf \times 15.5 \times 10.9 = 21119$

$P_b = 0.5 \times \tan^2(45 - \frac{30}{2}) \times 120 \times 15.5^2$
$= 4805$

$P_q = 0.33 \times 250 \times 15.5 = 1279$

1) The eccentricity, e, of the resultant loads:

$$e = \frac{\Sigma \text{ Driving Moments}}{\Sigma \text{ Resisting Forces}} = \frac{P_b(H/3) + P_q(H/2)}{W + V_q}$$

$$= \frac{K_{a,b} H^2(\gamma_b H + 3\gamma_s h_s)}{6L(\gamma_r H + \gamma_s h_s)} \le L/6$$

$$e = \frac{4805 \times (\frac{15.5}{3}) + 1279 \times (\frac{15.5}{2})}{21119 + 2725}$$

$= 1.46' \le L/6$ (OK)

Size = 23' × 10.9'

2) The magnitude of the vertical stress, $\sigma_{v\,max}$:

$$\sigma_{v\,max} = \frac{V_q + W}{L - 2e} = \frac{\gamma_r H + \gamma_s h_s}{1 - 2e/L} \le q_a$$

where: $q_a = q_{ult}/2$

Load angle $\theta = \tan^{-1}\left(\frac{4805 + 1279}{2725 + 21119}\right)$
$= 14^\circ$

Influenced Zone
$Z = 0.5 \times 10.9 \times \tan(45 + \frac{32}{2}) = 9.8'$

1) Two layer bearing capacity calculation:

We use Vesic (1975) method in CBEAR/PC Program for the upper granular soils, the $q_{ult} = 17220$ psf. (see attached)

For clay, we use $N_c = 5.14$ $s_c' = 0.2(\frac{10.9}{23}) = 0.09$ $s_q = d_q = 1$

$d_c' = 0.4 \tan^{-1}(\frac{1.5+5}{10.9}) = 0.22$

$q_{ult}'' = 5.14 \times 600 \times (1 + 0.09 + 0.22) + 6.5 \times 120 \times 1 \times 1$
$= 4820$ psf.

```
ÚÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ Federal Highway Administration ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¿
                        CBEAR/PC - BEARING CAPACITY ANALYSIS
                                Using Vesic    (1975)

 Project Name    : CR375W Bridge/MSE W  Client          : BLA
 File Name       : MSE1                 Project Manager : SS
 Date            :  6/ 3/04             Computed by     : PC


                        FOUNDATION AND SOIL DESCRIPTIONS


 * FOUNDATION GEOMETRY        WIDTH =  10.90    LENGTH =  23.00

 LEFT SIDE: ELEV. (ft)  X-COORD. (ft)  RIGHT SIDE: ELEV. (ft)  X-COORD. (ft)
            100.00        100.00                   100.00        110.90

                         FOUNDATION TYPE :  RECTANGULAR


 * SOIL DATA

 LEFT SIDE: ELEV. (ft)  X-COORD. (ft)  RIGHT SIDE: ELEV. (ft)  X-COORD. (ft)
            101.50        50.00                    101.50        150.00

    FRICTION ANGLE (deg)       COHESION (psf)    TOTAL UNIT WEIGHT (psf)
         32.00                      0.00                120.00


                            * SURCHARGE DESCRIPTION

   SURCH.   SURFACE   UNIT WEIGHT        SURCH.   SURFACE   UNIT WEIGHT
     NO.   ELEVATION     TOTAL             NO.   ELEVATION     TOTAL
              (ft)       (pcf)                      (ft)       (pcf)

      1     *******       ***               2     *******       ***

 ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

 * WATER TABLE DESCRIPTION               * APPLIED LOAD DESCRIPTION

  WATER TABLE ELEVATION :  90.00 (ft)     APPLIED LOAD         :   23.8  (Kips)
                                          X-COORD. of LOAD     :  104.0  (ft)
  UNIT WEIGHT OF WATER  :  62.40 (pcf)    Z-COORD. of LOAD     :   11.5  (ft)
                                          ANGLE of INCLINAT.   :   14.0  (deg.)


 * EFFECTIVE BASE DIMENSIONS :   WIDTH =   8.00     LENGTH =  23.00

     *** SUMMARY OF BEARING CAPACITY FACTORS ***
                                                          BEARING CAPACITY
  FACTORS               FNC           FNQ           FNG        (Ksf)

  BEARING CAP.       35.4903       23.1768       30.2147      23.0838
  SHAPE - CONC.       1.3095        1.2961        0.8104      20.7341  SELECTED
  SHAPE ECC.          1.2271        1.2173        0.8609      17.0277  SELECTED
  INCLINATION         1.0000        1.0000        0.9999      17.0269  SELECTED
```

```
³  BASE TILT        1.0000        1.0000        1.0000        17.0269  SELECTED  ³
³  GROUND SLOPE     1.0000        1.0000        1.0000        17.0269  SELECTED  ³
³  EMBEDMENT        1.0397        1.0380        1.0000        17.2199  SELECTED  ³
³                                                                                ³
³                   FNC      +    FNQ      +    FNG      =     Q                 ³
³  COMBINE EFFECTS                                                               ³
³  of FACTORS       0.000         5.271        11.949         17.220   (Ksf)     ³
³                                                                                ³
ÃÄÄÄ Hit arrow keys to display next screen. <F8>   Print. <F10>   Main Menu ÄÄÄÙ
```

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

To obtain the punching contribution with granular soils,

$$P_v = 1.5 \times 120 \times 5 + 120 \times \frac{5^2}{2} = 2400 \text{ lb/ft}$$

$$K_s = K_o = 1 - \sin 32 = 0.47 \qquad P = 2(23 + 10.9) = 67.8'$$

$$q'_{ult} = 4820 + \frac{67.8' \times 2400 \times 0.47 \times \tan 32^\circ}{23' \times 10.9'}$$

$$= 4820 + 191 = 5011 \text{ psf} \leq q_{ult} \text{ (OK)}$$

2) The magnitude of the vertical stress, σ_{vmax}.

$$\sigma_{vmax} = \frac{2725 + 2119}{10.9 - 2 \times 1.46'} = 2988$$

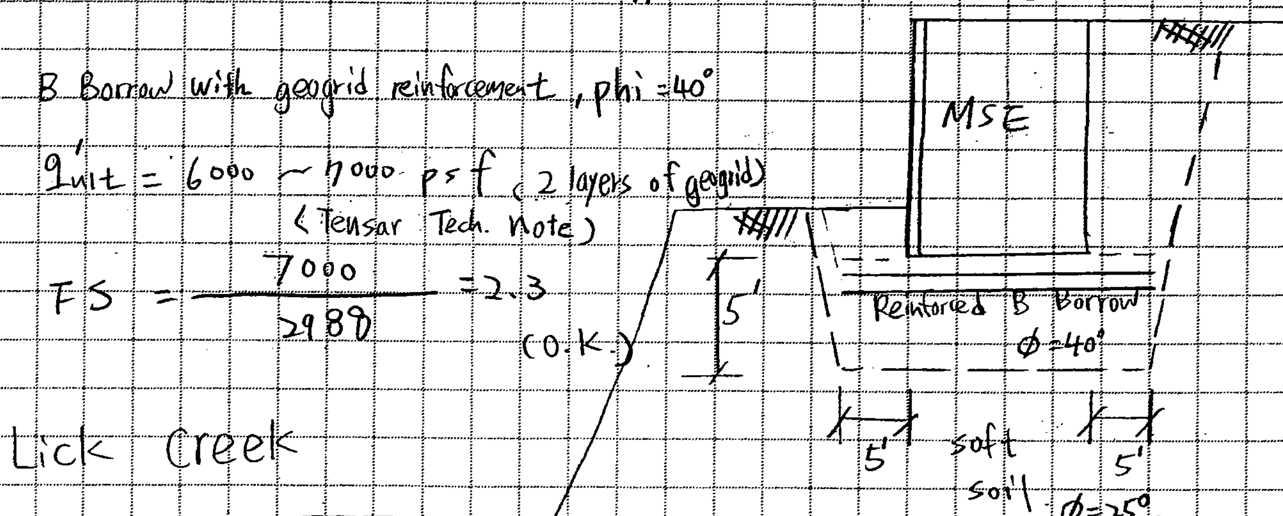
$$FS = \frac{5011}{2988} = 1.7 \text{ (NEED GROUND IMPROVEMENT)}$$
< 2.0

3) Therefore, 2 layers of geogrid are considered in conjunction with 5' undercut B Borrow backfill (approx. 12" c-c grid spacing)

B Borrow with geogrid reinforcement, phi = 40°

q'_{ult} = 6000 ~ 7000 psf (2 layers of geogrid)
(Tensar Tech. Note)

$$FS = \frac{7000}{2988} = 2.3$$
(O.K.)



H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

JOB 3-Span Bridge Replacement
SHEET NO. 5 OF 6
CALCULATED BY PC DATE 6/3/04
CHECKED BY ______ DATE ______
SCALE Sta. 92+50, Line "B"

$q_{ult}'' = 5.14 \times 750 \times (1+0.09+0.21) + 6.5 \times 120 \times 1 \times 1$

$= 5830$

$q_{ult}' = 5830 \text{ psf} + 191 \text{ psf} = 6021 \text{ psf}$

$F.S. = \frac{6021 \text{ psf}}{2988} = 2.0 \geq 2.0$ (min. requirement)

OK

A min. 5' undercut with 2 layer of Geogrid (App. 2' thick)

4) Check Global Stability in long term condition:

Soil Parameters used, and a traffic load of 250 psf is applied.

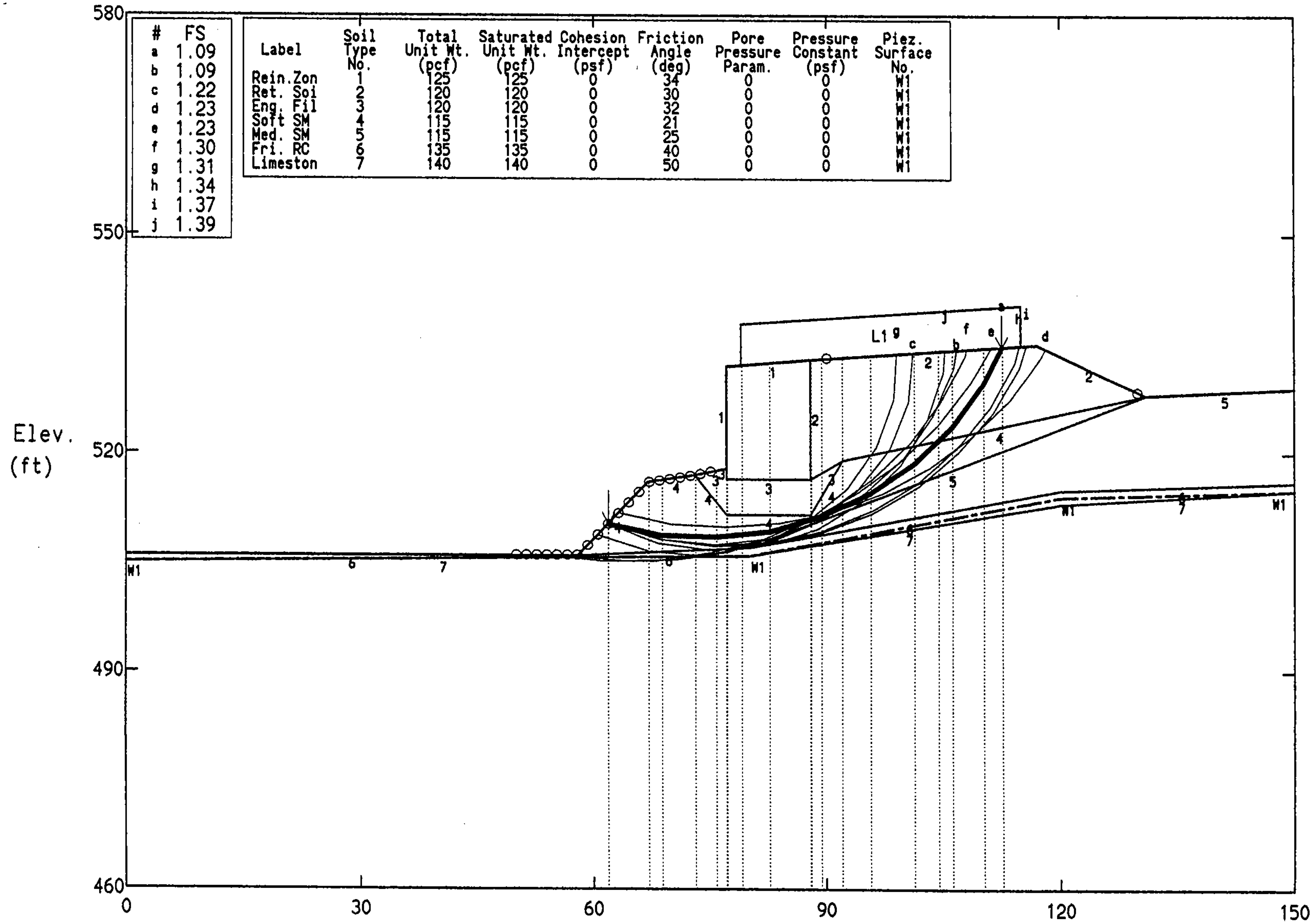
		γ	c	ϕ	@ c.
①	Reinforced Soil	125 pcf	0	34°	
②	Retained Soil	120 pcf	0	30°	
③	Engineered Fill	120 pcf	0	32°	
④	V. Loose SILT/SILTY LOAM	115 pcf	0	21°	
⑤	Loose SILTY/SANDY LOAM	115 pcf	0	25°	
⑥	Friable Limestone	135 pcf	0	40°	(8,000 psf)
⑦	Limestone	140 pcf	0	50°	(10,000 psf)

* STABL's result indicated a min. FS of 1.09 (see attached) which is lower than the min. requirement of 1.2

* Ground Improvement or Structural Modification is required.

BLA-CR375W Bridge Replacement, Orange Co Sta. 92+50, Line "B", Long-Term Cond.

Ten Most Critical. C:BLA2.PLT By: PC 8/27/2004 2:40pm



PCSTABL5M/SI FSmin=1.09 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

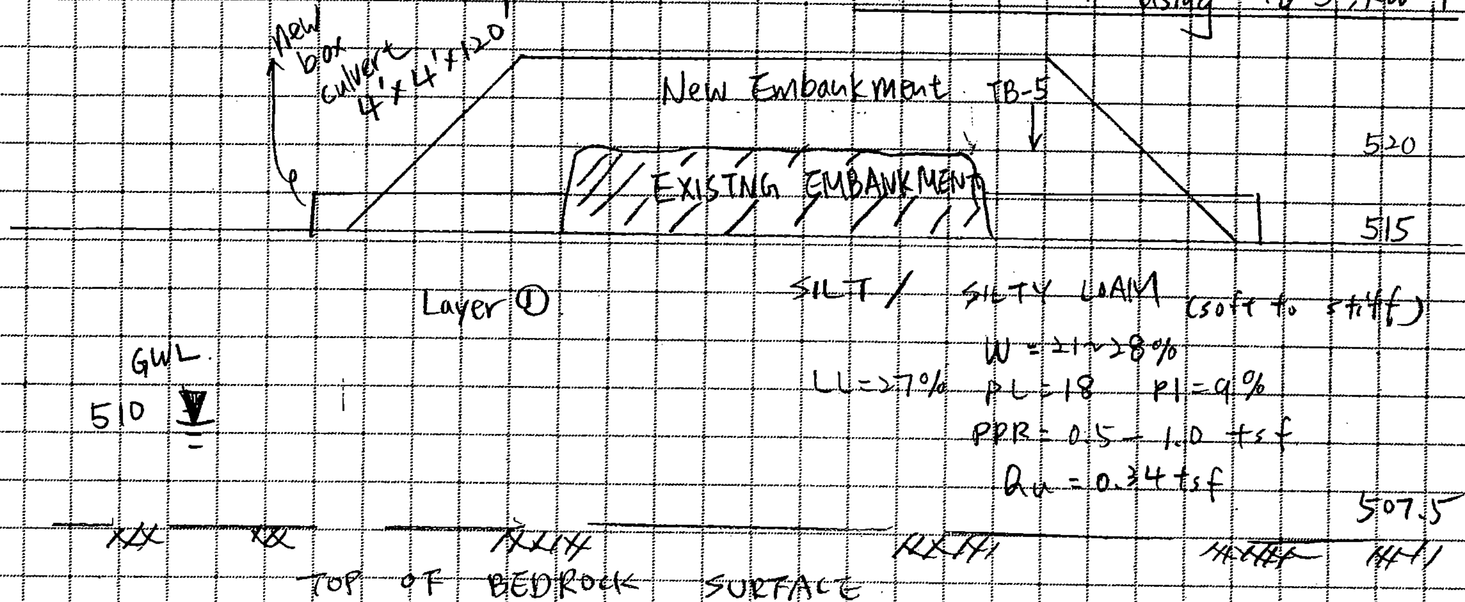
H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

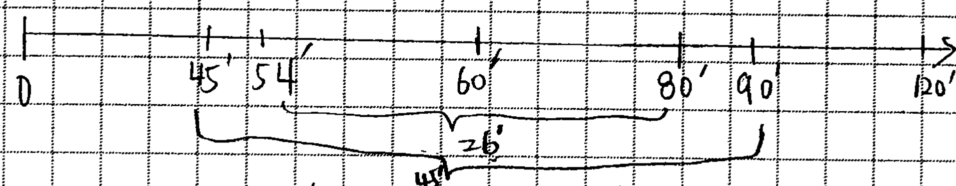
JOB 3-Span Bridge Replacement
SHEET NO. 1 OF 5 BRO-9959(
CALCULATED BY PC DATE 8/29/04
CHECKED BY ____ DATE ____
SCALE Sta. 91+64, Line "B"

alysis 2

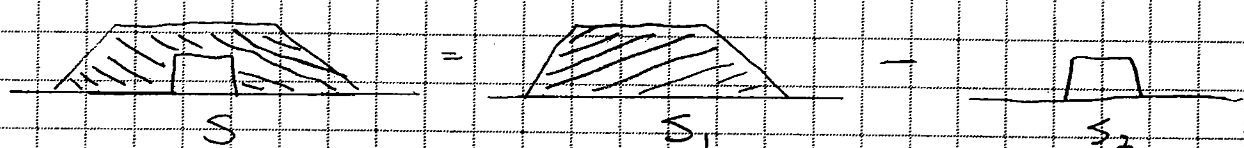
Embankment / Drainage Culvert Settlement Analyses:

Sta. 91+64 using TB-5, RW-1





Settlement Calculation = (Using Superposition method).



