KB Technologies®

Load Test Program
Kentucky Transportation Cabinet
U.S. Hwy 60/U.S. Hwy 231
Bridge Over Ohio River

Pier 8, Shaft #43 &
Peir 9, Shaft #42

Owner:
Federal Highway Administration/
Kentucky Transportation Cabinet

Contractor:
National Engineering and Contracting

Slurry:
SlurryPro® CDP™
KB Technologies Ltd

Loadtest by:
LOADTEST, Inc.
Project N0. LT-106

Test Date:
February 1993

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U.S. 60/U.S. 231
Bridge Over Ohio River
Pier 8, Shaft #43

Project Description

The project consisted of the construction of a four-lane bridge over the Ohio river connecting U.S. Route 60 in Davies County, Kentucky with U.S. Route 231 in Spencer County, Indiana. The structure will be located approximately nine miles upstream of Owensboro, Kentucky (river mile 745.5) and the two main piers were to be constructed adjacent to the river navigational channel to provide a navigational clearance of about 1100 feet. The width of the Ohio river at this location is about 2900 ft.

Drilled shafts socketed in the underlying shales were selected for this project due to the scour levels of only 4 to 8 feet above the top of the rock. The final design consisted of a 72 in diameter shaft, with a embedded length in the soft rock of 45 ft. Slake durability index testing of core shale samples indicated a range of 0 to 93 percent, with an average of 66 percent.

The wet method of construction was selected, and CDP™ was selected as the slurry base. CDP™ yielded the best result in inhibiting the shale dispersion or deterioration, according to results obtained in the laboratories of the Kentucky Transportation Cabinet. The slurry was prepared in a large premix tank to a Marsh Funnel viscosity equal to 85 seconds/quart, and the Ohio river water was used as make-up water. The depleted slurry was allowed to be disposed of in the river.

Document Structure

This document is the compilation of several technical reports. Such reports have been placed in the following sequence and a brief summary and relevant information, if any, is added.

1 **"Bi-Directional Load Testing of Shaft to 6000 Tons"**

By: J.W. Goodwin, P.E., LOADTEST, Inc.

ASCE Case History Report (Preprint)

This report gives a general overview of the load test program using an Osterberg cell calibrated to be loaded up to 6000 tons. Special emphasis is giving to the fact that although a combined load of 53 MN (6000 tons) was attained, failure was not reached neither in shear nor bearing. In addition, general site and design information, as well as test shaft dimension and details are included and general conclusion are presented.

2 Load Test Results

By: J.W. Goodwin, P.E. & J.H. Schmertmann, P.E., Ph.D.
LOADTEST, Inc.

This technical report provides all the results and details of the test and analysis of the data. Drawings showing the load test shaft location, dimensions and elevations are included as Figures A.1 and A.2 in Appendix A. A summary of load test results for drilled load test shaft No. 43, pier 8 are presented in Table 4, and reproduced in the following table.

Equivalent Top Load at 1" Settlement	**(Tons)**
Shear	3800
Bearing	1500
Total (axial)	5300
Creep Limit	> 3000
At FHWA Failure Criteria	
Shear	5800
Bearing	2800
Total (axial)	8600
Maximum Rock Shear (Below Failure)	**(psi)** 88

Based upon the load test results, the socket length at pier 8 was reduced by 6 feet.

3 Subsurface Log

By: Kentucky Transportation Cabinet
Division of Materials, Geotechnical Branch

This are the relevant bore logs to the pier location.

4

Geotechnical Investigation

By: C.R. Lennertz, P.E.
The H.C. Nutting Company

This report incorporates all the geotechnical field investigations (bore log, grain size distribution analysis, etc.) and a geological description of the Owensboro area. Unconfined compression tests for selected samples were carried out, and the results were used by Fuller, Mossbarger, Scott and May to generate the Geotechnical Engineering Report.

5

Geotechnical Engineering Report

By: S. L. Murray, P.E. & A. D: May, P.E.
Fuller, Mossbarger, Scott and May
Civil Engineers, Inc.

This report includes results of field borings and laboratory testing, and recommendation for design and construction. The results indicate that Pier 8 shales were found to have an average unconfined compressive strength of 340 psi, average Rock Designation Quality (RQD) equal to 65, and an average Kentucky Modified RQD of 33.

6

Foundation Evaluation Report

By: S. L. Murray, P.E.
Fuller, Mossbarger, Scott and May
Civil Engineers, Inc.

This report comprises a predesign of both H-piles and drilled shafts in terms of ultimate and allowable axial, uplift and lateral capacities.
The approximate capacities and shafts lengths were determined employing procedures outlined in AASHTO's 1991 Interim Specifications Section 4, "Foundations".
Ultimate capacities based on strain and/or designed settlements (3 inches) were determined by using procedures outlined in Publication No. FHWA-HI-88-042, "Drilled Shafts Construction Procedures and Design Methods."
It is important to point out that although separation of the shear and end bearing (point) capacities, are not shown in summary presented the introductory letter, the ultimate axial capacity of the shaft was calculated by adding the ultimate side resistance to the ultimate tip resistance. In the determination of the side resistance of a 5.5 feet diameter drilled shaft, a value of unit side shear, q_{sr}, equal to 60 psi was used. In a subsequent report [Drilled Shafts Predicted Capacities (Final)], the q_{sr}

was corrected to 20 psi, which is appropriate for smooth surfaces assuming q_u = 500 psi (from Figure 4.6.5.3.1A in the AASHTO's 1991 Interim Specifications Section 4, "Foundations". A summary of the values presented is presented below.

Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Ultimate Axial Capacity (Kips)	Allowable Axial Capacity (Kips)	Allowable Uplift Capacity (Kips
5.5	234	7,500	3,000	740

7 Drilled Shafts Predicted Capacities (Final)

By: S. L. Murray, P.E.
Fuller, Mossbarger, Scott and May
Civil Engineers, Inc.

This report includes all the revisions and the calculation for new dimensions in the diameter of the drilled shafts (from 5.5 to 7.5 feet). Final predesign capacities are provided, and the concrete cover for reinforcing steel in the socket was reduced from 6.5 inches to 6.5 inches. The Axial and uplift capacities are presented below.

Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Ultimate Axial Capacity (Kips)	Allowable Axial Capacity (Kips)	Allowable Uplift Capacity (Kips
5.5	234	7,500	3,000	740
7.5	234	11,250	4,500	1,300

8 Mixing and Handling

By: KB Technologies

This document present the recommended slurry specification as determined for the specific site and soil conditions. Values for Marsh funnel viscosity and density were determined base on laboratory results performed at KB Technologies facilities.
The slurry was prepared in a large premix tank located near the excavation and was continuously recycled

9 **Drilled Shaft Soil Excavation Log**

By: Kentucky Transportation Cabinet
Department of Highways, Division of Construction

Log of the excavation sequence presenting the properties of the slurry at different times during construction. Description of soil excavated at different depths.

10 **Drilled Shaft Concrete Placement Log**

By: Kentucky Transportation Cabinet
Department of Highways, Division of Construction

Log of the concrete pouring. Includes the theoretical and actual volume curve of concrete placed in the shaft.

11 **Borehole Caliper Log**

By: Western Atlas International
Atlas Wireline Services

After the shaft was excavated to designed depth, the contractor introduced a "back-scratcher" tool consisting of a core barrel with cable projections of 8 inches to produce grooves. Such asperities were intended to increase mobilized the skin friction of the structure. Furthermore, the rock socket was caliper to determine the socket diameter by using a wireline caliper tool. The calipering tool consisted of a four retractable arms, with a maximum reach of 2.3 m (7.55 feet). The results are presented graphically and numerically.

DRAFT

BI-DIRECTIONAL LOAD TESTING OF SHAFTS TO 6000 TONS

By Jeffrey W. Goodwin[1], Member, ASCE

ABSTRACT: This paper describes the use of an Osterberg Cell device to perform bi-directional load testing on a full size production drilled shaft for a major bridge foundation near Owensboro, Kentucky. A fully instrumented, 1.83 m (72 inch) diameter shaft was constructed with a rock socket in soft sandy shale, and with the top of shaft 17 m (57 feet) below the surface of the Ohio River. The test did not use a surface reaction system, yet still reached a combined load of 53 MN (6000 tons) without failure occurring in either shear or end bearing.

INTRODUCTION

Bi-directional load testing using Osterberg Cell technology permitted the static load testing of a drilled shaft to a what the writer believes to be a world record of 53 MN (6000 tons). This paper provides some of the details regarding the site, test methods, instrumentation, and results.

The Osterberg method loads the shaft from the base using a patented jacking device (Osterberg, 1989). Pressurizing the device simultaneously loads the rock beneath the shaft in end bearing and the rock and soil above in upward (negative) shear. No surface reaction system of any kind is required for this test method, because the end bearing provides the reaction for the shear test, and the shear resistance provides the reaction for the end bearing test.

As part of a drilled shaft research program in conjunction with the Federal Highway Administration and the Kentucky Transportation Cabinet, these load tests were performed near Owensboro, Kentucky to determine the load carrying capacity of drilled shafts socketed into soft shales. Two bi-directional load tests were performed on drilled shafts constructed at production locations. Both test shafts were constructed with the top of shaft approximately 17 m below the surface of the Ohio River. Each test shaft incorporated one calibrated 0.86 m (34 inch) diameter Osterberg Cell, and was instrumented with telltales to the top and bottom of the device and with 32 Sister Bar strain gages. This paper describes in detail only one of the two, similar tests.

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[1]Project Engineer, LOADTEST, Inc., 4509 NW 23rd Ave, Suite 19, Gainesville, FL 32606

GENERAL SITE AND DESIGN INFORMATION

The Owensboro Bridge is being constructed as an alternate for US 231, between Owensboro, Kentucky and Rockport, Indiana. The 1375 m (4510 feet) long bridge will have a center span of 366 m (1200 feet) and main piers supported by drilled shafts. Each of the two main bridge piers, Nos. 8 and 9, will be supported by 44 - 1.78 m (70 inch) diameter drilled shafts constructed within cofferdams. The piers are designed to support the main towers for either a concrete or steel cable stayed bridge (final design had not yet been chosen).

The Ohio River fluctuates rapidly at the site due to the release of water from upstream dam facilities. Normal pool elevation is approximately EL 110 m. The mudline within the cofferdams was at EL 104.2 prior to excavation. Excavation to the bottom of the foundation seal at EL 90.5 was through Pleistocene Age sand and gravel deposits which extend until the top of rock was reached at approximately EL 82. Rock consisting of Middle Pennsylvanian Age sandy shale with limestone and sandstone laminations is present at both pier locations. At Pier 8, a 1.2 m (4 ft) thick layer of coal and underclay was encountered at approximately EL 78.

A geotechnical investigation, performed by Fuller, Mossbarger, Scott & May of Lexington, Kentucky, indicated that the shales varied considerably in quality from Pier 8 to Pier 9. Pier 8 shales were found to have an average unconfined compressive strength of 2.3 MPa (340 psi), Rock Quality Designation (RQD) average of 65, and an average Kentucky Modified RQD of 33. At Pier 9, the shales were found to have an average unconfined compressive strength of 10.0 MPa (1450 psi), average RQD of 79, and average Kentucky Modified RQD of 35. The Kentucky Modified RQD uses the sum of recovered NX size rock core greater than or equal to 102 mm (4 inches) in length that <u>can not be broken by hand</u> divided by the length of the core run (as a percentage). Slake durability index testing of core samples indicated a range of 0 to 93 percent with an average value of 66 percent at Pier 8, and range of 16 to 94 percent with an average of 75 percent at Pier 9.

Due to the calculated scour levels of only 1.2 to 2.4 m above the top of rock, the design engineers chose a drilled shaft foundation socketed into the underlying shales. Each main pier shaft had a maximum axial compression service load of 12.6 MN (1425 tons), a net uplift load of 2.65 MN (300 tons), and a lateral load of 0.67 MN (75 tons). After considering group effects and the factor of safety, each test shaft required an ultimate axial compression load capacity of 37.1 MN (4200 tons).

Final shaft design required the placement of an 1.88 m diameter permanent casing from the bottom of footing to the top of rock. A 1.78 m diameter rock socket was designed with a length of 13.7 m (45 ft) at Pier 8 and 4.6 m (15 ft) at Pier 9. The differences in socket lengths were due to the overall better quality of rock encountered at Pier 9 and the presence of the zone of coal and underclay at Pier 8. This paper will describe in detail the load test at Pier 9.

LOAD TEST SHAFT CONSTRUCTION

The foundation contractor, National Engineering & Contracting Co., of Strongsville, Ohio constructed the load test shaft from a barge using a crane-mounted drill rig. The Contractor began drilling inside a previously installed 30.5 m (100 ft) long, 1.88 m (74 inch) I.D. casing using a 1.85 m (73 inch) diameter auger bit. The shaft was drilled under wet conditions using a polymer slurry, to protect the shales from deterioration and slaking due to contact with free water. Sand, gravel, and cobbles were removed from the shaft until the top of rock was encountered. The Contractor then advanced a nominal 1.78 m (70 inch) diameter rock socket the specified depth of 4.6 m (15 ft). After excavating to the target elevation, the walls of the socket were scraped to remove the softened shale interface using a specially constructed "back-scratcher" tool consisting of a core barrel with cable projections of 203 mm (8 inches). The shaft bottom was cleaned using a flat bottom cleanout tool, followed by replacement of the slurry column to remove suspended solids.

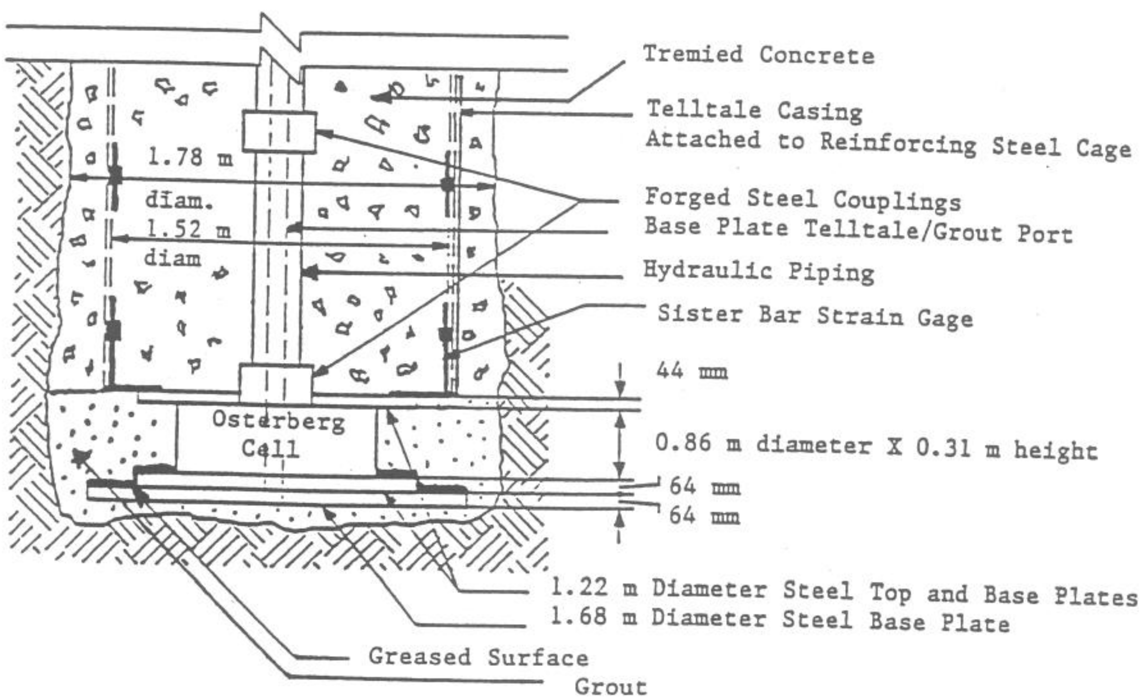

FIG. 1. Osterberg Cell and Instrumentation Configuration

When shaft cleanout was complete, the socket diameter was measured using a wireline caliper tool. Calipering services were provided under separate contract by Atlas Wireline Services of Olney, Illinois, with a calipering tool capable of extending out to a 2.3 m diameter. The calipering of the rock socket was performed to assess the roughness of the socket walls. Results of the calipering at Pier 9 indicated a relatively consistent average socket diameter of 1.83 m (72 inches), with a maximum diameter of 1.88 m (74 inches) and a minimum diameter of 1.78 m (70 inches).

A diagram of the Osterberg Cell configuration at Pier 9 is included as Figure 1. Piping was installed from the top of the jacking device to the work platform. Pressure piping consisted of 102 mm (4 inch) O.D. schedule 80 steel pipe. Inside the 102 mm piping was a 38 mm (1½ inch) O.D. schedule 80 steel pipe, to be used as a telltale and grouting port for the Osterberg device. Beneath the device were bearing plates consisting of a 1.22 m (48 inch) diameter by 64 mm thick steel plate and a 1.68 m (66 inch) diameter X 64 mm (2½ inch) thick steel plate. Above the device was a 1.22 m diameter by 44 mm (1-3/4 inch) thick steel plate. The top plate was attached to the reinforcing steel cage by welded steel flanges.

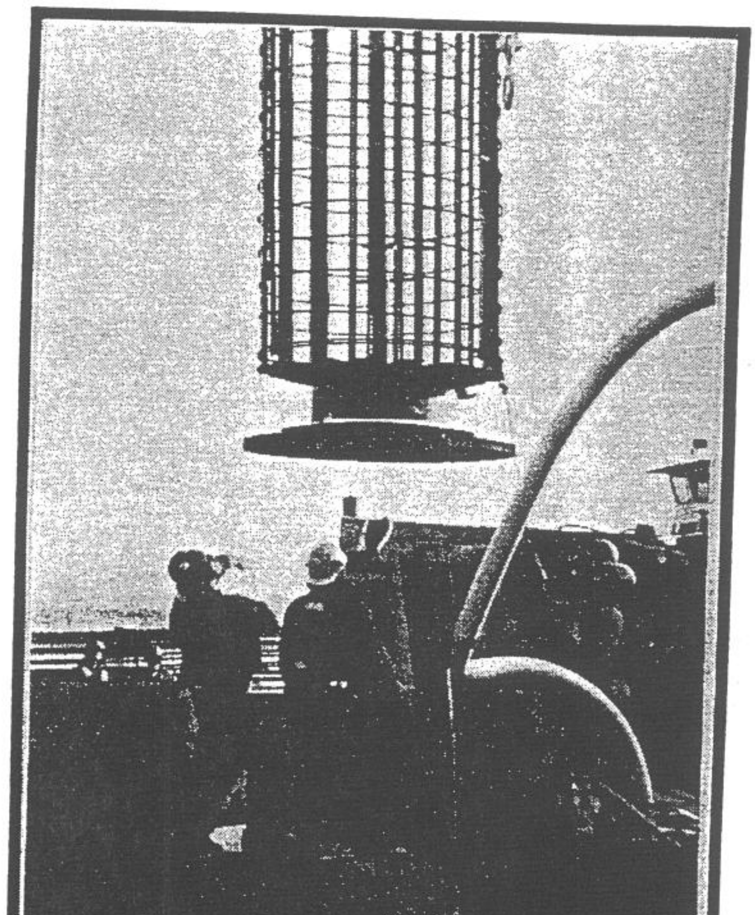

FIG. 2. Placement of Osterberg Cell Assembly into Shaft

After the test shaft was approved for concreting, the Contractor lowered a 1.5 m (60 inch) O.D. reinforcing steel cage into the shaft as shown in Figure 2. It held an Osterberg Cell, top and bottom bearing plates, instrumentation, and related cables and piping. Prior to placement, the top of the steel base plates were coated with grease to keep the concrete from adhering to the steel. Upon placement, the bottom plate was held approximately 76 mm (3 inches) above the bottom of the shaft to allow the pumping of high strength Embeco 885 grout through the center pipe of the Osterberg Cell beneath and around the device to provide a high quality bearing surface. Upon completion of grouting, concrete was placed into the shaft using a 152 mm (6 inch) diameter pump line. This concrete was placed from the top of the plates to a level of 1.83 m above top of shaft, with the excess concrete air-lifted out to produce a final top-of-shaft grade of EL 95.5.

OSTERBERG CELL AND INSTRUMENTATION

The Osterberg method loads the shaft bi-directionally from the base using the hydraulically pressurized jacking device. Prior to delivery to the site, the Osterberg Cell was calibrated to 5.3 MN. The linear result was then extrapolated to 26.5 MN based on the linear results from a full calibration of a similar device to 26.5 MN, performed at the National Institute for Standards and Technology. Load was determined by measuring the applied hydraulic pressure at the surface and comparing this pressure with the load calibration curve of the device. Calibrated, high pressure Bourdon gages and transducers were used to read pressure.

Downward movement of the base plate of the device was measured by dial gages attached to the combination grout port and telltale pipe nested within the vertical hydraulic pressure pipe. From a connector on the bottom plate, the telltale pipe projected upward through the device and then inside the hydraulic pipe to the surface where it was connected to a machined rod exiting the hydraulic pipe through a high pressure seal at the work platform. The raw movement data from this telltale pipe was later corrected for Poisson's Ratio effects on the pipe caused by the surrounding hydraulic fluid pressure. Upward movement of the top of the shaft was measured by dial gages attached to the hydraulic pressure pipe and shaft casing. Backup top-of-shaft movements were also measured using an independently supported wireline, and also a surveyor's level.

Thirty two (32) Geokon "Sister Bar" strain gages were installed in 8 levels, 4 gages per level in the load test shaft. The sister bar gages are assembled as 1.37 m long sections of #4 steel reinforcement bar (grade 60). The gages were attached parallel to the #10 reinforcing steel, at an offset of approximately 90 degrees from each other. The shielded cables were taped at various intervals along the reinforcing steel cage and center piping and exited at the top of the casing. Bumpers, made from short sections of #10 steel reinforcement, were tied horizontally to the reinforcing steel cage to protect the gages from the pump line during concrete placement.

The Geokon Model 4911 sister bar strain gage assembly used at the site contained a single vibrating wire to measure the strain in the active length of the bar. The vibrating wire gage was mounted in and protected by a high strength tube at the center of the bar. The gages were monitored by an engineer from LOADTEST, Inc. using a Geokon datalogging device. The frequency vs. strain calibration for each sister bar was determined during manufacturing. Sister bars also include a thermistor to allow the user to measure and compensate for any temperature induced strains during the test. Because the sister bar's strain measurement depended on frequency, any resistance changes due to factors such as longer cables or moisture and voltage leaks did not affect the measurement. Radio and electrical interference were eliminated by grounding the shielded cable.

The instrumentation for the test shaft also included two unstressed telltales, each resting on the metal plate above the Osterberg Cell. Movements of the telltales were read relative to the 1.88 m casing, to measure the compression of the shaft. Each telltale consisted of flush jointed, 3.05 m (10 ft) lengths of 7.9 mm (5/16 inch) solid steel rods extending to the work platform and protected by oil filled 19 mm (3/4 inch) diameter steel piping attached to the reinforcing cage.

LOAD TEST PROCEDURES AND RESULTS

Bi-directional load testing using the Osterberg Cell was performed in general accordance with *ASTM D-1143 Quick Load Test Procedures.* Load was applied to the device by hydraulic pressure via a hydraulic pump driven by a regulated air compressor system. Load was applied in increments of approximately 0.9 MN, and unloaded in increments of 4.4 MN. The pressure was held constant at each loading increment for a total of 4 minutes, and for 3 minutes per increment during unloading. Gages were read at 0.5, 1, 2, and 4 minutes after the load was applied, with an average of 1.0 minute required to increase the load to the next increment.

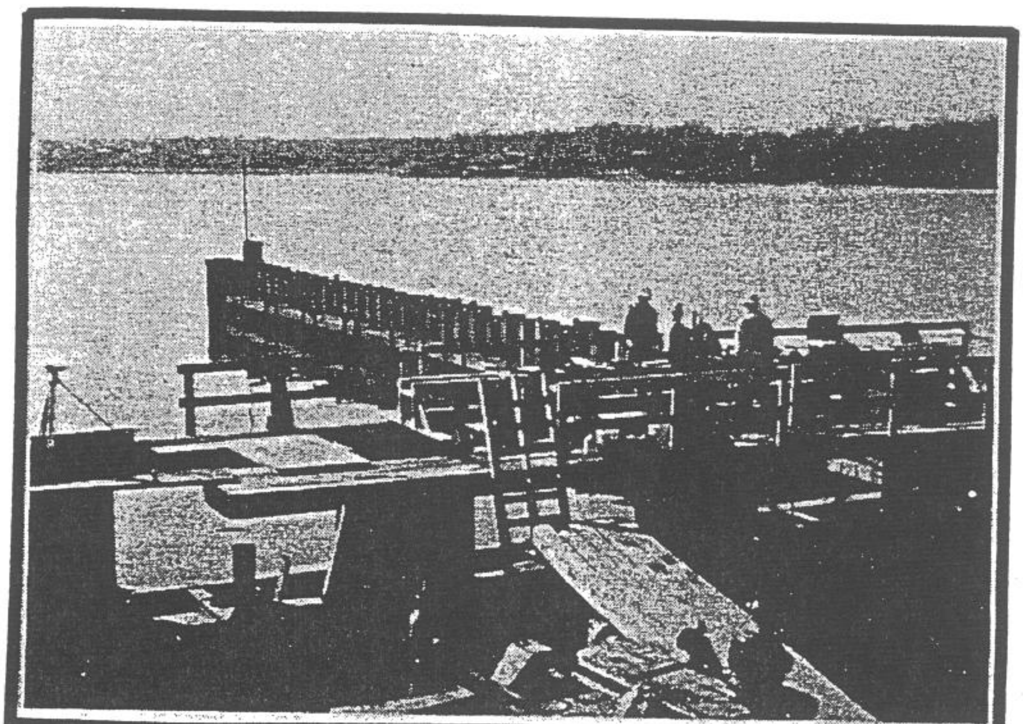

FIG. 3. Work Platform and Load Test Reference Beam Set Up

The load test was performed one week after the installation of the test shaft. A photograph of the reference beam set up is included as Figure 3. Shaft movements recorded from the dial gages, wireline, and survey readings were all in good agreement. The dial gage readings were judged to be the most accurate and their results are discussed herein. The presentation of strain gage data for distribution of shaft resistance is also included.

The Osterberg Cell was installed in a fully closed condition. A maximum load of 27.6 MN was applied, resulting in a vertical upward movement of 7.34 mm (0.289 inches) at the top of the shaft. Downward movement of the base plate at the maximum applied load was 18.39 mm (0.724 inches). The Poisson's Ratio correction added 15.77 mm (0.621 inches) to the measured downward movement, giving a maximum movement for the base plate of 34.16 mm (1.345 inches). After the maximum test load was reached, the load was then incrementally released to zero.

Acquisition of the strain gage data required the use of an electronic data logger to enable computerized reading and storing of 32 strain gages and 32 thermistors per 1 minute interval during the test. The zero load readings were subtracted to give true strain which was converted to load based upon the shaft dimensions and modulus values. To determine the modulus value of the concrete in the shaft, concrete test cylinders were broken at the time of testing and found to have an average compressive strength of 31 MPa (4500 psi). This compressive strength value, when used in the ACI formula for concrete modulus with additions for shaft reinforcing steel, yields a shaft Young's modulus of 32 GPa (4.65E6 psi).

The device and 1.22 m diameter cover plate did not span the entire cross section of the shaft. This resulted in significantly non-uniform strains at the two levels of strain gages closest to the device. Because the load-shedding analysis obtained from the strain gage readings assumes a uniform strain condition across any horizontal plane of the shaft, the closest levels of strain gage readings were corrected, using the theory of elasticity, to what they should have been had strains at these levels been uniform. This produced a major correction for the level of gages closest to the device, a small correction for the next closest level, and no correction for the remaining six levels of strain gages.

The strain gages provide point measurements of strain. Telltales were used to directly measure the compression of the shaft and thus mechanically integrated the strains. The telltale data gave good comparison checks vs. the corrected strain gage data integrated at each of the load increments. These checks, particularly at the highest loads where the integrated strain agreed within 0.02 mm (0.0008 inches) of the total telltale compression of 0.70 mm (0.0277 inches), gave confidence in the overall accuracy of the corrected strain gage data. They also provide some confirmation that the jacking device loaded the shaft in accordance with the calibration used.

Three load-movement curves are obtained directly from a bi-directional load test using an Osterberg Cell. These are the load-movement curve for the bottom of the device moving downward and the top of the device and shaft moving upward. An equivalent surface load-settlement curve is constructed modeling a conventional surface load test. Creep-movement curves are constructed to make an estimate of the friction and bearing loads at which significant creep begins.