Layer ①: $\gamma = 115$ pcf

$RR = 0.001598 + 0.0007160\,W + 0.0004613\,PL = 0.024$ (W: 21, PL: 18)

$Cr = 0.00194(9 - 4.6) = 0.009$

USE 0.027

B.K. Hough Chart (F.ASCE)

$\text{Average } N = \frac{4+5+12}{3} = 7 \text{ bpf}$ $\quad RR = \frac{1}{30} = 0.033$

$P_c = \frac{340}{0.1 + 0.04 \times 9} = 2500 \text{ psf}$

NOTES:
In addition, the compression of the existing 5' (+8') tall embankment due to the new fill placement should also be considered (0.5") see attached. S_3

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

STRIP SYMMETRICAL VERTICAL EMBANKMENT LOADING

Project Name : BRO-9959 () CR375W
File Name : 9164B-Proposed
Date : 8/29/04

Client : BLA
Project Manager : SS
Computed by : PC

Settlement for X-Direction

Embankment slope a	= 45.00 (ft)	Height of fill H	= 17.00 (ft)
Embankment top width	= 45.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment bottom width	= 135.00 (ft)	p load/unit area	= 2040.00 (psf)
Ground Surface Elev.	= 515.00 (ft)	Foundation Elev.	= 515.00 (ft)
Water table Elev.	= 510.00 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COMP.	RECOMP. RATIO	SWELL.	UNIT WEIGHT (pcf)
1	COMP.	7.5	0.027	0.027	0.027	115.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	1.50	514.25	200.00	2500.00
2	1.50	512.75	258.75	2500.00
3	1.50	511.25	431.25	2500.00
4	1.50	509.75	588.15	2500.00
5	1.50	508.25	667.05	2500.00

	X =	0.00	X =	15.00	X =	30.00	X =	45.00
Layer	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)
1	10.82	0.01	680.01	0.31	1359.99	0.43	2029.18	0.51
2	32.44	0.02	680.17	0.27	1359.81	0.39	2007.54	0.46
3	53.97	0.02	680.78	0.20	1359.15	0.30	1985.92	0.36
4	75.37	0.03	682.05	0.16	1357.75	0.25	1964.33	0.31
5	96.58	0.03	684.12	0.15	1355.45	0.23	1942.78	0.29
		0.11		1.10		1.61		1.93

	X =	60.00
Layer	Stress (psf)	Sett. (in.)
1	2039.99	0.51
2	2039.72	0.46
3	2038.75	0.37
4	2036.67	0.32
5	2033.20	0.30
		1.95

3' undercut to reduce about 1" settlement.

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

STRIP SYMMETRICAL VERTICAL EMBANKMENT LOADING

Project Name	: BRO-9959 () CR375W	Client	: BLA
File Name	: 9164B-Existing	Project Manager	: SS
Date	: 8/29/04	Computed by	: PC

Settlement for X-Direction

Embankment slope a	= 1.00	(ft)	Height of fill H	= 5.00	(ft)
Embankment top width	= 29.00	(ft)	Unit weight of fill	= 120.00	(pcf)
Embankment bottom width	= 31.00	(ft)	p load/unit area	= 600.00	(psf)
Ground Surface Elev.	= 515.00	(ft)	Foundation Elev.	= 515.00	(ft)
Water table Elev.	= 510.00	(ft)	Unit weight of Wat.	= 62.40	(pcf)

N§.	LAYER TYPE	THICK. (ft)	COMP.	RECOMP. RATIO	SWELL.	UNIT WEIGHT (pcf)
1	COMP.	7.5	0.027	0.027	0.027	115.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	1.50	514.25	200.00	2500.00
2	1.50	512.75	258.75	2500.00
3	1.50	511.25	431.25	2500.00
4	1.50	509.75	588.15	2500.00
5	1.50	508.25	667.05	2500.00

	X =	0.00	X =	15.00	X =	30.00
Layer	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)
1	122.90	0.10	599.97	0.29	477.10	0.26
2	220.07	0.13	599.16	0.25	379.82	0.19
3	250.00	0.10	596.27	0.18	349.51	0.13
4	263.43	0.08	590.43	0.15	335.26	0.10
5	270.61	0.07	581.30	0.13	326.65	0.08
		------		------		------
		0.48		1.01		0.75

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

Prepared by H. C. Nutting 8/29/04

SETTLEMENT CALCULATIONS

CLIENT BLA
PROJECT CR375W over Lick Creek
SETTLEMENT CASE Sta. 91+64, Line "B" at the drainage box culvert-compression within the existing 5' tall embankment due to new fill
W.O. #: 50043.009

1/2 Width : 22.5 ft
1/2 Length: 100 ft

Po: 120 psf
Δ P: 1440 psf

	Elevation (ft)	Depth (ft)	Thickness (ft)	Center (ft)	γ (pcf)	Po (ksf)	Iz	Δ P (ksf)	Log10 (1+dP/Po)	Cr / (1+e)	Settlement (in.)	Sum (in.)
	520.0	0	*****	*****	*****	0.120	*****	*****	*****	*****	*****	*****
	519.5	0.5	0.5	0.3	115.0	0.149	1.000	1.440	1.029	0.012	0.07	0.07
	519.0	1.0	0.5	0.8	115.0	0.206	1.000	1.440	0.902	0.012	0.06	0.14
Existing Embankment Fill	518.5	1.5	0.5	1.3	115.0	0.264	1.000	1.440	0.810	0.012	0.06	0.20
	518.0	2.0	0.5	1.8	115.0	0.321	1.000	1.440	0.739	0.012	0.05	0.25
	517.5	2.5	0.5	2.3	115.0	0.379	1.000	1.439	0.681	0.012	0.05	0.30
	517.0	3.0	0.5	2.8	115.0	0.436	0.999	1.439	0.633	0.012	0.05	0.35
	516.5	3.5	0.5	3.3	115.0	0.494	0.999	1.438	0.592	0.012	0.04	0.39
	516.0	4.0	0.5	3.8	115.0	0.551	0.998	1.437	0.557	0.012	0.04	0.43
	515.5	4.5	0.5	4.3	115.0	0.609	0.997	1.436	0.526	0.012	0.04	0.47
	515.0	5.0	0.5	4.8	115.0	0.666	0.998	1.435	0.499	0.012	0.04	0.50
	514.5	5.5	0.5	5.3	115.0	0.724	0.995	1.433	0.474	0.000	0.00	0.50
	514.0	6.0	0.5	5.8	115.0	0.781	0.993	1.431	0.452	0.000	0.00	0.50
This layer is calculated using EMBANK program	513.5	6.5	0.5	6.3	115.0	0.839	0.992	1.428	0.432	0.000	0.00	0.50
	513.0	7.0	0.5	6.8	115.0	0.896	0.990	1.425	0.413	0.000	0.00	0.50
	512.5	7.5	0.5	7.3	115.0	0.954	0.987	1.422	0.396	0.000	0.00	0.50
	512.0	8.0	0.5	7.8	115.0	1.011	0.985	1.418	0.381	0.000	0.00	0.50
	511.5	8.5	0.5	8.3	115.0	1.069	0.982	1.414	0.386	0.000	0.00	0.50
	511.0	9.0	0.5	8.8	115.0	1.126	0.979	1.410	0.352	0.000	0.00	0.50
	510.5	9.5	0.5	9.3	115.0	1.184	0.975	1.405	0.340	0.000	0.00	0.50
GWL	510.0	10.0	0.5	9.8	115.0	1.241	0.972	1.399	0.328	0.000	0.00	0.50
	509.5	10.5	0.5	10.3	52.6	1.283	0.968	1.394	0.319	0.000	0.00	0.50
	509.0	11.0	0.5	10.8	52.6	1.309	0.964	1.388	0.314	0.000	0.00	0.50
	508.5	11.5	0.5	11.3	52.6	1.336	0.959	1.381	0.308	0.000	0.00	0.50
	508.0	12.0	0.5	11.8	52.6	1.362	0.955	1.375	0.303	0.000	0.00	0.50
	507.5	12.5	0.5	12.3	52.6	1.388	0.950	1.368	0.298	0.000	0.00	0.50
	507.0	13.0	0.5	12.8	52.6	1.415	0.945	1.360	0.293	0.000	0.00	0.50
	506.5	13.5	0.5	13.3	52.6	1.441	0.939	1.353	0.288	0.000	0.00	0.50
	506.0	14.0	0.5	13.8	52.6	1.467	0.934	1.345	0.282	0.000	0.00	0.50
	505.5	14.5	0.5	14.3	52.6	1.494	0.928	1.336	0.278	0.000	0.00	0.50
	505.0	15.0	0.5	14.8	52.6	1.520	0.922	1.328	0.273	0.000	0.00	0.50
	504.5	15.5	0.5	15.3	52.6	1.546	0.916	1.319	0.268	0.000	0.00	0.50
Limestone Rock	504.0	16.0	0.5	15.8	52.6	1.572	0.910	1.310	0.263	0.000	0.00	0.50
	503.5	16.5	0.5	16.3	52.6	1.599	0.904	1.301	0.259	0.000	0.00	0.50
	503.0	17.0	0.5	16.8	52.6	1.625	0.897	1.292	0.254	0.000	0.00	0.50
	502.5	17.5	0.5	17.3	52.6	1.651	0.890	1.282	0.250	0.000	0.00	0.50
	502.0	18.0	0.5	17.8	57.6	1.679	0.884	1.273	0.245	0.000	0.00	0.50
	501.5	18.5	0.5	18.3	57.6	1.708	0.877	1.263	0.240	0.000	0.00	0.50
	501.0	19.0	0.5	18.8	57.6	1.737	0.870	1.253	0.236	0.000	0.00	0.50
	500.5	19.5	0.5	19.3	57.6	1.765	0.863	1.243	0.232	0.000	0.00	0.50
	500.0	20.0	0.5	19.8	57.6	1.794	0.856	1.233	0.227	0.000	0.00	0.50
	499.5	20.5	0.5	20.3	57.6	1.823	0.849	1.223	0.223	0.000	0.00	0.50
	499.0	21.0	0.5	20.8	57.6	1.852	0.842	1.213	0.219	0.000	0.00	0.50
	498.5	21.5	0.5	21.3	57.6	1.881	0.835	1.202	0.215	0.000	0.00	0.50
	498.0	22.0	0.5	21.8	57.6	1.909	0.828	1.192	0.211	0.000	0.00	0.50
	497.5	22.5	0.5	22.3	57.6	1.938	0.821	1.182	0.207	0.000	0.00	0.50
	497.0	23.0	0.5	22.8	57.6	1.967	0.814	1.172	0.203	0.000	0.00	0.50
	496.5	23.5	0.5	23.3	57.6	1.996	0.806	1.161	0.199	0.000	0.00	0.50
	496.0	24.0	0.5	23.8	57.6	2.025	0.799	1.151	0.196	0.000	0.00	0.50
	495.5	24.5	0.5	24.3	57.6	2.053	0.792	1.141	0.192	0.000	0.00	0.50
	495.0	25.0	0.5	24.8	57.6	2.082	0.785	1.131	0.188	0.000	0.00	0.50
	494.5	25.5	0.5	25.3	57.6	2.111	0.778	1.120	0.185	0.000	0.00	0.50
	494.0	26.0	0.5	25.8	57.6	2.140	0.771	1.110	0.182	0.000	0.00	0.50
	493.5	26.5	0.5	26.3	57.6	2.169	0.764	1.100	0.178	0.000	0.00	0.50
	493.0	27.0	0.5	26.8	57.6	2.197	0.757	1.090	0.175	0.000	0.00	0.50
	492.5	27.5	0.5	27.3	57.6	2.226	0.750	1.080	0.172	0.000	0.00	0.50
	492.0	28.0	0.5	27.8	57.6	2.255	0.743	1.070	0.169	0.000	0.00	0.50
	491.5	28.5	0.5	28.3	57.6	2.284	0.736	1.060	0.166	0.000	0.00	0.50
	491.0	29.0	0.5	28.8	57.6	2.313	0.730	1.051	0.163	0.000	0.00	0.50
	490.5	29.5	0.5	29.3	57.6	2.341	0.723	1.041	0.160	0.000	0.00	0.50
	490.0	30.0	0.5	29.8	57.6	2.370	0.716	1.031	0.157	0.000	0.00	0.50
	489.5	30.5	0.5	30.3	57.6	2.399	0.709	1.022	0.154	0.000	0.00	0.50
	489.0	31.0	0.5	30.8	57.6	2.428	0.703	1.012	0.151	0.000	0.00	0.50
	488.5	31.5	0.5	31.3	57.6	2.457	0.696	1.003	0.149	0.000	0.00	0.50
	488.0	32.0	0.5	31.8	57.6	2.485	0.690	0.994	0.146	0.000	0.00	0.50
	487.5	32.5	0.5	32.3	57.6	2.514	0.684	0.984	0.143	0.000	0.00	0.50
	487.0	33.0	0.5	32.8	57.6	2.543	0.677	0.975	0.141	0.000	0.00	0.50
	486.5	33.5	0.5	33.3	57.6	2.572	0.671	0.966	0.139	0.000	0.00	0.50
	486.0	34.0	0.5	33.8	57.6	2.601	0.665	0.957	0.136	0.000	0.00	0.50
	485.5	34.5	0.5	34.3	57.6	2.629	0.659	0.948	0.134	0.000	0.00	0.50
	485.0	35.0	0.5	34.8	57.6	2.658	0.653	0.940	0.131	0.000	0.00	0.50
	484.5	35.5	0.5	35.3	57.6	2.687	0.647	0.931	0.129	0.000	0.00	0.50
	484.0	36.0	0.5	35.8	57.6	2.716	0.641	0.923	0.127	0.000	0.00	0.50
	483.5	36.5	0.5	36.3	57.6	2.745	0.635	0.914	0.125	0.000	0.00	0.50
	483.0	37.0	0.5	36.8	57.6	2.773	0.629	0.906	0.123	0.000	0.00	0.50
	482.5	37.5	0.5	37.3	57.6	2.802	0.623	0.898	0.121	0.000	0.00	0.50
	482.0	38.0	0.5	37.8	57.6	2.831	0.618	0.889	0.119	0.000	0.00	0.50
	481.5	38.5	0.5	38.3	57.6	2.860	0.612	0.881	0.117	0.000	0.00	0.50
	481.0	39.0	0.5	38.8	57.6	2.889	0.607	0.873	0.115	0.000	0.00	0.50
	480.5	39.5	0.5	39.3	57.6	2.917	0.601	0.866	0.113	0.000	0.00	0.50
	480.0	40.0	0.5	39.8	57.6	2.946	0.596	0.858	0.111	0.000	0.00	0.50

← within existing embankment!

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

JOB 3-Span Bridge Replacement
SHEET NO. 5 OF 5
CALCULATED BY PC DATE 8/29/04
CHECKED BY DATE
SCALE Sta. 91+64, Line "B"

* Settlement due to the placement of 12'~17' of New Embankment Fill at Sta. 91+64, Line "B"

Offset	60'	45'	30'	15'	CL
S_1	0.11"	1.10"	1.61"	1.93"	1.95"
S_2				+ 0.48"	- 1.01"
S_3 (within existing embankment)				+ 0.5"	+ 0.5"
	0.11"	1.10"	1.61"	1.95"	1.44"

$\underbrace{1.61'' \quad 1.95'' \quad 1.44''}_{\text{Pavement area}}$

** We anticipate that approximate $1\frac{1}{2}$" ~ 2" settlement may occur near the existing drainage structure where max. fill of 17' is proposed.

*** SILTY LOAM encountered (12.5' thick) is similar to the silty loam at Sta. 94+70 (Analysis 4) and therefore, we anticipate that a min. of 4 weeks waiting period after the completion of new fill placement prior to any pavement construction.

**** A min. 3' undercut and replacement with INDOT B Borrow is recommended immediately below the new box culvert. With this option, approx. $\frac{3}{4}$" ~ 1" settlement can be reduced.

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

Analysis 3

Embankment Slope Stability Analyses:

Sta. 91+64, Line "B" Using TB-5 RW-1

Soil parameter used:

		γ	Short-Term c	Short-Term ϕ	Long-Term c	Long-Term ϕ
①	SILT	115 pcf	500	0	0	22°
②	SILTY LOAM	115 pcf	750	0	0	25°
③	FRIABLE LIMESTONE	135 pcf	0	40°	0	40°
④	LIMESTONE	140 pcf	0	50°	0	50°
⑤	Engineered FILL.	120 pcf	1000	0	0	32°

Long-term Slope Stability

Case 1: General condition which new fill is placed on top of existing grade.

Long Term Global Stability FS = 1.1 < 1.3 required
(BLA 3)

Case 2: 3' Undercut below the new box culvert and new embankment fill area. Long Term Global Slope Stability FS = 1.2 < 1.3 required.
(BLA 4)

Case 3: 3' undercut below the new box culvert + 4 layers of Geogrid (BX 1100) extending 20'~30' behind the embankment slope facing creek. The geogrid elev. ranged between 510, 512, 514, and 516.

Long Term Global Slope Stability F.S. = 1.4 > 1.3 required (OK)

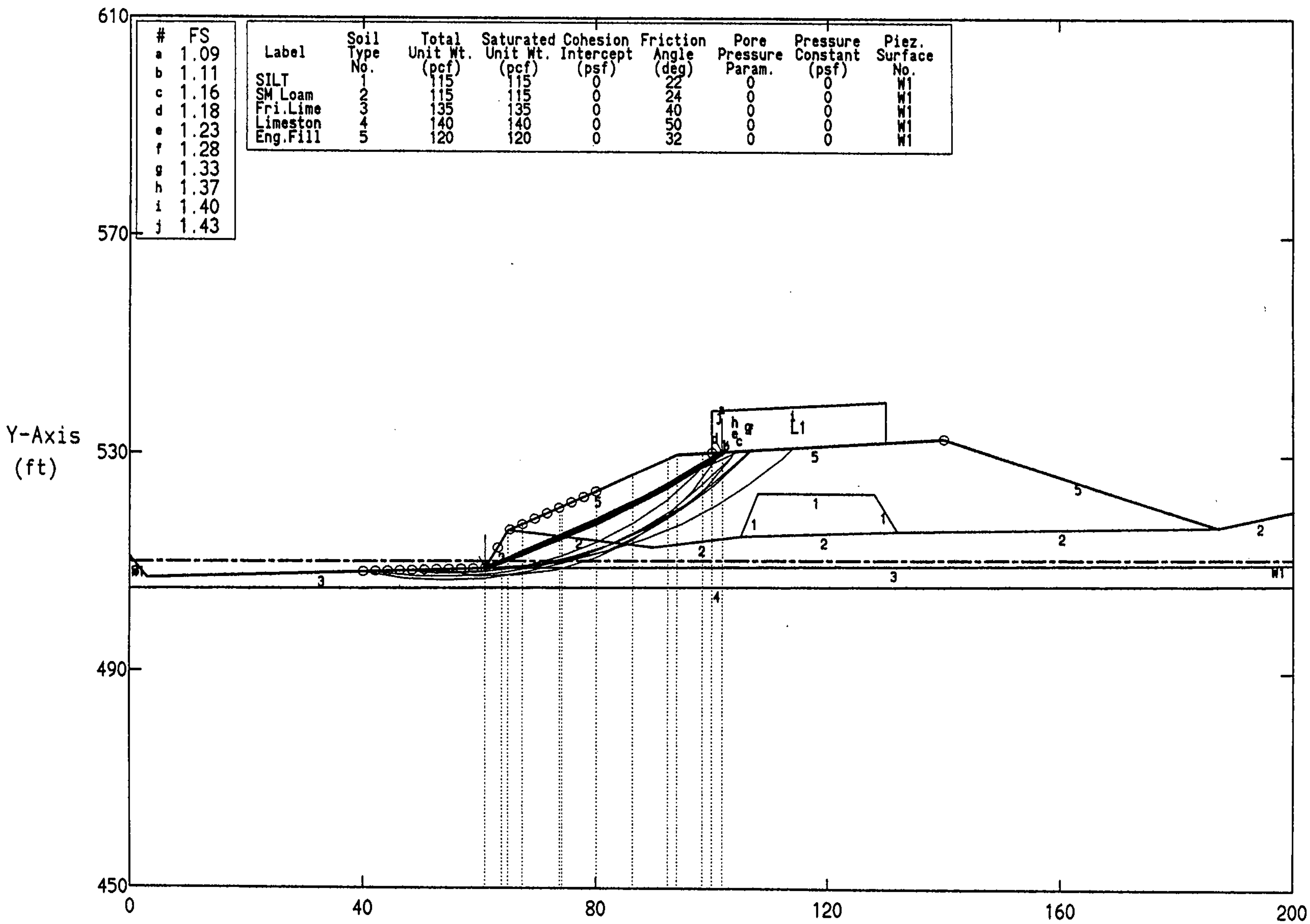
Short Term Global Slope Stability F.S. = 1.5 > 1.3 required (OK)

BLA-CR375W Embankment Fill, Orange Co. 91+64, Without Culvert-Long Term

Ten Most Critical. C:BLA3.PLT By: PC 8/30/2004 9:55am

#	FS
a	1.09
b	1.11
c	1.16
d	1.18
e	1.23
f	1.28
g	1.33
h	1.37
i	1.40
j	1.43

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SILT	1	115	115	0	22	0	0	W1
SM Loam	2	115	115	0	24	0	0	W1
Fri.Lime	3	135	135	0	40	0	0	W1
Limeston	4	140	140	0	50	0	0	W1
Eng.Fill	5	120	120	0	32	0	0	W1



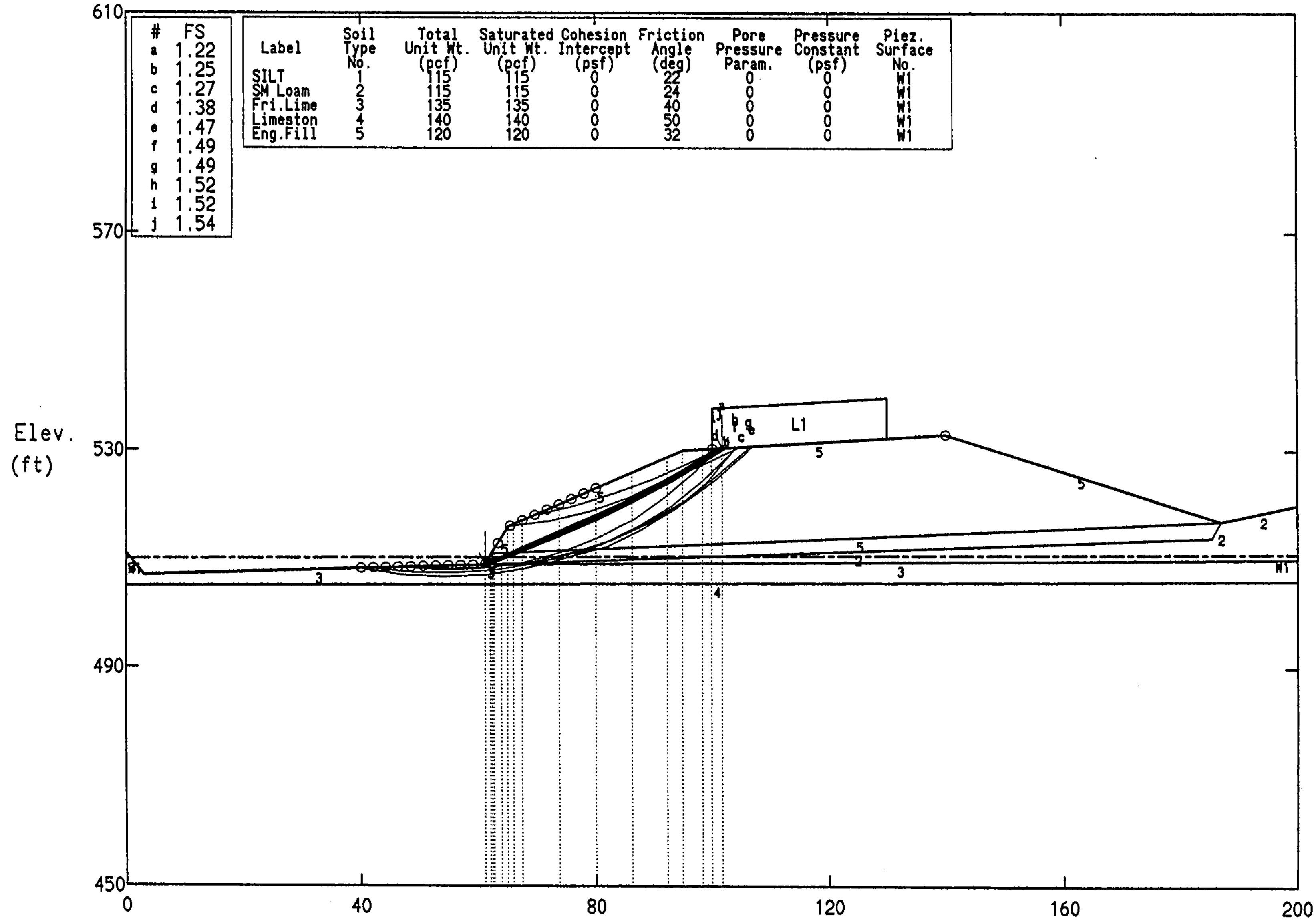
PCSTABL5M/SI FSmin=1.09 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

7

BLA-CR375W Embankment Fill, Orange Co. 91+64, With 3'Undercut&Culvert Long Term

Ten Most Critical. C:BLA4.PLT By: PC 8/30/2004 10:56am



#	FS
a	1.22
b	1.25
c	1.27
d	1.38
e	1.47
f	1.49
g	1.49
h	1.52
i	1.52
j	1.54

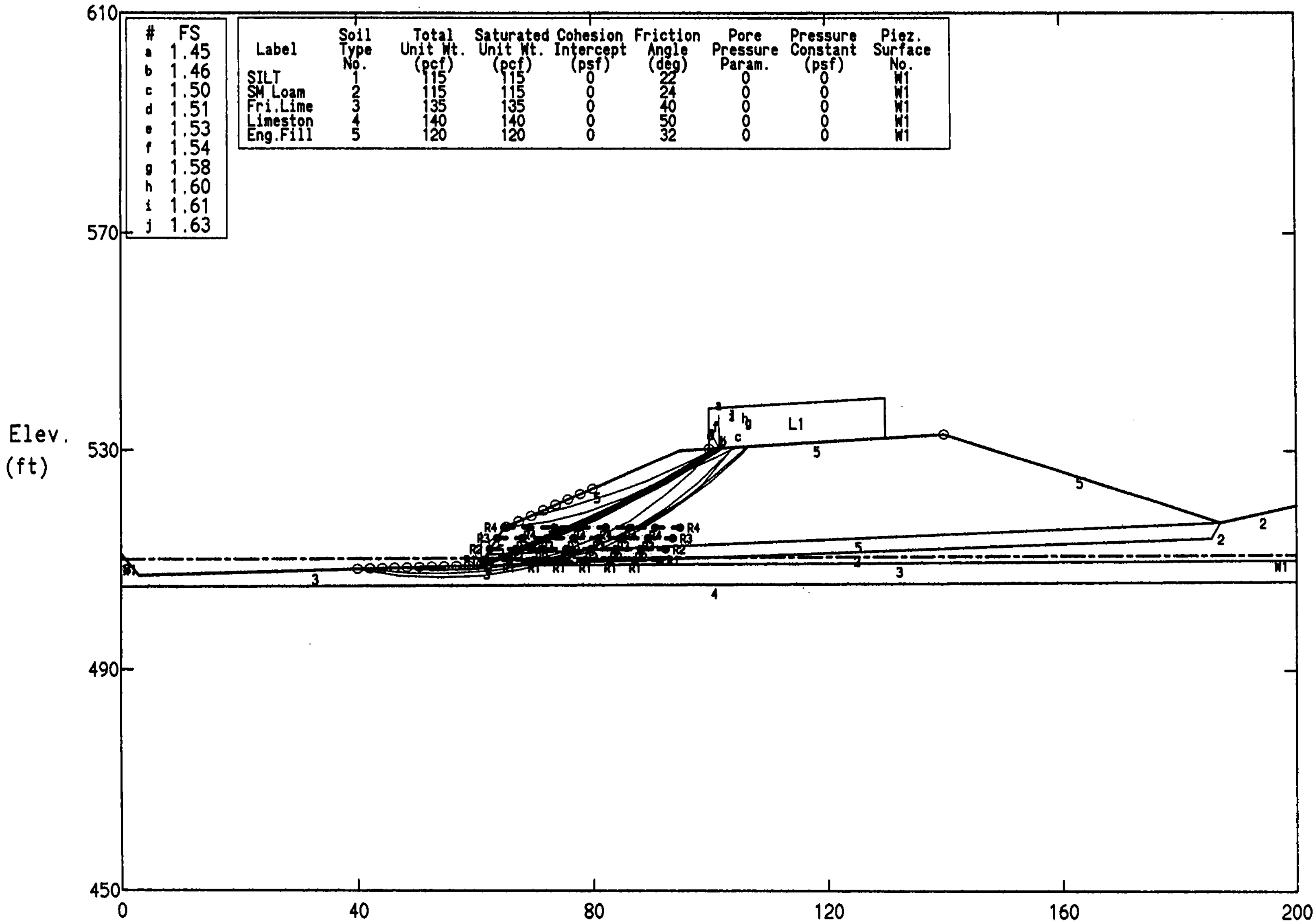
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SILT	1	115	115	0	22	0	0	W1
SM Loam	2	115	115	0	24	0	0	W1
Fri.Lime	3	135	135	0	40	0	0	W1
Limeston	4	140	140	0	50	0	0	W1
Eng.Fill	5	120	120	0	32	0	0	W1

PCSTABL5M/SI FSmin=1.22 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

BLA-CR375W Embankment Fill, Orange Co. 91+64, 4 layer Geogrid BX, Long Term

Ten Most Critical. C:BLA5.PLT By: PC 8/30/2004 2:27pm



#	FS
a	1.45
b	1.46
c	1.50
d	1.51
e	1.53
f	1.54
g	1.58
h	1.60
i	1.61
j	1.63

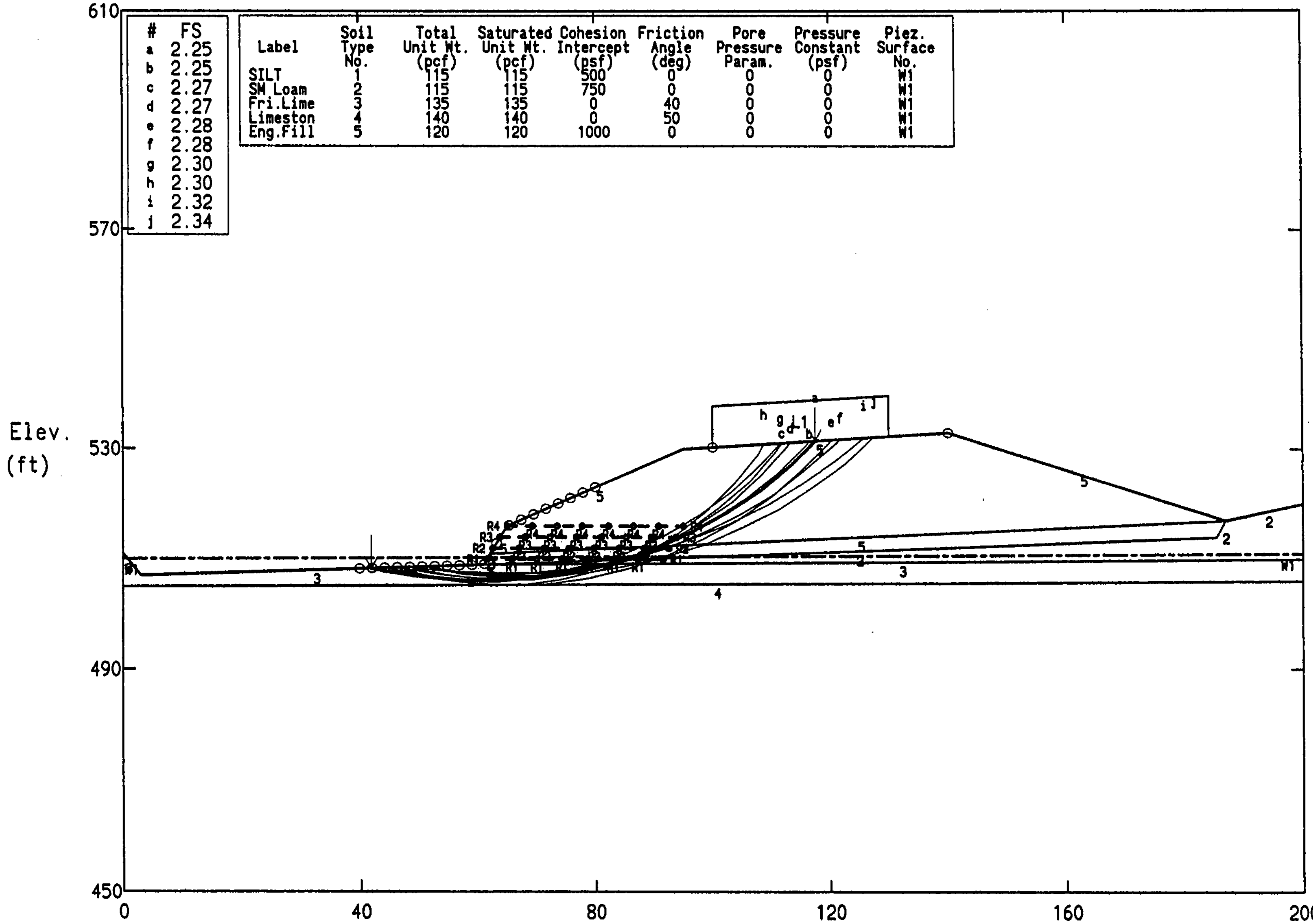
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SILT	1	115	115	0	22	0	0	W1
SM Loam	2	115	115	0	24	0	0	W1
Fri.Lime	3	135	135	0	40	0	0	W1
Limeston	4	140	140	0	50	0	0	W1
Eng.Fill	5	120	120	0	32	0	0	W1