Figure 4 presents the downward movement of the base of the device and upward movement of the top of the shaft during the bi-directional load test. As previously explained, the measured downward movement data was corrected for Poisson's ratio effects on the center-pipe telltale. Loads for the upward movements are net axial loads after subtracting the weight of the shaft.

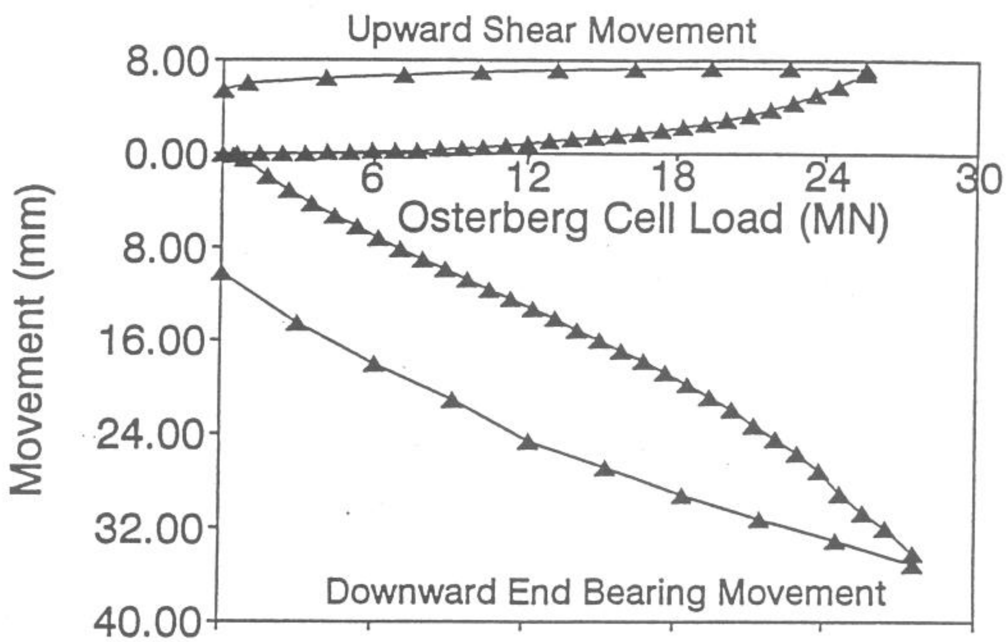

FIG. 4. Upward Shear and Downward Bearing Movement vs. Load

Figure 5 presents the reconstructed, equivalent surface load-settlement curve for the top of the shaft. This has been reconstructed using the following assumptions:

1. The end bearing load-movement curve in a conventionally loaded shaft is the same as the load-movement curve developed by the bottom of the Osterberg Cell. See Schmertmann (1993) for an example.

2. The shear-movement curve for the upward movement in a bi-directional load test is the same as from the downward movement in a conventional test. See Schmertmann (1993) for an example.

3. The compression of the shaft is considered negligible and the shaft is assumed rigid. At Pier 9, the measured compression of the shaft was 0.70 mm (0.0277 inches) when the shear movement was 7.34 mm (0.289 inches) at the top of the shaft.

Using these assumptions allows the engineer to calculate, for any selected settlement, the sum of the shear and end bearing components of resistance from the bi-directional load test to give the equivalent top loading at that top settlement in a conventional test. This can be done up to the maximum movement of that component, upward shear or downward bearing, which moves the least. The heavy solid line in Figure 5 shows this reconstruction over the range where the measured movement data was available from both components. Further construction of the curve requires the extrapolation of the component of lessor movement. The dashed line shows this movement continuing, based on the corrected bottom-of-plate movements plus a least squares hyperbolic curve fit and extrapolation of the top-of-device movements to the maximum corrected downward movement of the bottom plate.

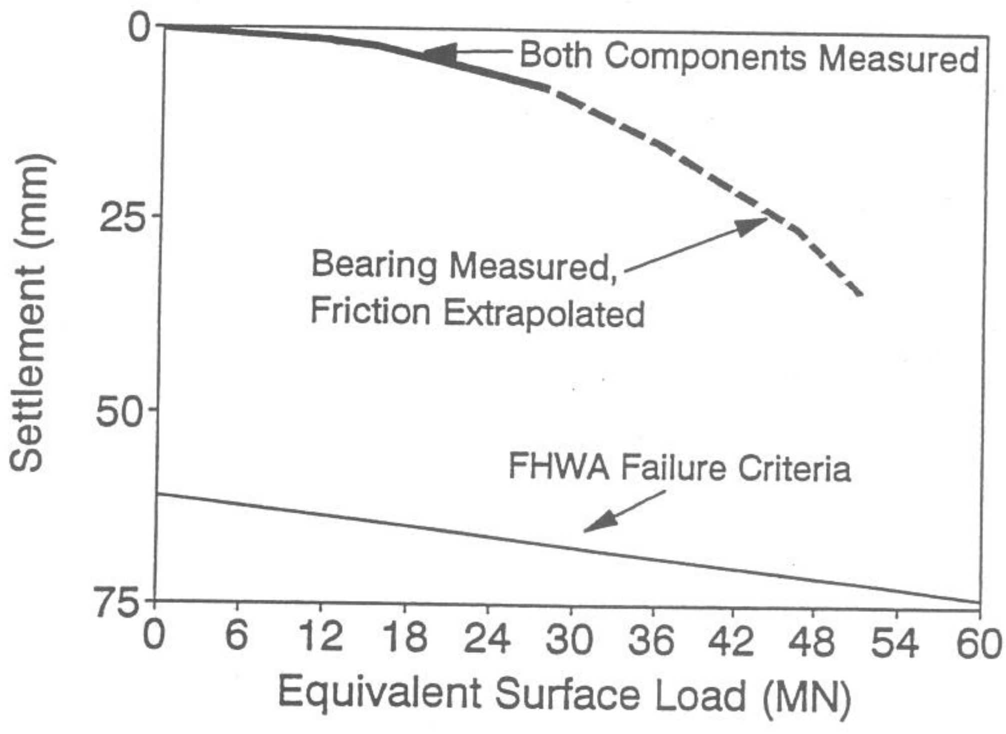

FIG. 5. Reconstructed Equivalent Surface Load-Settlement Curve

The surface dial gages were read at intervals to allow the calculation of the creep of the friction and bearing components. In this test, neither component reached a distinct creep limit.

The purpose of the various levels of multiple strain gages is to permit the construction of a load vs. depth diagram for the test shaft. This gives a visual picture of how the load transferred to the adjacent rock and soil. It also permits calculating the average unit shear resistance for the different layers of rock and soil between the levels of gages. Figure 6 shows the results for the test shaft.

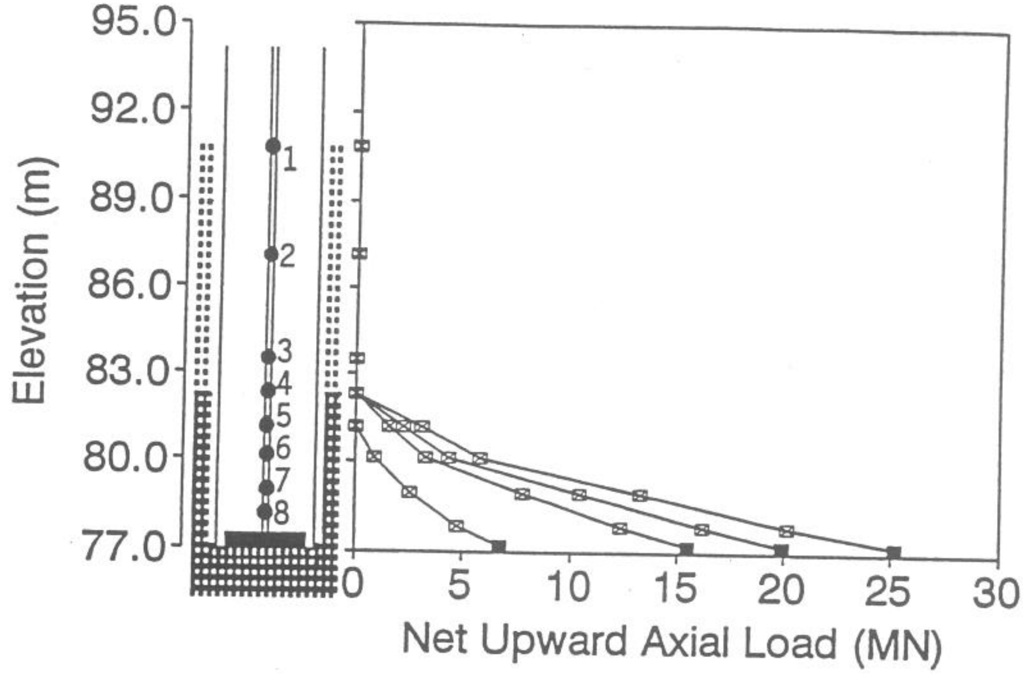

FIG. 6. Shear Load Transfer

Upon completion of load testing, the expanded interior of the Osterberg Cell was grouted in the open position to prevent the jacking device from compressing while carrying service loads. Upon uncoupling the inner bottom-of-plate telltale from the device, grout was pumped into the device through this pipe, displacing the hydraulic fluid in the system.

CONCLUSIONS

Bi-directional load testing using Osterberg Cell technology permitted the successful static load testing of two production drilled shafts to an equivalent surface load of 53 MN (6000 tons), setting a possible world record for the static load testing of a drilled shaft. Neither the ultimate shear resistance nor ultimate end bearing had been reached at the maximum capacity of the device. The equivalent surface load-settlement curve suggests a conventional FHWA failure criteria load of at least 57 MN (6500 tons). Based upon the load test results, the socket length at Pier 8 was reduced by 1.8 m (6 feet). The socket length at Pier 9 was not changed due to lateral load constraints. The post test grouting of the devices was performed to enable the shafts to carry their service loads by using end bearing as well as shear.

ACKNOWLEDGEMENTS

The above project could not have been completed without the assistance and dedication of the many engineers involved. Special thanks go to Dr. John Schmertmann, Mr. Jack Hayes and Dr. David Crapps of LOADTEST, Mr. Tony Simmonds of Geokon, Mr. Daryl Greer and Mr. William McKinney of the Kentucky Transportation Cabinet, Mr. Barry Berkovitz of the Federal Highway Administration, and the staff of National Engineering & Contracting Company.

APPENDIX REFERENCES

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1. INTRODUCTION:

1.1 PROJECT INFORMATION: Two Osterberg Cell load tests were performed on production test shafts by LOADTEST, INC., at the request of the foundation Contractor, National Engineering And Contracting Company. The shafts were drilled under the direction of Mr. Walter Gratz of National Engineering and Contracting, with shaft inspections directed by Mr. Daryl Greer and Mr. William McKinney, Jr. of the Kentucky Transportation Cabinet. Load testing was performed to proof test design criteria for the production drilled shaft program.

1.2 SITE CONDITIONS: The site for the new Ohio River crossing extends from the east edge of Rockport, Indiana, across the river and to where it will rejoin US 231 east of Owensboro, Kentucky. To support the main bridge span, two foundation piers constructed within cofferdams will be built. Each pier will consist of 44 - 72 inch nominal diameter drilled shafts which will be socketed into the underlying rock. One production shaft within each pier was designated for testing by the Kentucky Transportation Cabinet. The two main bridge piers were designated Pier 8 which was located nearest the Kentucky (south) shore of the river, and Pier 9, located nearest to the Indiana (north) shore.

1.3 GENERALIZED SITE GEOLOGY: The generalized geology of the site consists of sand and gravel overlying interbedded shales and limestones. At the time of testing, the Ohio River pool elevation was at about EL. 360± (all elevations in feet MSL). Typically the river depth is about 30± feet. At the location of the drilled shafts, the cofferdam was pre-excavated to the elevation of the bottom of seal for the pier, at about EL 297. Interbedded sand, gravel and cobbles were present from the level of pre-excavation to the top of rock at about EL 270±. Thinly laminated shale with interbedded limestone was located at both pier locations. At Pier 8, a 4 feet thick layer of coal and associated underlying soft clay was encountered at about EL 256. No coal was encountered at the location of Pier 9 during the project geotechnical investigation, which was performed by others.

1.4 SUMMARY OF TEST CHRONOLOGY: Table 1 provides a quick reference for the chronology of the load test shaft installation and testing.

1.5 BIBLIOGRAPHY: This report ends with a bibliography for those readers who are unfamiliar with Osterberg Cell testing or who would like to expand their knowledge of this new method.

2. INSTALLATION OF LOAD TEST SHAFTS:

2.1 DRILLED LOAD TEST SHAFT No. 43, PIER 8: Drawings indicating the load test shaft location, dimensions and elevations are included as Figure Nos. A.1 & A.2 in Appendix A. Photographs of the load test shaft construction and test setup are included in Appendix E. The Contractor began construction of the load test shaft on February 3, 1993. The drilling equipment used was a crane-mounted HAINS drill. The Contractor began drilling inside a previously installed 100 feet long, 74 inch I.D. casing using a 73-inch diameter auger bit. The shaft was drilled under wet conditions using a polymer slurry. The primary function of the slurry was to protect the shales from deterioration and slaking from contact with free water. Sand, gravel, and cobbles were removed from the shaft until the top of rock was encountered at about EL 369.

The Contractor then advanced a nominal 70" I.D. rock socket to a target elevation of EL 219. After the shaft reached the target elevation, the walls of the socket were scraped to remove softened shale build up using a specially constructed "back-scratcher" tool consisting of a core barrel with 8 inch long cable projections. The shaft bottom was then cleaned using flat bottom cleanout tool and then the slurry column was replaced using a submersible pump. Final shaft tip elevation for 8-43 was EL 218.6.

When cleanout was complete, the rock socket diameter was measured using a wireline caliper tool. Calipering services were provided under separate contract by Atlas Wireline Services of Olney, Illinois, with a calipering tool capable of extending out to 90 inch diameter. The calipering of the rock socket was performed to access the roughness of the socket walls. Results of the calipering indicated an average socket diameter of 73 inches, with a maximum diameter of 76 inches and a minimum of 71 inches.

The reinforcing steel cage with the Osterberg Cell, bearing plates, and sister bar strain gages attached was lowered into the shaft. Additional pressure piping as required to reach the top of the casing was added as the reinforcing steel was lowered into the shaft. The Osterberg Cell and reinforcing steel was lowered until it touched the bottom of the shaft, and then it was raised approximately 0.2 feet to facilitate grouting beneath and around the cell.

High strength Embeco 885 grout was pumped through the center pipe of the Osterberg Cell to fill the area beneath the cell up to above the top bearing plate. A total of 39 cubic feet of grout was placed to fill this area. During pumping operations, the pump broke down at about the 2/3 point of placement, and the remainder was placed by tremmie.

Concrete was then placed into the shaft using a 6" diameter pump line. The concrete was placed to a level of 6 feet above top of shaft, then air lifted down to final grade at EL 313.5.

2.1.1 OSTERBERG CELL CALIBRATION AND INSTALLATION: Osterberg Cell number 92-562-2 was installed in shaft No. 8-43. The cell was 12¼" high and 33-½" diameter. One hundred fifty-one feet of piping was installed from the top of the cell to above the casing surface. Pressure piping consisted of 4" O.D. schedule 80 steel pipe. Inside the 4" piping was a 1½" O.D. schedule 80 steel pipe, to be used as a telltale. Beneath the cell were bearing plates consisting of a 66" diameter X 2½" thick steel plate and 48" diameter by 2½" thick steel plate. Above the cell was a 48" diameter by 1-3/4" thick steel plate. The top plate was attached to the reinforcing steel cage by welded steel flanges.

Calibration records for this Osterberg Cell are included in Appendix B at the end of this report. Note that the cell was calibrated to 600 tons and then this linear calibration curve was extrapolated to 3000 tons. We justify this extrapolation on the basis of the linear results from an actual calibration, performed at the National Institute for Standards and Technology (NIST), of a similar cell to the full 3000 tons (see Appendix G for summary report).

2.1.2 SISTER BAR STRAIN GAGE INSTALLATION: A total of thirty two(32) Geokon "Sister Bar" strain gages were installed in load test shaft 8-43. The gages were installed 4 per level for a total of 8 levels of strain gages. As shown in the schematic included as Figure A of Appendix C, the sister bar gages resembled a 4.5' long section of #4 steel reinforcement bar (grade 60). The gages were attached, parallel to the No. 10 reinforcing steel, at an offset of approximately 90 degrees from each other. The shielded cables were taped at various intervals along the reinforcing steel and center piping and exited at the top of the casing. To protect the gages from the pump during concrete placement, short (12") sections of #10 steel reinforcement were tied horizontally to the reinforcing steel.

OWENSBORO - INDIANA STATE LINE ROAD (US231) BRIDGE

TABLE 2: UNIT SHEAR RESISTANCE PIER 8 SHAFT 43

ELEVATION (FEET)	CELL LOAD (TONS)	WEIGHT SHAFT (TONS)	NET LOAD (TONS)	UNIT SHEAR RESISTANCE (PSI)
297.8	19	36	0	
				0
277.5	75	82	0	
				1
267.6	123	105	18	
				24
262.0	316	118	198	
				24
251.5	681	142	539	
				44
237.5	1542	174	1368	
				51
227.9	2225	195	2030	
				66
222.5	2717	206	2511	
				88
220.2	2998	211	2787	

TABLE 3: UNIT SHEAR RESISTANCE PIER 9, SHAFT 42

ELEVATION (FEET)	CELL LOAD (TONS)	WEIGHT SHAFT/ STONE (TONS)	NET CELL LOAD (TONS)	UNIT SHEAR RESISTANCE (PSI)
297.8	-5	135	0	
				0
285.6	19	163	0	
				0
274	-4	189	0	
				0
270.2	14	198	0	
				70
266.5	560	206	354	
				59
262.7	874	214	660	
				183
259.3	1725	221	1504	
				143
255.3	2509	229	2280	
				205
253.1	3126	233	2893	

The sister bars installed for load test shaft 8-43 were installed with a vertical orientation. Four sister bars were placed at each of 8 levels. Figure A.2 indicates the placement intervals of the sister bar strain gages.

2.2 DRILLED LOAD TEST SHAFT No. 42, PIER 9: Drawings indicating the load test shaft location, dimensions and elevations are included as Figure Nos. A.3 & A.4. The Contractor began construction of the load test shaft on February 6, 1993. The shaft was drilled using the same equipment and methods as for shaft No. 8-43. Sand, gravel, and cobbles were removed from the shaft until the top of rock was encountered at about EL 369.

Below EL 369, the Contractor advanced a nominal 70" I.D. rock socket to a target elevation of EL 251.5. After the shaft reached the target elevation, the walls of the socket were scraped to remove softened shale build up using a specially constructed "back-scratcher" tool consisting of a core barrel with 8 inch long cable projections. The shaft bottom was then cleaned using flat bottom cleanout tool and then the slurry column was replaced using a submersible pump. The final shaft tip elevation for shaft No. 9-42 was EL 251.3.

When cleanout was complete, the rock socket diameter was measured using a wireline caliper tool as in shaft 8-43. Results of the calipering indicated an average socket diameter of 72 inches, with a maximum diameter of 74 inches and a minimum of 70 inches.

The reinforcing steel cage with the Osterberg Cell, bearing plates, and sister bar strain gages attached was lowered into the shaft. Additional pressure piping as required to reach the top of the casing was added as the reinforcing steel was lowered into the shaft. The Osterberg Cell and reinforcing steel was lowered until it touched the bottom of the shaft, and then it was raised approximately 0.2 feet to facilitate grouting beneath and around the cell.

High strength Embeco 885 grout was pumped through the center pipe of the Osterberg Cell to fill the area beneath the cell up to above the top bearing plate. A total of 38 cubic feet of grout was placed to fill this area.

Concrete was then placed into the shaft using a 6" diameter pump line. The concrete was placed to a level of 6 feet above top of shaft, then air lifted down to final grade at EL 313.5.

2.2.1 OSTERBERG CELL CALIBRATION AND INSTALLATION: Osterberg Cell number 92-562-3 was installed in shaft No. 9-42. The cell was 12¼" high and 33-½" diameter. One hundred seventeen feet of piping was installed from the top of the cell to above the casing surface. Pressure piping consisted of 4" O.D. schedule 80 steel pipe. Inside the 4" piping was a 1½" O.D. schedule 80 steel pipe, to be used as a telltale. Beneath the cell were bearing plates consisting of a 66" diameter X 2½" thick steel plate and 48" diameter by 2½" thick steel plate. Above the cell was a 48" diameter by 1-3/4" thick steel plate. The top plate was attached to the reinforcing steel cage by welded steel flanges. To provide support to the pressure pipe, for shaft 9-42, stone was placed from the top of the concrete shaft to within 4 feet of the top of the casing.

Calibration records for this Osterberg Cell are included in Appendix B at the end of this report. As explained in section 2.1.1, we also calibrated this cell to 600 tons and extrapolated the linear curve to 3000 tons.

2.2.2 SISTER BAR STRAIN GAGE INSTALLATION: A total of thirty two (32) Geokon "Sister Bar" strain gages were installed in load test shaft 9-42. The gages were installed 4 per level for a total of 8 levels of strain gages. As shown in the schematic included as Figure A of Appendix C, the sister bar gages resembled a 4.5' long section of #4 steel reinforcement bar (grade 60). The gages were attached, parallel to the No. 10 reinforcing steel, at an offset of approximately 90 degrees from each other. The shielded cables were taped at various intervals along the reinforcing steel and center piping and exited at the top of the casing. To protect the gages from the pump during concrete placement, short (12") sections of #10 steel reinforcement were tied horizontally to the reinforcing steel.

The sister bars installed for load test shaft 9-42 were installed with a vertical orientation. Four sister bars were placed at each of 8 levels. Figure A.4 indicates the placement intervals of the sister bar strain gages.

2.3 GROUTING OF CELLS UPON COMPLETION OF TESTING: Upon completion of load testing, the Osterberg Cells were grouted in the open position using Embeco 885 grout, to prevent the cell from compressing and to allow the shafts to carry production loads. The Osterberg Cell at Pier 9, Shaft 42 was grouted under our observation on February 15, 1993. The Osterberg Cell at Pier 8, Shaft 43 was grouted under the observation of the Kentucky Department of Highways on March 3, 1993. Grouting was performed using a pumped tremmie line of Embeco 885 grout, which displaced the hydraulic fluid in the system.

TABLE 1: SUMMARY OF TEST CHRONOLOGY

ITEM	DATE/TIME SHAFT 8-43	DATE/TIME SHAFT 9-42
PULLOUT TESTS	12/03/92 - 12/14/92	12/09/92 - 12/18/92
SHAFT EXCAVATION STARTED	02/03/93 [07:06AM]	02/05/93 [04:00PM]
SHAFT EXCAVATION COMPLETED	02/04/93 [06:00PM]	02/06/93 [09:00AM]
OSTERBERG CELL AND REINFORCING STEEL CAGE INSTALLED	02/05/93 [02:00AM]	02/06/93 [03:00PM]
SHAFT CONCRETE PLACED	02/05/93 [06:00AM]	02/06/93 [10:00PM]
OSTERBERG CELL LOAD TEST PERFORMED	02/10/93 [03:00PM]	02/13/93 [02:00PM]
OSTERBERG CELL GROUTED	02/15/93 [02:00PM]	03/03/93 [01:00AM]

3. OSTERBERG CELL LOAD TEST FEATURES:

3.1 LOAD TESTING TYPES AND REACTIONS: The load test shafts were tested in one stage for both shear and bearing. The Osterberg Cell technique loads the shaft from the bottom by hydraulically pressurizing the cell. The cell load was determined by measuring the applied hydraulic pressure at the surface and comparing this pressure with the cells' load calibration curves. Calibrated high pressure analog gages and hydraulic transducers were used to read pressure. Pressurizing the cell, simultaneously loaded the rock beneath the shaft in end bearing and the rock and soil above in upward (negative) shear. The end bearing thus provided reaction for the shear test, and the shear resistance provided reaction for the end bearing test.

3.2 CONDUCT OF OSTERBERG LOAD TESTS: Movement of the cell bottom was indicated by an axial "telltale" pipe nested within the vertical hydraulic pressure pipe. From a connector on the bottom plate, the 1½" telltale pipe projected upward through the cell and then through the 4" hydraulic pipe to the surface. At the surface it was connected to a machined rod which exited the hydraulic pipe through a high pressure seal. The relative vertical movement of the bottom telltale relative to the top of the shaft was measured using an Ames dial gage, having a travel of 4.000" and read to the nearest 0.001". The dial stem of each gage was plumbed to ensure accurate vertical measurement and was supported by the wooden reference beam assembly. The raw deflection data from the inner telltale was corrected for Poisson's ratio effects on the rod caused by the surrounding hydraulic fluid pressure. This correction was **added** to the measured deflection and was dependent upon the piping material's inner and outer diameters, length, and applied hydraulic pressure.

Movement of the top of the shaft was measured using a dial gage similarly installed on the reference beam system. The dial stem of the gage was plumbed to ensure accurate vertical measurement and rested on a plate attached to the outer pipe. Movement was measured in the upward direction. Movement of the top of the cell was measured by two telltales extending from the top of the cell to the surface inside a 1-inch metal pipe. Movement of the top of the Cell was measured in the upward direction. Readings of the top of shaft movement were also taken using a wireline and rule and a surveyors level.

3.3 SISTER BAR STRAIN GAGES: The Geokon Model 4911 sister bar used at the site contained a single vibrating wire strain gage to measure the strain in the active length of the bar. The vibrating wire gage was mounted in and protected by a high strength tube (0.5" O.D.) at the center of the bar. The gages were monitored by an engineer from LOADTEST, Inc. using a Geokon datalogging device. The datalogger magnetically "plucked" the vibrating wire inside each gage with an electric pulse applied to a coil mounted around the center of the wire. Each subsequent vibration of the wire induced an electrical pulse in the coil which the readout device sensed and counted to display the vibration frequency. The squared frequency varied with the strain in the wire. Geokon determined the strain frequency calibration for each sister bar during manufacture. The sister bar also included a thermistor to allow the user to measure and compensate for any temperature induced strains during the test.

Since the sister bar's strain measurement depended on frequency, resistance changes, such as longer cables, moisture and voltage leaks did not affect the measurement. Radio and electrical interference were eliminated by grounding the shielded cable.

3.4 TELLTALES: The instrumentation for each shaft included two unstressed telltales, each resting on the metal plate above the Osterberg Cell, at the elevations shown on <u>Figure Nos. A.2 & A.4</u>. Each consisted of flush jointed, 10 feet lengths of 5/16 inch steel rods extending to the work platform and protected by oil filled 3/4 inch diameter steel piping attached to the reinforcing steel cage. Movements were measured using mechanical dial gages with 0.0001 inch divisions and attached to the outside casing of the shaft. The telltales were used to measure the compression of the shaft.

4. LOAD TEST RESULTS:

4.1 DRILLED SHAFT No. 43, PIER 8:

4.1.1 OSTERBERG CELL LOAD TEST: The Osterberg Cell load test at 8-43 was begun on Wednesday February 10, 1993 at approximately 3:00 P.M. Prior to the time of testing, a concrete cylinder formed during the construction of the shaft was tested by others and found to have a compressive strength of approximately 4500 psi. Grout from beneath the cell was found to have a compressive strength in excess of 8000 psi.

The Osterberg Cell was installed in a fully closed condition. Load was applied in increments of approximately 100 tons. A maximum load of 2998 tons was applied to the shaft resulting in a vertical upward movement of 0.285 inches. Downward movement at the maximum applied load was 3.192 inches. The maximum capacity of the cell was reached after the 2998 ton load and the load was then released to zero. Due to Poisson's Ratio effects, a correction of up to 0.776 inches was **added** to the field end bearing deflection.

In calculating the upward load applied, the weight of the shaft (211 tons) was subtracted from the load at the test cell. Note that the total shaft weight was used for conservatism, due to the uncertainty of ground water conditions within the shales. If we had used the full buoyant forces, the total shaft shear resistance would have increased by 86 tons for shaft No. 43 at pier 8. A tabulation of all field data, including Poisson's Ratio corrections and load corrections is included as Appendix F.

4.1.2 SISTER BAR STRAIN GAGES: The sister bar strain gages were read using the datalogger device at 1 minute intervals. Thermistor readings were taken simultaneously. Differences from the zero load reading were taken and corrected to the true strain by the formula:

True Strain = Gage Factor(R1-R0) + 1.3(T1-T0)

where R0 = Zero Load Reading

R1 = Reading at Load Increment

T0 = Temperature at Zero Load (degrees C)

T1 = Temperature at Load Increment

True strain was converted to load based upon the shaft dimensions, and modulus values. To determine the modulus value of the concrete in the shaft, a cylinder was broken at the time of testing and found to have a compressive strength of 4500 psi. This compressive strength value when used in the ACI formula for concrete modulus, with additions for shaft reinforcing steel, yields a shaft modulus of elasticity of 4.65E6 psi.

In the test, the cell and 48" diameter cover plate did not cover the entire cross section of the shaft. This resulted in significantly non-uniform strains at the two levels of strain gages closest to the cell. Because the load-shedding analysis obtained from the strain gage readings assumes an uniform strain condition across any horizontal plane of the shaft, we corrected the closest levels of strain gage readings to what they should have been had strains at these levels been uniform. We used the theory of elasticity for this purpose. This produced a major correction for the first level of gages closest to the cell, and a very small correction for the second level. The remaining levels of strain gages did not require such corrections.

A plot of the load distribution between the gage intervals and the Osterberg Cell are included as Figure 3. Detailed tabulations of the sister bar data incorporated the use of an electronic data acquisition system for the computerized reading and storing of 32 strain gages and 32 thermistors per minute for the duration of the load test. Due to the high volume of data retrieved, the data is not tabulated in this report. Spreadsheet files of the raw strain gage data are available upon request.

4.1.3 TELLTALES: Appendix F includes the measured telltale movements, at each of the two telltales, 4 minutes after applying each load increment. Unfortunately, the 2 telltales in load test shaft 8-43 did not provide reliable data for unknown reasons. Their great length (150 ft), the difficulties of holding the protective piping sufficiently plumb within the shaft during concreting, and the possible effects of non-uniform stress conditions at the top of the cell may have caused these telltales to perform poorly. We did not incorporate these results in our analyses.

4.2 DRILLED TEST SHAFT No. 42, PIER 9:

4.2.1 OSTERBERG CELL LOAD TEST: The Osterberg Cell load test at 9-42 was begun on Saturday February 13, 1993 at approximately 2:00 P.M. Prior to the time of testing, a concrete cylinder formed during the construction of the shaft was tested by others and found to have a compressive strength of approximately 4800 psi. Grout from beneath the cell was found to have a compressive strength in excess of 8000 psi.

The Osterberg Cell was installed in a fully closed condition. Load was applied in increments of approximately 100 tons. A maximum load of 3126 tons was applied to the shaft resulting in a vertical upward movement of 0.289 inches. Downward movement at the maximum applied load was 1.345 inches. The maximum capacity of the cell was reached after the 3126 ton load and the load was then released to zero. Due to Poisson's Ratio effects, a correction of up to 0.622 inches was **added** to the field end bearing deflection.

In calculating the upward load applied, the weight of the shaft and stone (233 tons) was subtracted from the load at the test cell. Note that the total shaft weight was used for conservatism, due to the uncertainty of ground water conditions within the shales. If we had used the full buoyant forces, the total shaft shear resistance would have increased by 102 tons for shaft No. 42 at pier 9. A tabulation of all field data, including Poisson's Ratio corrections and load corrections is included as Appendix F.

4.2.2 SISTER BAR STRAIN GAGES: The sister bar strain gages were read using the datalogger device at 1 minute intervals. Thermistor readings were taken simultaneously. Data was analyzed and corrected as previously outlined.

4.2.3 TELLTALES: Appendix F includes the measurements at each of the telltales, again 4 minutes after the application of each load increment. For this load test, the telltales compared reasonably well with each other. Furthermore, integrating the corrected strain gage data gave good comparison checks at each of the loads illustrated in Table 4.1. The telltales integrate the strains mechanically. These good checks, particularly at the highest loads, give us confidence in the overall accuracy of the strain gage data.

4.3 LOAD DEFLECTION CURVES: One obtains various load deflection curves from an Osterberg Cell load test. One obtains directly the load deflection curve for the bottom of the load cell moving downward and the top of the shaft (or load cell) moving upward. One can also reconstruct an equivalent load deflection curve had the load been applied at the top of the shaft as in a conventional load test. Also, one can construct creep deflection curves so as to make an estimate of the creep load limit. Each of these is discussed below:

4.3.1 AS OBTAINED AND CORRECTED: Figures D.1.1 and D.2.1 in Appendix D show the downward movements of the load cells for tests 8-43 and 9-42, respectively. Figures D.1.2 and D.2.2 show the corresponding upward movements of the top of the load cells. As explained previously in section **4.1.1**, the measured downward movement data was corrected for Poisson's ratio effects on the center-pipe telltale. The loads for the upward movements are the net loads after subtracting the weight of the shaft.

4.3.2 RECONSTRUCTED TOP LOADING CURVES: Figures 2 and 5 present our reconstructed, equivalent load settlement curves for the top of each of the shafts. These have been reconstructed using the following assumptions:

1. The end bearing load deflection curve in a conventionally loaded shaft is the same as the load deflection curve developed by the bottom of the Osterberg cell.

2. The shear-deflection curve for the upward movement in an Osterberg cell test is the same as from the downward movement in a conventional test. This is known to be a usually conservative assumption.

3. The compression of the shaft is considered negligible and the shaft is assumed rigid. (Note the telltales showed the compression of shaft 9-42 at maximum cell load was only about 0.03".)

4. The component, end bearing or side friction, with the least amount of movement is extrapolated via hyperbolic curve fitting to a deflection point matching the maximum movement in the other component.

Using these assumptions allows the engineer to select a given settlement and then simply add the shear and end bearing components of resistance from the Osterberg cell test to give the equivalent top loading at that deflection settlement in a conventional test. This can be done up to the maximum deflection in the Osterberg cell test.

The heavy solid lines in Figures 2 and 5 show this reconstruction over the range where the measured deflection data was available from both components. The continuing dashed line gives our estimate of the likely equivalent load-settlement curve wherein we used the aforementioned hyperbolic extrapolation to give the shear component deflections matching those measured in end bearing. The Figures also show the FHWA failure criterion. In the case of test 8-43 the failure criterion intersects the aforementioned extrapolated curve at an equivalent top loading of approximately 8,600 tons. For test 9-42 we did not get an intersection. The equivalent surface loading at FHWA failure appears to be greater than 6,500 tons.

4.3.3 CREEP LIMITS: Because of the surface dial gage setups we could not measure creep separately for the friction and bearing components, but instead measured the combined effect. Figures 3 and 6 show the amount of movement measured over the 2 to 4 minute load interval vs. the load applied, for tests 8-43 and 9-42, respectively. The point on these curves where one might interpret a distinct upward break, or abrupt increase in the rate of shaft movement at constant load over the 2 minute time interval, denotes a load below which creep movement of the shaft is unlikely to occur.

Although test 8-43 showed more creep, neither test reached a distinct creep limit. Both showed a distinct "trough" reduction in the creep rate between cell loads of 2500 - 3000 tons. We do not know the reason for this but suspect that it may somehow be related to the local concrete failure effects at the high concrete stresses and stress gradients that probably occurred near the edges of the 48" diameter top bearing plate. It may also have been related to the very high, near yield point stresses in the pressure pipe which provided the measurement point for the dial gages used to measure creep.

4.4 SHEAR LOAD TRANSFER: The purpose of the various levels of multiple strain gages was to permit the construction of a load vs. depth diagram for each shaft. This gives a visual picture of how the load transferred to the adjacent soil and rock. It also permits calculating the average unit shear resistance for different layers of soil and rock between the levels of gages. We have done this using the methods previously described. Figures 1 and 4 show the results for the test shafts 8-43 and 9-42, respectively. Tables 2 and 3 give the unit shear resistance between the levels of gages, as determined from the data in Figures 1 and 4.

5. CONCLUSIONS:

5.1 Load test shaft Nos. 8-43 & 9-42 were successfully tested to the limits of the Osterberg Cells, reaching equivalent surface loads of 6000 tons.

5.2 Based upon the load deflection characteristics of the test and our reconstruction of the load test data, we conclude that an equivalent conventional load test would have given, at a deflection of 1 inch, a total resistance of about 5300 tons at Pier 8 and 5250 tons at Pier 9 as shown on the accompanying Table 4.

5.3 Load distribution data from the sister bar strain gages indicated that the unit shaft resistance at Pier 8 varied from 22 psi in the upper portion of the rock socket to 88 psi in the interval directly above the Osterberg Cell. At Pier 9 the shear resistance varied from 70 psi in the upper portion of the rock socket to 205 psi in the interval directly above the Osterberg Cell. However; at both load test locations, neither the ultimate shear resistance or end bearing had been reached at the time when the maximum capacity of the Osterberg Cell had been reached.

5.4 A definable creep limit was not reached during either load test.

TABLE 4: SUMMARY OF LOAD TEST RESULTS

LOAD TEST SHAFT No.	8-43	9-42
DATE TESTED	02/13/93	02/10/93
EQUIVALENT TOP LOAD At 1" Settlement	(Tons)	(Tons)
Shear	3800	3250
Bearing	1500	2000
	-----	-----
Total	5300	5250
CREEP LIMIT	>3000	>3000
AT FHWA FAILURE CRITERIA		
Shear	5800	-
Bearing	2800	-
	-----	-----
Total	8600	>6500
MAXIMUM ROCK SHEAR (Below Failure)	(PSI) 88	(PSI) 205

SISTER BAR STRAIN GAGE RESULTS

OSTERBERG CELL LOAD TEST, PIER 8 - 43

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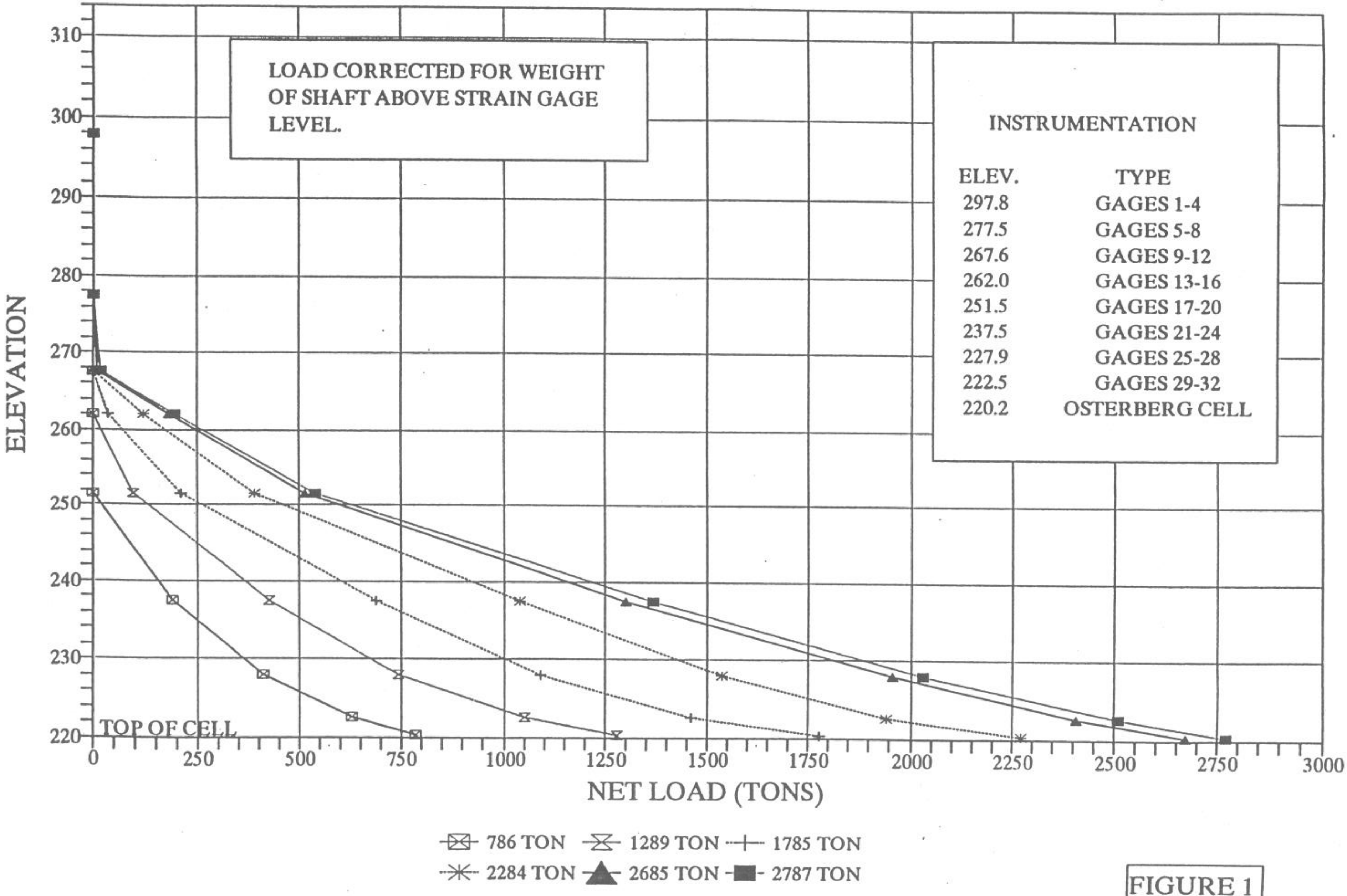
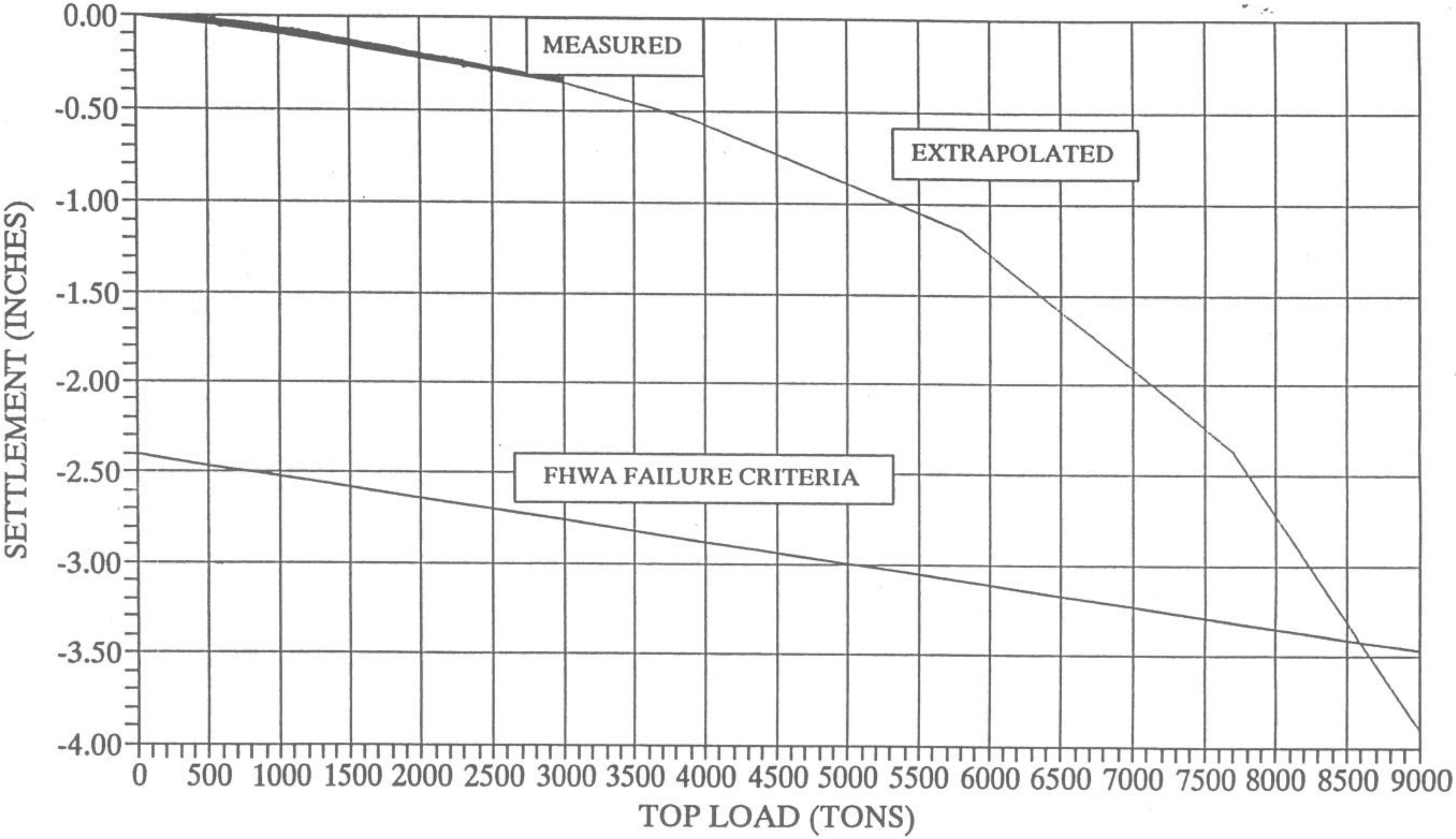


FIGURE 1

EQUIVALENT TOP LOAD SETTLEMENT CURVE
GENERATED FROM OSTERBERG CELL LOAD TEST

OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE
PIER 8 SHAFT 43



DOTTED LINE REPRESENTS EQUIVALENT TOP LOAD SETTLEMENT CURVE GENERATED USING HYPERBOLIC EXTRAPOLATION OF SHEAR DEFLECTION CURVE.

FIGURE 2

COMBINED CREEP CURVE
OSTERBERG CELL LOAD TEST, PIER 8 - 43

OWENSBORO - INDIANA STATE LINE ROAD (US231) BRIDGE

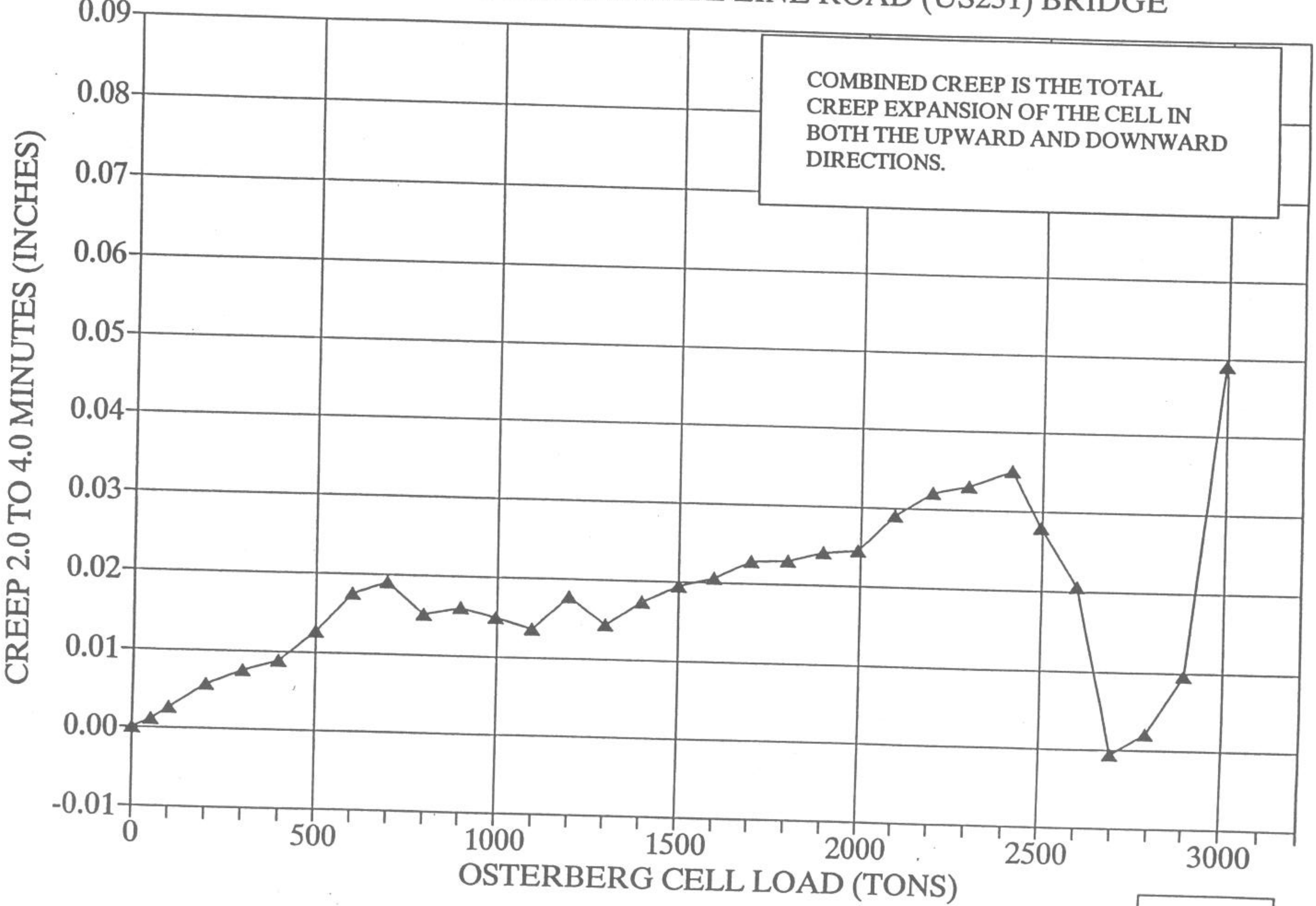


FIGURE 3

SISTER BAR STRAIN GAGE RESULTS

OSTERBERG CELL LOAD TEST, PIER 9 - 42

OWENSBORO - INDIANA STATE LINE ROAD (US231) BRIDGE

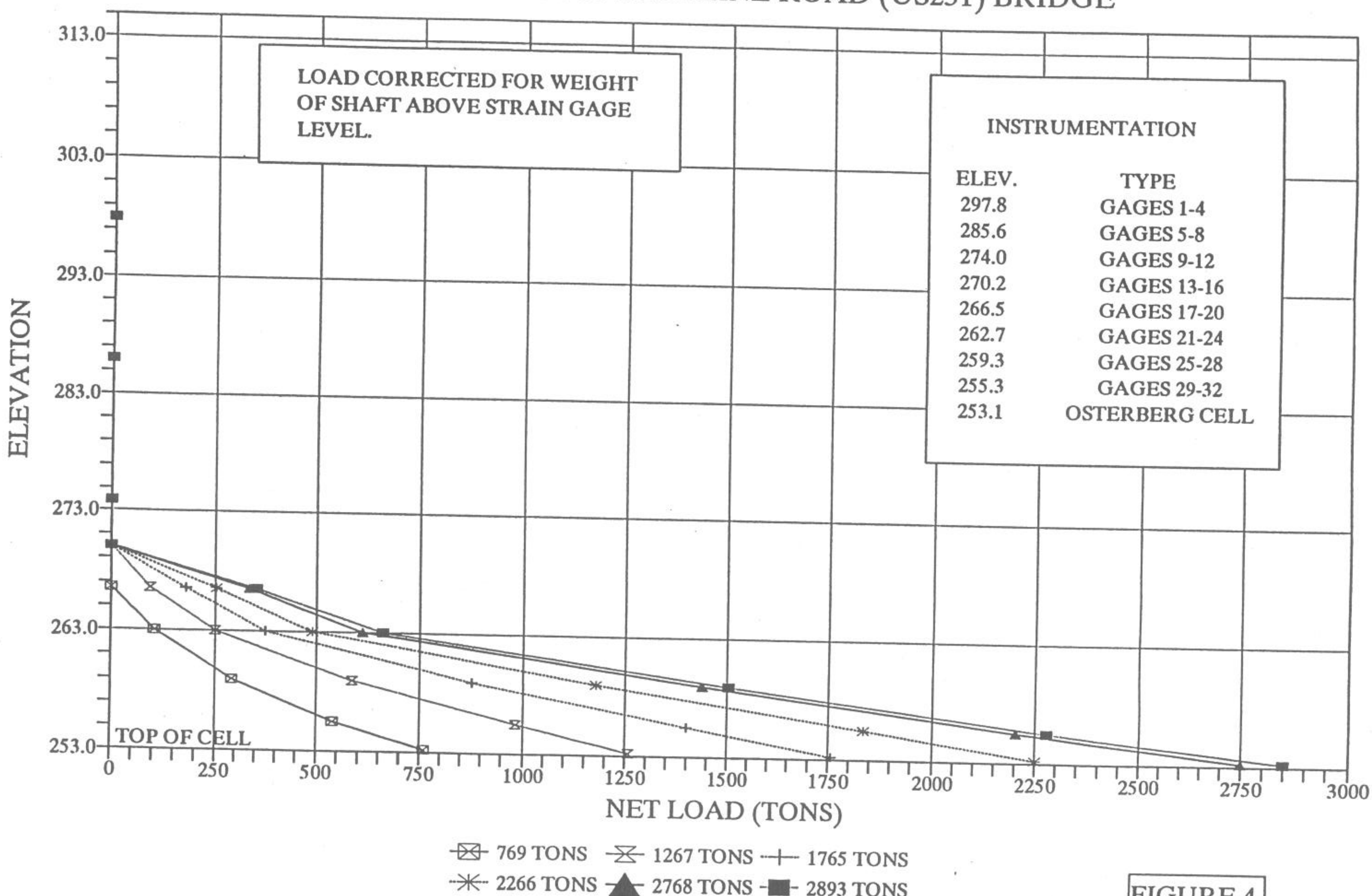
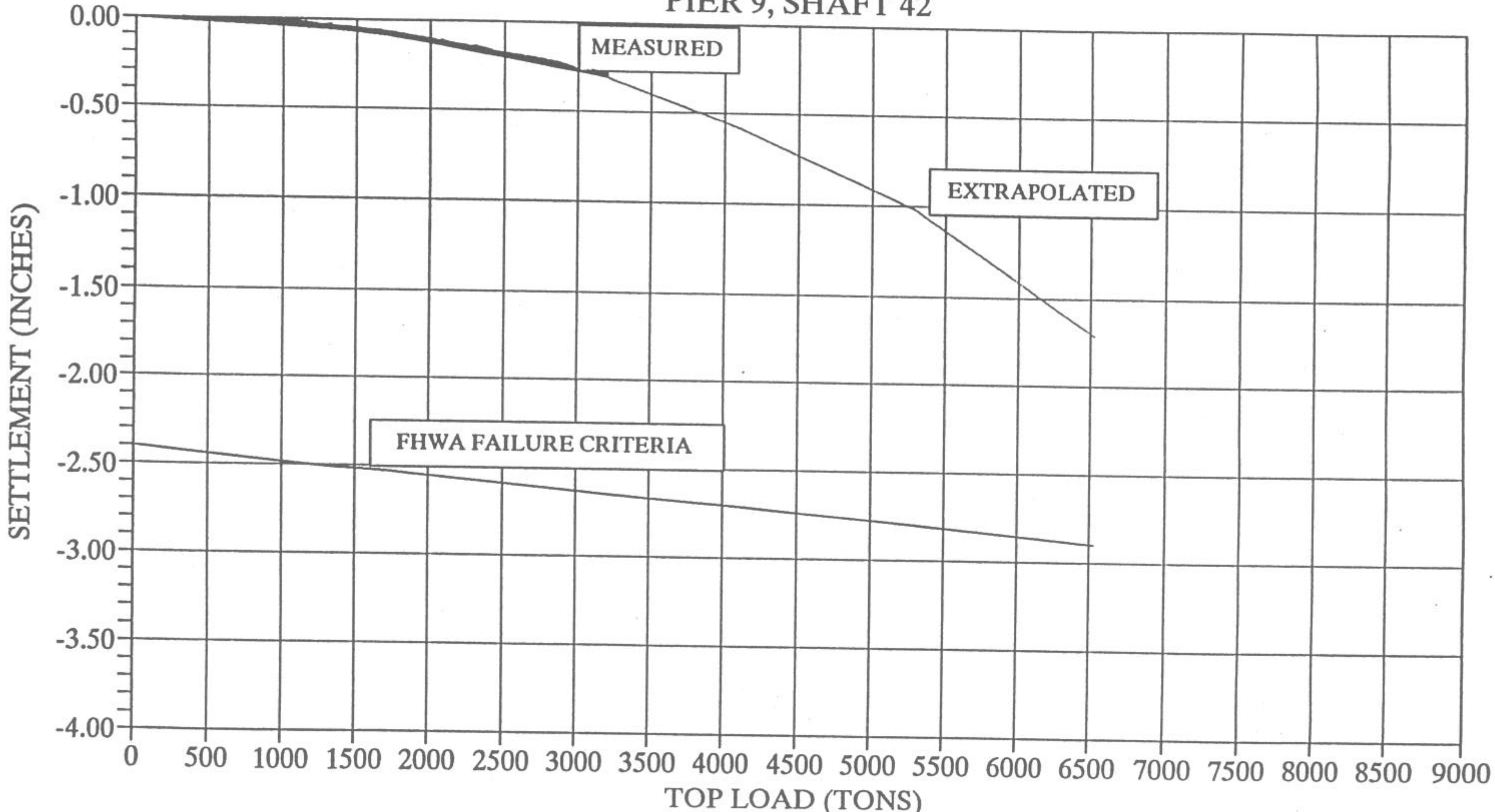


FIGURE 4

EQUIVALENT TOP LOAD SETTLEMENT CURVE
GENERATED FROM OSTERBERG CELL LOAD TEST

OWENSBORO - INDIANA STATE LINE ROAD (US 231) BRIDGE
PIER 9, SHAFT 42



DOTTED LINE REPRESENTS EQUIVALENT TOP LOAD SETTLEMENT CURVE GENERATED USING HYPERBOLIC EXTRAPOLATION OF SHEAR DEFLECTION CURVE.