STABL6H FSmin=1.45 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

BLA-CR375W Embankment Fill, Orange Co. 91+64, 4 layer Geogrid BX, Short Term

Ten Most Critical. C:BLA6.PLT 3/18/2005 2:52pm



STABL6H FSmin=2.25 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

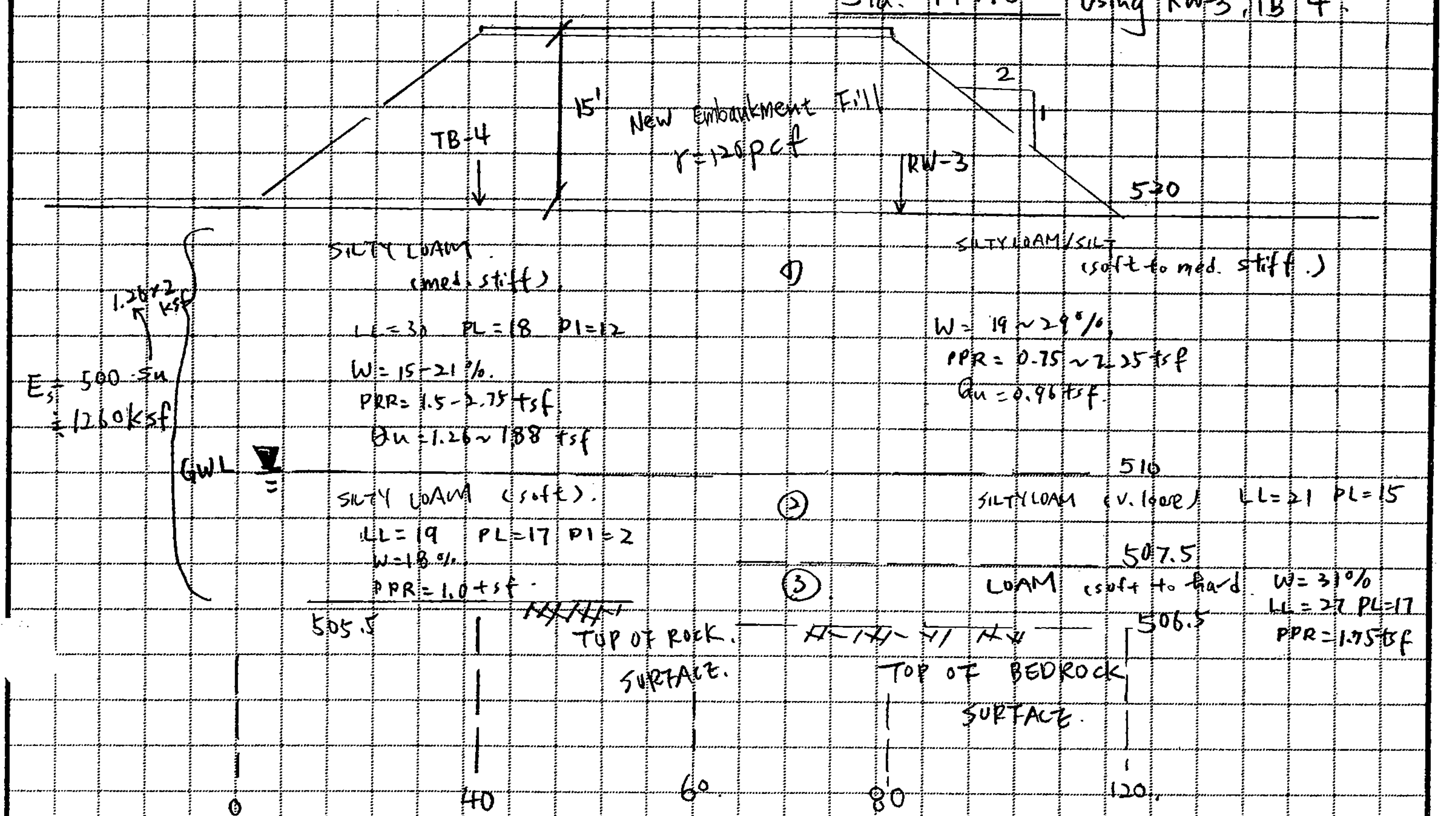
H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

JOB: Proposed 3-Span Bridge Replacement
SHEET NO. 1 OF 5 BRO-9959()
CALCULATED BY: PC DATE: 8/28/04
CHECKED BY: DATE:
SCALE: Sta. 94+70, Line "B"

alysis 4:

Embankment settlement Analyses:

Sta. 94+70 Using RW-3, TB-4.



Layer 1: $\gamma = 115$ pcf.

$CR = 0.022378 + 0.0061715\, \overset{\text{use }25\%}{W} + 0.0012949\, \overset{18\%}{PL} - 0.0010568\, \overset{30\%}{LL} = 0.168$

$RR = 0.001598 + 0.0007160\, \underset{25\%}{W} + 0.0004613\, \underset{18\%}{PL} = 0.037$

$P_c = \dfrac{960}{0.1 + 0.004\, \underset{12}{PI}} \doteqdot 6400$ psf or $Cr = 0.00194(\overset{12}{I_p} - 4.6) = 0.014$ 0.020

Layer 2: $\gamma = 115$ pcf

$CR = 0.022378 + 0.0061715\, \overset{18}{W} + 0.0012949\, \overset{17}{PL} - 0.0010568\, \overset{19}{LL} = 0.135$

$RR = 0.001598 + 0.0007160\, \underset{18}{W} + 0.0004613\, \underset{17}{PL} = 0.022$ (immediate Settlemen

$P_c = \dfrac{1000}{0.1 + 0.004\, \underset{2}{PI}} \doteqdot 9200$ psf

Layer 3: $\gamma = 125$ pcf

$CR = 0.02378 + 0.0061715\, \overset{31}{W} + 0.0012949\, \overset{17}{PL} - 0.0010568\, \overset{27}{LL} = 0.207$

$RR = 0.001598 + 0.000\,7160\, \underset{31}{W} + 0.0004613\, \underset{17}{PL} = 0.031$

$P_c = \dfrac{1756}{0.1 + 0.004\, \underset{10}{PI}} = 12500$ psf. $Cr = 0.00194(\overset{10}{I_p} - 4.6) = 0.016$ 0.01

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

STRIP SYMMETRICAL VERTICAL EMBANKMENT LOADING

Project Name	: BRO-9959 () CR375W	Client	: BLA
File Name	: 9470B	Project Manager	: SS
Date	: 8/29/04	Computed by	: PC

Settlement for X-Direction

Embankment slope a	= 40.00 (ft)	Height of fill H	= 20.00 (ft)
Embankment top width	= 40.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment bottom width	= 120.00 (ft)	p load/unit area	= 2400.00 (psf)
Ground Surface Elev.	= 520.00 (ft)	Foundation Elev.	= 520.00 (ft)
Water table Elev.	= 510.00 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COMP.	RECOMP. RATIO	SWELL.	UNIT WEIGHT (pcf)
1	COMP.	10.0	0.168	0.020	0.020	115.00
2	COMP.	2.5	0.135	0.022	0.022	115.00
3	COMP.	1.0	0.207	0.020	0.020	125.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	2.00	519.00	200.00	6040.00
2	2.00	517.00	345.00	6120.00
3	2.00	515.00	575.00	6200.00
4	2.00	513.00	805.00	6280.00
5	2.00	511.00	1035.00	6360.00
6	1.25	509.38	1182.88	8025.00
7	1.25	508.13	1248.63	9175.00
8	1.00	507.00	1312.80	11400.00

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 20.00 Stress (psf)	Sett. (in.)	X = 40.00 Stress (psf)	Sett. (in.)	X = 60.00 Stress (psf)	Sett. (in.)
1	19.09	0.02	1200.00	0.41	2380.90	0.53	2399.97	0.53
2	57.17	0.03	1199.97	0.31	2342.73	0.43	2399.25	0.43
3	94.93	0.03	1199.86	0.23	2304.63	0.34	2396.60	0.34
4	132.16	0.03	1199.62	0.19	2266.65	0.28	2391.02	0.29
5	168.67	0.03	1199.19	0.16	2228.83	0.24	2381.81	0.25
6	197.69	0.02	1198.68	0.10	2198.26	0.15	2371.38	0.16
7	219.56	0.02	1198.16	0.10	2174.85	0.14	2361.53	0.15
8	238.87	0.02	1197.61	0.07	2153.87	0.10	2351.31	0.11
		------		------		------		------
		0.21		1.57		2.21		2.26

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

Settlement Analysis of Spread Footings on Cohesive Soils

Title: BLA CR 375W over Lick Creek

Footing Shape: Continuous
(Press ALT-S to change)

Units of Measurement: English
(Press ALT-U to change)

Stress Distribution: Boussinesq
(Press ALT-D to change)

Footing Width	=	40.00	ft
Footing Depth	=	1.00	ft
Applied Load	=	96	k/ft
Soil Modulus	=	1260000	lb/ft2
Groundwater Depth	=	10.00	ft
3-D Adjus. Coeff.	=	1.000	

RESULTS

SETTLEMENT

Distortion	2.29	in
Consolidation	1.64	in
Total	3.92	in

BEARING PRESSURE

Gross	q =	2515	lb/ft2
Net	q' =	2400	lb/ft2

Date: **********
Time: 02:43 PM

Layer Depth (ft) Top	Layer Depth (ft) Bottom	Unit Weight (lb/ft3)	Compressibility C	Initial Eff. Stress at Midpoint (lb/ft2)	Change in Eff. Stress at Midpoint (lb/ft2)	Consol Settlement (ft)	Strain (%)
0.00	1.00	115.0	0.020	58	0	0.000	0.00
1.00	1.50	115.0	0.020	144	2400	0.011	2.12
1.50	2.00	115.0	0.020	201	2400	0.009	1.89
2.00	2.50	115.0	0.020	259	2400	0.009	1.72
2.50	3.00	115.0	0.020	316	2399	0.008	1.59
3.00	3.50	115.0	0.020	374	2399	0.007	1.48
3.50	4.00	115.0	0.020	431	2397	0.007	1.39
4.00	4.50	115.0	0.020	489	2396	0.007	1.31
4.50	5.00	115.0	0.020	546	2394	0.006	1.24
5.00	5.50	115.0	0.020	604	2391	0.006	1.18
5.50	6.00	115.0	0.020	661	2387	0.006	1.13
6.00	6.50	115.0	0.020	719	2383	0.005	1.08
6.50	7.00	115.0	0.020	776	2378	0.005	1.04
7.00	8.00	115.0	0.020	863	2369	0.010	0.98
8.00	9.00	115.0	0.020	978	2354	0.009	0.91
9.00	10.00	115.0	0.020	1093	2336	0.008	0.84
10.00	11.00	115.0	0.020	1176	2315	0.008	0.80

Layer Depth (ft) Top	Layer Depth (ft) Bottom	Unit Weight (lb/ft3)	Compressibility C	Initial Eff. Stress at Midpoint (lb/ft2)	Change in Eff. Stress at Midpoint (lb/ft2)	Consol Settlement (ft)	Strain (%)
11.00	12.00	125.0	0.020	1234	2290	0.008	0.77
12.00	13.00	125.0	0.020	1297	2263	0.007	0.75
13.00	14.00	140.0	0.000	1367	2233	0.000	0.00
14.00	15.00	140.0	0.000	1444	2201	0.000	0.00
15.00	16.00	140.0	0.000	1522	2168	0.000	0.00
16.00	17.00	140.0	0.000	1599	2132	0.000	0.00
17.00	18.00	140.0	0.000	1677	2096	0.000	0.00
18.00	19.00	140.0	0.000	1755	2059	0.000	0.00
19.00	20.00	140.0	0.000	1832	2021	0.000	0.00
20.00	21.00	140.0	0.000	1910	1983	0.000	0.00
21.00	22.00	140.0	0.000	1987	1945	0.000	0.00
22.00	23.00	140.0	0.000	2065	1907	0.000	0.00
23.00	24.00	140.0	0.000	2143	1869	0.000	0.00
24.00	25.00	140.0	0.000	2220	1831	0.000	0.00
25.00	26.00	140.0	0.000	2298	1794	0.000	0.00
26.00	27.00	140.0	0.000	2375	1758	0.000	0.00
27.00	28.00	140.0	0.000	2453	1722	0.000	0.00
28.00	29.00	140.0	0.000	2531	1688	0.000	0.00
29.00	30.00	140.0	0.000	2608	1653	0.000	0.00
30.00	31.00	140.0	0.000	2686	1620	0.000	0.00
31.00	32.00	140.0	0.000	2763	1587	0.000	0.00
32.00	33.00	140.0	0.000	2841	1556	0.000	0.00
33.00	34.00	140.0	0.000	2919	1525	0.000	0.00
34.00	36.00	140.0	0.000	3035	1480	0.000	0.00
36.00	38.00	140.0	0.000	3190	1423	0.000	0.00
38.00	40.00	140.0	0.000	3345	1370	0.000	0.00
40.00	42.00	140.0	0.000	3501	1320	0.000	0.00
42.00	44.00	140.0	0.000	3656	1272	0.000	0.00
44.00	46.00	140.0	0.000	3811	1227	0.000	0.00
46.00	48.00	140.0	0.000	3966	1185	0.000	0.00
48.00	50.00	140.0	0.000	4121	1146	0.000	0.00
50.00	52.00	140.0	0.000	4277	1108	0.000	0.00
52.00	54.00	140.0	0.000	4432	1073	0.000	0.00
54.00	56.00	140.0	0.000	4587	1040	0.000	0.00

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

JOB 3-Span Bridge Replacement.
SHEET NO. 5 OF 5
CALCULATED BY PC DATE 8/28/04
CHECKED BY DATE
SCALE Sta. 94+70. "B"

* Based on our settlement analyses, we anticipate that approximate $1\frac{3}{4}$" to 2" consolidation settlement may occur at the centerline. In addition, approximate 1"~2" immediate settlement (elastic) is anticipated during 15' Fill placement.

** Time rate of consolidation settlement

V. soft to soft silty loam C_v = 150 - 350 ft^2/year (Based on U.S. NAVY (1971))

LL = 30 $C_v \doteqdot 7\times10^{-3}$ cm^2/s $\doteqdot$ 240 ft^2/year → use 250 ft^2/year.

Due to the presence of sand/silt seams throughout the 12.5' zone, we assumed double (two-way) drainage paths in the analyses.

① 50% of SILTY LOAM settlement (about 1")

$$t = \frac{0.197 \times \left(\frac{12.5}{2}\right)^2}{250} = 0.030 \text{ year} = 11 \text{ days} \quad (2 \text{ weeks})$$

② 90% of settlement (about $1\frac{3}{4}$" settlement).

$$t = \frac{0.848 \left(\frac{12.5}{2}\right)^2}{250} = 0.132 \text{ year} = 48 \text{ day} \quad (6 \text{ weeks}).$$

③ 95% of settlement (about 2" settlement)

$$t = \frac{1.163 \left(\frac{12.5}{2}\right)^2}{250} = 0.181 \text{ year} = 66 \text{ days} \quad (9 \text{ weeks})$$

* We recommend that a min. of 4 weeks waiting period after the completion of FILL PLACEMENT prior to any pavement construction in this area (Sta. 94+70).

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

Analysis 5:

Embankment Slope Stability Analyses:

Sta 94+70, Line "B"
Using TB-4 RW-3

Soil Parameters used

		Short Term			Long Term	
		γ	c	ϕ	c	ϕ
①.	SILTY LOAM (cohesive) ML	115 pcf	750 psf	0	0	24°
②	SILTY LOAM SM (granular)	115 pcf	500 psf	0	0	22°
③.	LOAM C(CL)	125 pcf	1750 psf	0	100 psf	27°
④.	Friable bedrock	135 pcf	0	40°	0	40°
⑤	Engineered Fill	120 pcf	1000	0	0	32°

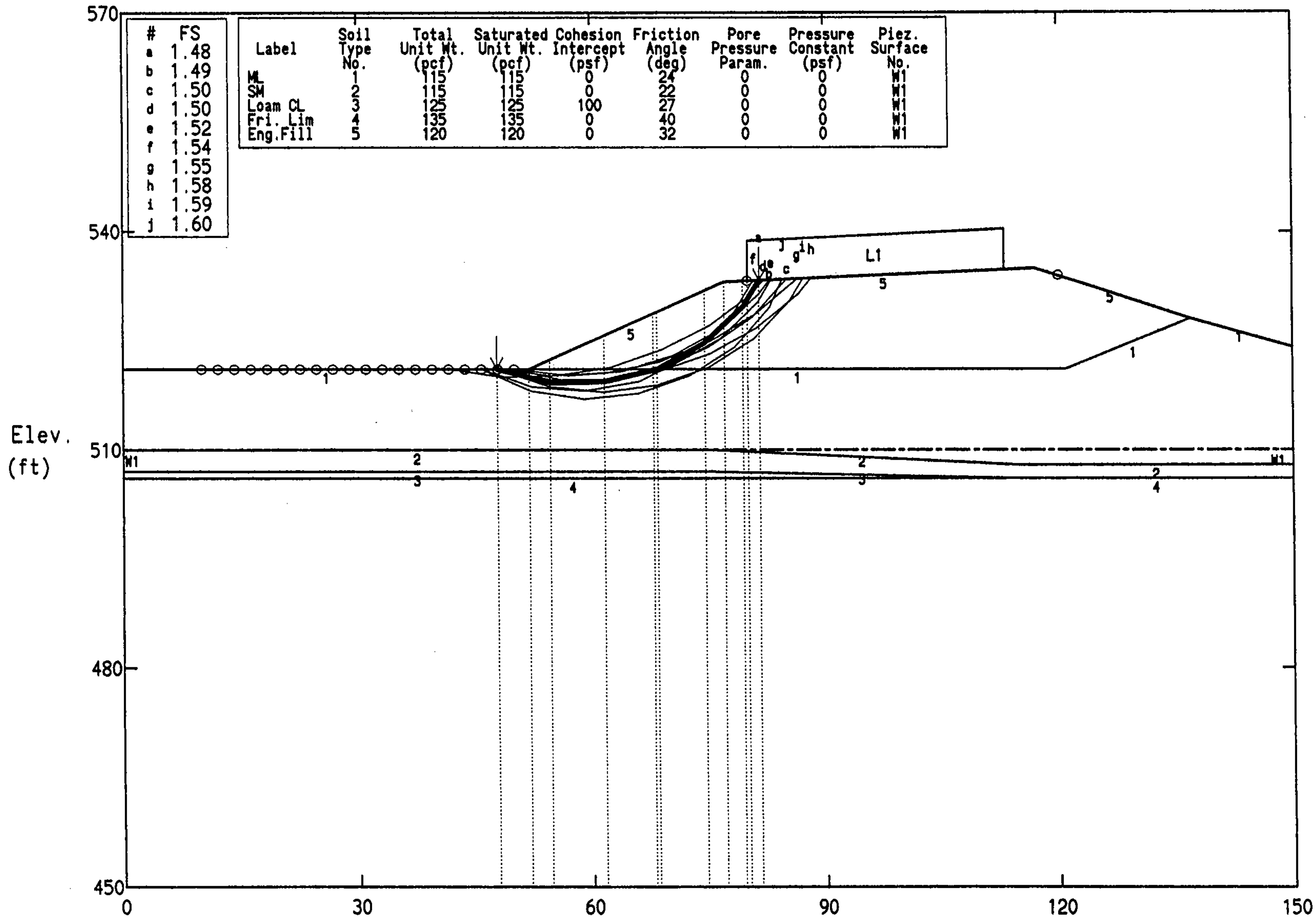
* Long-term global stability analyses: FOS = 1.4 > 1.3 required (OK).