FIGURE 5

COMBINED CREEP CURVE

OSTERBERG CELL LOAD TEST, PIER 9 -42

OWENSBORO - INDIANA STATE LINE ROAD (US231) BRIDGE

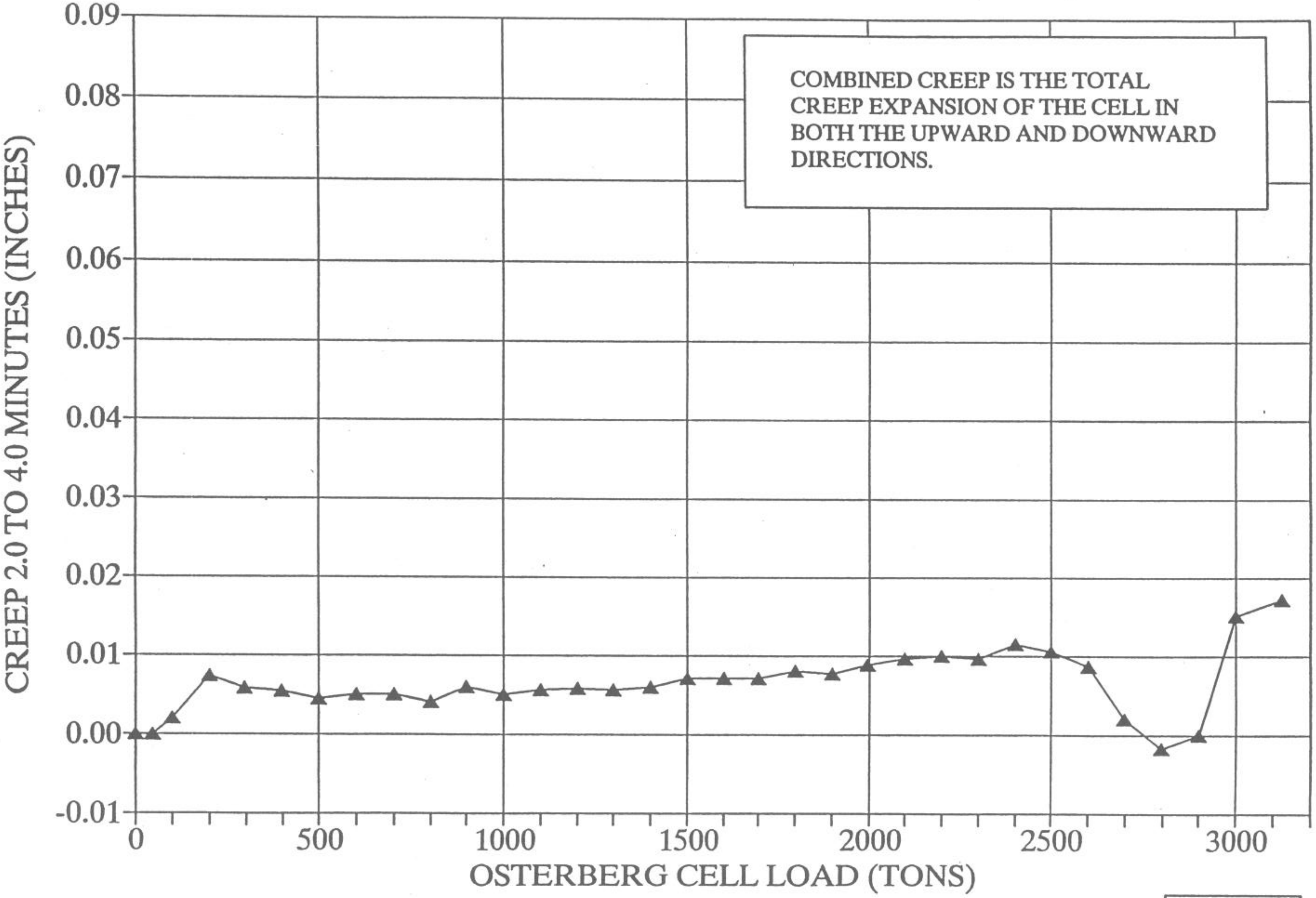


FIGURE 6

6. BIBLIOGRAPHY:

Crowther, C.L., 1988, LOAD TESTING OF DEEP FOUNDATIONS, 1988, John Wiley & Sons, New York.

Osterberg, J.O., 1989. *New Load Cell Testing Device*, Proceedings, 14th Annual Conference. Deep Foundations Institute. pp. 17 - 28.

Osterberg, J.O., 1989. *Breakthrough in Load Testing Methodology*, FOUNDATION DRILLING. November 1989. Published by the Association of Drilled Shaft Contractors.

Osterberg, J.O., 1992. *Rock Socket Friction in Drilled Shafts: It's Greater Than You Think*, FOUNDATION DRILLING. January, 1992. Published by the Association of Drilled Shaft Contractors.

Osterberg, J.O., 1992. THE OSTERBERG LOAD CELL FOR TESTING DRILLED SHAFTS AND DRIVEN PILES, *Draft Copy*, January, 1992. Prepared for the Federal Highway Administration.

March 8, 1993

National Engineering & Contracting Company
12608 Almeda Drive
Strongsville, Ohio 44136

Attention: Mr. Walter Gratz

Subject: Report on Osterberg Cell Load Testing, Drilled Test Shaft Nos. 8-43 & 9-42, Owensboro - Indiana State Line Road (US 231) Bridge, Owensboro, Kentucky (LOADTEST Project No. LT-106)

Enclosed are the results of Osterberg Cell load testing performed at the subject site. We forwarded copies of the test results and of this report directly to Mr. Daryl Greer of the Kentucky Transportation Cabinet.

We performed the following work in accordance with the scope of services detailed in our work order dated February 25, 1992, and additional services which were detailed in our letter of November 19, 1992.

1.) Assisted University of Florida personnel with the installation and performance of four plug pullout tests, two at each shaft location. Services with respect to reporting the results of pullout tests are being performed by the University of Florida, under the direction of Dr. Frank Townsend, and will be forwarded upon completion by them.

2.) Supply and assistance in installation of Osterberg Load Cells in two drilled shafts.

3.) Instrumentation of test shafts using thirty two (32) sister bar strain gages and three (3) telltales per shaft.

4.) Conduct an Osterberg load test at each pier location to test the capacity of the test shafts, in both shear and end bearing.

5.) Report the results of the Osterberg Cell load tests.

Others provided the services for geotechnical engineering design, preparation of project specifications, means and methods, and observation of the installation of drilled shafts.

We are pleased to have been of service to you on this project, and look forward to providing continued service to you.

If you have any questions, please do not hesitate to contact either of the undersigned.

Very truly yours,
for LOADTEST, INC.

____
Jeffrey W. Goodwin, P.E.

____
John H. Schmertmann, P.E., Ph.D.

OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE

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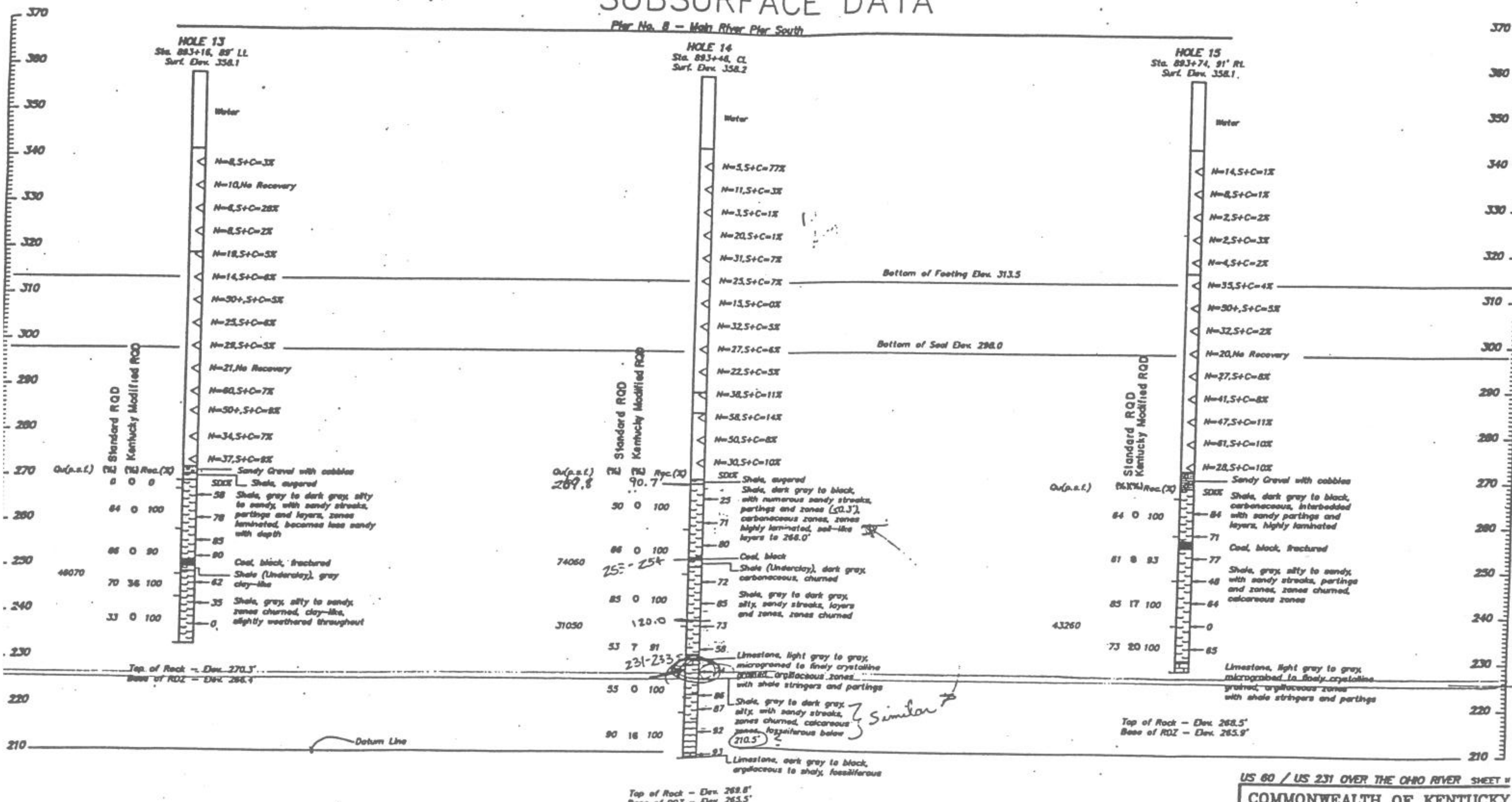
APPENDICES

'SUBSURFACE DATA

Pier No. 8 – Main River Pier South



NOTE:
The Rock Quality Designations (RQD) shown on this sheet were determined by summing the lengths of all pieces of core equal to or longer than 4 inches and dividing by the total length of the coring run. The resultant of that calculation was multiplied by 100 to express the RQD in percent. Breaks caused by drilling operations were ignored and the resulting pieces were counted as intact lengths.

Sheet 1 of 2

US 60 / US 231 OVER THE OHIO RIVER SHEET

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

COUNTY OF
DAVIESS
US 60 / US 231
ROAD

STATION 899+45.00
P.E. PROJECT NO.

22535

Rev. 5

SUBSURFACE LOG

unty: DAVIESS Item No.: - -	Location: PIER 8, STA.893+69, Rt.75
Fed. Project No.: KBD 10-1(2)	Hole Number: SHAFT 44 Total Depth: 154
State Project No.: FSP 030 0231 017-018 C	Date Started: 6/22/92 Date Completed: 6/23/9
Road Name: US 231	Depth to Water Immediate: - - ft.
Surface Elevation: 359.4 (WATER)	Depth to Water: - - ft. Date: - -
Project Type: BRIDGE OVER OHIO RIVER	Driller: LAW, NASHVILLE Geologist: M. BLEVINS

Lithology Elev.	Lithology Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std.RQD) REMARKS
268.4	91.0							
-4.9	104.5	Shale - gray to black, clayey with sandstone lamination that decrease with depth	0	9.0	8.9	99		(32)
53.7	105.7	Coal (1.0') - with underclay						
247.8	111.6	Shale - light gray, silty and sandy with a few clay shale partings at 110.0 to 111.6'	0	10.0	10.0	100		(49)
245.6	113.8	Shale - gray, very clayey						
240.0	119.4	Shale - light gray, sandy with clay shale laminations	8	10.0	10.0	100		(47)
233.5	125.9	Shale - grayish brown and red, clayey, broken with slickensides						
225.3	134.1	Shale - gray, silty and clayey with slickensides	0	10.0	10.0	100		(8)
219.4	140.0	Shale - light gray, sandy	5	10.0	10.0	100		(32)
206.1	153.3	Shale - dark gray to black, slightly carbonaceous and clayey with sandy laminations	0	10.0	10.0	100		(49)
205.4	154.0	Limestone - dark gray w/fossils	25	4.0	4.0	100		(95)

Depth scale: 100, 110, 120, 130, 140, 150

END CORE

KENTUCKY TRANSPORTATION CABINET
DIVISION OF MATERIALS
GEOTECHNICAL BRANCH

Rev. 5

SUBSURFACE LOG

County DAVIESS Item No. - -	Location PIER 8, STA.893+21, Rt.75
Fed. Project No. KBD 10-1(2)	Hole Number SHAFT 41 Total Depth 144
State Project No. FSP 030 0231 017-018 C	Date Started 6/17/92 Date Completed 6/18/9
Road Name US 231	Depth to Water Immediate - - ft.
Surface Elevation 357.6 (WATER)	Depth to Water - - ft. Date - -
Project Type BRIDGE OVER OHIO RIVER	Driller LAW, NASHVILLE Geologist M. BLEVINS

Lithology Elev.	Lithology Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std. RQD) REMARKS
266.6	91.0							
264.8	92.8	Shale - dk gray to black, clayey						
253.9	103.7	Shale - dark gray, clayey and sandy with sandstone laminations in zones	0	9.0	8.0	89		(4)
252.4	105.2	Coal (1.0')	0	10.0	9.6	96		100 (24)
244.5	113.1	Shale - gray, silty and clayey						
240.3	117.3	Shale - gray, silty with coarse sand grains	0	10.0	10.0	100		110 (26)
235.3	122.3	Shale - gray, silty and clayey with slickensides						
229.0	128.6	Shale - gray, silty with sandstone laminations	10	10.0	8.7	87		120 (38)
226.9	130.7	Limestone - gray w/shale laminations						
		Shale - gray to black, clayey to silty with sandy zones, grading to a gray to black silty shale at 138.5'	0	10.0	9.8	98		130 (32)
213.6	144.0		0	4.0	3.6	90		140 (0)
		END CORE						

DIVISION OF MATERIALS

GEOTECHNICAL BRANCH

Rev. 5

SUBSURFACE LOG

County DAVIESS Item No. - -	Location PIER 8, STA.893+69, Centerline
Fed. Project No. KBD 10-1(2)	Hole Number SHAFT 24 Total Depth 150.0
State Project No. FSP 030 0231 017-018 C	Date Started 6/23/92 Date Completed 6/24/9
Road Name US 231	Depth to Water Immediate - - ft.
Surface Elevation 359.3 (WATER)	Depth to Water - - ft. Date - -
Project Type BRIDGE OVER OHIO RIVER	Driller LAW, NASHVILLE Geologist M. BLEVINS

Lithology Elev.	Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std. RQD) REMARKS
269.3	90.0							
		Shale - dark gray, clayey with many sandstone laminations	0	10.0	10.0	100	14 @ 92' 46 @ 97'	(27)
[illegible]2.2	107.1		0	10.0	10.0	100	59 @ 102' 80 @ 110'	100 (14)
250.8	108.5	Coal						
241.6	117.7	Shale - gray, silty and clayey with coarse sand grains in zones	0	10.0	10.0	100	77 @ 115' 3 @ 119'	110 (46)
237.4	121.9	Shale - gray, clayey with many slickensides	0	1.5	0.6	40		120 121.5 (0)
234.4	124.9	Shale - brown and gray, clayey w/ many slickensides	0	2.0	2.0	100	6 @ 123'	123.5 (0)
			0	1.5	1.0	67		(0) 125
229.6	129.7	Shale - gray, silty with sandy zones and clay shale partings	18	6.5	6.2	95	28 @ 127'	(43)
227.9	131.4	Limestone - gray w/shale laminations						131.5
		Shale - gray, clayey with sandy zones and partings, grading to a black carbonaceous shale at 137.8'	0	10.0	10.0	100	75 @ 134' 52 @ 140'	(26)
209.3	150.0		0	8.5	8.5	100	80 @ 145' 85 @ 149'	141.5 (87)
		END CORE						

Rev. 5

SUBSURFACE LOG

County DAVIESS Item No. - -
Fed. Project No. KBD 10-1(2)
State Project No. FSP 030 0231 017-018 C
Road Name US 231
Surface Elevation 357.6 (WATER)
Project Type BRIDGE OVER OHIO RIVER

Location PIER 8, STA.893+21, Centerline
Hole Number SHAFT 21 Total Depth 149.0
Date Started 6/18/92 Date Completed 6/19/9
Depth to Water Immediate - - ft.
Depth to Water - - ft. Date - -
Driller LAW, NASHVILLE Geologist M. BLEVINS

Lithology Elev.	Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std. RQD) REMARKS
266.6	91.0							
265.2	92.4	Shale-dk.gray to black, clayey						
		Shale-dark gray, clayey & sandy w/zones of sandstone laminations	0	9.0	9.0	100		(43)
								100
250.5	107.1		0	10.0	9.8	9.8		(42)
249.0	108.6	Coal (1.3')(0.2')						
								110
237.0	120.6	Shale-gray, silty to sandy w/sandstone laminations	0	10.0	10.0	100		(77)
								120
230.6	127.0	Shale-gray silty & clayey w/a sandy zone 124.6'-126.2'	0	7.0	7.0	100		(61)
229.2	128.4	Shale - greenish gray, clayey, slickensided						127
227.9	129.7	Limestone-gray w/Shale Laminations						
220.8	136.8	Shale-gray, clayey, to silty w/sandy zones (or calcareous)	10	10.0	10.0	100		(47)
								137
		Shale-gray to black, silty	0	7.0	5.9	84		(34) Lost Core Pulling Out
								144
208.6	149.0		0	5.0	5.0	100		(52)
		END CORE						

KENTUCKY TRANSPORTATION CABINET
DIVISION OF MATERIALS
GEOTECHNICAL BRANCH

Rev. 5

SUBSURFACE LOG

County DAVIESS Item No. - -	Location PIER 8, STA.893+69, Lt.75
Fed. Project No. KBD 10-1(2)	Hole Number SHAFT 4 Total Depth 144.4
State Project No. FSP 030 0231 017-018 C	Date Started 6/24/92 Date Completed 6/24/9
Road Name US 231	Depth to Water Immediate - - ft.
Surface Elevation 358.4 (WATER)	Depth to Water - - ft. Date - -
Project Type BRIDGE OVER OHIO RIVER	Driller LAW, NASHVILLE Geologist M. BLEVINS

Lithology Elev.	Lithology Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std.RQD) REMARKS
269.9	88.5							
		Shale - dark gray, clayey with sandy laminations and partings, increasingly silty below 103.9'	0	1.5	1.5	100		90 (67)
			0	10.0	10.0	100		(34)
252.5	105.9							100
251.2	107.2	Coal - (1.3')	0	10.0	10.0	100		(44)
		Shale - light gray, silty with sandy zones, becoming very sandy below 112.6'						110
242.2	116.2		9	6.8	6.8	100		(75)
240.3	118.1	Shale -dark gray, clayey; very slickensided						116.8
236.2	122.2	Shale - brown to gray, clayey with a/few slickensides	0	3.2	3.2	100		(25) 120
232.1	126.3	Shale - gray, clayey to silty	0	5.7	5.7	100		(63) shale expands badly
230.9	127.5	Limestone - gray w/shale laminations						125.7
		Shale - gray, clayey to silty with sandy zones above 131'	26	4.3	4.3	100		(65)
223.6	134.8							130
		Shale - gray to black, silty and carbonaceous	0	10.0	10.0	100		(43)
4.2	144.2							140
	144.4	Limestone - dk gray, fine grained	0	4.0	4.0	100		(45)
214.0		END CORE						

KENTUCKY TRANSPORTATION CABINET

DIVISION OF MATERIALS
GEOTECHNICAL BRANCH

Rev. 5

SUBSURFACE LOG

County DAVIESS Item No. - -
Fed. Project No. KBD 10-1(2)
State Project No. FSP 030 0231 017-018 C
Road Name US 231
Surface Elevation 357.6 (WATER)
Project Type BRIDGE OVER OHIO RIVER

Location PIER 8, STA.893+21, Lt.75
Hole Number SHAFT 1 Total Depth 147.5
Date Started 6/25/92 Date Completed 6/25/9
Depth to Water Immediate - - ft.
Depth to Water - - ft. Date - -
Driller LAW, NASHVILLE Geologist M. BLEVINS

Lithology Elev.	Lithology Depth	Description (Overburden / Rock Core)	Sample No. / KY RQD	DEPTH / Run	Rec. (ft.) / Rec. (ft.)	SPT BLOWS / Rec. (%)	SAMPLE TYPE / SDI (%)	(Std. RQD) REMARKS
270.6	87.0							
268.6	89.0	Shale - augered						
		Shale - gray to black, clayey with sandstone laminations above 100', grading to a silty carbonaceous shale below 105'	0	1.0	0.8	80		90 (0)
			0	9.0	9.0	100		(51)
			0	1.0	1.0	100		99 (0) 100
250.2	107.4		0	10.0	10.0	100		(26)
248.8	108.8	Coal						110
242.6	115.0	Shale - light gray, silty and sandy with coarse sand grains	0	5.5	5.5	100		(73) 115.5
237.2	120.4	Shale - gray to brown, clayey with slickensides	0	4.5	4.5	100		(60) 120
232.0	125.6	Shale - light gray, clayey and silty with a few zones of coarse sand grains	0	5.0	5.0	100		125
231.0	126.6	Limestone - gray w/a few shale laminations	16	5.0	5.0	100		(58) 130
219.0	Sh. Tip 138.6	Shale - gray, clayey with sandy zones above 130.6'; grading to a black, silty, carbonaceous shale below 133.4'	0	10.0	10.0	100		(25) 140
211.9	145.7							
211.4	146.2	Limestone - dark gray, fine grained with fossils	12	7.5	7.5	100		(57)
210.1	147.5	Shale - dark gray, clayey						
		END CORE						

REPORT OF
GEOTECHNICAL INVESTIGATION
OHIO RIVER BRIDGE AT OWENSBORO, KENTUCKY
DAVIESS COUNTY, KENTUCKY & SPENCER COUNTY, INDIANA

PREPARED FOR

BURGESS & NIPLE, LTD.
ARCHITECT & ENGINEER

FEBRUARY, 1990

INVESTIGATION BY
THE H. C. NUTTING COMPAN

THE H. C. NUTTING COMPANY
GEOTECHNICAL, GEO-ENVIRONMENTAL AND TESTING ENGINEERS
SINCE 1921

CORPORATE CENTER
4120 AIRPORT ROAD
CINCINNATI, OHIO 45226
(513) 321-5816

June 7, 1990

Order No. 70131.007 bj

Mr. Raymond Grover, P.E.
Burgess & Niple, Ltd.
5085 Reed Road
Columbus, Ohio 43220

Re: Geotechnical Report
Ohio River Bridge at
Owensboro, Kentucky

Dear Ray:

In accordance with our recent discussion, we are submitting herewith 4 bound and 1 unbound copy of the report for the reference project together with 10 mylar sheets which represent the geotechnical drawings.

We appreciate the opportunity to work with you. If any questions develop as you proceed with the project, please contact me.

Very truly yours,

THE H. C. NUTTING COMPANY

C. R. Lennertz, P.E.
Chief Engineer

THE H. C. NUTTING COMPANY
GEOTECHNICAL, GEO-ENVIRONMENTAL AND TESTING ENGINEERS
SINCE 1921

BLUEGRASS REGION
1445 JAMIKE DRIVE
ERLANGER, KENTUCKY 41018
(606) 283-9914

June 7, 1990

Order No. 70131.007 bj

Burgess & Niple, Ltd.
2734 Chancellor Drive
Suite 205
Crestview Hills, Kentucky - 41017

Attn: Mr. Raymond Grover

Re: Geotechnical Report
Ohio River Bridge at
Owensboro, Kentucky

Gentlemen:

We are pleased to submit herewith the preliminary geotechnical report for a proposed new bridge over the Ohio River near Owensboro, Kentucky.

The field investigation included seventeen test borings made along the proposed alignment identified as Alternative A-2. This alignment extends from existing U.S. Rt. 60 in Kentucky to existing U.S. Rt. 231 in Indiana and crosses the Ohio River just east of Rockport, Indiana.

The work included five test borings made from a barge in the Ohio River, one test boring at the intersection of U.S. 231 and Indiana Rt. 66 and eleven test borings in Kentucky. Appropriate laboratory tests were performed on representative samples obtained in these borings.

This report includes detailed logs of each of the seventeen borings, data sheets for laboratory tests and geotechnical discussions and recommendations. In addition to the material found within this report, we have prepared ten sheets of geotechnical drawings which include a graphic log for each of the seventeen test borings together with a summary of the laboratory test data.

We appreciate the opportunity of working with you on this project.

Respectfully submitted,

THE H. C. NUTTING COMPANY

C. R. Lennertz

C. R. Lennertz, P.E.
Chief Engineer

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PROJECT

PROJECT LOCATION

The proposed project is to construct a new bridge and its approaches over the Ohio River. The project is to be located northeast of Owensboro, Kentucky and will connect U.S. Rt. 60 in Kentucky with U.S. Rt. 231 in Indiana. Figure 1 shows the general location of the proposed project with respect to major roads and cities. Figure 2 shows the location of the proposed project with respect to county landmarks. The bridge will provide an improved route from the Owensboro area to Interstate 64 in Indiana.

PROJECT DESCRIPTION

The project starts approximately 0.6 miles southwest of the intersection of U.S. Rt. 60 and Rockport Ferr Road in Daviess County, Kentucky. The project begins with an at-grade intersection of the bridge approach embankment with U.S. Rt. 60. Approximately 1800 ft. of U.S. Rt. 60 will be widened to accommodate turning traffic movement at the intersection. From this location it is proposed to construct a road embankment to provide access to the proposed bridge. The proposed embankment on the Kentucky side of the Ohio River is approximately 3.3 miles long and reaches a maximum height of approximately 37 ft. The proposed embankment includes up to 130 ft. long structures over 5 creeks. The roadway embankment approach to the bridge in Indiana is approximately 0.2 miles long and reaches a maximum height of approximately 25 ft.

The width of the Ohio River channel at the proposed bridge site is approximately 2900 ft. The main river crossing, beginning on the southern Ohio River bank in Kentucky and extending to the north side of U.S. Rt. 66 in Indiana, is comprised of 14 approach spans in addition to the main 3 span unit. Three of the four main bridge piers and five approach span piers will be constructed in the river.

GEOLOGY OF THE OWENSBORO AREA

GEOLOGICAL HISTORY OF THE OHIO RIVER VALLEY IN THE OWENSBORO AREA

The Ohio River flows in a broad valley that was carved out of Pennsylvanian age rock by a preglacial river. The course of this ancient river meanders back and forth across the present valley and passes through the City of Owensboro. The valley has been filled with Wisconsin age glacial outwash deposits which are up to 170 ft. thick. The outwash deposits are primarily sand and gravel, generally moderately compacted, with some silt and clay layers that are significantly over-consolidated.

The glacial age outwash deposits are overlain by recent alluvium and overbank deposits that have been measured up to approximately 60 ft. thick. The recent alluvium and overbank deposits consist mostly of fine sand, silt and silty clay with lesser amounts of fine to coarse sand and a minor amount of gravel.

The bedrock is of middle Pennsylvanian age identified as Carbondale and Tradewater Formations. Sandstone is most prevalent, however, the formation includes shale and siltstones with some thin coal beds. The elevation of the top of rock varies significantly in the area, with the top elevation at specific locations relating to the ancient stream course, and the existing upland areas that rise above the flood plain. Rock elevations as low as 210 have been measured close to the mapped area of the ancient stream course. Sandstone cliffs rise above elevation 400 along the valley wall within the City of Rockport.