** Short-term global stability analyses: FOS = 2.0 > 1.3 required (OK)

*** Based on the embankment slope analyses, only undercut and replacement at the soft zones revealed during construction is required near Sta. 94+70. Settlement monitoring program should be performed prior to any new pavement construction.

BLA-CR375W Embankment Fill, Orange Co. 94+70, B Embankment Slope, Long Term

Ten Most Critical. C:BLA7.PLT By: PC 8/30/2004 3:56pm



PCSTABL5M/SI FSmin=1.48 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

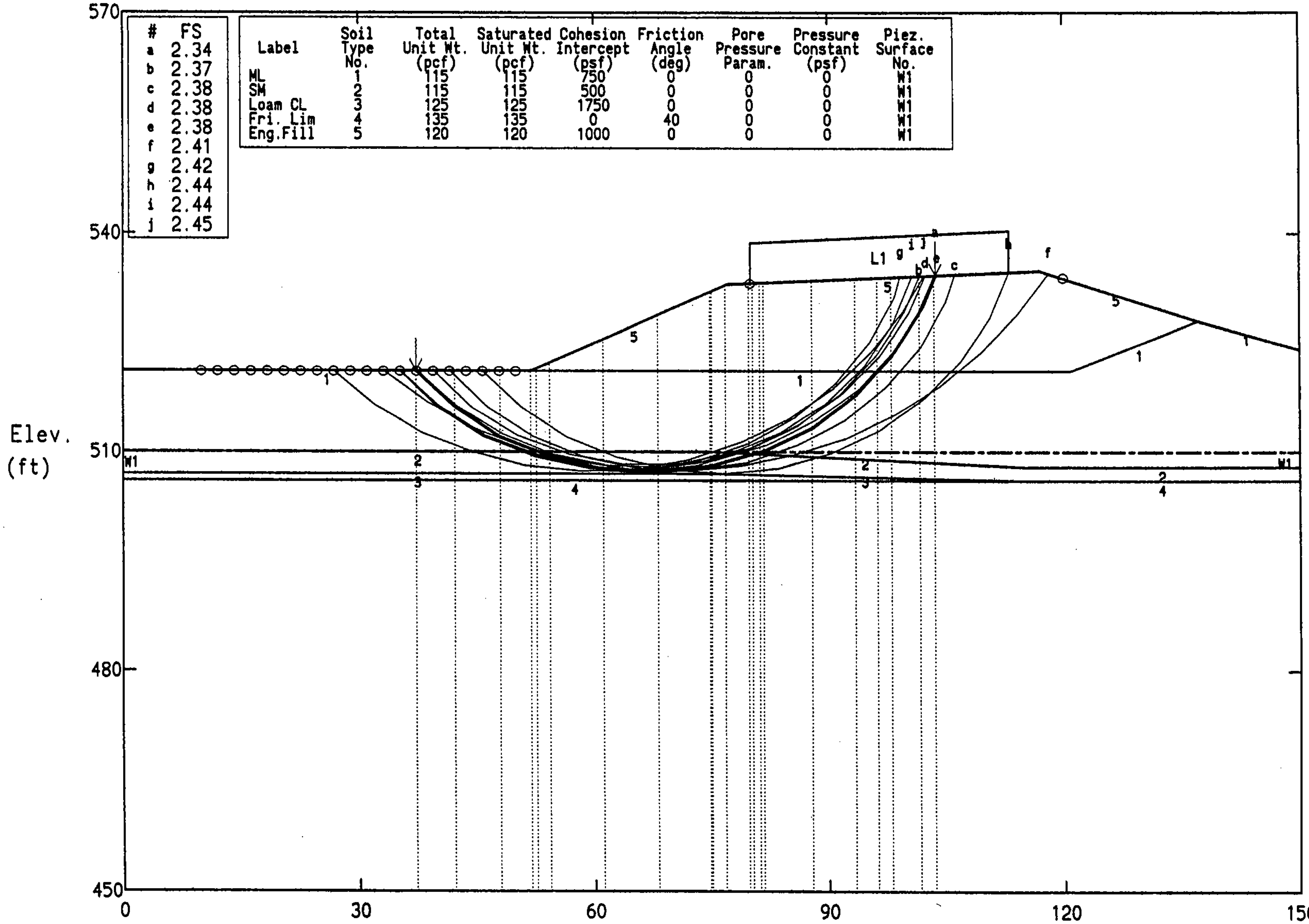
2

BLA-CR375W Embankment Fill, Orange Co. 94+70, B Embankment Slope, Short Term

Ten Most Critical. C:BLA8.PLT 3/18/2005 2:53pm

#	FS
a	2.34
b	2.37
c	2.38
d	2.38
e	2.38
f	2.41
g	2.42
h	2.44
i	2.44
j	2.45

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
ML	1	115	115	750	0	0	0	W1
SM	2	115	115	500	0	0	0	W1
Loam CL	3	125	125	1750	0	0	0	W1
Fri. Lim	4	135	135	0	40	0	0	W1
Eng.Fill	5	120	120	1000	0	0	0	W1



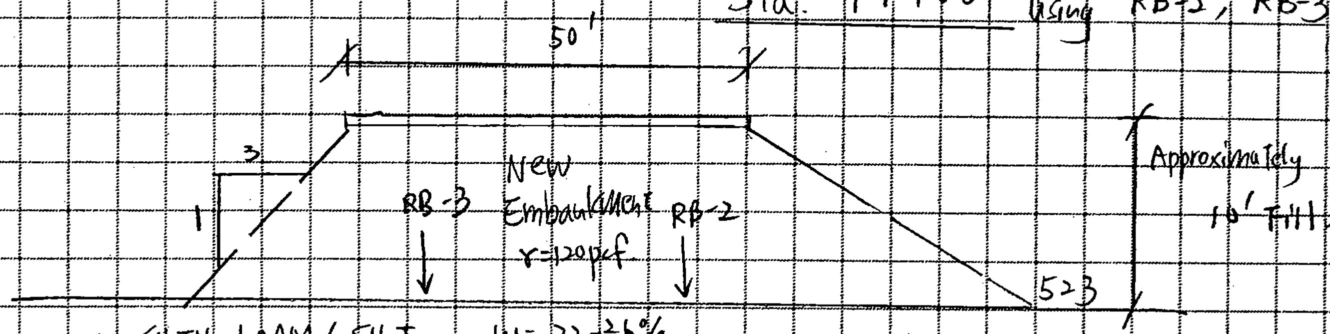
PCSTABL5M/SI FSmin=2.34 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

Analysis 6:

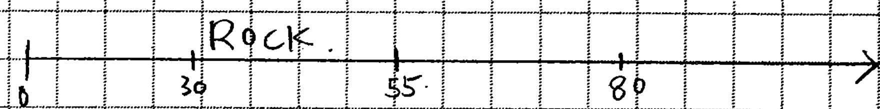
Embankment Settlement Analyses: Sta. 97+00 using RB-2, RB-3



① SILTY LOAM / SILT. W = 22-26%. LL = 28 PL = 20 PI = 8. PPR = 1.75 tsf. 518

② CLAY (limestone fragments) W = 22-30% LL = 49 PL = 23 PI = 26 PPR = 2.25 tsf Qu = 1.78 tsf 513

Relatively Firm Soils.



Layer 1: $\gamma = 115$ pcf (overconsolidated soils).

$RR = 0.001598 + 0.0007160\,\underset{22\%}{W} + 0.0004613\,\underset{20\%}{PL} = 0.026$ } USE 0.020

$Cr = 0.00194(8 - 4.6) = 0.007.$ (same as Sta 94+70).

$P_c = \frac{1750}{0.1 + 0.004 \times 8} \doteq 13600$ psf

Layer 2: $\gamma = 120$ pcf (overconsolidated soils)

$RR = 0.001598 + 0.0007160\,\underset{25\%}{W} + 0.0004613\,\underset{23\%}{PL} = 0.030$ ✓ (use)

$Cr = 0.00194(26 - 4.6) = 0.041$ $P_c = \frac{1780}{0.1 + 0.004 \times 26} \doteq 8700$ psf

B.K. Hough Chart (F ASCE) $RR = \frac{1}{32-38} = 0.026 \sim 0.031$ (32-38 → compressibility Index.)

Average $N = \frac{9+15+7+22}{4} = 14$ bpf.

* Based on EMBANK settlement analyses:

We anticipate that approximate 1.5" of consolidation settlement may occur.

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

STRIP SYMMETRICAL VERTICAL EMBANKMENT LOADING

Project Name	: BRO-9959 () CR375W	Client	: BLA
File Name	: 9700B	Project Manager	: SS
Date	: 8/29/04	Computed by	: PC

Settlement for X-Direction

Embankment slope a	= 30.00 (ft)	Height of fill H	= 10.00 (ft)
Embankment top width	= 50.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment bottom width	= 110.00 (ft)	p load/unit area	= 1200.00 (psf)
Ground Surface Elev.	= 523.00 (ft)	Foundation Elev.	= 523.00 (ft)
Water table Elev.	= 510.00 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COMP.	RECOMP. RATIO	SWELL.	UNIT WEIGHT (pcf)
1	COMP.	5.0	0.020	0.020	0.020	115.00
2	COMP.	5.0	0.030	0.030	0.030	120.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	2.50	521.75	200.00	12462.50
2	2.50	519.25	431.25	11387.50
3	2.50	516.75	725.00	10312.50
4	2.50	514.25	1025.00	9237.50

	X =	0.00	X =	30.00	X =	60.00
Layer	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)	Stress (psf)	Sett. (in.)
1	15.91	0.02	1184.09	0.50	1199.98	0.51
2	47.48	0.03	1152.45	0.34	1199.36	0.35
3	78.38	0.04	1121.30	0.37	1197.13	0.38
4	108.19	0.04	1090.92	0.28	1192.50	0.30
		------		------		------
		0.13		1.49		1.54

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

H.C. NUTTING CO.
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816

JOB 3-Span Bridge Replacement.
SHEET NO. 3 OF 3
CALCULATED BY PC DATE 8/29/04
CHECKED BY DATE
SCALE Sta. 97+00, Line "B"

** Time rate of consolidation settlement.

Based on the encountered 5' silty loam over 5' clay (A-7-6) strata, we anticipate the lower 5' clay zone will control the consolidation ~~settlement~~ waiting period.

med. stiff to v. stiff clay (with limestone fragments)

$C_v = 100 - 150 \ ft^2/year$

After U.S. NAVY 1971, LL = 49%. $C_v = 3 \times 10^{-3} \ cm^2/s = 103 \ ft^2/year$

We assumed a single drainage path (upper boundary) in the analyses. Use $120 \ ft^2/year$.

① 50% of clay consolidation settlement = $(\frac{3}{4}'' \sim 1'')$

$$t = \frac{0.197 \times (\frac{5}{1})^2}{120} = 0.041 \text{ year} = 14 \text{ days } (2 \text{ weeks})$$

② 90% of consolidation settlement = $(1\frac{1}{4}'')$

$$t = \frac{0.848 \, (\frac{5}{1})^2}{120} = 0.17 \text{ year} = 64 \text{ days} = (8 \sim 9 \text{ weeks})$$

*** We recommend that a min. of 2 weeks waiting period after the completion of Embankment FILL PLACEMENT prior to any new pavement construction near Sta. 97+00.

INDIANA DEPARTMENT OF TRANSPORTATION (INDOT)

SPECIAL PROVISION OF MICROPILE FOUNDATIONS

FOR

INDOT PROJECT NO. BRO-9959(), DES. NO. 9982490

ORANGE COUNTY BRIDGE NO. 34

ORANGEVILLE TOWNSHIP, ORANGE COUNTY, INDIANA

1. DESCRIPTION

This Work is for the furnishing of all materials, products, accessories, tools, equipment, services, transportation, labor and supervision, and manufacturing techniques required for testing and installing drilled and grouted micropiles and pile top attachments for this project.

The micropile contractor shall select installation means and methods and shall install a system of micropiles as shown on the Drawings. The Engineer has already established the diameters, bond zone length, locations, and required load capacities and tolerable deflections of the micropiles. The micropile load capacities and deflections shall be verified by testing as specified herein.

In selecting the micropile installation means and methods, the micropile contractor is advised that the casing must extend through the overburden soil and into the limestone bedrock to provide full-length side support. The top of competent rock is estimated to be from 1 foot to 4 feet below the limestone surface and will be determined in the field by the Engineer. The bond zone is the pile length to be in competent rock shown on the drawings. The bottom of pile elevations shown on the Drawings is for estimating purposes only. The actual installed pile tip elevation shall be determined in the field to provide the minimum bond length of 10 feet or to the design length, whichever is greater, in competent limestone bedrock, based on the actual top of competent bedrock surface encountered.

Production pile installation, except reaction piles and load test piles, shall not begin prior to conducting the load tests.

The Engineer or Consulting Engineer hired by Owner will monitor and record the installation of all micropiles and will record the load test data and interpret the results of the load tests.

2. QUALIFICATIONS OF MICROPILE PILING CONTRACTOR

The qualified contractor shall have installed micropile foundations with dimensions and capacity similar to those shown on the Drawings for a minimum of five projects in the last five years, involving construction totaling at least 100 micropiles prior to the bid date for this project.

The Contractor shall have previous micropile drilling and grouting experience in soil/rock similar to project conditions. The Contractor shall submit construction details, structural details and load test results for at least three previous successful micropile load tests from different projects of similar scope to this project. The on-site foremen and drill rig operators shall also have experience on at least three projects over the past five years installing micropiles of equal or greater capacity than required in these plans and specifications. The Contractor must also provide resumes of key personnel who will be present on site and who will each have at least three years of relevant experience.

3. DEFINITIONS

Admixture: Substance added to the grout to control bleed and/or shrinkage, improve flowability, reduce water content, or retard setting time.
Alignment Load (AL): A minimum initial load (5 percent of DL maximum) applied to micropile during testing to keep the test equipment correctly positioned.
Bonded Length: The length of the micropile that is bonded to the rock and conceptually used to transfer the applied axial load to the surrounding rock. Also known as the load transfer length.

Bond-breaker: A sleeve placed over the steel reinforcement to prevent load transfer.

Casing: Steel tube introduced during the drilling process in overburden soil to temporarily stabilize the drillhole. This is usually withdrawn as the pile is grouted, although in certain types of micropiles, some casing is permanently left in place to provide added pile reinforcement.

Centralizer: A device to support and position the reinforcing steel in the drillhole and/or casing so that a minimum grout cover is provided.

Contractor: The person/firm responsible for performing the micropile work.

Coupler: The means by which load capacity can be transmitted from one partial length of reinforcement to another.

Creep Movement: The movement that occurs during the creep test of a micropile under a constant load.

Design Load (DL): The maximum un-factored load expected to be applied to the micropile during its service life.

Encapsulation: A corrugated or deformed tube protecting the reinforcing steel against corrosion.

Engineer: The Owner or Owner's authorized agent.

Micropile: A small-diameter, bored, cast-in-place composite pile, in which the applied load is resisted by steel reinforcement, cement grout and frictional grout/ground bond.

Maximum Test Load: The maximum load to which the micropile is subjected during testing. Recommended as 2.5xDL for verification load tests and as 1.67xDL for proof load tests.

Overburden: Material, natural or placed, that may require cased drilling methods to provide an open borehole to underlying strata.

Post-grouting: The injection of additional grout into the load transfer length of a micropile after the primary grout has set. Also known as regrouting or secondary grouting.

Primary Grout: Portland-cement-based grout injected into the micropile hole prior to or after the installation of the reinforcement to direct the load transfer to the surrounding ground along the micropile.

Proof Load Test: Incremental loading of a production micropile and recording the total movement at each increment.

Reinforcement: The reinforcing steel bar component of the micropile that accept and/or resist applied loading.

Sheathing: Smooth or corrugated piping or tubing that protects the reinforcing steel against corrosion.

Spacer: A device to separate elements of multiple-element reinforcement.

Verification Load Test: Pile load test performed to verify the design of the pile system and the construction methods proposed, prior to installation of production piles.

4. AVAILABLE INFORMATION

Available information developed by the Owner, or by the Owner's duly authorized representative, include the following items:

1. Plans and Micropile Design Drawings prepared by the designer. The plans include the plan view, profile and typical cross sections for the proposed micropile locations. The estimated top of bond zone elevation is shown on the Micropile Design Drawings. The actual top of bond zone elevation is expected to differ from the estimate by possibly 4 feet or more. The governing criterion for installation is the minimum bond zone length as shown on the Micropile Design Drawings.
2. Geotechnical Report INDOT Project No. BRO-9959(), Des. No. 9982490 and Str: Orange No. 34 (BLA Project No. 199-0047-0BD), dated September 7, 2004, included and referenced in the bid documents, contains the results of test borings and other site investigation data obtained in the vicinity of the proposed micropile locations.

5. CONSTRUCTION SITE SURVEY

Before bidding the Work, the Contractor shall review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, and location of existing structures and above ground facilities.

The Contractor is responsible for field locating and verifying the location of all utilities shown on the plans prior to starting the Work. Maintain uninterrupted service for those utilities designated to remain in service throughout the Work. Notify the Engineer of any utility locations different from shown on the plans that may require micropile relocations or structure design modification. Subject to the Engineer's approval, additional cost to the Contractor due to micropile relocations resulting from utility locations different from shown on the plans, will be paid as Extra Work.