The relationship of the bedrock surface to the present topography and its impact to the present river channel can best be seen from a study of the Owensboro East and Rockport quads. The quad sheets dramatically show the influence of the bedrock hill at Rockport with present channel widths in the range of 2500 to 2800 ft. upstream and downstream fro- Rockport, while the channel narrows to about 1700 ft. at Rockport.

Preglacial Pliocene deposits have been previously mapped in the area and were penetrated by borings referred to in this study. These deposits represent alluvium deposited through a long period of erosion prior to the invasion by glacial age outwash. The Pliocene deposits include sand and gravel beds resting on bedrock benches above the level of the present river and on lower benches that have been buried by glacial material.

The principal soils that will impact this project are valley train and lacustrine deposits associated with Wisconsin age glaciation and recent alluvium and overbank deposits. Geologists have mapped Wisconsin age deposits into Pretazewell, Tazewell, and Cary from oldest to youngest. During each invasion of the Ohio drainage basin by continental ice sheets, meltwater and debris produced a valley train. During each withdrawal of the ice sheets, the valley train deposits were dissected and partly removed by stream degradation. Because aggradation during glaciation exceeded interglacial degradation, alluvium gradually filled the preglacial bedrock valley to a maximum depth of about 200 ft.

An outcrop of Pretazewell deposits, named the Beds at Hubert Court, consists of fine silty sand overlain by clayey, humic, fossiliferous silts and silty clay. This outcrop, exposed for a distance of several hundred feet along the Ohio River in the eastern part of Owensboro, is the only documented occurrence of Pre-Tazewell Wisconsin deposits in the Owensboro area.

The most extensive ice sheet of Wisconsin age was named the Tazewell. This deposited the highest and most widespread valley train. Small tributary streams were dammed by debris accumulating within the main valley, resulting in significant deposits of backwater sediment. Low bedrock divides were buried. During this period, the Ohio River shifted its course from the ancient bedrock channel which passes south through the present City of Owensboro to the present channel north of Bon Harbor Hills. Upstream, the river shifted its course from the eastern valley wall to impinge against the bedrock at Rockport.

Retreat of the Tazewell ice sheet initiated a period of stream degradation with consequent dissection of the valley train and the lacustrine deposits in the lower courses of the tributary valleys. The surface of these eroded remnants of deposits of Tazewell age now form the extensive terrace on which the City of Owensboro is located.

During Cary glaciation, degradation was supplanted by aggradation and a valley train again developed. Because of the limited expansion of the ice sheet within the Ohio drainage basin, its valley train deposits failed to refill the valley to the level of the Tazewell deposits by some 15 to 20 ft. The surface of the Cary deposits is generally near or slightly above elevation 380 as compared to elevations of 400 to 410 for the Tazewell age terrace.

Outwash in the Wabash Valley intersects the Ohio approximately 50 miles downstream and may have had significant impact on deposits in the study area during the Cary period. During recession of the Cary ice sheet, the Wabash River carried torrents of meltwater from a spill-over of the glacial Lake Maumee. Aggradation of sediments at the mouth of the Wabash River resulted in ponding of the Ohio River. This resulted in a mantle of alluvial-deltaic type deposits of fine sand, silts, and clays over the Cary granular outwash. The amount of aggradation from this event in the Owensboro area is not known, but it is believed to have been relatively insignificant.

Following the effects of the Cary ice sheet, the Ohio River again became a degrading stream in this area. Valley train deposits of Cary age were dissected leaving a low poorly defined terrace rising only a few feet above the broad flood plain that marks the final episode and geomorphic history of the river. In post Cary time, the river, through limited incision and lateral shift, produced a broad flood plain marked by extensive point bars. These bars are especially well developed north of Owensboro where swell (scroll) and swale topography is prominent. The surface of flood plain alluviation defined by the high points on the scrolls which represent natural levees and alluvial islands indicate that alluvial channel deposits are only a few feet below the frequently flooded Cary age terrace. Flooding of the Cary age terrace has resulted in reworking of the glacial age materials and covering of the glacial age materials with post-glacial overbank deposits.

BEDROCK GEOLOGY

The bedrock is of middle Pennsylvanian age identified as Carbondale and Tradewater Formations. Sandstone is most prevalent, however, the formation includes shale and siltstones with some thin coal beds.

STRUCTURAL GEOLOGY

The general site area is located structurally on the eastern flank of the Illinois basin and is bounded on its northeast side by the Cincinnati Arch, on the west side by the north-northeast trending Wabash Valley Fault System, and on the south by the east-westward trending Rough Creek Fault System. The described area includes parts of Daviess and Hancock Counties of Kentucky and Spencer and Perry Counties of Indiana. Figure 3 shows the general location of fault systems in western Kentucky and Indiana. Faults in the general site area are not thought to be closely related to the Rough Creek Fault Zone of Kentucky and their relationship to the Wabash Valley fault system is at best unclear.

Both the Wabash Valley Fault System and faults in the general site area are post-Pennsylvanian. Faults in the general site area are thought to be pre-Pleistocene in age and thus represent an ancient fault system apparently unrelated to recent seismic activity in the region. Faults of the Wabash Valley Fault System in Indiana occur as well defined compound faults usually less than 0.5 miles wide but as much as 1.5 miles wide. The faults are normal type, and strata may be down thrown either westward or eastward. Faults in the general site area are similar in nature to those in the Wabash Valley Fault System in that both trend generally to the northeast and are all of normal type with high angle dips on the fault planes.

In general, the faults in Spencer and Daviess Counties are downthrown eastward, and the faults in Perry and Hancock Counties are downthrown westward. Outcrops near faults in Perry County showed steeply dipping inclined bedding, some dipping in opposite directions than would be expected from normal drag features near faulting. The erratically dipping strata suggests that faulting is indeed complex with more than one fault plane occurring within a zone.

At least one fault passes beneath the southeastern portion of the proposed project. The Africa Fault extends across the Ohio River from Daviess County, Kentucky into Perry County, Indiana, striking approximately N. 40° E at a location approximately one mile west of Owensboro. Approximately 3 miles into Indiana, the fault changes direction, striking approximately N. 20° E., and crosses the Ohio River again, reentering Daviess County, Kentucky. The maximum vertical displacement of the Africa Fault appears to be about 70 ft. Throw on the fault decreases with depth which suggests that the fault dies out with depth.

SEISMOLOGY

The proposed bridge site is located in a seismically active region commonly referred to as the Central Mississippi Valley seismic region, which is considered to be the most seismically active region in the central United States.

Figure 4 is a Modified Mercelli intensity map for the United States and shows the site area is located on the border of Zones

2 and 3. This indicates that there is risk for an earthquake at VII on the Modified Mercelli scale and that moderate to major damage should be anticipated.

Figure 5 shows the location and frequency of earthquakes in the entire Mississippi Valley Seismic Region. The earthquakes are deep-seated and none are thought to be associated with the Paleozoic faulting of the Wabash Valley Fault System or those faults in the general site area.

Figure 6 shows a cumulative frequency vs. intensity plot for the entire Mississippi Valley Region. Review of Figure 3 showing epicentral locations indicates that the general site area is less susceptible to earthquakes than the overall Mississippi Valley Region.

Figure 7 shows the generalized isoseismal map of the December 16, 1811 New Madrid Earthquake. Evident from the map is diminishing intensities from the epicenter region to the general site area.

The project is located within the seismic performance category Zone B, having an acceleration coefficient equal to 0.10 to 0.20, and a Type III soil profile, per "Seismic Design Guidelines for Highway Bridges"; Report No. FHWA/RD-81/081.

DRILLING AND SAMPLING

Seventeen test borings were made by The H. C. Nutting Company during November and December of 1989. Eleven of the borings were made on the Kentucky side of the river along the line of the proposed embankment and structurally supported bridge approaches. One boring was made on the Indiana side of the Ohio River along the line of the proposed embankment. Five borings were made from a barge-mounted drill rig in the Ohio River along the line of the proposed bridge.

Borings on the Kentucky side of the river were extended to depths ranging from 31.5 ft. to 130.3 ft. Two of the borings on the Kentucky side of the river were taken to auger refusal in bedrock at depths of 130.3 ft. and 115 ft. The one boring drilled on the Indiana side of the river was taken to auger refusal in bedrock which was encountered at a depth of 115 ft. The five borings drilled in the Ohio River were taken to bedrock which was encountered at depths ranging from 85.5 to 97.5 ft. below the Ohio River pool. Split spoon samples at all of the borings were collected at intervals of 2.5 ft. for the upper 10 ft. of soil encountered and at intervals of 5 ft. for the remaining depth of the soil profile. In addition to split spoon sampling, 19 to 23.5 ft. of bedrock was cored in each of the river borings.

Test boring samples were returned to our Soil Mechanics Laboratory where they were examined by the Project Engineer. Laboratory tests were performed on selected representative samples to provide specific data for classifying the soils in accordance with the Unified Soil Classification System. Additional laboratory tests were performed for evaluating moisture and density, bearing capacity, and settlement properties of the encountered materials. Laboratory tests performed included moisture content, sieve, hydrometer, Atterberg limit, unconfined compression, consolidation, and triaxial tests. The following table summarizes the laboratory tests performed as a part of this investigation.

Test Type	No. of Tests
Complete Classification	11
Sieve Analysis	31
Atterberg Limit	10
Unconfined Compression	9
3-Point Triaxial	1
Consolidation	2
Moisture Content	45

All of the classification test data are summarized on the soil profile and soil cross-section sheets. The detailed data developed in the moisture content, unconfined compression, consolidation, and 3-point triaxial tests are summarized in the appendices.

SUBSURFACE INVESTIGATION

Soil classification data developed during the investigation is presented on geotechnical drawings presented as profiles along the length of the project. Laboratory test data is presented in tabular form in the appendix.

GEOMORPHOLOGY

The site of the proposed bridge and roadway embankment is located in the flood plain of the present-day Ohio River. The project starts at U.S. Rt. 60 in Kentucky. U.S. Rt. 60 follows the edge of the Tazewell terrace at this location. In order to construct an at-grade intersection an embankment is required with a height equal to the difference in elevation between the Tazewell terrace and the flood plain of the present Ohio River. The project ends on the flood plain of the Indiana side of the Ohio River.

Louis G. Ray (1965) interpreted the topography of the Ohio River flood plain on the Kentucky side of the river from U.S. Rt. 60 to Station 810+00 as being inherited from erosion and reworking of a previously existing Cary terrace by flood waters of the recent Ohio River system. The topography from Station 810+00 to the present bank of the Ohio River was interpreted as scroll and swale topography inherited from northwesterly migration of the present Ohio River.

RECENT ALLUVIAL AND OVERBANK DEPOSITS

Subsurface evidence developed in the present investigation shows that the fine poorly graded sands of the present alluvial system can be traced further to the southeast than previously interpret-ed by Ray (1965) from the topography. Therefore, it appears the elevation and extent of the Cary terrace is even more obscure than previously imagined. The alluvial sands are overlain by 15 to 25 ft. of overbank deposits part of which were previously interpreted by Ray to be reworked Cary age deposits. Beyond the southeastern limit of the alluvial sands the overbank deposits may mimic the previous Cary terrace as suggested by Ray (1965) because the topography in this area has an appearance similar to that of a braided fluvial system.

The recent overbank deposits consist of brown to gray lean clay with minor amounts of fat clay, sand, silt and clayey sand. The scrolls of the overbank deposits represent natural levees and are often capped by materials containing greater proportions of sand than materials found in the swales. In swales, such as at the location of boring B-9 at Station 831+00, the overbank materials have a tendency to be soft. Otherwise, the overbank deposits are generally medium stiff to stiff.

The poorly graded sands of the recent alluvial system can be correlated laterally from boring B-4 at Station 760 to boring B-21 at Station 920+00 on the Indiana side of the present Ohio River channel. The poorly graded sands of the recent alluvial deposits can be differentiated from the underlying outwash deposits on the basis that they are generally finer grained and contain lesser amounts of erratics and silt mixed with coarse sand. Differentiation on of the recent alluvial deposits with glacial outwash is difficult in the vicinity of the present river channel, perhaps due to the effects of dredging and reworking of outwash deposits. The recent alluvial sands and silts are 5 to 20 ft. thick on the Kentucky side of the river and thicken to as great as 60 ft. beneath the upper point bar of the present Ohio River channel. Towards the lower point bar of the present channel, the recent alluvial deposits thin. In boring B-18 the thalweg of the present Ohio River channel appears to have completely truncated the surface of the glacial outwash. On the Indiana side of the Ohio River, north of the present cut bank, the recent alluvial deposits are approximately 20 ft. thick.

On the Kentucky side of the Ohio River, the recent alluvial sand deposits are medium dense to very loose and are mostly loose. On the Indiana side of the Ohio River, the recent alluvial deposits penetrated medium dense. Groundwater was encountered at depths of approximately 5 to 30 ft. on either side of the river.

In the area of the point bar of the present Ohio River channel the alluvial deposits vary from loose to very dense. A break in the density of the fine sands making up the point bar is observed in the upper 13.5, 14.0, and 13.0 ft. of borings B-14, B-15, and

B-16, respectively, at an elevation of approximately 329. The sands above an elevation of 329 to 330 are loose and somewhat finer grained than the underlying medium dense to very dense fine alluvial sands. The loose sands above elevation 330 in addition to being loose and finer in texture, contain a greater proportion of sand grains of low specific gravity derived from carbonaceous rocks such as black shale, coal, and carbonaceous shale. The shallow loose sands are interpreted to have been deposited on a surface created by machine dredging during modern times. Their texture is finer and they are less dense because they were deposited in the relatively quiet water of the Ohio River subsequent to pooling of the Ohio River by man-made dams. The modern shallow loose deposits are identified on the profile drawings. The modern alluvium was not sampled in boring 17. The thalweg of the Ohio River bed at the location of boring 18 is at elevation 330, below the modern alluvium. Recent alluvium was not identified at this location either and it appears the thalweg has truncated the outwash.

WISCONSIN AGE VALLEY-TRAIN GLACIAL OUTWASH

Directly beneath the recent poorly graded fine alluvial sands and also beneath the reworked Cary age glacial outwash deposits at the southeastern part of the project, a thick sequence of undifferentiated Wisconsin age valley train outwash was encountered. The outwash is generally coarser grained, generally contains greater proportions of silt mixed with coarse sand, and contains more erratics than the overlying units. Contained within the glacial outwash are lenses of poorly graded fine sands, silty sands, and silts that are generally less than 10 ft. thick with limited continuity. These lenses were encountered most commonly at elevations between 270 and 310. Approximately 30 ft. of poorly graded fine sand was encountered at an elevation of approximately 337 in boring B-1 at Station 715+00. This unit of fine sand is thicker and appears to have greater lateral continuity than the lenses found along the length of the project between elevations of 270 and 310. This thicker more continuous unit of fine sand could be correlated as far north as boring B-9 at Station 831+00 where it is much thiner, approximately 15 ft. Beyond boring B-9 it is absent. Approximately 25 ft. of lacustrine lean clay was encountered from elevation 302 to 272 in boring B-1 at Station 715+00. This unit is likely the result of sedimentation in backwater during Tazewell time.

The fine to coarse sands which constitute by far the greatest proportion of the valley train deposits are loose to very dense in approximately the upper 20 ft. At greater depths the fine to coarse sands are medium dense to dense and are, for the most part, very dense. The thin fine sand, silty sand, and silt lenses encountered at elevations between 270 and 310 along the length of the project are medium dense to very dense. The thicker and more continuous fine sand encountered along the southern portion of the project at an elevation of approximately 337 is dense to very dense except where it thins at Station 831+00 in boring B-9. At this location the fine sand is loose to medium dense. The 25 ft. of lakebed clay encountered in boring B-1 at Station 715 is soft to medium stiff.

In the immediate vicinity of the present Ohio River channel boring B-14 through B-18 completely penetrated the glacial outwash consisting predominately of fine to coarse sand with silt. Glacial outwash encountered in these borings was dense to very dense except in upper portions in some of the borings and within some fine sand seams where loose to medium dense materials were encountered.

In boring B-14 the glacial outwash is very dense with the exception of a 5 ft. thick medium dense fine sand seam encountered at elevation 291. Another fine sand seam was encountered in boring B-14, 15 ft. thick at elevation 306 and this sand seam was very dense. In boring B-15 the glacial outwash is dense to very dense. A 5 ft. thick bed of dense fine sand was penetrated at elevation 290.5. In boring B-16 the glacial outash is dense to very dense and no fine sand seams were encountered. In boring B-17 the upper 5 ft. of glacial outwash is medium dense. Also, a 10 ft. thick medium dense sandy silt seam was encountered at elevation 290. The remaining majority of outwash penetrated in boring B-17 was dense to very dense. Boring B-18 penetrated 9.5 ft. of loose glacial outwash directly beneath the bed of the Ohio River. At elevation 305, boring B-18 penetrated 5 ft. of medium dense fine sand. The remaining majority of glacial outwash in boring B-18 was dense to very dense.

PRE-GLACIAL OHIO RIVER VALLEY AND EXISTING BEDROCK SURFACE

The bedrock topography defines the configuration of the preglacial deep stage Ohio River valley. Data made available through oil well drilling shows the deepest part of the ancient Ohio River valley passes beneath the project at approximately Station 710+00. At Station 710+00 the elevation of bedrock is approximately 210 ft. The elevation of the top of bedrock rises steadily from the southern end of the project along the project line to Station 877+87 at the southern bank of the Ohio River where boring B-13 encountered bedrock at elevation 273. From the southern bank of the Ohio River to Station 920+00 on the northern bank of the Ohio River, bedrock was encountered between elevations 264.6 and 274.8, thus defining a bedrock plateau that once flanked the deepest cut part of the ancient Ohio River valley.

Boring B-21 at Station 920+00 on the Indiana side of the Ohio River penetrated coarse sand and gravel contianing angular rock fragments of the same lithology as the country rock directly above bedrock. Similar material was encountered in boring B-17 immediately above the top of bedrock. This material is interpreted as preglacial alluvium deposited in the bed of the ancient Ohio River.

PENNSYLVANIAN AGE FLUVIO-DELTAIC DEPOSITS

The bedrock was cored in borings made at the location of the present Ohio River channel. The amount of core taken in each of these borings ranged from 19 to 23.5 ft. Lithologies encountered were typical of Pennsylvanian age, fluvio-deltaic type deposits. In general, the rock sequence encountered consisted of a basal unit of a well indurated gray claystone and siltstone, underclay which is heavily bioturbated. Overlying the underclay, a coal seam was encountered consisting of the coal lithotype clarain which is characterized by having a semi-bright silky luster with dull intercollations and a blocky appearance when fractured. Directly above and beneath the coal bed a thin layer of carbonaceous shale which parts readily along bedding planes was often encountered. Gray shale generally overlies the coal and carbonaceous shale. The gray shale parts readily along bedding planes and generally coarsens upwards into siltstones containing laminations of fine sandstone. At the top of the sequences sandstone generally becomes abruptly predominant. The sandstone generally contains thin shale and siltstone laminations. The general sequence described is not always complete and often one or more lithologic units are entirely missing.

High angle joints were encountered in some of the sandstone beds, indurated claystone and siltstone underclay unit, and also within the coal unit. Soft seams were encountered in some of the underclay units.

The gray shale showed a low resistance to slaking when submersed in water. An unconfined compression test performed on the gray shale showed a compressive strength of 452 tsf. An unconfined compression test performed on gray silty shale and siltstone with fine sandstone laminations showed an unconfined compressive strength of 439 tsf.

The following table summarizes top of bedrock elevations, elevations at which coal seams were encountered, and the amount of rock cored in borings B-13 through B-21. Also, the minimum thickness of rock above the base of the underclay is given for those borings that encountered coal and underclay. The underclay was not completely penetrated in any of the borings.

Boring	Elevation of Top of Rock	Thickness of Rock Cored	Elevation of Coal Seam	Thickness of Coal Seam	Thickness of Rock above Coal Seam	Minimum Thickness of Rock above Base of Underclay
13	272.6'	---	---	---	---	---
14	264.6'	22.0'	244.6'	2.25'	20.0'	24.5'+
15	265.5'	19.0'	252.5'	2.00'	13.0'	20.5'+
16	268.0'	21.5'	251.5'	1.75'	16.5'	16.5+
17	274.3'	23.5'	---	---	---	---
18	274.8'	19.0'	---	---	---	---
21	274.6	---	---	---	---	---

CONCLUSIONS AND RECOMMENDATIONS

The road embankment is to be constructed from the start of the project at U.S. Rt. 60 for a distance of 3.3 miles to the southeastern abutment of the Ohio River Bridge. The embankment reaches a maximum height of approximately 37 ft. The proposed embankment includes five bridges to connect breaks in the embankment that allow passage of the several creeks that traverse the broad Ohio River flood plain. The proposed roadway embankment on the Indiana side of the river is approximately 0.2 mile long and reaches a maximum height of 25 ft.

The main river crossing beginning on the southern Ohio River bank in Kentucky and extending to the north side of U.S. Rt. 66 in Indiana, is comprised of 14 approach spans in addition to the main 3-span unit. Three of the four main bridge piers and five of the approach span piers will be constructed in the river.

RIVER PIERS

It is our opinion that the depth of the bedrock surface beneath the bed of the Ohio River channel is too deep to consider founding the piers directly on bedrock. Accordingly, we have considered the geotechnical aspects of the following three basic types of foundations for the river piers.

H-Piles
Drilled Shafts
Mat/Spread Footings

Preliminary scour analysis indicates a design scour elevation of 324, however, a somewhat greater scour depth may be required for the north pier. A pile foundation system would consist of a mat supported on H-piles. The piles would be driven inside a cofferdam with a tremie seal subsequently constructed to permit dewatering and construction of the foundation mat. The piles would include both battered and straight piles. The piles would be driven to practical refusal on bedrock. The borings do not indicate any conditions that would suggest a problem in the installation of the piles.

We recommend that the capacity of H-piles be based on an allowable stress of 9000 psi of cross-sectional area under gravity loads. The allowable pile load may be increased by 50% when considering the worst combination of loads, including barge impact. We recommend that pile estimates be based on the following tip elevations at the locations of the five river borings.

Boring No.	Estimated H-Pile Tip Elevation
14	262'
15	263'
16	266'
17	272'
18	272'

Drilled shafts may prove to be an economical alternative to H-piles. Preliminary analysis indicates that high capacities would have to be developed for drilled shafts supporting the main span. The allowable design load of drilled shafts is a function of the penetration into the bedrock and the relationship of the shafts to the coal seam and underclay penetrated on the south side of the river in borings Nos. 14, 15 and 16. We recommend that preliminary design analysis for drilled shafts for the primary piers be based on an allowable bearing capacity of 50 tons per sq. ft. for gravity load, again with a 50% increase under the worst combination of loads including barge impact. For the north pier we recommend that preliminary estimates be based on penetration of approximately 20 ft. into rock with an approximate bottom elevation of 255. For the south pier we recommend that preliminary estimates be based on an approximate bottom elevation of 240 in order to penetrate the coal and underclay.

Preliminary analysis indicates that loads on intermediate piers are such that a substantially lower shaft capacity would be required. We recommend that preliminary estimates assume an allowable end bearing capacity of 25 tons per sq. ft. with a minimum penetration of 5 ft. into bedrock. Again, the bearing capacity may be increased by 50% under the worst combination of loading.

Preliminary studies have also considered a cellular foundation constructed on a mat or combined footing. The estimates have been based on the bottom of the tremie seal at elevation 314. This elevation would place the bottom of the tremie seal on compact sand at each of the five river borings. It is recommended that preliminary analysis be based on an allowable net bearing capacity under gravity load of 6000 lbs. per sq. ft. with an allowable 50% increase under the worst combination of load.

We make note of the presence of the silt layer penetrated in B-17 starting at approximately elevation 290. If preliminary cost estimates favor a cellular foundation, it is our opinion that additional borings, with emphasis on evaluation of the properties of the overburden materials, should be made prior to proceeding with structural design of this foundation.

LAND PIERS - MAIN BRIDGE

We recommend that foundation studies consider both cast-in-place concrete piles driven to capacity in the sand and high capacity H-piles driven to refusal on bedrock. For high capacity H-piles we recommend that loads be determined as above for river piers. Estimates for high capacity H-piles should be based on the following tip elevations.

Boring	Estimated Pile Tip Elevation
Boring 13 on Kentucky Side of the Ohio River	272'
Boring 21 on Indiana Side of the Ohio River	274'

CAST-IN-PLACE PILES

The capacity of cast-in-place concrete piles should be determined by the diameter and specified concrete strength. We recommend that preliminary estimates be based on a capacity of 55 tons for 12" piles and 70 tons for 14" piles. We recommend that these estimates be based on the following tip elevations.

Boring	Estimated Tip Elevation for Cast-in-Place Concrete Piles
1	340
4	345
5	340
9	320
11	335
13	335
21	340

EMBANKMENT CONSTRUCTION

It is technically feasible to develop borrow pits in the overbank deposits which consist mostly of lean clay and are 10 to 20 ft. thick. Atterberg limit tests for the lean clays of the overbank deposits showed liquid limits ranging from 26 to 42% and averaging 33%. There does not appear to be any correlation between depth and liquid limit values for the lean clay overbank deposits. Moisture contents for the upper 6 ft. of the lean clays ranged from 11% to 24% with most of the clays having moisture contents of 21% to 24%. Below a depth of 6 ft. moisture contents in the lean clays ranged rom 17% to 29%, with most of the clays having moisture contents of approximately 26%. Boring B-9 located at Station 831+00 encountered somewhat different conditions in that fat clay was encountered at a depth of 5 ft. and moisture contents for the overbank deposits were somewhat higher than the other borings. Moisture contents of the overbank deposits for boring B-9 were lowest at a depth of 1.0 ft. at 27% and ranged as high as 40% at a depth of 5 ft. in the fat clay unit. Optimum moisture contents for the lean clays are expected to range from 15% to 19%, the higher the liquid limits, the higher the optimum moisture content. Fat clays such as encountered in boring 9 can be expected to have optimum moisture contents in the range of 22% to 27%.

Based on the moisture contents and Atterberg limits of soils encountered in the borings, shallow borrow pits with depths of approximately 6 ft. are recommended. The borrow pits should be located in topographically high areas with respect to swale areas. Most of the test borings were made near swales to test critical soils. More favorable materials are likely away from the swales.

Swale areas are not recommended for borrow pits and should be avoided because they are likely to contain soils with higher moisture contents. In order to bring the upper 6 ft. of overbank deposits within 2% to 3% above optimum moisture content, it will be necessary to aerate most of the soils and reduce the moisture contents by 2% to 5%.

An alternative source of embankment materials is granular material within the channel of the present Ohio River channel. Granular materials directly beneath the bed of the present Ohio River channel consist predominately of fine poorly graded sands. Optimum moisture contents for these materials can be expected to fall in the range of 10% to 13%.

Based on analysis of the soils encountered in boring B-9 at Station 831+00, it is our opinion that some control will be required on the rate of filling in order to avoid an over-stress condition during embankment construction. Total stress analysis for a 35 ft. high embankment with 2:1 slopes showed a safety factor with respect to rotational failure of 1.03. Effective stress analysis for the same embankment showed a safety factor of 1.69. Stage construction, allowing dissipation of excess pore pressure in order to develop the soil's frictional strength is recommended.

Settlement analysis for the described embankment yielded results on the order of 12" to 16". The foundation materials consist of clay and, therefore, settlement can be expected to be time dependent. Settlements are of such magnitude that some time period will be required after the embankments are brought to rough subgrade elevation before paving can commence.

FURTHER GEOTECHNICAL INVESTIGATION

Further geotechnical investigation for the proposed project should include a boring at the location of each land pier for the main bridge and for the secondary bridges. It is also recommended that intermediate borings be made in embankment areas between structures, particularly at low points in the existing ground line.

It is recommended that information obtained in the additional borings be used in analysis to determine the length for cast-in-place concrete piles, shear strength and compressibility characteristics of soft overbank deposits, and moisture content profiles for fine grained soil. Analysis should include time-rate settlement predictions for embankments for use in determining construction rates in order to maintain a minimum safety factor of 1.2 during construction and allowing for time delays between completion of embankments and beginning of paving.

Additional geotechnical investigation on the river should consider the most likely type of foundation as indicated by preliminary cost studies. We have already stated that it is our opinion that if a cellular foundation supported on sand appears to be cost-effective, then we recommend that a rather intensive boring investigation be made at the locations of the piers prior to commencing detailed design of this foundation.

It is our opinion that the existing five borings provide sufficient information for designing and estimating the cost of an H-pile foundation. An item could be included in the construction contract for borings to confirm conditions at pier locations.

If a drilled shaft foundation is designed, we recommend that additional borings be made prior to the installation of the shafts to establish the bottom elevation of the shafts. It would be most desirable to make these borings during the detailed design stage of the project, however, design could be based on estimated bottom elevations as given above and core borings included as a part of the construction contract for the purpose of confirming the bearing elevation.

LIMITATIONS OF LIABILITY

OUR WARRANTY

We warrant that the services performed by The H. C. Nutting Company are conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions. NO OTHER WARRANTIES, EXPRESSED OR IMPLIED, ARE MADE. While the services of The H. C. Nutting Company are a valuable and integral part of the design and construction teams, we do not warrant, guarantee, or insure the quality or completeness of services provided by other members of those teams, the quality, completeness, or satisfactory performance of construction plans and specifications which we have not prepared, nor the ultimate performance of building site materials.

SUBSURFACE EXPLORATION

Subsurface exploration is normally accomplished by test borings; test pits are sometimes employed. The method of determining the boring location ·d the surface elevation at the boring is noted in report. This information is represented on a drawing or on the boring log. The location and elevation of the boring should be considered accurate only to the degree inherent with the method used.

The boring log includes sampling information, description of the materials recovered, approximate depth of boundaries between soil and rock strata and groundwater data. The log represents conditions specifically at the location and time the boring was made. The boundaries between different soil strata are indicated at specific depths; however, these depths are in fact approximate and dependent upon the frequency of sampling. The transition between soil strata is often gradual. Water level readings are made at the times and under conditions stated on the boring logs. Water levels change with time and season. The borehole does not always remain open sufficiently long for the measured water level to coincide with the groundwater table.

LABORATORY AND FIELD TESTS

Tests are performed in accordance with specific ĪTM Standards unless otherwise indicated. All determinations included in a given ASTM Standard are not always required and performed. Each test report indicates the measurements and determinations actually made.

ANALYSIS AND RECOMMENDATIONS

The geotechnical report is prepared primarily to aid in the design of site work and structural foundations. Although the information in the report is expected to be sufficient for these purposes, it is not intended to determine the cost of construction or to stand alone as a construction specification.

Report recommendations are based primarily on data from test borings made at the locations shown on a boring location drawing included. Soil variations may exist between borings and these variations may not become evident until construction. If significant variations are then noted, the geotechnical engineer should be contacted so that field conditions can be examined and recommendations revised if necessary.

The geotechnical report states our understanding as to the location, dimensions and structural features proposed for the site. Any significant changes in the nature, design, or location of the site improvements MUST be communicated to the geotechnical engineer so that the geotechnical analysis, conclusions, and recommendations can be appropriately adjusted.

The geotechnical engineer should be given the opportunity to review all drawings that have been prepared based on his recommendations.

CONSTRUCTION MONITORING

Construction monitoring is a vital element of complete geotechnical services. The field engineer / inspector is the owner's "representative" observing the work of the contractor, performing tests as required in the specifications, and reporting data developed from such tests and observations. THE FIELD ENGINEER OR INSPECTOR DOES NOT DIRECT THE CONTRACTOR'S CONSTRUCTION MEANS, METHODS, OPERATIONS OR PERSONNEL. He does not interfere with the relationship between the owner and the contractor and, except as an observer, does not become a substitute owner on site. He is responsible for his own safety but has no responsibility for the safety of other personnel at the site. He is an important member of a team whose responsibility is to watch and test the work being done and report to the owner whether that work is being carried out in general conformance with the plans and specifications.

A description of terminology and symbols used in the logs of test borings, and a copy of ASTM D 2487-83, "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.

GEOTECHNICAL REPORT
TWO MAIN RIVER PIERS
U.S. 60 / U.S. 231
BRIDGE OVER OHIO RIVER
DAVIESS COUNTY, KENTUCKY
SPENCER COUNTY, INDIANA

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July 12, 1991

0.1.1.90300B-R.72

Haworth, Meyer and Boleyn, Inc.
643 Teton Trail
Frankfort, Kentucky 40601

Attn: J.C. Pyles, P.E.

Re: Geotechnical Report
Two Main River Piers
U.S. 60/U.S. 231
Bridge over Ohio River
Daviess County, Kentucky
Spencer County, Indiana

Gentlemen:

Submitted herein is our geotechnical engineering report for the referenced project. This report addresses foundation considerations for the two main river piers, Pier Nos. 8 and 9, for the proposed structure. Laboratory testing and engineering analyses are continuing for the remaining substructure elements. When testing and analyses are complete, a final geotechnical report will be issued, presenting recommendations for the entire structure.

Results of field borings and laboratory testing, and our recommendations for design and construction of the main river piers are presented in this report. Subsurface data sheets showing graphic boring logs are also provided herein.

We appreciate the opportunity to support your design process. If you have any questions concerning this report, of if we can be of further assistance, please contact our office.

Respectfully submitted,
FULLER, MOSSBARGER, SCOTT AND MAY
CIVIL ENGINEERS, INC.

Scott L. Murray

Scott L. Murray, P.E.
Senior Project Engineer

Aubrey D. May

Aubrey D. May, P.E.
Vice President

GEOTECHNICAL REPORT

TWO MAIN RIVER PIERS

U.S. 60 / U.S. 231

BRIDGE OVER OHIO RIVER

DAVIESS COUNTY, KENTUCKY

SPENCER COUNTY, INDIANA

Prepared For

Haworth, Meyer and Boleyn, Inc.
Frankfort, Kentucky

Prepared By

Fuller, Mossbarger, Scott and May
Civil Engineers, Inc.
Lexington, Kentucky

July, 1991

GEOTECHNICAL ENGINEERING REPORT

U.S. 60 / U.S. 231

OVER THE OHIO RIVER

DAVIESS COUNTY, KENTUCKY

SPENCER COUNTY, INDIANA

1. LOCATION AND DESCRIPTION OF PROJECT

A new, four-lane bridge is being proposed to span the Ohio River and connect U.S. Route 60 in Daviess County, Kentucky with U.S. Route 231 in Spencer County, Indiana. The structure will be located approximately nine miles upstream of Owensboro, Kentucky, at approximate river mile 745.5.

Concrete and steel design alternates with approximate abutment-to-abutment lengths of 4500 feet are being considered. The two main river piers adjacent to the river navigation channel will be positioned to provide a navigational clearance of approximately 1100 feet. This report addresses geotechnical aspects for design and construction of the two main river piers.

2. SITE GEOLOGIC CONDITIONS

Available geologic mapping (Geology of the Rockport and Lewisport Quadrangles, Kentucky, U.S.G.S. 1964) indicates soils at the site consist of Wisconsin age glacial outwash deposits, overlain by more recent alluvial and overbank soils deposited during flooding of the Ohio River Valley. Sands, silts, clays and gravels comprise the deposits which are often present within the region in thicknesses exceeding 100 feet.

Underlying these depositional soils is bedrock of the Carbondale and Tradewater Formations, representing the Middle Pennsylvanian geologic age. These formations consist primarily of sandstones, shales and siltstones. Relatively thin (less than 3 feet thick) zones of coal and underclays are also occasionally present within these formations. The sandstones are generally light gray, fine grained, micaceous and commonly contain shale zones and partings. Shales are often light to dark gray, clayey to sandy, highly laminated, soft and fossiliferous. The geologic mapping does not indicate the presence of faults within the immediate site of the proposed structure. However, the project site is located within the Mississippi Valley seismic region, considered to be one of the most seismically active regions in the central United States.

3. SUMMARY OF BORINGS

3.1. General

As shown on the accompanying subsurface data sheets, three disturbed sample borings with rock coring were performed at each main pier location. The borings are numbered 13-15 and 16-18 for Pier Nos. 8 and 9, respectively. Each boring was drilled utilizing truck mounted equipment supported by a floating plant operation. Standard penetration testing (SPT) was conducted in each boring at depth intervals of five feet until the top of rock was encountered. The SPT tests provide a general indication of the soil consistencies, and retrieve samples for subsequent laboratory testing and evaluation. The project engineer, supervising on-site operations, provided a log of the soil profile at each boring location based on visual examinations of the SPT samples.

Summaries of test boring information for the two main river piers are provided in the following tables (all measurements expressed in feet):

TABLE I

SUMMARY OF BORING LOCATIONS

Pier No.	Boring No.	Station	Offset
8	13	893+16	89' Lt.
	14	893+46	CL
	15	893+74	91' Rt.
9	16	905+16	93' Lt.
	17	905+46	2' Rt.
	18	905+80	96' Rt.

Survey control and actual field locations for the test borings were provided by Burgess & Niple, Ltd.

TABLE II

SUMMARY OF BORING RESULTS

	Boring No.	Water Surface Elevation	Depth to River Bottom	River Bottom Elevation	Depth to Top of Rock	Top of Rock Elevation		Depth to Bottom of Boring	Bottom of Boring Elevation	
Lt	13	358.1	16.7	341.4	87.8	270.3	71	125.1	233.0	57'
CL	14	358.2	15.8	342.4	88.4	269.8	72.6	149.6	208.6	61'
Rt	15	358.1	15.1	343.0	89.6	268.5	74.5	130.0	228.1	40'

Soil thickness

Rock drill

TABLE II (Cont'd)

SUMMARY OF BORING RESULTS

Boring No.	Water Surface Elevation	Depth to River Bottom	River Bottom Elevation	Depth to Top of Rock	Top of Rock Elevation		Depth to Bottom of Boring	Bottom of Boring Elevation
16	358.2	23.7	334.5	84.7	273.5	61'	115.3	242.9
17	358.2	23.2	335.0	84.4	273.8	61'	135.0	223.2
18	358.2	22.2	336.0	84.3	273.9	62'	119.9	238.3

Graphic logs, showing the results of standard penetration testing, and soil and rock core testing, are presented on the subsurface data sheets located in Appendix D at the end of this report.

3.2. Soil Conditions Encountered

A review of the drilling data indicates soil deposits with thicknesses ranging from approximately 61.0 to 75.0 feet are present. Specifically, soil deposits of 71.1, 72.6 and 74.5 feet in thickness are noted at Boring Nos. 13, 14 and 15, respectively, while deposits of 61.0, 61.2 and 62.1 feet are present at Boring Nos. 16, 17 and 18, respectively. This information suggests that soil thicknesses within a given main pier location are relatively uniform.

The results of SPT testing indicate that the upper 20 to 30 feet of soils encountered at the boring locations exhibit loose to medium consistencies. This depth range most likely corresponds to the zone of soils subjected to scouring and transportation during severe flow events. These soils are generally described as sand, or sand with gravel, brown to gray, wet, non-plastic, with loose to medium consistencies. The results of classification testing show these deposits generally contain low percentages of fine grained soils (silts and clays).

These upper loose sands grade into deposits of medium to dense sands with gravel, which extend downwardly to the top of rock. Occasionally, larger gravel and/or cobbles are present just above the top of rock. These soil deposits generally become more dense with depth and exhibit SPT N-values ranging from 15 to 50+. Size of Gravel ?

3.3. Bedrock Conditions Encountered

As shown in Table II, the elevations of the top of rock range from 268.5 to 270.3 feet at Boring Nos. 13-15, and from 273.5 to 273.9 at Boring Nos. 16-18. Bedrock types and qualities are generally consistent at the locations of Boring Nos. 13-15 (Pier 8), but are significantly different from those encountered at Boring Nos. 16-18 (Pier 9).

Generally, light gray to dark gray or black shale, silty to sandy, and of soft to medium hardness is present at the locations of Boring Nos. 13-15. A relatively thin seam of coal and underclay is noted beginning at depths of 19.0, 17.3 and 11.6 feet below the top of rock at these locations, respectively. The upper three to four feet of bedrock is highly weathered and soft and contains clayey zones. The base of the rock disintegration zone (RDZ) is noted in these borings at depths of 3.9, 4.3 and 2.6 feet below the top of rock, respectively. The shale, from the RDZ to the top of the coal seam, can generally be described as dark gray to black, silty to sandy, with numerous sandy streaks and partings, and is highly laminated. Below the coal seam and underclay, the shale is gray, silty to sandy, with sandy streaks and partings, with churned zones. Relatively good recovery rates ranging from 90 to 100 percent are noted for the coring runs in Boring Nos. 13-15. However, rock quality designations (RQD's) are consistently low for the core samples obtained at these boring locations, and range from 0 to 36 percent. The RQD is defined as the sum of all core pieces longer than four inches, divided by the total length of the coring run. The resultant is multiplied by 100 to express the RQD in percent. This classical definition of RQD for rock cores is modified by the Kentucky Department of Highways. For Kentucky highway projects, if a given length of core can be broken, by hand pressure, into pieces smaller than four inches in length, its interval is excluded from the numerator in the RQD calculation. Modified RQD values are shown on boring logs presented on subsurface data sheets.

Bedrock conditions at the location of Pier 8 differ significantly from those determined at Pier 9. Cores obtained from Pier 8 consist primarily of interbedded units of shales and sandstones, generally exhibiting higher RQD values than cores from Pier 9. Weathering is generally deeper at the Pier 9 boring locations with RDZ depths of 6.0, 7.3 and 6.4 feet below the top of rock being noted, respectively. Even though each boring drilled at the location for Pier 9 was advanced to considerable depth below the elevations at which coal seams were observed in Boring Nos. 13-15, the coal seam was not encountered in these Pier 9 borings. The coal seam was present in borings performed for all substructure elements south of Pier 8, and in borings drilled for Pier Nos. 10 and 11, located north of Pier No. 9.

Quantitative testing of core samples further indicates significant differences between bedrock encountered at Pier 8 and Pier 9. Selected core samples obtained from both pier locations were subjected to slake durability index (SDI) and compression testing. The results of SDI tests are shown at their respective sample intervals on the graphical logs. SDI values range from 0 to 93 percent with an average of 66 percent for cores from Boring Nos. 13-15. Core samples obtained from Boring Nos. 16-18 exhibit SDI values ranging from 16 to 94 percent, with an average of 75 percent. The results of compression tests are as follows:

TABLE III

COMPRESSION TEST RESULTS
ROCK CORE SAMPLES

Boring No.	Depth of Sample (ft.)	Compressive Strength (psi)	Elev	Qu, tsf
13	108.7 - 109.1	±320	249.1	23.0
14	106.4 - 106.8	±514	251.6	37.0
14	120.3 - 120.7	±216	237.7	15.6
15	119.5 - 119.9	±300	238.6	21.6
16	101.1 - 101.5	±1287	257	92.7
17	104.0 - 104.4	±1668	254.1	120.1

4. FOUNDATION SYSTEM CONCLUSIONS AND RECOMMENDATIONS

4.1 General

4.1.1. According to Report No. FHWA/RD-81/081 titled "Seismic Design Guidelines for Highway Bridges" the project lies within the seismic performance category Zone B. An acceleration coefficient equal to 0.10 to 0.20 is stipulated for Zone B.

4.1.2. The differences in bedrock lithologies at Pier No. 8 as compared to those at Pier No. 9, most notably the absence of the coal seam at the location of Pier No. 9, suggest that a geologic unconformity of bedrock strata occurs between Pier Nos. 8 and 10. An unconformity is a surface of erosion or non-deposition that separates younger strata from older rocks. An unconformity is typically the result of changes that occurred in the depositional environment in which bedrock strata were formed. An extreme explanation for the observed differences would be that unmapped fault zones are present between Pier No. 8 and Pier No. 9, and between Pier No. 9 and Pier No. 10.

4.1.3. Due to the large vertical loads and possible large lateral loads anticipated for the main river piers, it is recommended that the foundation systems for these piers consist of high capacity H-piles bearing on bedrock, or drilled shafts bearing on bedrock. Lateral loadings or uplift capacities may ultimately control the selection of the foundation systems. Recommendations for both alternatives are provided to support the design team in the selection process.

4.2. High Capacity Steel H-Piles Bearing on Bedrock

4.2.1. The following table provides a summary of estimated tip elevations for H-piles.

TABLE IV

ESTIMATED PILE TIP ELEVATIONS

Substructure Element	Boring No.	Estimated Pile Tip Elevation (ft.)
Pier 8	13,14,15	266.0
Pier 9	16,17,28	267.5

The estimated pile tip elevations provided above correspond approximately to the base of the rock disintegration zone (RDZ) observed in the rock core samples obtained at the boring locations.

4.2.2. Because of the dense sands and gravels present above rock, and because driving of piles through weathered rock is likely before achieving refusal, it is recommended that pile tip reinforcements be utilized to help avoid pile damage.

4.2.3. It is recommended that the capacity of H-piles be based on an allowable stress of 9,000 psi of cross-sectional area, not including the area of any tip reinforcement. This capacity recommendation is in accordance with the Division of Bridges Guidance Manual issued in June, 1988. A 1991 Interim update of Standard Specifications for Highway Bridges has been issued by AASHTO. Section 4.5.7.3 of this AASHTO reference suggests the maximum allowable stress for steel H-piles may be increased to .33 Fy in conditions where pile damage is unlikely. The update states that static or dynamic load tests and evaluation confirming satisfactory results should be performed when using 0.33 Fy.

4.2.4. The following table provides an estimate of the uplift capacity that a single pile will exhibit as a result of side resistance between the pile and surrounding soil. The available uplift capacity is based on utilizing two-thirds of the predicted side resistance, as outlined on page 46, Section 3.7.1. of "Pile Foundation Analysis and Design" by H.G. Poulos and E.H. Davis. These values are based on preliminary bottom of foundation seal elevations provided by HMB, Inc. If the elevations are revised, the following capacities will also need to be revised. Calculations to determine the uplift capacities are presented in Appendix A.

TABLE V

UPLIFT CAPACITY OF A SINGLE STEEL H-PILE

what el. lowest scour line

& rock

Pier No.	Approximate Length of Pile Embedment in Soil (ft.)	Pile Size	Single Pile Uplift Capacity* (Tons)
8	48	Hp 12 x 74	50
		Hp 14 x 73	60
		Hp 14 x 89	65
9	36	Hp 12 x 74	35
		Hp 14 x 73	40
		Hp 14 x 89	45

* Uplift capacities include a safety factor of 2.

4.3. Drilled Shafts on Rock

Design and/or economic considerations may dictate the use of drilled shafts for the pier foundations.

The following recommendations concerning drilled shafts are based on a review of available data and were developed utilizing procedures outlined in Publication No. FHWA-HI-88-042, titled, "Drilled Shafts: Construction Procedures and Design Methods", issued in August, 1988. Analyses conducted to determine the recommended shaft capacities are provided in Appendix B.

4.3.1. It is recommended that the capacity of drilled shafts utilized at the location of Pier No. 8 be based on load transfer in side resistance between concrete and bedrock. This will require drilled shafts be socketed into rock. The diameter of the shaft and the length of the socket should be based on the load requirements determined by the design team. The allowable capacity of shafts should be determined by the following equation:

$$Qs\ (\text{allowable}) = 3.39\ (D)(L)$$

Where:

Qs = Allowable Side Resistance, in Tons
D = Diameter of Shaft, in Feet
L = Length of Socket Below the RDZ, in feet.

Qs does not include any end-bearing capacity, and is based on a safety factor of 3. Why no end brg

4.3.2. It is recommended that the sides of the sockets be roughened as necessary prior to placement of concrete. This will help provide adequate bonding between concrete and rock.

NO - Why dewater?

4.3.3. It should be anticipated that dewatering of the excavations may be necessary. A slurry drilling operation may be necessary to perform drilled shaft installations.

4.3.4. If the length of the socket is such that less than three feet of bedrock will be present between the bottom of the socket and the top of the coal seam encountered in Boring Nos. 13-15, the socket should extend a minimum of five feet below the coal seam and underclay. No capacity should be attributed to the section of shafts located within coal and/or underclay zones.

4.3.5. If the coal seam and underclay are penetrated to develop required socket lengths, it is likely that significant amounts of water will enter the open shafts through the coal seam. It may become necessary to case or otherwise seal the coal seam to prevent water infiltration. Drilled shaft capacity may only be calculated for the length of socket in actual contact with bedrock.

4.3.6. It is recommended that the capacities of drilled shafts utilized at the location of Pier No. 9 be based on load transfer in end bearing. The recommended allowable end bearing capacity is 50 tons per square foot. A minimum two-foot socket extending below the RDZ is recommended. Providing this alternative is selected, the following bearing elevations are applicable at the corresponding boring locations:

Why not use EB & SF for both?

Boring No.	Recommended End Bearing Elevation
16	265.5
17	264.5
18	265.5

4.3.7. Should socketing of the drilled shafts at Pier No. 9 be necessary to resist uplift, the shaft uplift capacities should be based on load transfer in side resistance. The diameter of the shaft and the length of the socket should be based on the uplift requirements as determined by the design team. The allowable uplift capacity of the shafts should be determined by the following equation:

$$Qs\ (\text{allowable}) = 7.3\ (D)(L)$$

Where:

Qs = Allowable Side Resistance, in Tons
D = Diameter of Shaft, in Feet
L = Length of Socket Below the RDZ, in feet.

The equation provided for determining Qs includes a safety factor of 3.

4.3.8. The soils encountered in Boring Nos. 13-18 (Pier Nos. 8 and 9) primarily consist of sands and sands with gravel. It should be anticipated that casing of these materials will be necessary to maintain an open excavation for installation of the drilled shafts. The casing will need to be seated into rock prior to any rock excavation. This will help control infiltration of water into the excavation.

The use of temporary casing would require the excavations be filled with concrete with good workability, i.e., high slump, to a level such that there could be no inward movement of soil or water before the seal is broken at the bottom of the temporary casing. The level of concrete being placed would have to be maintained at a distance above the bottom of the casing as the casing is retrieved such as to prevent soils from caving into the excavation and detrimentally effecting the structural integrity of the drilled shafts. This procedure would be difficult at best and is not recommended. It is recommended that permanent casing be installed down to the top of rock.

5. LATERAL RESISTANCE PROVIDED BY SOIL TO PILE OR TO DRILLED SHAFT FOUNDATION SYSTEM

The application of a lateral load to a drilled shaft or pile may result in some lateral deflection. A lateral deflection will, in turn, cause a soil reaction acting in an opposite direction to the deflection. The magnitude of the soil reaction is a function of the deflection, but the deflection is also dependent on the soil reaction. The behavior of drilled shafts and piles under lateral loading conditions requires the solution of a soil-structure-interaction problem. Two different methods for solving this problem are outlined in Publication No.

FHWA-HI-88-042. The first method discussed, and most widely accepted, is the "p-y method" of analysis. With this method, sets of p-y curves are developed to model the soil reaction. The term "p" is the force per unit length along the drilled shaft or pile, and "y" is the lateral deflection.

The second method outlined in this publication is "Broms Method", which is used in obtaining approximate solutions. This method computes the lateral load at which a deep foundation will fail. In cohesionless soils, Broms assumes the ultimate lateral resistance is equal to three times the Rankine passive earth pressure. At a depth, z, below the ground surface, the soil resistance per unit of length P_z can be obtained by the following equations:

$$P_z = 3B_b(\gamma)\,z\,(Kp)$$

$$Kp = \tan^2(45 + \phi/2)$$

Where:

γ = Unit Weight of Soil, pcf
Kp = Rankine Coefficient of Passive Earth Pressure
ϕ = Angle of Internal Friction of Soil, degrees

Recommended soil profiles to be used in determining lateral resistances are provided in Appendix C. One critical profile at each pier location is provided to simplify the design process. Wet unit weights, angles of internal friction and passive earth pressure coefficients are shown on the profiles. Publication Nos. FHWA-IP-84-11, July, 1984, and FHWA/RD-85/106, March, 1986, provide in-depth discussions of design procedures for lateral resistance concerns.

FOUNDATION EVALUATION

TWO MAIN RIVER PIER FOUNDATIONS

NEW BRIDGE OVER OHIO RIVER

DAVIESS COUNTY, KENTUCKY

SSP 030 8530 032D

ITEM NO. 2-116.0

3

Prepared For

Haworth, Meyer and Boleyn, Inc.
Frankfort, Kentucky

October, 1991

FULLER, MOSSBARGER, SCOTT & MAY

CIVIL ENGINEERS, INC.

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October 14, 1991

0.1.1.90300-3-L.2

— 102-0032001

Mr. Tim Pyles, P.E.
Haworth, Meyer and Boleyn, Inc.
643 Teton Trail
Frankfort, Kentucky 40601

Re: New Bridge Over Ohio River
Two Main River Pier Foundations
Pier Nos. 8 and 9
Daviess County, Kentucky
SSP 030 8530 032D
Item No. 2-116.0

Dear Tim:

On September 27, 1991, you informed this office that the design scour elevation for the two main river piers had again been lowered, the most recent value being 278.5 feet. Should scour occur to this elevation, only about 8 and 4 feet of soil will remain above the top of rock at the locations of Pier Nos. 8 and 9, respectively. Such shallow depths of soil would not provide adequate lateral support for steel piles driven to refusal on bedrock. In view of this revised design scour depth, foundation systems utilizing drilled shafts will be more appropriate. The shafts will need to be socketed to some depth into bedrock to develop the high axial and lateral capacities necessary for design. The data provided with this letter outlines the procedures used to evaluate axial, uplift and lateral capacities for the drilled shafts. As requested by Mr. Barry Berkovitz in his letter dated September 4, 1991, axial capacities were evaluated based on design settlements. Both side load transfer and end bearing were calculated and combined to estimate ultimate axial capacities. The "STIFF" and "COM 624" computer programs were utilized to determine lateral capacities of the drilled shafts.

The results of our evaluations for Pier Nos. 8 and 9 are provided in Appendix A and B, respectively, and are summarized in Tables 1 and 2 below. Also included with this letter are graphic logs for Borings Nos. 13-18.

You provided FMSM with the following design information to be used in our evaluation of shaft capacities*:

- Maximum Axial Service Load - 2850 kips ~300 Barge impact load
- Maximum Uplift Service Load - 600 kips 110k
- Maximum Lateral Service Load - 150 kips X44 =6600 Allowable lateral
- Maximum Tolerable Settlement - 3 inches ?
- Shaft Diameter - 6.0 feet to base of RDZ
- Rock Socket Diameter - 5.5 feet
- Configuration of Foundation Seal, Footing and Shafts (see page 1 of 15 for Pier 8, Appendix A)
- Steel Reinforcement Configuration for Shafts (see input data for "STIFF")
- Boundary conditions for structural modeling and use with COM624, using the boundary condition of KCB = 2, and the slope of the elastic curve input as zero. ?

Costs savings Done one to the other to save $

TABLE 1

AXIAL AND UPLIFT CAPACITIES

Pier No.	Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Ultimate Axial Capacity* (kips)	Allowable Axial Capacity* (kips)	Allowable Uplift Capacity* (kips)
8	5.5	234.0	7,500	3,000	740
9	5.5	251.5	11,750	4,700	1,060

Geot $ 1/2 mil

TABLE 2

LATERAL CAPACITIES

Pier No.	Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Maximum Lateral Service Load* (kips)	Deflection of Top of Shaft (inches)
8	5.5	234.0	200	1.1
9	5.5	251.5	240	0.9

* Capacities listed are per shaft capacities.

7054767

The following points should be noted about the analyses:

- Because the center to center spacing of the shafts is less than eight shaft diameters, the shafts were considered as a group. Appropriate modifications were made to the axial and uplift capacities, and to the modulus of subgrade reaction to account for group effects. These modifications are outlined on pages 159-160 of FHWA-IP-84-11. The capacities provided in Table Nos. 1 and 2 are for single shafts acting in a group.

- A safety factor of 2.5 was utilized to determine allowable capacities and maximum lateral service loads. This safety factor was selected since it is likely the shaft sockets will be constructed in the wet. These construction procedures may cause the types of bedrock encountered at the site to degrade somewhat. Also direct inspection of the rock sockets will not be possible as a result of these procedures.

- A zone consisting of the coal seam and underclay and two feet above the coal and two feet below the underclay, was neglected in estimating the ultimate side load transfer for shafts at Pier No. 8. ?

- Side load transfer to the soil and RDZ material remaining after scour, was neglected in estimating the axial and uplift capacities for the shafts.

- Settlements on the order of magnitude of 1 inch are estimated for both piers for service loads approaching the allowable axial capacities.

- Since shaft deflections, as predicted by COM624, were less than 0.2 inches at all positions below the RDZ, no reductions in axial or uplift capacities were necessary.

These data and results will be included in the final report for the total structure. At this time, we are awaiting determination of scour depths to be used at the remaining pier and abutment locations before continuing our foundation analysis.

If you have any questions concerning this information, please feel free to contact our office.

Respectfully submitted,

FULLER, MOSSBARGER, SCOTT AND MAY
CIVIL ENGINEERS, INC.