Prior to start of any micropile construction activity, the Contractor and Engineer shall jointly inspect the site to observe and document the pre-construction condition of the site, existing structures and facilities.

6. SUBMITTALS AND MEETING

6.1-Pre-Construction Submittals: At least 45 calendar days before the planned start of micropile construction, the Contractor shall submit five copies of the completed project reference list and a personnel list. The project reference list shall include a brief project description with the Owner's name and current phone number and load test reports. The personnel list shall identify the project supervisor, drill rig operators, and on-site foremen to be assigned to the project. The personnel list shall contain a summary of each individual's experience and be complete enough for the Engineer to determine whether each individual satisfies the required qualifications. The Engineer will approve or reject the Contractor's qualifications within 15 calendar days after receipts of a complete submission. Additional time required due to incomplete or unacceptable submittals will not be cause for time extension or impact or delay items. All costs associated with incomplete or unacceptable submittals shall be borne by the Contractor.

The Contractor shall submit a general description of his proposed schedule, construction methods and equipment prior to production pile installation. Work shall not begin until the appropriate submittals have been received, reviewed, and accepted by the Engineer.

The submittal(s) shall include:

1. Proposed start date and the time schedule, detailed description of the drilling methods, construction procedure and sequence, personnel, testing and equipment to assure quality control to allow the Engineer to monitor the construction and quality of the micropiles;
2. If welding of casing is proposed, submit the proposed welding procedure, and certification of a qualified welding specialist;
3. Information on headroom and space requirements for installation equipment that verify the proposed equipment can perform at the site;
4. Plan describing how surface water, drill flush, and excess waste grout will be controlled and disposed;
5. Certified mill test reports for the reinforcing steel or coupon test results for permanent casing without mill certification. The ultimate strength, yield strength, elongation, and material properties composition shall be included. For API N-80 pipe casing, coupon test results may be submitted in lieu of mill certification;
6. Proposed grouting plan. The grouting plan shall include complete description, details, and supporting calculations for the following:
   (a) Grout mix design and type of materials to be used in the grout, including certified test data and trial batch reports;
   (b) Methods and equipment for accurately monitoring and recording the grout length, grout volume and grout pressure as the grout is being placed;
   (c) Grouting rate calculations, when requested by the Engineer. The calculations shall be based on the initial pump pressures or static head on the grout and losses throughout the placing system, including anticipated head of drilling fluid (if applicable) to be displaced;
   (d) Estimated curing time for grout to achieve specified strength. Previous test results for the proposed grout mix completed within one year of the start of grouting may be submitted for initial verification and acceptance and start of production work. During

production, grout shall be tested in accordance with the "grout testing";

(e) Procedure and equipment for Contractor monitoring of grout quality.

7. Detailed plans for the method proposed for the testing of the micropiles prior to beginning the tests. This shall include all necessary drawings, structural design calculations and details to clearly describe the method, such as reaction load system capacity and equipment setup, types and accuracy of apparatus to be used for applying and measuring the test loads and pile top movement in accordance with the "Pile Load Tests" section of this special provision;

8. Calibration reports and data for each test jack, pressure gauge and master pressure gauge and load cell to be used. The calibration tests shall be have been performed by an independent testing laboratory, and tests shall have been performed within 90 calendar days of the date submitted. Testing shall not commence until the Engineer has reviewed and accepted the jack, pressure gauge, master pressure gauge and electronic load cell calibration data.

Work other than test pile installation shall not begin until the construction submittals have been received, reviewed and accepted in writing by the Engineer. Provide submittal items 1 through 4 at least 21 calendar days prior to initiating micropile construction, item 6 as the work progresses for each delivery and submittal items 5, 7 and 8 at least 7 days prior to start of micropile load testing or incorporation of the respective materials into the work. The Contractor shall allow the Engineer 7 calendar days to review the construction submittals after a complete set has been received. Additional time required due to incomplete or unacceptable submittals shall not be cause for delay or impact claims. All costs associated with incomplete or unacceptable Contractor submittals shall be the responsibility of the Contractor.

6.2-Pre-construction Meeting: A pre-construction meeting will be scheduled by the Engineer and held prior to the start of micropile construction. The Engineer, prime Contractor, micropile specialty Contractor, micropile designer, excavation

Contractor and geotechnical instrumentation specialist shall attend the meeting. Attendance is mandatory. The pre-construction meeting will be conducted to clarify the construction requirements for the Work, to coordinate the construction schedule and activities, and to identify contractual relationships and delineation of responsibilities amongst the prime Contractor and various Subcontractors – specially those pertaining to excavation for micropile structures, anticipated subsurface conditions, micropile installation and testing, micropile structure survey control and site drainage control.

6.3-Installation Records: The Engineer will prepare an installation record for each micropile. The Contractor shall assist the Engineer, as required, to obtain installation data. The records will include the following minimum information:

1. Pile Identification
2. Pile drilling start and finish times
3. Existing ground surface elevation
4. Top of Rock Elevation
5. Bottom of Tremie Concrete Elevation
6. Material Type below Tremie Concrete
7. Final tip elevation
8. Cut-off elevation
9. Description of unusual installation behavior or conditions
10. Grout pressures attained, if applicable
11. Grout quantities pumped, including start and finish times
12. Grout compression test results
13. Pile materials and dimensions
14. Reinforcing steel sizes and lengths
15. Characteristics of all materials encountered during the drilling process, and their specific location(s) within the holes
16. The location of special features such as mud seams, open cracks, broken rock, etc.
17. Points where abnormal loss or gain to drill water has occurred
18. Groundwater levels or other items of interest for grouting

19. All significant actions of the bit
20. If any weak material, such as coal, clay, weathered rock or the like is encountered within the required bond length, the hole shall be extended to compensate for the weak material.

7. MATERIALS

7.1-Water: Water for mixing grout will be potable and free from substances which may be in any way deleterious to grout or steel and from a non-variable source and shall be in accordance with AASHTO T26.

7.2-Admixtures: Admixtures shall conform to the requirements of ASTM C494/AASHTO M194. Admixtures, which control bleed, improve flowability, reduce water content and retard set may be used in the grout subject to the review and acceptance of the Engineer. Accelerators will not be permitted. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations. Their use will only be permitted after appropriate field tests on fluid and set grout properties. Expansive admixtures shall only be added to the grout used for filling sealed encapsulations and anchorage covers. Admixtures containing chlorides are not permitted.

7.3-Cement: All cement shall be Portland cement conforming to Subsection 701.1, ASTM C150 Type II and shall be the product of one manufacturer.

7.4-Fine Aggregate: Fine Aggregate shall meet the requirements of ASTM C144/AASHTO M45.

7.5-Grout: Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 2,500 psi and a 28-day compressive strength of 5,000 psi per AASHTO T106/ASTM C109.

7.6-Grout Protection: Provide a minimum 1 inch grout cover bare for the or epoxy coated bars (excluding bar couplers) or minimum 0.5" grout cover over the encapsulation of encapsulated bars.

7.7-Reinforcing Bars: All reinforcing steel shall be deformed bars in accordance with ASTM A615/AASHTO M31, Grade 60 or Grade 75 or ASTM A722/AASHTO M275, Grade 150. When a bearing plate and nut are required to be threaded onto the top end of reinforcing bars for the pile top to footing anchorage, the threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g., Dywidag or Williams continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, the next larger bar number designation from the shown on the Drawings shall be provided, at no additional cost. Bar tendon coupler, if required, shall develop the ultimate tensile strength of the bars without failure.

7.8-Pipe/Casing: Casings shall be steel, smooth, clean, watertight, and of ample strength to withstand both handling and driving stresses and the pressure of both concrete and the surrounding earth materials. The outside diameter of casing shall not be less than the specified size of micropiles. No extra compensation will be allowed for concrete required to fill an oversized casing or oversized excavation. All temporary casings shall be removed from micropile installation. Any length of permanent casing installed below the micropile cutoff elevation shall remain in place.

When the micropile extends through a body of water, the portion through a body of water may be formed with a removable casing. Removable casing shall be stripped from the pier in a manner that will not damage the concrete. Casings can be removed when the concrete has attained sufficient strength provided: curing of the concrete is continued for the full 72 hours period in accordance with specification; the pile grout is not exposed to salt water or moving water for 7 days; and the grout reaches a compressive strength of at least 2500 psi as determined from grout cube breaks.

Permanent steel casing/pipe drill casing shall be of the flush joint type and shall have the diameter and at least minimum wall thickness shown on the Drawings. The permanent steel casing/pipe:

1. shall meet the requirements of N-80 - API Specification with minimum yield strength of 80,000 psi.
2. may be new “Structural Grade” (a.k.a. “Mill Secondary”) steel pipe meeting above but without Mill Certification, free from defects (dents, cracks, tears) and with two coupon tests per truckload delivered to the fabricator.

For permanent casing/pipe that will be welded, the following material conditions apply:

1. the carbon equivalency (CE) as defined in AWS D1.1, Section X15.1, shall not exceed 0.45, as demonstrated by mill certifications.
2. The sulfur content shall not exceed 0.05%, as demonstrated by mill certifications.

For permanent casing/pipe that will be shop or field welded, the following fabrication or construction conditions apply:

1. the steel pipe shall not be joined by welded lap splicing;
2. welded seams and splices shall be complete penetration welds;
3. partial penetration welds may be restored in conformance with AWS D1.1;
4. the proposed welding procedure certified by a welding specialist shall be submitted for approval.

Threaded casing joints shall develop at least the required nominal resistance used in the design of the micropile.

7.9-Centralizers and Spacers: Centralizers and spacers shall be fabricated from schedule 40 PVC pipe or tube, steel, or material that is non-detrimental to the reinforcing steel. Wood shall not be used. Centralizers and spacers shall be securely attached to the reinforcement; sized to position the reinforcement within 10 mm of plan location from center of pile; sized to allow grout tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole and casing and between adjacent reinforcing bars.

7.10-Encapsulation: Encapsulation (double corrosion protection) shall be shop fabricated using high-density, corrugated polyethylene tubing conforming to the requirements of ASTM D3350/AASHTO M252 with a nominal wall thickness of 0.8 mm. The inside annulus between the reinforcing bars and the encapsulating tube shall be a minimum of 5 mm and be fully grouted with non-shrink grout.

7.11-Epoxy Coating: The permanent steel casing and reinforcing bars should be epoxy coated. The minimum thickness of coating applied electrostatically to the reinforcing steel shall be 0.3 mm. Epoxy coating shall be in accordance with ASTM A775/AASHTO N282 or ASTM A934. Bend test requirements are waived.

Bearing and nuts encased in the pile concrete footing need not be epoxy coated unless the footing reinforcement is epoxy coated.

7.12-Plates and Shapes: Structural steel plates and shapes for pile tops attachments shall conform to ASTM A36/AASHTO M183, or ASTM A572/AASHTO M223, Grade 350.

7.13-Sheathing: Smooth plastic sheathing, including joints, shall be watertight. Polyvinyl chloride (PVC) sheathing shall conform to ASTM D1784, Class 13464-B.

8. CONSTRUCTION METHODS AND EQUIPMENT

8.1-General: The Engineer has designed the micropiles. The estimated top of bond zone elevation is shown on the Drawings. The actual top of bond zone elevation is expected to differ from the estimate by possibly 4 feet or more. The governing criterion for installation is the minimum bond zone length as shown on the drawings. Ground water levels are anticipated to be at the level of the river.

8.2-Site Drainage Control: The Contractor shall control and properly dispose of drill flush and construction related waste, including excess grout, in accordance with the standard specifications and all applicable local codes and regulations. Provide positive control and discharge of all surface water that will affect construction of the micropile installation. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water at no additional cost. Upon substantial completion of the Work, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

Immediately contact the Engineer if unanticipated existing subsurface drainage structures are discovered during excavation or drilling. Suspend work in these areas until remedial measures meeting the Engineer's approval are implemented. Cost of remedial measures or repair work resulting from encountering unanticipated subsurface drainage structures, will be paid for as Extra Work.

8.3-Excavation: Coordinate the work and the excavation so that the micropile structures are safely constructed. Perform the micropile construction and related excavation in accordance with the Plans, Design Drawings and approved submittals. No excavations steeper than those specified in the Design Drawings herein or shown on the Plans will be made above or below the micropile structure locations without written approval of the Engineer.

8.4-Protection of Existing Utilities: The Contractor shall control his operations to prevent damage to existing overhead and underground utilities. Preventive measures shall include, but not limited to, selecting construction methods and procedures that will prevent caving of the micropile boreholes.

8.5-Allowable Tolerances: Centerline of piling shall not be more than 3 inches from indicated plan position. Pile alignment shall be within 2% of design alignment. Top elevation of pile shall be within +/- 1" of the design vertical elevation. Centerline of reinforcing steel shall not be more than 0.5" from indicated location. Micropiles not constructed within the required tolerances are unacceptable. The Contractor shall be responsible for correcting all unacceptable micropile installation to the satisfaction of the Engineer. Materials and work necessary, including engineering analyses and redesigns, to complete corrections for out of tolerance micropile installations shall be furnished without either cost to the Owner or an extension of the completion dates of the project.

8.6-Installation: The micropile contractor shall select the drilling method, the grouting procedure, and the grouting pressure used for the installation of the micropiles. The procedures shall not damage adjacent facilities or newly installed piles.

<u>8.6.1 Drilling:</u> The drilling equipment and methods shall be suitable for drilling through the conditions to be encountered, without causing damage to any overlying or adjacent structures or services. The drilling equipment and methods shall provide an open borehole to the defined nominal diameter and full length, as shown on Drawings, prior to placing grout and reinforcement. Temporary casing or other approved method of pile drillhole support will be required in caving or unstable ground to permit the pile shaft to be formed to be the minimum design drillhole diameter. The Contractor's proposed method(s) to provide drillhole support and to prevent detrimental ground movements shall be reviewed by the Engineer. Detrimental ground movement is defined as movement that requires remedial repair measures. Water or polymer drilling mud (for flushing the hole during drilling) can be used to provide temporary side support and facilitate rock coring during pile installation. Bentonite slurry may impair grout/ground bond capacity and shall not be used. Costs of removal or remedial measures due to encountering unanticipated subsurface obstructions will be paid at the contract unit price.

8.6.2 Ground Heave or Subsidence: During construction, the Contractor shall observe the conditions in the vicinity of the micropile construction site on a daily basis for signs of ground heave or subsidence. Immediately notify the Engineer if signs of movements are observed. Contractor shall immediately suspend or modify drilling or grouting operations if ground heave or subsidence is observed, if the micropile is adversely affected, or if adjacent structures are damaged from the drilling or grouting. If the Engineer determines that the movements require corrective action, the Contractor shall take corrective actions necessary to stop movement or perform repairs. When due to the Contractor's methods or operations or failure to follow the specified/approved construction sequence, as determined by the Engineer, the costs of providing corrective actions will be borne by the Contractor.

8.6.3 Pipe Casing and Reinforcing Bar Placement and Splicing: Reinforcement may be placed either prior to grouting or placed into the grout-filled drillhole before temporary casing (if used) is withdrawn. Reinforcement surface shall be free of deleterious substances such as soil, mud, grease or oil that might contaminate the grout or coat the reinforcement and impair bond. Pile casing and reinforcement groups, if used, shall be sufficiently robust to withstand the installation and grouting process and the withdrawal of the drill casings without damage or disturbance.

The Contractor shall check pile top elevations and adjust all installed micropiles to the planned elevations. Centralizers (spaced not to exceed 10 ft) shall be provided on central reinforcement. The uppermost centralizer shall be located 3 ft maximum from the top of the central reinforcement. Centralizers shall permit the free flow of grout without causing misalignment of the reinforcement.

The central reinforcement steel with centralizers shall be lowered, not dropped, into the stabilized drillholes to the desired depth. The reinforcing steel shall be inserted into the drillholes to the desired depth without difficulty. Partially inserted reinforcing bars shall not be driven or forced into the hole.

Due to relatively short pile lengths expected, the use of pile splices are not anticipated. Pile splices are not permitted unless authorized by the Engineer. If used, pile splices shall be constructed to develop the full strength of the pile section and shall be secured in proper alignment and in a manner to avoid eccentricity or kink angle between the axes of the two lengths spliced. Splices and threaded joints shall meet the requirements of the "Material Section". Threaded pipe casing joints shall be located at least two casing diameters (OD) from a splice in any reinforcing bar. Proposed pile splice detail shall be submitted for review with the submittals.

8.6.4 Grouting: Micropiles shall be primary grouted the same day the load transfer bond length is drilled. The Contractor shall use a stable neat cement grout or a sand cement grout with a minimum 28-day unconfined compressive strength of 5,000 psi. Admixtures, if used, shall be mixed in accordance with manufacturer's recommendations. The grouting equipment used shall produce a grout free of lumps and undispersed cement. The Contractor shall have verifiable means and methods of measuring the grout quality, quantity and pumping pressure during the grouting operations. Expansion additives in grout will not be allowed. Grout shall not be re-tempered or used after it has begun to set. Quality control and testing of grout shall conform to the applicable sections of this special provision.

The grout pump shall be equipped with a pressure gauge to monitor grout pressures. A second pressure gauge shall be placed at the point of injection into the pile top. The pressure gauges shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used, whichever is greater. The grout shall be kept in agitation prior to mixing. Grout shall be placed within one hour of mixing. The grouting equipment shall be sized to enable each pile to be grouted in one continuous operation. The grout shall be injected from the lowest point of the drillhole and injection shall continue until uncontaminated grout flows from the top of the pile. The grout may be pumped through grout tubes, casing, hollow-stem augers, or drill rods. Temporary casing, if used, shall be extracted in stages ensuring that, after each length of casing is removed the grout level is brought back up to the ground level before next length is removed. The tremie pipe or casing shall always extend below the level of the existing grout in the drillhole. The grout pressures and grout takes shall be controlled to prevent excessive heave or fracturing of rock or soil formations. Upon completion of grouting, the grout tube may remain in the hole, but must be filled with a 5,000-psi minimum compressive strength grout without voids from bottom to top of micropile. The entire bond zone shall be completely filled with grout.