Scott L. Murray

Scott L. Murray, P.E.
Senior Project Engineer

Aubrey D. May

Aubrey D. May, P.E.
Vice President

ADM/SLM/sh

8

RESULTS OF "STIFF" AND "COM624" ANALYSES

TWO MAIN RIVER PIERS

(PIER NOS. 8 & 9)

REVISED 10/24/91 & 10/25/91

NEW BRIDGE OVER THE OHIO RIVER

DAVIESS COUNTY, KENTUCKY

SSP 030 8530 032D

ITEM NO. 2-116.0

4

Prepared For

Haworth, Meyer and Boleyn, Inc.
Frankfort, Kentucky

October, 1991

FULLER, MOSSBARGER, SCOTT & MAY

CIVIL ENGINEERS, INC.

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October 28, 1991

0.1.1.90300-R.3

Mr. Tim Pyles, P.E.
Haworth, Meyer and Boleyn, Inc.
643 Teton Trail
Frankfort, Kentucky 40601

Re: New Bridge Over the Ohio River
Two Main River Pier Foundations
Pier Nos. 8 and 9
Revised "STIFF" and "COM624" Computer Analyses
Daviess County, Kentucky
SSP 030 8530 032D
Item No. 2-116.0

Dear Tim:

The "STIFF and "COM624" computer analyses for Pier Nos. 8 and 9 have been revised as a result of recent conversations with you and comments offered during the plan review meeting of October 24, 1991. The "STIFF" analyses were revised as follows:

- Steel casing was not used in determining EI values to be utilized for the shaft or the socket.

- The concrete cover for the reinforcing steel in the socket was reduced from 6.5 inches to 3.5 inches.

- The selection of EI values for use with "COM624" was performed utilizing procedures outlined in FHWA-IP-84-11, "Handbook on Design of Piles and Drilled Shafts Under Lateral Loads", pages 81-86.

"COM624" analyses were revised as follows:

- EI values obtained from revised "STIFF" analyses were utilized.

- An axial service load of 3,000 kips per shaft was used for 6-foot diameter shafts. We understand from conversations with you that the lateral service load has been revised from 150 kips per shaft to 115 kips per shaft.

The data provided with this letter contain the "STIFF" and "COM624" results for these revisions. The data also provide results of "STIFF" and "COM624" analyses for 8-foot diameter shafts, with 7.5-foot diameter rock sockets. These analyses were requested by Mr. V. J. Chandra of Parsons, Brinckerhoff, Quaade and Douglas, Inc. Mr. Chandra provided FMSM with the following service loads for 8-foot diameter shafts:

- Axial Service Load - 4,500 kips
- Lateral Service Load - 250 kips

FMSM has performed axial capacity and settlement estimates for the 8-foot diameter shafts. The results of these calculations were faxed to Mr. Chandra on October 25, 1991, and are also included with this letter. Mr. Chandra noted that PBQ&D, Inc. is designing the structure for a maximum tolerable settlement of 1 inch. The results of the enclosed analyses are summarized in the following tables:

TABLE 1

AXIAL AND UPLIFT CAPACITIES

Pier No.	Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Ultimate Axial Capacity* (kips)	Allowable Axial Capacity* (kips)	Allowable Uplift Capacity* (kips)	Estimated Settlement at Allowable Axial Capacity (inches)
8	5.5	234.0	7,500	3,000	740	±1
8	7.5	234.0	11,250	4,500	1,300	±1
9	5.5	251.5	11,750	4,700	1,060	±1
9	7.5	251.5	23,700	7,900	2,000	±1

* Capacities listed are per shaft capacities.

TABLE 2

LATERAL CAPACITIES

Pier No.	Diameter of Rock Socket (feet)	Bottom Elevation of Shaft (feet)	Maximum Lateral Service Load * (kips)	Deflection of Top of Shaft (inches)
8	5.5	234.0	126	±0.9
8	7.5	234.0	330	±0.7
9	5.5	251.5	136	±0.7
9	7.5	251.5	360	±0.6

* Capacities listed are per shaft capacities.

If you have any questions concerning these analyses, please feel free to contact our office.

Respectfully submitted,

FULLER, MOSSBARGER, SCOTT AND MAY
CIVIL ENGINEERS, INC.

Scott L. Murray

Scott L. Murray, P.E.
Senior Project Engineer

SLM/sh

c: V. J. Chandra
Henry Phillips

January 20, 1993
GG251

Via Fax. Total Pages: 2

Mr. John De Luca
National Engineering & Contracting Co.
12608 Alameda
Strongsville, OH 44136
216-238-3335

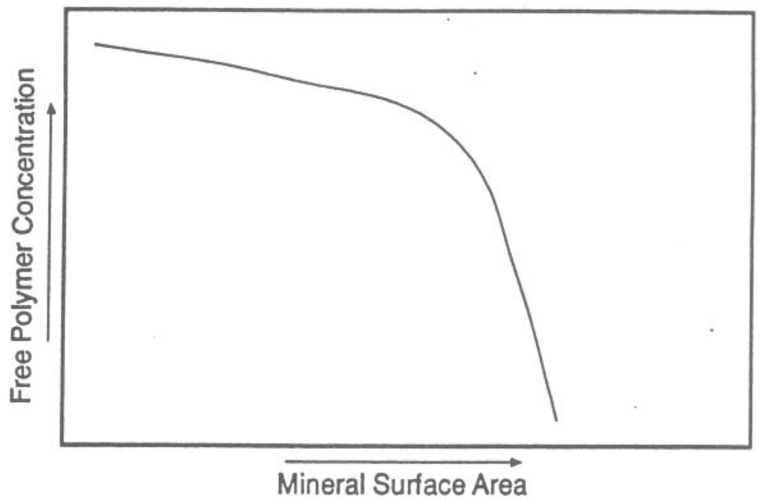
Subject: **Slurry Specifications presented by Kentucky Transportation Department for on the Bridge**

Dear Mr. De Luca:

We were informed yesterday by Mr. Wally Gratz that Kentucky Transportation Cabinet has set slurry specifications for the Ohio river bridge at a Marsh funnel viscosity of 40 to 45 seconds, a pH of 5 to 9, a sand level of 0.25 percent maximum, drilling density not to exceed 67 pcf, and density for tremie not to exceed 65 pcf.

KB Technologies takes issue with the Marsh Funnel viscosity specification, as we feel it is not appropriate for a CDP fluid system, and may cause less-than-optimum results to be obtained in the construction process and in quality of the finished piers. KB Technologies has consistently recommended that Marsh Funnel viscosity (MFV) be maintained in a range of 55 to 60 seconds per quart during drilling and displacement. This recommended MFV value is intended to assure adequate free polymer concentration in the fluid to: (1) stabilize and inhibit hydration of the claystone formation; (2) assure optimum borehole geometry and cleanliness; (3) optimize drilling rate and minimize exposure time of the claystone to the drilling fluid; (4) maximize filming encapsulation of cuttings, to minimize degradation of the cuttings into silt fines; and (5) to avoid any possibility of depletion of free polymer in the fluid due to adsorption onto mineral surface area. Depletion of free polymer can occur when the fluid becomes laden with clay fines and is subjected to the dynamics of the drilling operation.

KB Technologies has observed in the laboratory the effects of polymer depletion under dynamic conditions in the presence of increasing loads of fine solids. These effects are also well-documented and generally accepted in polymer-colloid surface chemistry. As mineral cuttings are degraded into finer particles, the surface area of the solids increases exponentially. This increase in surface area creates an exponentially higher number of reactive sites to which the dissolved polymer bonds. This bonding ties up the free polymer so that it is not available to do any other work, e.g. filming of borehole walls, inhibiting hydration of claystone formation, encapsulation of cuttings, etc. When free polymer content reaches a critical point (as illustrated in the graph below), the system in effect collapses. When this occurs, the quality of the construction project is jeopardized.



Potential problems can be easily avoided by following KB's recommendations with regard to dosage rates and viscosity parameters. It is important to adhere to both parameters because Marsh Funnel viscosities can be influenced by factors other than free polymer content. All initial slurry makeup should be at KB Technologies' recommended dosage rate of 0.1 percent (8.3 pounds per 1,000 gallons of makeup water). During drilling Marsh Funnel viscosities should be maintained at 55 to 60 seconds per quart by adding fresh CDP as required. These procedures, combined with maintenance of density at 67 pcf maximum, will assure ample free polymer under normal operating conditions expected on this job.

We request that the SlurryPro CDP system be used in accord with our recommendations in order to achieve the desired results.

Sincerely yours,
KB Technologies Ltd

K.Gifford Goodhue, Jr.
President

cc: Mr. Wally Gratz

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

116.8

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-5-93
Approved By ______ Date ______

Page 1 of 6
Pier No. 9
Shaft No. 42
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		367.8	94.8	274.0
	ROCK	267.2	101.1	251.5
	SHALE	275.0		

Soil Auger Diam. 70"
River Surf. Elev. ______
Water Table Elev. 358.6
Reference Elev. ______
Drilling Mud · CDP

Notes: 358.7 WATER LEVEL DAY SHIFT. (10'S)

Depth	Elev.	Time		Soil Description & Notes
		7:30 PM	IN	MOVED BARGED TO PIER 9, GETTING READY TO DRILL
		8:50 PM	OUT	PLUMB IS A. O.K.
73.2	295.1	8:50 PM	IN	DRILLING STARTS , 3/4 LOAD, SAND,CLAY,COBBLESTONES
		8:53 PM	OUT	SOIL AUGER WITH SIDE TEETH
		8:54 PM	IN	FULL LOAD- SAND, CLAY, COBBLESTONES
		9:00 PM	OUT	MOVING MORE EQUIPMENT. WIELDING ON SLURRY HOSE
		9:11 PM	IN	FULL LOAD- SAND CLAY & COBBLESTONES
		9:18 PM	OUT	
		9:19 PM	IN	FULL LOAD, SAND, CLAY , COBBLESTONES
		9:27 PM	OUT	STILL WORKING ON SLURRY HOSE
		9:32 PM	IN	FULL LOAD- SAND, CLAY, COBBLESTONES
		9:39 PM	OUT	MOVED DUMP BED 9:40 PM
		9:41 PM	IN	STOPPED 9:48 PM TO WELD SLURRY PIPE CASTING
		9:57 PM	OUT	FULL LOAD, SAND, CLAY,COBBLESTONES
		9:58 PM	IN	ADDED SLURRY, 10:PM BREAK TIME, 10:25 PM DRILLS AGAIN
84.4	283.9	10:30 PM	OUT	AGAIN 3/4 LOAD, SAND, CLAY, STONES- REPLACED TEETH
		10:57 PM	IN	1/2 LOAD SAND, CLAY, COBBLESTONES
		11:03 PM	OUT	
		11:05 PM	IN	SAME AS ABOVE
		11:11 PM	OUT	ADDED SLURRY- CHANGING TEETH
		11:13 PM	IN	FULL LOAD, CLAY, SAND, STONES
		11:20 PM	OUT	
		11:22 PM	IN	3/4 LOAD, CLAY, SAND, STONES
88.2	280.1	11:30 PM	OUT	SOUNDING
		11:34 PM	IN	FULL LOAD, CLAY,SAND, STONES
		11:40 PM	OUT	

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Department of Highways
Division of Construction

116.8

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-5-93
Approved By ______ Date ______

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Pier No. 9
Shaft No. 42
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.3	94.8	274.0
	ROCK	267.2	101.1	251.5
	SHALE	275.0		

Soil Auger Diam. 70"
River Surf. Elev. ______
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
		11:41 PM	IN	MOVED DUMP BED, FULL LOAD
		11:50 PM	OUT	CLAY, SAND, STONES, CLEANED AUGER
2-6-93		12:06 AM	IN	VIS. 50, DEN. 63, PH 6.4 SAND CON. TRACE CHECKING SLURRY, ADDING SLURRY
94.0	274.3	12:15 AM	OUT	3/4 LOAD, SAND/GREY SHALE, CLEANING AUGER, CHANGED TO SOIL AUGER/TEETH
		12:43 AM	IN	FULL LOAD, GREY SHALE,SAND,SMALL STONES
		12:52 AM	OUT	ADDED SLURRY
		12:54 AM	IN	3/4 FULL GREY SHALE/ SAND & COBBLESTONES
		1:05 AM	OUT	MOVED DUMP BED- LUNCH
		1:45 AM	IN	3/4 LOAD, GREY SHALE WITH SAND & COBBLESTONES
96.9	271.4	1:52 AM	OUT	CLEANED AUGERS & SOUNDED
		1:56 AM	IN	FULL LOAD, GREY SHALE WITH SAND & COBBLESTONES
		2:04 AM	OUT	
		2:05 AM	IN	3/4 LOAD, GREY SHALE WITH SAND & COBBLESTONES
		2:12 AM	OUT	CLEANED AUGERS & ADDED SLURRY
		2:15 AM	IN	1/2 LOAD, GREY SHALE WITH SAND & COBBLE STONES
		2:21 AM	OUT	PLUMB IS O.K.
99.6	268.7	2:23 AM	IN	FULL LOAD, GREY SHALE WITH SAND & COBBLESTONES
		2:33 AM	OUT	CLEANED AUGERS & SOUNDED
		2:38 AM	IN	MOVED DUMP BED- 3/4 LOAD, GREY SHALE
		2:47 AM	OUT	SAND, SMALL COBBLESTONES, CLEANED AUGER, ADDED SLURRY
		2:50 AM	IN	1/2 LOAD, GREY SHALE, SOME SAND, COBBLES & CLEANED AUGER, TEST#
		2:58 AM	OUT	
		3:05 AM		TEST = VISC. 54, DEN. 63.5, SAND TRACE% PH 6.4
		3:12 AM	IN	1/3 LOAD, GREY SHALE, SMALL COBBLES, SMALL AMOUNT OF SAND
		3:25 AM	OUT	ADDING SLURRY, CLEANED AUGER, CHANGING TO SOIL AUGER W/ ROCK TEETH

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Department of Highways
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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-6-93
Approved By ______ Date ______

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Pier No. 9
Shaft No. 42
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.3	94.8	274.0

Soil Auger Diam. 70"
River Surf. Elev. ______
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time	Soil Description & Notes
		3:39 AM: IN	2/3 LOAD, GREY SHALE, SAND, & COBBLESTONES
		3:52 AM: OUT:	
		3:53 AM: IN	1/2 LOAD GREY SHALE & SAND- CLEANED AUGER
102.7	256.6	4:06 AM: OUT:	ADDED SLURRY
		4:10 AM: IN	1/3 LOAD GREY SHALE & SAND- CLEANED AUGER
		4:23 AM: OUT:	
		4:26 AM: IN	EMPTIED DUMP BED, 1/2 LOAD GREY SHALE
		4:41 AM: OUT:	CLEANED AUGER, BREAK TIME- GREASING RINGS
		5:00 AM: IN	DARK GREY SHALE, 1/2 LOAD, CLEANED AUGER & SOUNDING
104.1	264.2	5:14 AM: OUT:	
		5:17 AM: IN	1/2 LOAD, DARK GREY SHALE, SOME SMALL STONES,
		5:28 AM: OUT:	CLEANED AUGER, & PLUMB IS O.K.
		5:31 AM: IN	1/2 LOAD DARK GREY SHALE, SMALL STONES & ADDED SLURRY
		5:42 AM: OUT:	
		5:43 AM: IN	1/2 LOAD DARK GREY SHALE, & CLEANED AUGER
		5:54 AM: OUT:	
106.2	262.1	5:57 AM: IN	1/3 LOAD DARK GREY SHALE, SOUNDING & CLEANED AUGER
		6:05 AM: OUT:	MOVED DUMP BED
		6:08 AM: IN	1/2 LOAD DARK GREY SHALE
		6:17 AM: OUT:	
		6:19 AM: IN	SAME AS ABOVE & ADDED SLURRY & CLEANED AUGER
		6:29 AM: OUT:	
		6:31 AM: IN	1/2 LOAD DARK GREY SHALE
		6:40 AM: OUT:	
		6:42 AM: IN	2/3 LOAD DARK GREY SHALE, CLEANED AUGER, GREASING KELLY RIG
		6:55 AM: OUT:	

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116.4

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-6-93
Approved By ______ Date ______

Pier No. 9
Shaft No. 42
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		367.9	94.3	273.6

Soil Auger Diam. 70"
River Surf. Elev. ______
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time	Soil Description & Notes
		7:00 AM: IN	
109.3	259.0	7:15 AM: OUT	1/2 LOAD DARK GREY SHALE AND SAND
		: IN	REPLACING TEETH ON AUGER (E)
		: OUT	
		7:34 AM: IN	BACK IN
		7:50 AM: OUT	1/2 LOAD, SANDY SHALE DARK GREY COLOR
		: IN	SAMPLING VISC. 50 SEC/DEN 63/PH.6.8/SAND CONTENT - TRACE
		: OUT	
		8:01 AM: IN	BACK IN .
111.1	256.8	8:19 AM: OUT	1/2 LOAD SANDY SHALE DARK GREY COLOR
		8:23 AM: IN	BACK IN, 8:24 ADDING SLURRY, 8:26 STOPPED
		8:44 AM: OUT	1/3 LOAD SAND SHALE DARK GREY COLOR
		8:46 AM: IN	BACK IN, ROCK TOOTH AUGER BIT (E)
		9:03 AM: OUT	DRILL RIG DOWN, WORKING ON CABLE
		9:09 AM: IN	BACK DRILLING
		9:14 AM: OUT	1/4 LOAD SAND SHALE DARK GREY COLOR
		9:16 AM: IN	BACK IN, ADD SLURRY 9:17, STOPPED 9:18
115.0	252.9	9:49 AM: OUT	2/3 LOAD SAND SHALE DARK GREY COLOR
		9:52 AM: IN	BACK IN, PLUMB CHECK, KY TO IND PLUMB UP & DOWN RIVER 1/16 TO 1' OUT
116.2	251.7	10:09 AM: OUT	1/2 LOAD SAND SHALE DARK GREY COLOR
		: IN	STARTED ADDING SLURRY 10:12, STOPPED 10:14
		: OUT	CHANGING TO ONE EYE (C)
		10:22 AM: IN	BACK IN TO CLEAN BOTTOM WITH (C) BIT.
116.6	251.3	10:31 AM: OUT	NO MUCH IN BUCKET, BOTTOM CHECK GOOD
		: IN	10:35 ADDING SLURRY, 10:36 STOPPED, CHANGING TO WIRE BRUSH (A)
		: OUT	

KENTUCKY TRANSPORTATION CABINET

Department of Highways
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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ________ Date 2-6-93
Approved By ________ Date ________

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Pier No. 9
Shaft No. 42
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Offset ________

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		367.9	94.3	273.6

Soil Auger Diam. 70"
River Surf. Elev. 358.2
Water Table Elev. ________
Reference Elev. ________
Drilling Mud CDP

Notes: ________

Depth	Elev.	Time		Soil Description & Notes
116.6	251.3	10:50 AM	IN	FOR WALL BRUSHING (WIRE BRUSH (A))
		11:04 AM	OUT	COMPLETE WALL CLEANING
		11:17 AM	IN	STARTED CALIPERING SHAFT
		11:48 AM	OUT	FINISHED CALIPERING SHAFT
		11:54 AM	IN	BACK IN WITH ONE EYE (C) FOR BOTTOM CLEANING
		11:58 AM	OUT	VERY LITTLE IN ONE ENE, SANDY SHALE DARK GREY COLOR
			IN	BOTTOM CHECKED GOOD
			OUT	
		12:05 PM	IN	SAMPLING VISC 48 SEC/DEN.64/PH. 6.6/SAND CONTENT - TRACE
			OUT	
		12:15 PM	IN	PLACING PUMP TO CHANGE SLURRY OUT
		12:48 PM	OUT	STARTED CHANGING SLURRY
		1:10 PM	IN	STILL CHANGING SLURRY
		1:45 PM	OUT	STILL CHANGING SLURRY
		2:12 PM	IN	COMPLETED CHANGING SLURRY
		2:25 PM	OUT	SAMPLING SLURRY, VISC. 56 SEC/DEN. 62.5/PH. 6.8/SAND CONTENT - TRACE
			IN	
		2:38 PM	OUT	PULLED SLURRY PUMP
		2:40 PM	IN	BOTTOM CHECK GOOD
			OUT	
		3:38 PM	IN	STARTED PLACING REBAR CAGE WITH LOAD CELL INTO CASING
		4:00 PM	OUT	STILL PLACING REBAR CAGE
		4:10 PM	IN	ATTACHING LOAD TEST EQUIPMENT
		4:45 PM	OUT	STILL PLACING REBAR CAGE WITH LOAD CELL
		5:15 PM	IN	ATTACHING LOAD TEST EQUIPMENT
		5:45 PM	OUT	PLACING REBAR CAGE IN CASING

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-6-93
Approved By ______ Date ______

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Pier No. 9
Shaft No. 42
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		367.9	94.3'	273.6

Soil Auger Diam. 70"
River Surf. Elev. 358.2
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
		6:12 PM	IN	REBAR CAGE IN PLACE
		6:25 PM	OUT	REBAR CAGE COMPLETE, LOAD CELL IN PLACE
			IN	
			OUT	
		6:30 PM	IN	ATTACHING GROUTING EQUIPMENT
		6:38 PM	OUT	MIXING GROUT
		6:43 PM	IN	STARTED PUMPING GROUT
			OUT	ARE PLANNING TO USE 90 BAGS OF GROUT
		7:15 PM	IN	PUMPING GROUT
		7:50 PM	OUT	FINISHED PUMPING GROUT
		8:05 PM	IN	SETTING STEEL FRAME FOR CONCRETE PIPE
			OUT	
		8:10 PM	IN	TRIED TO LOOSEN DYWIDAG BARS, BUT 1 TOO TIGHT TO DROP CASE
			OUT	
		8:20 PM	IN	PLACING TREMIE PIPE IN SHAFT
		8:50 PM	OUT	PLACING TREMIE PIPE IN SHAFT
		9:10 PM	IN	PLACING LAST PIECE OF TREMIE PIPE
		9:47 PM	OUT	READY FOR CONCRETE AND GROUT
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	

Gal = 116.8

9:00pm
Crazy Horse Injures Knee (left)
Right foot Down Hole Left Leg Doubled up

T' wheeler 360.0 middl

KENTUCKY TRANSPORTATION CABINET
Department of Highways
Division of Construction

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE — Page 1 of ___
Project No. KBD-00101-002-000 — Pier No. 9
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY — Shaft No. 42
Inspected By Tony Colagiovanni — Date 2/5/93 — Station ___
Approved By ___ — Date _/_/_ — Offset ___

360.0
1.1
358.8
9.85
368.33
36

Casing Information				
ID	OD	Top Elev.	Length	Bot.Elev.
		~~367.~~	~~100.6~~	~~255.5~~
		368.3	101.1	251.5

bottom of casing 273.2

Soil Auger Diam. ___
Grnd. Surf. Elev. ___
Water Table Elev. ___
Reference Elev. ___
Drilling Mud ___

Notes ___

359.38
~~9.85~~
368.85

Depth	Elev.	Time		Soil Description & Notes
73.2'		8:50pm	In	Mixed Slurry inside Caisson
		8:55pm	Out	3/4 Full Sandy overburden
		8:57	In	
		9:02	Out	Full Sandy overburden
		9:13	In	Replace teeth on Auger
		9:19	Out	Full Sandy Clay overburden
		9:21	In	
		9:29	Out	Full Sand, Clay overburden
		9:35	In	
		9:41	Out	Full Sand, Clay overburden
		9:43	In	
		9:59	Out	Full Sand, Clay overburden
		10:27	In	ADD Slurry
		10:29	Out	3/4 FULL SAND CLAY OVERBURDEN
84.4		11:00	In	Replace teeth on Auger
		11:06	Out	1/2 to 3/4 Sand Clay overburden
		11:08	In	ADD Slurry
		11:13	Out	1/2 to 3/4 Sand Clay overburden
		11:16	In	
		11:23	Out	Full Sand Clay overburden
		11:25	In	
		11:32	Out	Full Sand Clay overburden
88.3		11:34	In	
		11:40	Out	Full SAND + GRAVEL
		11:42	In	
		11:50	Out	Full Sand + Gravel + Shale

.62
9.45
L12

DRILLED SHAFT SOIL EXCAVATION LOG				Page 2 of
Depth	Elev.	Time		Soil Description & Notes
			In	ADD Slurry
			Out	Visc = 50 Dens = 63 S = Trace PH = 6.4
		12:10 AM	In	
		12:19	Out	Full Sand, Gravel Shale
94.0			In	Change Augers due to material wedging on
			Out	Auger; Change to (2 lift shovel tooth)
		12:47	In	
		12:55	Out	Full Sand, gravel, Shale
		12:57	In	ADD Slurry
		1:07	Out	Full Sand, gravel, shale
		1:40	In	1/2 AUGUR
		1:50	Out	Gray shale & gravel
97.0		1:56	In	
		2:08	Out	Full Gray Shale + gravel
		2:09	In	
		2:16	Out	3/4 Gray shale
		2:19	In	ADD Slurry
		2:25	Out	1/2 Gray shale
		2:26	In	Kelly checked for Plumb, O.K.
		2:37	Out	Full Gray Shale
99.6		2:41	In	
		2:50	Out	1/2 Full Gray Shale
		2:53	In	ADD Slurry
		3:02	Out	1/4 Full Gray shale
99.6			In	Visc = 54 Dens. = 63.5 PH = 6.4
			Out	
		3:15	In	1/4 Gray shale
		3:22	Out	
		3:38	In	swith to Keimeetal auger
		3:50	Out	1/2 auger gray shale
		3:55	In	1/3 auger gray shale
		4:04	Out	
102'8"		4:10	In	1/2 Auger Gray shale
		4:23	Out	
		4:27	In	1/2 Auger Gray Shale
		4:41	Out	
104'1"		5:00	In	Greased swivel on inner kelly
		5:10	Out	1/3 Auger Gray Shale
		5:16	In	
		5:25	Out	1/2 Auger Gray Shale

KENTUCKY TRANSPORTATION CABINET
Department of Highways
Division of Construction

36790
251 5
~~[illegible]~~
1164

CASING LENGTH 94³

DRILLED SHAFT ROCK EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date _/_/_
Approved By ______ Date _/_/_

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Pier No. ___
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Offset ___

Rock Auger Diameter ___
Core Tool Diameter ___
Overream Tool Diameter ___
Notes ___

Reference Elev. ___
Msd. Top of Rock Elev. ___
Msd. Shaft Bottom Elev. ___

Depth	Elev.	Rpm	Time	Rock Description & Notes
			5:31 IN	1/2 AUGER GRAY SHALE
			5:40 OUT	
			5:43 IN	1/2 AUGER GRAY SHALE
			5:53 OUT	
			5:56 IN	ADD SLURRY
			6:03	1/3 AUGER GRAY SHALE
106'2"			6:07 IN	1/3 AUGER GRAY SHALE
			6:16 OUT	
			6:18 IN	1/2 AUGER GRAY SHALE
			6:27 OUT	
			6:30	ADD SLURRY
			6:40	1/3 AUGER GRAY SHALE
			6:43	1/2 AUGER GRAY SHALE
			6:53	
			6:59	2ND SHIFT
109³			7:10	1/2 AUGER GREY SHALE W/ AUGER
			7:34	~~[illegible]~~ REPLACE TEETH
			7:52	1/2 AUGER
			8:00	V=50 S= TRACE PH 6.8 D=63
			8:22	
111.1			8:24	1/2 AUGER GREY SHALE, SANDY
			8:45	
			8:47	VERY LITTLE ON AUGER
			9:15	LONG CYCLE DUE TO CABLE PROBLEMS
			9:17	1/3 AUGER
115⁰			9:50	DARK GREY SHALE
			9:55	1/3 BUCKET
116 =			10:09	GREY SHALE

DRILLED SHAFT ROCK EXCAVATION LOG

Project Name ______________________ Page ___ of ___
Project No. ______________________ Pier No. ______
Contractor ______________________ Shaft No. ______
Inspected By ______________ Date _/_/_ Station ______
Approved By ______________ Date _/_/_ Offset ______

Depth	Elev.	Rpm	Time	Rock Description & Notes
			10:20	1-EYE
116.6			10:31	
			10:50	CHANGE TO BACKSCRATCHER
			11:04	
			11:05	[illegible]
			11:50	CALLIPER
			11:[illegible]	ONE-EYE
			11:59	
				D= 64 PH 6.7 S= <1/4% V-48
			12:05	BEGIN INSTALLING PUMP
			2:39	
				V= 56 S= TRACE PH 6.8 D=62.5
			3:15	PICKING UP CAGE
			3:36	CAGE UP
			5:20	CAGE INSTALLED
			6:30	GROUT PUMP ATTACHED
			6:37	MIXING & PLACING
			6:37	mixing & placing grout
			7:50	
			7:50	clean up Grout equipment, set up to
			9:35	Place concrete