8.6.5 Grout Testing: Grout within the micropiles shall attain a minimum compressive strength of 4,000 psi prior to load testing. Previous test results for the proposed grout mix completed within one year of the start of work may be submitted for initial verification of the required compressive strengths for installation of pre-production verification test piles and initial production piles. The Contractor shall make 2 sets of 6 two-inch cubes for each day of grouting (one set near the beginning of the day and one set near the end of the day) or for every 10 piles, whichever occurs more frequently. The Contractor shall test two cubes after 7 days cure, two cubes after 28 days cure, and keep two in reserve. Cubes shall be cured and tested according to ASTM C 109.

Grout consistency as measured by grout density shall be determined by the Contractor per ASTM C188/AASHTO T133 or API RP-13B-1 at a frequency of at least one test per pile, conducted just prior to start of pile grouting. The Baroid Mud Balance used in accordance with API RP-13B-1 is an approved device for determining the grout density of neat cement grout. Grout samples shall be taken directly from the grout plant. Provide grout cube compressive strength and grout density test results to the Engineer within 24 hours of testing.

8.6.6 Pile Damage: If a micropile is deemed unacceptable by the Engineer due to improper or inadequate construction or to damage caused by the Contractor, that micropile shall be load tested and/or replaced in a manner acceptable to the Engineer. Load testing and replacement shall be at the Contractor's expense. Any modification, which requires changes to the structure, shall have prior review by and the acceptance of the Engineer.

8.6.7 Obstructions: Surface and subsurface obstructions at drilled pile locations shall be removed by the Contractor. Such obstructions may include man-made materials such as old concrete foundations and natural materials such as boulders. Special procedures and/or tools shall be employed by the Contractor after the hole cannot be advanced using conventional earth or rock augers, fitted with soil or rock teeth, drilling bucket and/or underreaming tools. Such special procedures/tools may include but are not limited to: chisels, boulder breakers, core barrels, air tools, hand excavation, temporary casing, and increasing the hole diameter in the overburden soils. Blasting shall not be permitted. Obstruction removal shall be paid separately.

8.6.8 Lost Tools: Drilling tools that are lost in the excavation shall not be considered obstructions and shall be promptly removed by the Contractor without compensation. All costs due to lost tool removal shall be borne by the Contractor including but not limited to, costs associated with hole degradation due to removal operations or the time the hole remains open.

8.7-Micropile Installation Records: The contractor shall prepare and submit to the Engineer full-length installation records for each micropile installed. The records shall be submitted within one work shift after that pile installation is completed. The data shall be recorded on the micropile installation log included at the end of this special provision. A separate log shall be provided for each micropile.

9. PILE LOAD TESTS:

Perform verification and proof testing of piles at the locations specified herein or designated by the Engineer. Pile load tests shall be performed according to ASTM D 1143, except as modified herein. One verification compressive test shall be performed for prior to the commencement of production pile installation. 5% of the total number of production piles installed shall be subjected to the proof testing.

9.1-Verification Load Test: Verification load tests shall be performed to verify that the Contractor installed micropiles will meet the required compression load capacities and to verify that the length of the micropile load transfer bond zone is adequate. The test piles with reaction/support piles or anchors shall be constructed prior to the commencement of the installation of the remaining production micropiles. The test piles shall be loaded in accordance with the sequence indicated below to the maximum Test Load (TL) with no horizontal load. Test verification piles designated for compression load testing to a maximum test load of 2.5 times the micropile Design Load shown on the Plans or Design Drawings.

The test piles shall not be production piles located as shown on the Contract Plans. The test piles shall be installed using the same procedures as the production piles. The maximum verification and proof test loads applied to the micropile shall not exceed 80% of the structural capacity of the micropile structural elements, to include steel yield or buckling in compression, or grout crushing in compression. The micropile load test results will be reviewed and accepted by the Engineer prior to installing the remaining production micropiles.

The load testing program submittal shall be furnished to the Engineer prior to the start of load testing. This submittal shall provide the following information as a minimum:

1) Sketch of the Load Test Set-up
2) Type and Accuracy of apparatus for measuring load
3) Type and Accuracy of apparatus for applying load
4) Type and Accuracy of apparatus for measuring the pile deformation
5) Type and capacity of reaction load system
6) Hydraulic jack and load cell calibration reports (both are required).

The drilling and grouting methods, casing diameter and depth of bond zone of the test pile shall be identical to the production piles. The jack shall be positioned at the beginning of the test such that the unloading and repositioning of the jack during the test will not be required. Axial pile load tests shall be made by loading the micropile in the following steps and recording the head movement at each step:

Load	Hold Time (Minutes)
0	0
12.5% TL	5
25% TL	5
0	5
12.5% TL	1
25% TL	1
37.5% TL	5
50% TL	5
0	5
37.5% TL	1
50% TL	1
62.5% TL	5
75% TL	5
0	5
50% TL	1
62.5% TL	1
75% TL	1
87.5% TL	5
100% TL	300
0	5

Measurement of pile movement shall be obtained at each increment. The load hold period shall start as soon as the test load is applied and the pile movement, with respect to a fixed reference, shall be measured and recorded at 1 minute 2,3,4,5,30, 60, 90, 120, 150, 180, 210, 240, 270, and 300 minutes (load cycle maximum test load only).

The piles shall sustain the compression test load (100% TL) with no more than 2" of total vertical downward movement at the top of the pile. In addition, the butt movement at the design load must be no greater than 1".

For the test, the final rate of butt movement during the load hold period at 100% TL shall not exceed 0.1" per hour, and the creep rate shall be decreasing. In the event that the tests indicate that the compressive capacity is not adequate and the design rock socket length must be increased, the change will be paid for at the contract unit price. Alternately, the Engineer may elect to change the design loads to agree with the results of the tests and add piling to the contract at the contract unit prices. In the event of failing tests, additional tests at the contract unit price per test may be directed by the Engineer.

9.2-Proof Load Test: Perform proof load tests on the first set of production piles installed at each designated substructure unit prior to the installation of the remaining production piles in that unit. The first set of production piles is the number required to provide the required reaction capacity for the proof tested pile. The initial proof test piles shall be installed at the following substructure units 1 through 4. Proof testing shall be conducted at a frequency of 5% of the subsequent production piles installed, beyond the first 20, in each abutment and pier. Location of additional proof test piles shall be as designated by the Engineer.

Test piles designated for compression or tension proof load testing to a maximum test load of 1.67 times the micropile Design Load shown on the Plans or Design Drawings. Proof tests shall be made by incrementally loading the micropile in accordance with the following schedule:

Load	Hold Time (Minutes)
AL	1
25% DL	1
50% DL	1
75% DL	1
100% DL	1
133% DL	10 or 60 minute Creep Test
167% DL	1
AL	1

Measurement of pile movement shall be obtained at each increment. The load hold period shall start as soon as the test load is applied and the pile movement, with respect to a fixed reference, shall be measured and recorded at 1 minute 2,3,4,5,10,30, and 60 minutes (load cycle maximum test load only). Depending on performance, either a 10-minute or 60 minute creep test shall be performed at the 1.33 DL Test load. Where the pile top movement between 1 and 10 minutes exceed 0.04 inch, the Maximum Test Load shall be maintained an additional 50 minutes. Movements shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes. The alignment load (AL) shall not exceed 5% of DL. Dial gauges shall be reset to zero after the initial AL is applied.

The acceptance criteria for micropile proof load tests are:

1. The micropile shall sustain the compression 1.0 DL test load with no more than 0.25 inch total vertical movement at the top of the pile, relative to the position of the pile prior to testing.
2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 0.04 inch/log cycle time (1 to 10 minutes) or 0.08 inch/log cycle time (6 to 60 minutes). The creep rate shall be linear or decreasing throughout the creep load hold period.
3. Failure does not occur at the 2.0 DL maximum test load. Failure is defined as a slope of the load versus deflection (at the end of increment) curve exceeding 0.025 inches/kip.

If a proof-tested micropile fails to meet the acceptance criteria, the Contractor shall immediately proof test another micropile within that footing. For failed piles and further construction of other piles, the Contractor shall modify the construction procedure. These modifications may include installing replacement micropiles, incorporating piles at not more than 50% of the maximum load attained, postgrouting, modifying installation methods, increasing the bond length, and changing the micropile type. Any modification shall require the Engineer's review and acceptance. Any modification of construction procedures, or cost of additional verification test piles and verification and/or proof load testing, or replacement production micropiles, shall be at the Contractor's expense.

10. METHOD OF MEASUREMENT:

The number of linear foot of piles of the type specified to be paid for will be the actual length of piles remaining in the finished structure. All cut-offs remain the property of the Contractor for disposition.

The contractor will not be paid for grout used that is in excess of two times the theoretical amount required to fill an empty core hole from the bottom of the rock socket to the top of the final pile tip elevation.

11. BASIS OF PAYMENT:

The quantities, determined as provided above, will be paid for at the contract unit prices bid for the items listed below, which prices and payments shall be full compensation for furnishing all materials and doing all the work prescribed in a workmanlike and acceptable manner, including all labor, materials, tools, equipment, supplies and incidentals necessary to complete the work. The cost of preboring, filling of voids and splicing, except as noted in this special provision, shall be included in the price bid for the piles.

5.6-PAY ITEMS:

Item 1, "Mobilization and Demobilization"...... ....Lump Sum
Item 2, "Micropile", 8 inch diameter.. per linear foot
Item 3, "Micropile Verification Load Test," ....per pile
Item 4, "Micropile Proof Load Test," ....per pile
Item 5, "Unexpected Obstruction Drilling" .... Hour
Item 6, "Excess Grout"... Hour

END

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

APPALACHIAN REGION, 912 MORRIS STREET, CHARLESTON, WV 25031, (304) 344-0821, FAX (304) 342-4711
CENTRAL OHIO REGION, 790 MORRISON ROAD, COLUMBUS, OH 43230, (614) 863-3113, FAX (614) 863-0475
INDIANA REGION, 349 WALNUT STREET, STE 8, LAWRENCEBURG, IN 47025, (812) 539-4300, FAX (812) 539-4301
BLUEGRASS REGION, 470-B CONWAY CT., STE B-8, LEXINGTON, KY 40511, (859) 455-8530, FAX (859) 455-8630

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-1
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/11/2004
Boring Location	Station 92+70, 20' Rt., Line "B" Orange County, IN	Date Completed	4/11/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)
517.89	0.0		
517.29	0.6	0.6	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)
510.39	7.5	6.9	Brown SILT, moist, very soft to soft, - with frequent silt seams/layers; with noted tree stems A-4, Lab No. 2852
507.89 ▼	10.0	2.5	Brown SILTY LOAM, very moist to wet, very loose, - with frequent wet silt and sand seams A-4, Lab No. 2868
507.39	10.5	0.5	
505.89	12.0	1.5	Brown SANDY LOAM, wet, medium dense, - with frequent wet silt seams and limestone fragments A-4 (0), Lab No. 2873
			Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained
497.89	20.0	8.0	Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close spaced, rough, tight, and moderately dipping joints; occasional steeply dipping joints at 13.5' and 15' to 16', occurs well distributed in 4" to 18" layers;, occasional open joints and fractured rock fragments at 12', 17.5', and 19.5'
495.89	22.0	2.0	Brown and gray SILTSTONE, hard, fine-grained, trace calcareous, moderately close spaced, smooth, tight, and moderately dipping joints; occasional high angle joints at 21'
			BORING COMPLETED @ 22.0'

SAMPLE

NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
1	SS	0.0-1.5 ~~1.0-1.5~~	1-1-2 (3)	100						0.75
2	SS	2.5-4.0	1-1-2 (3)	100		19				0.75
3	SS	5.0-6.5	1-2-2 (4)	100		23				0.5
4	SS	7.5-9.0	WOH	100		24	22	4		
5	SS	10.0-11.2	1-11-50/.3'	100						
1	RC	12.0-17.0		100	50				355 to 689	
2	RC	17.0-22.0		100	52				678	

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490
(1) Cave-in at a depth of 20' following removal of augers
(2) WOH: Weight of Hammer

Water Level Observations

Immediate	DRY	ft.
At Completion	10.0	ft. ▽
After 24 Hrs.	10.0	ft. ▼
Water used in drilling	12.0	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

LOG OF TEST BORING

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25301
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
750 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 8
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT, STE B-4
LEXINGTON, KY 40511
(859) 455-8530
FAX (859) 455-8630

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-2
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/11/2004
Boring Location	Station 93+30, 15' Lt., Line "B" Orange County, IN	Date Completed	4/11/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.		DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)
517.34	0.0		
516.84	0.5	0.5	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)
507.34 ▼	10.0	9.5	Brown SILT, moist, very soft, - with occasional silt seams/layers A-4, Lab No. 2852
506.34	11.0	1.0	Brown and trace gray SILTY LOAM, very moist, very soft,
505.34	12.0	1.0	- with occasional wet silt seams A-4 (6), Lab No. 2877
493.34	24.0	12.0	Gray and trace brown FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints Gray LIMESTONE, very hard, fine-grained, calcareous, moderately close to widely spaced, rough, tight, and moderately dipping joints, occasional steeply dipping joints at 15'~16', 21'~22', and 23'~24', occurs well distributed in 12" to 24" layers,, occasional fractured rock zones (high angle joints) at 15' and 19', but generally well cemented
490.34	27.0	3.0	Brown and gray SILTSTONE, hard, fine-grained, trace calcareous, moderately close spaced, smooth, tight, and moderately dipping joints; occasional high angle joints and fractured rock zone at 25'
			BORING COMPLETED @ 27.0'

SAMPLE NO.	TYPE	DEPTH ft	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PPR tsf
1	SS	0.0-1.5	1-1-1 (2)	67						
2	SS	2.5-4.0	WOH-1	80		22				0.5
3	SS	5.0-6.5	1-1-2 (3)	27		26				0.5
4	SS	7.5-9.0	1-1-1 (2)	33		27				0.75
5	SS	10.0-11.2	WOH-50/.3'	83		31	28	10		0.25
1	RC	12.0-17.0		100	54				921	
2	RC	17.0-22.0		98	60				851	
3	RC	22.0-27.0		100	72					

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

WOH: Weight of Hammer

Water Level Observations

Immediate	DRY	ft.
At Completion	10.0	ft. ▽
After 24 Hrs.	10.0	ft. ▼
Water used in drilling	12.0	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

LOG OF TEST BORING

Client	Bernardin, Lochmueller & Associates, Inc.
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek
Boring Location	Station 94+110, 20' Rt., Line "B" Orange County, IN
Elevation Ref.	Interpolated from the provided Site Plan
Boring No.	TB-3
Date Started	4/9/2004
Date Completed	4/10/2004
Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf
518.19	0.0											
517.49	0.7	0.7 Dark brown silty clay (TOPSOIL), moist, soft (Visual)	1	SS	0.0-1.5 1.0-1.5	2-2-2 (4)	100					
		9.3 Brown SILTY LOAM, moist, soft, - with occasional silt seams/layers; became very moist to wet at 7.5' A-4, Lab No. 2868	2	SS	2.5-4.0	2-2-2 (4)	100		21			
			3	SS	5.0-6.5	2-2-3 (5)	67		21			
			4	SS	7.5-9.0	2-2-2 (4)	53		21			
508.19	10.0											
505.69	12.5	2.5 Brown SILTY CLAY LOAM, very moist, stiff, - with occasional wet silt seams and soft zones A-4, Lab No. 2877	5	SS	10.0-11.5	5-5-7 (12)	100		20			0.28
504.19	14.0	1.5 Brown and gray FRIABLE LIMESTONE AND CLAY SEAMS/LAYERS, - Limestone, moderately hard and fine-grained - Clay, very stiff and moist	6	SS	12.5-13.5	27-39-50/.1'	80					
503.19	15.0	1.0 Gray LIMESTONE, hard to very hard, fine-grained, moderately spaced, rough, tight, and moderately dipping joints	1	RC	14.0-15.1		100	45				784
501.19	17.0	2.0 Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and moderately dipping joints; occurs well distributed in 1" to 3" pieces.	2	RC	15.1-20.1		94	32				560
496.19	22.0	5.0 Gray LIMESTONE, very hard, calcareous, fine-grained, moderately close to widely spaced, rough, and moderately dipping joints; occasional steeply dipping (high angle) joints at 21'-22'; occurs well distributed in 6" to 24" layers	3	RC	20.1-25.1		100	54				
488.09	30.1	8.1 Gray LIMESTONE, very hard, calcareous, slightly crystalline, widely spaced, rough, and moderately dipping joints; well distributed in 12" to 30" layers; interbedded with occasional siltstone seams/layers	4	RC	25.1-30.1		98	88				876
		BORING COMPLETED @ 30.1'										

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

Water Level Observations

Immediate	DRY ft.
At Completion	8.0 ft.
After	24 Hrs. 8.5 ft.
Water used in drilling	14.0 ft.

BF = BACKFILLED NW = NO WATER (Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

LOG OF TEST BORING

APPALACHIAN REGION
912 MORRIS STREET
CHARLESTON, WV 25031
(304) 344-0821
FAX (304) 342-4711

CENTRAL OHIO REGION
750 MORRISON ROAD
COLUMBUS, OH 43230
(614) 863-3113
FAX (614) 863-0475

INDIANA REGION
349 WALNUT STREET, STE 5
LAWRENCEBURG, IN 47025
(812) 539-4300
FAX (812) 539-4301

BLUEGRASS REGION
470-B CONWAY CT. STE
LEXINGTON, KY 4051
(859) 455-8530
FAX (859) 455-8530

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No	TB-4
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 94+70, 15' Lt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	50043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)
518.19	0.0		
517.69	0.5	0.5	Dark brown silty clay (TOPSOIL), moist, medium stiff (Visual)
510.69	7.5	7.0	Brown SILTY LOAM, moist, medium stiff, - with occasional silt seams/layers A-6 (9), Lab No. 2884
508.19	10.0	2.5	Brown and trace gray SILTY LOAM, moist, medium stiff, A-6, Lab No. 2884
505.69 ▽	12.5	2.5	Brown and gray SILTY LOAM, moist to very moist, soft, - with occasional wet silt and sand seams A-4, Lab No. 2868
503.69	14.5	2.0	Gray FRIABLE LIMESTONE AND SILTY LOAM SEAMS/LAYERS, - Limestone, moderately hard and fine-grained - Silty Loam, medium dense and wet
502.69	15.5	1.0	
497.69	20.5	5.0	Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and moderately to steeply dipping (high angle) joints; occurs well distributed in 1" to 3" pieces
492.69	25.5	5.0	Gray LIMESTONE, very hard, calcareous, fine-grained, moderately to widely spaced, rough, and moderately dipping joints; occasional steeply dipping (high angle) joints and fractured rock zones at 19'~20'; occurs distributed in 12"-18" layers
			Gray LIMESTONE, very hard, calcareous, crystalline, moderately to widely spaced, rough, and moderately dipping joints; interbedded with occasional siltstone seams/layers
			BORING COMPLETED @ 25.5'

SAMPLE

NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	P. t
1	SS	0.0-1.5 1.0-1.5	4-4-4 (8)	100		15				2
2	SS	2.5-4.0	2-3-3 (6)	100		21	30	12		2.
3	SS	5.0-6.5	2-4-5 (9)	100		21			1.88	1
4	SS	7.5-9.0	3-4-4 (8)	100		18			1.26	1.
5	SS	10.0-11.5	2-2-2 (4)	100		18	19	2		1.
6	SS	12.5-13.6	24-42-50/.1'	91						
1	RC	14.5-15.5		100	0					
2	RC	15.5-20.5		100	78				742	
3	RC	20.5-25.5		100	66				776	

General Notes

Driller	J. Gilbert
Rig No.	550
Rig Type	ATV
Method	RC/SS
Inspector	

Remarks

Project No. BRO-9959(), Designation No. 9982490

Water Level Observations

Immediate	12.5	ft. ▽
At Completion	10.8	ft. ▼
After 24 Hrs.	10.5	ft. ▼
Water used in drilling	14.5	ft.

BF = BACKFILLED NW = NO WATER
(Measured from ground surface)

H.C. NUTTING COMPANY
CORPORATE CENTER - 611 LUNKEN PARK DRIVE
CINCINNATI, OH 45226 (513) 321-5816
FAX (513) 321-0294
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS SINCE 1921

LOG OF TEST BORING

APPALACHIAN REGION / CENTRAL OHIO REGION / INDIANA REGION / BLUEGRASS REGION

Client	Bernardin, Lochmueller & Associates, Inc.	Boring No.	TB-5
Project	Prop. 3-Span Bridge Replacement CR 375 W over Lick Creek	Date Started	4/10/2004
Boring Location	Station 91+80, 22.5' Rt., Line "B" Orange County, IN	Date Completed	4/10/2004
Elevation Ref.	Interpolated from the provided Site Plan	Work Order No.	52043.009

ELEV. ft.	DEPTH ft.	Thickness	DESCRIPTION OF MATERIALS (color, material description, moisture, stiffness/density/hardness)	NO.	TYPE	DEPTH ft.	BLOWS/6" (N Value)	REC. %	RQD %	W %	LL %	PI %	Qu tsf	PP ts
520.04	0.0													
519.04	1.0	1.0	Dark brown silty clay (TOPSOIL), moist, very soft (Visual)	1	SS	0.0-1.5 / 1.0-1.5	1-1-1 (2)	100		22				0.
515.04	5.0	4.0	Brown SILT, moist, very soft, A-4, Lab No. 2852	2	SS	2.5-4.0	1-1-1 (2)	20		20				
				3	SS	5.0-6.5	2-2-2 (4)	80		21	27	9		1
				4	SS	7.5-9.0	3-2-3 (5)	80		28				0.
507.54	12.5	7.5	Brown SILTY LOAM, moist, soft to stiff, - with occasional silt seams; with occasional limestone fragments/floaters at 10' A-4, Lab No. 2877	5	SS	10.0-11.5	32-7-5 (12)	100		25			0.34	(
506.04	14.0	1.5	Gray FRIABLE LIMESTONE, moderately hard to hard, fine-grained, with occasional clay-filled joints	6	SS	12.5-12.6	50/1	100						
505.04	15.0	1.0	Gray FRACTURED LIMESTONE, hard to very hard, calcareous, fine-grained, very closely spaced, rough, open, and steeply dipping (high angle) joints; occurs distributed in 1" to 3" pieces	1	RC	14.0-19.0		100	56				758	
501.04	19.0	4.0	Gray LIMESTONE, very hard, calcareous, crystalline, fine-grained, moderately close spaced, rough, and moderately dipping joints; occurs well distributed in 6" to 12" layers with occasional fractured, steeply dipping (high angle) joints at 18'-19'											
			BORING COMPLETED @ 19.0'											

General Notes		Remarks	Water Level Observations	
Driller	J. Gilbert	Project No. BRO-9959(), Designation No. 9982490	Immediate	DRY
Rig No.	550		At Completion	10.0
Rig Type	ATV		After 24 Hrs	8.0
Method	RC/SS		Water used in drilling	14.0
Inspector			BF = BACKFILLED NW = NO WATER (Measured from ground surface)	

The H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over Lick Creek
Project:BRO-9959 ( ), DES No. 9982490
Orange County Bridge File No. 34
Orange County, IN
W.O. # 50043.009

TABLE IA: CLASSIFICATION TEST DATA AND ATTERBERG LIMITS

TEST NO.	BORING NO.	SAMPLE NO.	DEPTH		DESCRIPTION	AASHTO	GRL %	CS %	FS %	SILT %	CLAY %	pH +	L.L. %	P.L. %	P.I. %
			Feet	Meters											
STA. 86+00, 10'RT, LINE B															
2848	RB-1	2/SS	2.5 - 4	0.75 - 1.4	Silty Clay	A-7-6(38)	0	0	2	54	44	4.14	56	22	34
STA. 97+00, 10.0'RT, LINE B															
2852	RB-2	2/SS	2.5- - 4	0.75 - 1.4	Silt	A-4(7)	0	0	3	87	10	6.40	28	20	8
STA. 92+25, 23.0'RT, LINE B															
2860	RW-1	3/SS	5 - 6.5	1.5 - 2.0	Loam	A-4(3)	3	0	37	47	13	6.96	25	15	10
STA 92+70, 20.0'LT, LINE B															
2863	RW-2	3/SS	5 - 6.5	1.5 - 2.0	Sandy Loam	A-6(2)	1	3	55	27	14	4.63	29	16	13
STA. 94+70, 25.0'RT, LINE B															
2868	RW-3	5/SS	10 - 11.5	3.0 - 3.5	Silty Loam	A-4(1)	0	0	37	52	11	5.94	21	15	6
STA. 92+70, 20.0'RT, LINE B															
2873	TB-1	5/SS	10 - 11.2	2.5 - 3.4	Sandy Loam	A-4(0)	0	1	58	36	5	7.36	NP	NP	NP
STA. 93+30, 15.0'LT, LINE B															
2877	TB-2	5/SS	10 - 11.2	2.5 - 3.0	Silty Loam	A-4(6)	2	1	18	62	17	7.04	28	18	10
STA. 94+70, 15.0'LT, LINE B															
2884	TB-4	2/SS	2.5 - 4	0.7 - 1.2	Silty Loam	A-6(9)	0	1	16	67	16	4.03	30	18	12

The H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over Lick Creek
Project:BRO-9959 ( ), DES No. 9982490
Orange County Bridge File No. 34
Orange County, IN
W.O. # 50043.009

TABLE IB: ADDITIONAL ATTERBERG LIMITS

TEST NO.	BORING NO.	SAMPLE NO.	DEPTH		DESCRIPTION	AASHTO	GRL %	CS %	FS %	SILT %	CLAY %	pH +	L.L. %	P.L. %	P.I. %
			Feet	Meters											
STA. 86+00, 10'RT, LINE B															
2847	RB-1	1/SS	1 - 1.5	0.3 - 0.4									77	29	48
STA. 97+00, 10.0'RT, LINE B															
2854	RB-2	4/SS	8.5 - 10	2.5 - 3.0									49	23	26
STA 92+70, 20.0'LT, LINE B															
2864	RW-2	4/SS	7.5 - 8.7	2.3 - 2.6									27	15	12
STA. 94+70, 25.0'RT, LINE B															
2869	RW-3	6/SS	12.5 - 13.7	3.8 - 4.2									27	17	10
STA. 92+70, 20.0'RT, LINE B															
2871	TB-1	4/SS	7.5 - 9	2.3 - 2.7									22	18	4
STA. 94+70, 15.0'LT, LINE B															
2887	TB-4	5/SS	10 - 11.5	2.5 - 3.5									19	17	2
STA. 91+80, 22.5'RT, LINE B															
2890	TB-5	3/SS	5 - 6.5	1.5 - 2.0									27	18	9

H. C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over
Lick Creek, Project:BRO-9959 ( ), Des. No. 9982490
Orange Co. Bridge File No. 34, Orange Co., IN
W.O. #50043.009

TABLE II: NATURAL MOISTURE CONTENT DETERMINATION

Test No.	Boring No.	Sample No.	Depth (Feet)	Depth (Meters)	Moisture Content (%)	**Microwave** Moisture Content (%)
STA. 86+00, 10.0'RT, LINE B						
2847	RB-1	1/SS	1-1.5	0.3 0.4	37.4	
2848		2/SS	2.5-4	0.75-1.4	25.8	
2849		3/SS	6-6.5	1.8-2.0	31.4	26.4
STA. 87+00, 10.0'RT, LINE B						
2851	RB-2	1/SS	1-1.5	0.3-0.5	22.4	
2852		2/SS	2.5-4	0.8-1.2	25.6	
2853		3/SS	5-6.5	1.5-2.0	22.2	
2854		4/SS	8.5-10	2.6-3.0	27.9	26.0
STA. 98+50, 10.0'LT, LINE B						
2855	RB-3	1/SS	1-1.5	0.3-0.5	21.8	
2856		2/SS	2.5-4	0.8-1.2	24.3	25.0
2857		3/SS	6-7.5	1.8-2.3	29.8	30.5
STA. 92+25, 23.0'RT, LINE B						
2858	RW-1	1/SS	1-1.5	0.3-0.5	26.2	
2859		2/SS	2.5-4	0.8-1.2	18.3	19.0
2860		3/SS	5-6.5	1.5-2.0	19.9	
STA. 92+70, 20.0'LT, LINE B						
2861	RW-2	1/SS	0-1.5	0-0.5	17.6	
2862		2/SS	2.5-4	0.8-1.2	19.3	18.4
2863		3/SS	5-6.5	1.5-2.0	19.0	
2864		4/SS	7.5-8.7	2.3-2.6	19.3	21.2
STA. 94+70, 25.0'RT, LINE B						
2865	RW-3	2/SS	2.5-4	0.8-1.2	29.3	
2866		3/SS	5-6.5	1.5-2.0	21.6	20.1
2867		4/SS	7.5-9	2.3-2.7	18.5	
2869		6/SS	12.5-13.7	3.8-4.2	31.1	
STA. 92+70, 20.0'RT, LINE B						
2870	TB-1	2/SS	2.5-4	0.8-1.2	19.4	18.7
2871		3/SS	5-6.5	1.5-2.0	23.2	
2872		4/SS	7.5-9	2.3-2.7	24.4	24.0

H. C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Bernardin Lochmueller & Assoc.
Bridge Replacement Carrying CR 375W Over
Lick Creek, Project:BRO-9959 ( ), Des. No. 9982490
Orange Co. Bridge File No. 34, Orange Co., IN
W.O. #50043.009

TABLE II: NATURAL MOISTURE CONTENT DETERMINATION

Test No.	Boring No.	Sample No.	Depth (Feet)	Depth (Meters)	Moisture Content (%)	**Microwave** Moisture Content (%)
STA. 93+25, 30.0'LT, LINE B						
2874	TB-2	2/SS	2.5-4	0.8-1.2	22.0	22.2
2875		3/SS	5-6.5	1.5-2.0	25.6	
2876		4/SS	7.5-9	2.3-2.7	26.7	26.1
2877		5/SS	10-11.2	3.0-3.4	30.9	
STA. 94+10, 20.0'RT, LINE B						
2878	TB-3	1/SS	1-1.5	0.3-0.5	20.6	
2879		2/SS	2.5-4	0.8-1.2	20.7	19.4
2880		3/SS	5-6.5	1.5-2.0	20.7	
2881		4/SS	7.5-9	2.3-2.7	21.1	23.4
2882		5/SS	10-11.5	3-3.4	20.2	
STA. 94+70, 15.0'LT, LINE B						
2883	TB-4	1/SS	1-1.5	0.3-0.5	15.3	
2884		2/SS	2.5-4	0.8-1.2	21.4	
2885		3/SS	5-6.5	1.5-2.0	20.8	22.6
2886		4/SS	7.5-9	2.3-2.7	18.3	18.3
2887		5/SS	10-11.5	3.0-3.4	18.3	
STA. 91+80, 22.5'RT, LINE B						
2888	TB-5	1/SS	1-1.5	0.3-0.5	22.4	
2889		2/SS	2.5-4	0.8-1.2	20.3	18.0
2890		3/SS	5-6.5	1.5-2.0	20.5	20.6
2891		4/SS	7.5-9	2.3-2.7	27.6	
2892		5/SS	10-11.5	3-3.5	24.5	

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carryng CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.0. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 87+00, 10.0'RT, LINE B									
2853	RB-2	3/SS	5-6.5	1.5-2.0	1.78	0	9.0	101.0	22.2
STA. 92+25, 23.0'RT, LINE B									
3158	RW-1	1/RC	13.5	4.1	792	0	2.4	165.9	0.2
3159		2/RC	17.5	5.3	823	0	0.8	165.0	0.2
STA. 92+70, 20.0'LT, LINE B									
3160	RW-2	1/RC	9.5	2.9	837	0	0.6	156.4	0.3
STA. 94+00, 25.0'RT, LINE B									
2867	RW-3	4/SS	7.5-9	2.3-2.7	0.96	0	5.4	111.1	18.5
3161		2/RC	16.0	4.9	817	0	0.8	165.0	0.2
3162		2/RC	20.0	6.1	201.8	0	0.5	160.7	0.4

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carrying CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.O. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 92+70, 20.0'RT, LINE B									
3150	TB-1	1/RC	12.5	3.8	689	0	0.8	161.5	0.6
3151		1RC	16.5	5.0	355	0	0.8	158.4	0.7
3152		2/RC	20.5	6.2	678	0	1.0	150.3	4.1
STA. 93+30, 15.0'LT, LINE B									
3153	TB-2	1/RC	13.5	4.1	921	0	1.0	164.4	0.2
3154		2/RC	16.0	4.9	851	0	1.0	162.6	0.7
STA. 94+10, 20.0'RT, LINE B									
2882	TB-3	5/SS	10-11.5	3-3.4	0.28	0	15.0	110.5	20.2
3147		1/RC	14.0	4.3	784	0	0.8	165.4	0.1
3148		2/RC	18.0	5.5	560	0	1.0	162.3	0.1
3149		4/RC	29.0	8.8	876	0	2.1	154.2	0.9

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

Indiana Dept of Transportation
Bridge Replacement Carrying CR 375W over Lick Creek
Project:BRO-9959 ( ), Des. No. 9982490
Orange County Bridge File No. 334, Orange Co. IN
W.O. #50043.009

TABLE III: TABULATION OF UNDISTURBED DATA

Test No.	Boring No.	Sample No.	Depth (Ft.)	Depth (Meter)	Triaxial Compressive Strength (tsf)	Confining Pressure (psi)	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
STA. 94+70, 15.0'LT, LINE B									
2885	TB-4	3/SS	5-6.5	1.5-2.0	1.88	0	7.1	104.6	20.8
2886		4/SS	7.5-9	2.3-2.7	1.26	0	10.7	114.9	18.3
3155		2/RC	15.5	4.7	742	0	1.3	157.3	0.7
3156		3/RC	22.0	6.7	776	0	1.4	168.4	0.2
STA. 91+70, 22.5'RT, LINE B									
2892	TB-5	5/SS	10-11.5	3-3.5	0.34	0	15.6	99.1	24.5
3157		1/RC	15.5	4.7	758	0	0.7	167.1	0.2

TABLE IV: SUMMARY OF HAND AUGER SOUNDINGS

CLIENT: BERNARDIN, LOCHMUELLER & ASSOCIATES, INC.
PROJECT: ORANGE COUNTY BRIDGE #34 RECONSTRUCTION
LOCATION: ORANGEVILLE TOWNSHIP, ORANGE COUNTY, INDIANA
DATE: March 8, 2004

PROJECT NO.: BRO-9959-( )
DESIGNATION NO.: 9982490
HCN W.O.: 50043.009
BLA P.N.: 199-0047-0BD

SOUNDING NO.	STATION	OFFSET LINE "A"	APPROX. ELEVATION (Ft.)	SOUNDING DEPTH (Ft.)	FIELD OBSERVATION (TOP TO BOTTOM)	NOTE
S-1	91+00	20' Rt.	526.0	3.5	0.5' Topsoil, 2.5' Br. SOFT Silty Clay, 0.5' scft to medium stiff Br. Clay	4' Proposed Fill along existing pavement
S-2	91+64	20' Lt.	516.5	2.0	2.0' Br. SOFT Silty Clay on Cobbles	17' Proposed Fill along existing drainage ditch
S-3	91+65	20' Rt.	514.0	1.5	1.5' Br. wet Silty Sand on Cobbles	
S-4	96+00	10' Lt.	520.5	2.5	1.5' to 2' Br. and Gr. SOFT Silty Clay, 0.5' to 1' medium stiff to stiff Silty Clay	12' Proposed Fill
S-5	98+00	10' Rt.	527.5	3.5	0.5' Topsoil, 2.5' Br. wet Silty Sand, 0.5' soft to medium stiff Br. Clay	5' Proposed Fill