SHIFT CHNAG

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-3-93
Approved By ______ Date ______

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Pier No. 3
Shaft No. 43
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
			IN	DEN. 63.5/VISC. 45 SEC ON SLURRY IN CASING
			OUT	
		7:25 AM	IN	DIRT TOOTH AUGER
		7:28 AM	OUT	SAND, 1/2 AUGER FULL
		7:31 AM	IN	7:32 ADDING SLURRY, 7:35 STOPPED
80.3	287.9	7:37 AM	OUT	DID NOT DRILL
		7:45 AM	IN	WORK ON DRILL CRANE
			OUT	
		8:45 AM	IN	STILL WORK ON DRILL CRANE
			OUT	
		9:15 AM	IN	STILL WORK ON DRILL CRANE
			OUT	
		9:19 AM	IN	DIRT TOOTH AUGER
		9:23 AM	OUT	1/2 LOAD, SAND AND SILT
		9:24 AM	IN	BACK IN
		9:29 AM	OUT	2/3 LOAD, SILT AND SAND
		9:30 AM	IN	9:34 STARTED ADDING SLURRY, 9:35 STOPPED
		9:36 AM	OUT	FULL LOAD, SILT AND SAND
		9:36 AM	IN	BACK IN, 9:37 ADDING SLURRY, 9:38 STOPPED
		9:41 AM	OUT	2/3 LOAD, SAND, SILT AND COBBLESTONE
		9:42 AM	IN	BACK IN
86.5	281.7	9:46 AM	OUT	1/2 LOAD, SAND AND COBBLESTONES
		9:48 AM	IN	BACK IN
		9:53 AM	OUT	2/3 LOAD, SAND AND COBBLESTONES
		9:54 AM	IN	BACK IN, 9:55 AM ADDING SLURRY, 9:56 STOPPED
		9:59 AM	OUT	2/3 LOAD, SAND AND COBBLESTONE

KENTUCKY TRANSPORTATION CABINET

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-3-93
Approved By ______ Date ______

Pier No. 8
Shaft No. 43
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
86.5	281.7	10:03 AM	IN	BACK IN
		10:08 AM	OUT	FULL LOAD, SAND AND COBBLESTONES
		10:09 AM	IN	BACK IN
		10:16 AM	OUT	2/3 LOAD, SAND AND COBBLESTONES
		10:16 AM	IN	BACK IN 10:17 STARTED ADDING SLURRY, 10:19 STOPPED
91.8	276.4	10:21 AM	OUT	2/3 LOAD, SAND, COBBLESTONE AND GRAVEL
		10:23 AM	IN	BACK IN
		10:29 AM	OUT	3/4 LOAD, SAND, COBBLESTONE AND GRAVEL
		10:30 AM	IN	BACK IN
		10:35 AM	OUT	FULL LOAD, SAND, COBBLESTONE AND GRAVEL
		10:37 AM	IN	BACK IN, 10:38 ADDING SLURRY, 10:39 STOPPED
		10:43 AM	OUT	FULL LOAD, SAND, GRAVEL AND COBBLESTONE
		10:44 AM	IN	BACK IN
		10:50 AM	OUT	FULL LOAD, SAND, GRAVEL AND COBBLESTONE
		10:51 AM	IN	BACK IN
		10:58 AM	OUT	2/3 LOAD, SAND, GRAVEL AND COBBLESTONE
		11:04 AM	IN	BACK IN
		11:10 AM	OUT	1/2 LOAD, SAND, COBBLESTONE AND GRAVEL
		11:11 AM	IN	BACK IN, 11:12 ADDING SLURRY, 11:13 STOPPED
99.3	268.9	11:18 AM	OUT	1/2 LOAD, SAND, DARK GREY SHALE AND COBBLESTONE
		11:35 AM	IN	VISC. 45 SEC/DEN. 64/PH 6.8/ SAND - TRACE
			OUT	CHANGED TO NEW TYPE AUGER, NEW TYPE TEETH (CARBIDE TIP TEETH)
		11:46 AM	IN	BACK IN
		11:53 AM	OUT	2/3 LOAD, DARK GREY SHALE, SAND AND COBBLESTONE
		12:00 PM	IN	DOWN FOR LUNCH
			OUT	

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-3-93
Approved By ______ Date ______

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Pier No. 8
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Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time	Soil Description & Notes
99.3	268.9	12:34 PM: IN	BACK IN, 12:33 STARTED ADDING SLURRY, 12:35 STOPPED
		12:39 PM: OUT	1/2 LOAD, DARK GREY SHALE, SAND AND COBBLESTONE
		12:40 PM: IN	BACK IN
102.1	266.1	12:53 PM: OUT	1/2 LOAD, DARK GREY SHALE AND SAND
		12:59 PM: IN	BACK IN, PLUMB CHECK UP & DOWN RIVER 1/16" TO 1' OUT
		: OUT	INDIANA TO KY 1/8" TO 1' OUT
		: IN	
		1:13 PM: OUT	2/3 LOAD, DARK GREY SHALE, LIGHT GREY SHALE AND SAND
		1:16 PM: IN	BACK IN, 1:18 STARTED ADDING SLURRY, 1:20 STOPPED
		1:27 PM: OUT	1/2 LOAD, DARK GREY SHALE, SAND AND LIGHT GREY SHALE
		1:29 PM: IN	BACK IN
		1:47 PM: OUT	1/2 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		1:57 PM: IN	BACK IN, 1:59 ADDING SLURRY, 2:02 STOPPED
105.5	262.7	2:10 PM: OUT	2/3 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		2:18 PM: IN	BACK IN, VICS. 46 SEC/DEN. 64/PH. 7.0/SAND 0.2%
		2:34 PM: OUT	1/2 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		2:38 PM: IN	BACK IN
		2:54 PM: OUT	1/3 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		2:59 PM: IN	BACK IN, 2:59 STARTED ADDING SLURRY, 3:02 STOPPED
		3:12 PM: OUT	1/3 LOAD DARK GREY SHALE AND LIGHT GREY SHALE
		3:13 PM: IN	BACK IN
108.4	259.8	3:29 PM: OUT	2/3 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		3:39 PM: IN	BACK IN
		3:55 PM: OUT	1/2 LOAD, DARK GREY SHALE AND LIGHT GREY SHALE
		3:57 PM: IN	BACK IN, STARTED ADDING SLURRY 3:57, STOPPED 4:00 PM
		4:17 PM: OUT	1/4 LOAD, DARK GREY SHALE, LIGHT GREY SHALE

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DRILLED SHAFT SOIL EXCAVATION LOG

149.2

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ________ Date 2-3-93
Approved By ________ Date ________

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Pier No. 8
Shaft No. 43
Station ________
Offset ________

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	260.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev. ________
Reference Elev. ________
Drilling Mud CDP

Notes: ________

Depth	Elev.	Time	Soil Description & Notes
110.9	257.3	4:22 PM: IN	CHANGING AUGERS- VIS. 47 SEC.- DEN. 64- PH 7.0 SAND .25%
		____: OUT	ROCK TOOTH AUGER
		4:37 PM: IN	BACK IN, PLUMB CHECK, UP & DOWN RIVER 1/16" TO 1" OUT, KY TO IN 1/8" TO 1" OUT
		4:48 PM: OUT	1/3 LOAD DARK GREY SHALE
		4:50 PM: IN	BACK IN
		5:01 PM: OUT	1/3 LOAD OF DARK GREY SHALE & COAL
		5:04 PM: IN	BACK IN
		5:13 PM: OUT	2/3 LOAD DARK GREY SHALE & COAL
		5:15 PM: IN	BACK IN- 5:16 PM STARTED ADDING SLURRY- STOPPED AT 5:19 PM
		5:23 PM: OUT	2/3 LOAD OF DARK GREY SHALE & COAL
		5:24 PM: IN	BACK IN
116.7	251.5	5:34 PM: OUT	1/3 LOAD OF DARK GREY SHALE (LIGHT & DARK)
		5:36 PM: IN	BACK IN
		5:48 PM: OUT	1/3 LOAD OF LIGHT GREY SHALE
		5:49 PM: IN	BACK IN, 5:51 PM STARTED ADDING SLURRY - STOPPED AT 5:53 PM
		6:05 PM: OUT	1/2 LOAD OF LIGHT GREY SHALE
		6:06 PM: IN	BACK IN, GREASED KELLY
119.5	248.7	6:27 PM: OUT	1/3 LOAD OF LIGHT GREY SHALE
		6:30 PM: IN	BACK IN, 6:31 PM STARTED ADDING SLURRY- STOPPED AT 6:33 PM
		6:48 PM: OUT	1/2 LOAD OF LIGHT GREY SHALE
		6:53 PM: IN	ADDING COOLANT TO DRILL CRANE
[illegible]0.3	247.9	7:02 PM: OUT	FUELING EQUIPMENT
		7:13 PM: IN	DRILLING STARTS, 2/3 LOAD LIGHT GREY SHALE
		7:23 PM: OUT	FUELING CRANE (FOR KELLY), ADDING SLURRY
121.4	246.8	7:30 PM: IN	STILL FUELING, TEST # VIS. 47- DEN. 65- SAND 0.75%- PH 6.7
		8:05 PM: OUT	

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By
Approved By
Date 2-3-93
Date

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Pier No. 8
Shaft No. 43
Station
Offset

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev.
Reference Elev.
Drilling Mud CDP

Notes:

Depth	Elev.	Time	Soil Description & Notes
		8:05 PM: IN	CHANGING TO SOIL AUGER, WITH ROCK BITS ON THE SIDE
		8:12 PM: OUT	
121.4	246.8	8:15 PM: IN	START DRILLING, CLEANED AUGER
		8:30 PM: OUT	3/4 LOAD GREY SHALE (LIGHT & DARK)
		8:34 PM: IN	3/4 LOAD, DARK GREY SHALE & LIGHT GREY SHALE
		8:41 PM: OUT	CLEANED AUGER
		8:46 PM: IN	DARK & LIGHT GREY SHALE
		8:55 PM: OUT	CLEANED AUGER
		8:57 PM: IN	SAME AS ABOVE- ADDED SLURRY
123.0	245.2	9:03 PM: OUT	CLEANED AUGER, SOUNDING, MOVED DUMP BED
		9:12 PM: IN	2/3 LOAD DARK & LIGHT GREY SHALE, CLEANING
125.4	242.8	9:26 PM: OUT	AUGER & SOUNDING
		9:28 PM: IN	2/3 LOAD DARK & LIGHT GREY SHALE
		9:40 PM: OUT	CLEANED AUGER
		9:45 PM: IN	9:58 PM CRANE OVER HEATED, 10:12 PM DRILL STARTS.
		: OUT	10:20 PM OVER HEATED AGAIN, 10:25
		: IN	DARK & LIGHT GREY SHALE, SOUNDING
127.6	240.6	10:28 PM: OUT	FULL LOAD, CLEANED AUGER, ADDED SLURRY
		10:38 PM: IN	3/4 LOAD, DARK & LIGHT GREY SHALE, CLEANED AUGER
		10:43 PM: OUT	MOVED DUMP BED
		10:45 PM: IN	1/3 LOAD DARK & LIGHT GREY SHALE
		10:55 PM: OUT	CLEANED AUGER & REPLACING TEETH
		11:08 PM: IN	11:23 PM CRANE OVER HEATED
129.0	239.2	: OUT	HOLE IN RADIATOR, DARK & LIGHT GREY SHALE
		: IN	SOUNDING, CLEANING AUGER
		11:42 PM: OUT	CRANE STILL OVERHEATING

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Department of Highways
Division of Construction

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By
Date 2-3 93
Approved By
Date

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Pier No. 8
Shaft No. 43
Station
Offset

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 359.4
Water Table Elev.
Reference Elev.
Drilling Mud CDP

Notes:

Depth	Elev.	Time	Soil Description & Notes
9.0	239.2	11:43 PM: IN	TEST #, VIS. 48 - DEN. 65
		: OUT	SAND 0.75%- PH 6.8
		12:00 AM: IN	OILING RINGS ON THE DRILL, 12:08 AM DRILL STARTS
		12:17 AM: OUT	1/2 LOAD DARK & LIGHT GREY SHALE, ADDING SLURRY
		12:20 AM: IN	CABLE TROUBLE 12:25, CHANGING AUGER
		12:45 AM: OUT	1/2 LOAD GREY SHALE
		12:55 AM: IN	LUNCH
		1:30 AM: OUT	
		1:31 AM: IN	WORKING ON CRANE (RADIATOR)
		2:01 AM: OUT	STILL WORKING ON CRANE
		2:01 AM: IN	BLOWING OUT RADIATOR
		2:30 AM: OUT	STILL WORKING ON CRANE
		2:30 AM: IN	REPAIRING HOLE IN RADIATOR
		3:00 AM: OUT	STILL WORKING ON CRANE, CHANGING AUGER
		3:02 AM: IN	ROCK AUGER USED, SOME BROWN SHALE
		3:28 AM: OUT	1/3 LOAD- DARK & LIGHT GREY SHALE- CLEANED AUGER
		3:32 AM: IN	1/2 LOAD BROWN & DARK SHALE, SOUNDING
130.7	237.3	3:40 AM: OUT	CLEANED AUGER, ADDED SLURRY
		: IN	WATCH STOPPED, SET UP TO 4:00 AM
		: OUT	LOST 20 MIN.
		4:02 AM: IN	BROWN & DARK GREY SHALE, MOVING DUMP BED
		4:13 AM: OUT	CLEANING AUGER
		4:20 AM: IN	MOSTLY BROWN & DARK GREY SHALE
		4:30 AM: OUT	CLEANING AUGER
		4:32 AM: IN	BROWN SHALE AND SOME DARK GREY SHALE
		4:45 AM: OUT	CLEANING AUGER, ADDED SLURRY

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By
Approved By
Date 2-4-93
Date

Page 7 of 12
Pier No. 8
Shaft No. 43
Station
Offset

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74'		368.2	100'	268.2

Soil Auger Diam. 70'
River Surf. Elev. 358.9
Water Table Elev.
Reference Elev.
Drilling Mud CDP

Notes:

Depth	Elev.	Time		Soil Description & Notes
		4:50 AM	IN	BROWN SHALE & SOME DARK GREY SHALE
135.0	233.2	5:09 AM	OUT	CLEANING AUGER, 1/2 LOAD
		5:17 AM	IN	SAME AS ABOVE
		5:32 AM	OUT	
		5:36 AM	IN	1/3 LOAD, BROWN SHALE& SOME DARK GREY SHALE
		5:49 AM	OUT	CLEANED AUGER , ADDED SLURRY
		5:53 AM	IN	1/2 LOAD, BROWN & DARK GREY SHALE
136.1	232.1	6:03 AM	OUT	CLEANING AUGER & SOUNDING
		6:07 AM	IN	1/2 LOAD- BROWN SHALE & DARK GREY SHALE
		6:24 AM	OUT	CLEANING AUGER
		6:25 AM	IN	1/4 LOAD BROWN SHALE & DARK GREY
		6:43 AM	OUT	
		6:45 AM	IN	GREASING KELLY RINGS- 6:53 AM DRILL STARTS
SHIFT CHANGE		7:25 AM	OUT	VERY LITTLE ON BIT
			IN	CHANGING AUGERS- CHANGED TO CORE BIT
			OUT	
		7:33 AM	IN	IN WITH CORE BIT
		8:54 AM	OUT	ADDING STEEL CABLE TO CORE BIT
		9:02 AM	IN	BACK IN WITH CORE BIT CABLE ATTACHED
		9:14 AM	OUT	NOTHING ON CORE BIT
		9:30 AM	IN	FUEL EQUIPMENT
			OUT	
		9:47 AM	IN	BACK IN WITH KELLY BAR TO BREAK LIMESTONE UP
		10:00 AM	OUT	ATTACHING 40' BIT WITH ROCK TEETH
		10:07 AM	IN	BACK IN SMALL BIT
		10:55 AM	OUT	SAMPLED TOP VIS. 48 SEC.- DEN. 65- PH 6.8

KENTUCKY TRANSPORTATION CABINET

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Division of Construction

149.2

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-4-93
Approved By ______ Date ______

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Pier No. 8
Shaft No. 43
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Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 358.9
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
136.1	232.1		IN	
		11:18 AM	OUT	VERY LITTLE DARK GREY SHALE
138.9	229.3	11:30 AM	IN	WITH KELLEY BAR TO BREAK ROCK, VISC. 49 SEC/DEN. 64/PH. 6.6/SAND .25%
			OUT	
			IN	
		11:46 AM	OUT	KELLEY OUT, ATTACHING BIT, (ROCK TOOTH AUGER)
		11:51 AM	IN	IN WITH ROCK AUGER
		12:26 PM	OUT	1/2 FULL LIMESTONE AND SHAVINGS
		12:27 PM	IN	
140.5	227.7	12:47 PM	OUT	1/2 LOAD, LIGHT GREY SHALE AND LIMESTONE
		12:56 PM	IN	BACK IN, 12:56 STARTED ADDING SLURRY, 12:58 STOPPED
			OUT	PLUMB CHECK, IND TO KY PLUMB UP AND DOWN RIVER 1/8 TO 1' OUT
			IN	
		1:13 PM	OUT	2/3 LOAD, LIGHT GREY SHALE, LIMESTONE AND SAND
		1:16 PM	IN	BACK IN, 1:18 STARTED ADDING SLURRY, 1:19 STOPPED
142.8	225.4	1:26 PM	OUT	1/8 LOAD, LIGHT GREY SHALE, LIMESTONE AND SAND
			IN	VISC. 49 SEC/DEN. 65/PH. 6.8/SAND CONTENT .5%
			OUT	
		1:37 PM	IN	BACK IN
		2:05 PM	OUT	STOPPED DRILL, STILL IN SHAFT WORKING ON CABLE ON DRILL CRANE
			IN	2:15 STARTED BACK DRILLING
142.8	225.4	2:21 PM	OUT	1/2 LOAD OF LIGHT GREY SHALE
		2:24 PM	IN	
144.7	223.5	2:50 PM	OUT	1/2 LOAD OF LIGHT GREY SHALE
		2:54 PM	IN	STARTED ADDING SLURRY AT 2:59, STOPPED 3:00
146.3		3:38 PM	OUT	FULL LOAD LIGHT GREY SHALE

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-4-93
Approved By ______ Date ______

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Pier No. 8
Shaft No. 43
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Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 358.9
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time	Soil Description & Notes
146.3	221.9	3:43 PM: IN	3:44 ADDED SLURRY, 3:46 STOPPED
147.6	220.6	4:02 PM: OUT	1/2 LOAD LIGHT GREY SHALE
		4:15 PM: IN	CHANGED TO ONE EYE OR MUCK BUCKET (ADDED ABOUT 30 LBS DRY SLURRY)
		4:19 PM: OUT	ADDING DRY SLURRY TO BRING UP VISC. FOR WIRE BRUSHING.(DARYL THINKS
		IN	BRUSHING MAY REMOVE SLURRY FROM WHAT IS ALREADY COATED BY INCREASING
		OUT	VISC. IT MAY COAT THE ROCK SOCKET AGAIN. WANT VISC. AT 55 SEC.
		IN	
		OUT	
		4:20 PM: IN	4:20 ADDING SLURRY, 4:22 STOPPED
148.3	219.9	4:23 PM: OUT	4:25 ADDING SLURRY, 4:27 STOPPED
		4:27 PM: IN	BACK IN, ONE EYE OR MUCK BUCKET
148.4	219.8	4:34 PM: OUT	VERY LITTLE IN BUCKET, LIGHT GREY SHALE
		IN	SAMPLE TOP VISC. 48 SEC., CHANGED TO ROCK AUGER
		OUT	
		4:45 PM: IN	BACK IN WITH ROCK AUGER
148.8	219.4	5:04 PM: OUT	2/3 LOAD LOAD GREY SHALE
		5:12 PM: IN	BACK IN WITH ONE EYE OR MUCK BUCKET
149.8	218.4	5:18 PM: OUT	VERY LITTLE IN BUCKET, LIGHT GREY SHALE
		5:28 PM: IN	BACK IN, WIRE BRUSH
		5:38 PM: OUT	WIRE BRUSH OUT, LIGHT COATING OF LIGHT GREY SHALE
		5:51 PM: IN	BACK IN WITH ONE EYE, ADDING SLURRY 5:44. STOPPED 5:47.
		5:57 PM: OUT	A LITTLE IN BUCKET, LIGHT GREY SHALE
		6:00 PM: IN	BACK IN WITH ONE EYE
149.9	218.3	6:07 PM: OUT	VERY LITTLE IN BUCKET, LIGHT GREY SHALE
		IN	SAMPLING VISC. 52 SEC/DEN. 64.5/PH. 6.8/SAND CONTENT .25%
		OUT	

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

149.2

DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-4-93
Approved By ______ Date ______

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Pier No. 8
Shaft No. 43
Station ______
Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. 358.9
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
149.9	218.3	6:12 PM:	IN	BOTTOM CHECK OK
			OUT:	
		6:24 PM:	IN	PLACING CALIPER IN SHAFT
		6:46 PM:	OUT:	FINISH CALIPERING
		6:49 PM:	IN	BOTTOM NEEDS CLEANING
			OUT:	
		6:56 PM:	IN	BACK IN WITH ONE EYE FOR BOTTOM CLEANING
		7:01 PM:	OUT:	VERY LITTLE IN BUCKET, LIGHT GREY SHALE
149.9	218.3	7:05 PM:	IN	BOTTOM CHECK, CLEAN
			OUT:	PLACING SLURRY PUMP IN SHAFT
149.9	218.3	7:27 PM:	IN	REPLACING SLURRY, 8:10 STILL REPLACING
			OUT:	SLURRY, 8:47 PLACING SLURRY
		8:15 PM:	IN	REMOVING DYWIDAG BARS FROM HOLE 42
			OUT:	3 OF THE BARS WIELD POINTS BROKE, PUMPING
			IN	DOWN SHAFT 42 BELOW BREAK POINTS TO REPAIR
			OUT:	
		9:15 PM:	IN	REMOVING SLURRY PUMP FROM HOLE 43
149.7	218.5	9:45 PM:	OUT:	SOUNDINGS 2" OF MATERIALS
		10:00 PM:	IN	CLEANING HOLE 43 WITH ONE EYE
		10:45 PM:	OUT:	HOLE READY FOR STEEL, 1/2" SEDIMENT ON BOTTOM
		10:45 PM:	IN	TEST # VISC. 52 SEC/DEN. 65/SAND-TRACE/PH. 6.8
149.9	218.3		OUT:	
		11:00 PM:	IN	STILL REPAIRING DYWIDAG BARS IN 42
		11:50 PM:	OUT:	FINISHED WITH BARS, GETTING READY FOR CAGE
		12:05 AM:	IN	RAISING AND PLACING CAGE
		2:00 AM:	OUT:	CAGE IN PLACE

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ________ Date 2-5-93
Approved By ________ Date ________

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Pier No. 8
Shaft No. 43
Station ________
Offset ________

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. ________
Water Table Elev. ________
Reference Elev. ________
Drilling Mud CDP

Notes: ________

Depth	Elev.	Time		Soil Description & Notes
		2:30 AM	IN	WELDING 1ST SECTION OF INSTRUMENTATION
		4:00 AM	OUT	WELDING 2ND SECTION OF INSTRUMENTATION
		5:10 AM	IN	3RD SECTION HAD TO BE CUT TO 8'
		6:20 AM	OUT	STILL WORKING ON BARS
		6:40 AM	IN	SETTING STEEL TO GRADE
SHIFT CHANGE		7:00 AM	OUT	STILL CUTTING OF INNER PIPE
		7:15 AM	IN	COMPLETED INSTALL REBAR CAGE AND LOAD CELL
			OUT	PREPARING TO PUMP GROUT
		7:43 AM	IN	STARTED PUMPING GROUT
			OUT	
		8:00 AM	IN	STILL PUMPING GROUT
		8:35 AM	OUT	GROUT PUMP DOWN
		8:45 AM	IN	STARTED POURING GROUT INTO PIPE
			OUT	(USING PIPE AS A TREMIE)
		9:00 AM	IN	STILL POURING GROUT INTO PIPE
		9:30 AM	OUT	STILL POURING GROUT INTO PIPE
		9:44 AM	IN	GROUTING COMPLETED, 90 BAGS
		9:55 AM	OUT	PLACING TEMPLATE FOR TREMIE
		10:14 AM	IN	PLUGGED TREMIE PIPE, STARTED PLACING PIPE IN SHAFT *SEE NOTE
			OUT	
		10:30 AM	IN	STILL PLACING TREMIE IN PIPE
		11:04 AM	OUT	PULL TREMIE PIPE, PLUG CAME OFF
		11:14 AM	IN	STARTED TREMIE PIPE BACK IN SHAFT
		11:45 AM	OUT	ATTACHING LAST SECTION TO TREMIE PIPE
		11:50 AM	IN	BATCHING GROUT TO PRIME CONCRETE PUMP
		11:54 AM	OUT	ATTACHING CONCRETE PUMP TO TREMIE

*NATIONAL WAS INFORMED WHERE TO PLACE TREMIE PIPE BY LOAD TEST. TREMIE PIPE WOULD NOT GO INTO SHAFT. NATIONAL MOVED 180 DEGREES TO OTHER SIDE OF CASING, HAD NO TROUBLE PLACING TREMIE.

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
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Inspected By ______ Date 2-5-93
Approved By ______ Date ______

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Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot. Elev.
74"		368.2	100'	268.2

Soil Auger Diam. 70"
River Surf. Elev. ______
Water Table Elev. ______
Reference Elev. ______
Drilling Mud CDP

Notes: ______

Depth	Elev.	Time		Soil Description & Notes
		11:56 AM	IN	CLEANING CONCRETE BUCKET
		12:10 PM	OUT	LOAD CONCRETE BUCKET WITH GROUT TO PRIME CONCRETE PUMP
			IN	
		12:14 PM	OUT	PRIMING PUMP
		12:20 PM	IN	SAMPLING CONCRETE
		12:22 PM	OUT	STARTED PUMPING CONCRETE INTO SHAFT
		12:41 PM	IN	SAMPLING CONCRETE, STOPPED CONCRETING
		1:06 PM	OUT	WORKING ON TREMIE PIPE
		1:13 PM	IN	SAMPLING CONCRETE
		1:30 PM	OUT	SAMPLING CONCRETE
		3:00 PM	IN	SAMPLING CONCRETE
		3:30 PM	OUT	REMOVING SECTIONS OF TREMIE PIPE
		4:20 PM	IN	SAMPLING CONCRETE
		4:50 PM	OUT	COMPLETED PLACING CONCRETE INTO SHAFT
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	
			IN	
			OUT	

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DRILLED SHAFT SOIL EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE   Page ___ of ___
Project No. KBD-00101-002-000   Pier No. 8
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY   Shaft No. 43
Inspected By ________________ Date _/_/_   Station ______
Approved By ________________ Date _/_/_   Offset ______

Casing Information

ID	OD	Top Elev.	Length	Bot.Elev.
		368'		

Soil Auger Diam. ______
Grnd. Surf. Elev. ______
Water Table Elev. ______
Reference Elev. ______
Drilling Mud ______

Notes ______________________________

Depth	Elev.	Time		Soil Description & Notes
			In	
			Out	
			In	
			Out	
			In	SAND USING 60° DIGGING AUGER
		7:06	Out	
		7:25	In	DELAY DUE TO COOLING SYSTEM ON 4000 W.
80.3	287.9	7:30	Out	SAND
		9:20	In	DELAY DUE TO COOLING SYSTEM ON 4000 W
		9:23	Out	SAND
		9:24	In	SAND & GRAVEL
		9:30	Out	
		9:31	In	SAND/GRAVEL
		9:35	Out	
		9:36	In	SAND/GRAVEL
		9:40	Out	
		9:42	In	SAND GRAVEL
86.5		9:47	Out	
		9:49	In	SAND/GRAVEL V IN TANKS = 65
		9:53	Out	
		9:54	In	SAND/GRAVEL
		9:59	Out	EMPTY THE BED
		10:04	In	SAND/GRAVEL
		10:09	Out	
		10:10	In	SAND/GRAVEL
		10:16	Out	

DRILLED SHAFT SOIL EXCAVATION LOG Page___ of ___

Depth	Elev.	Time		Soil Description & Notes
		10:17	In	SAND/GRAVEL
91.75		10:22	Out	
		10:24	In	SAND/GRAVEL
		10:29	Out	
		10:30	In	SAND/GRAVEL
		10:56	Out	
		10:38	In	
		10:44	Out	
		10:45	In	SAND w/ COBBLES
		10:50	Out	
		10:51	In	SAND w/COBBLES
		10:57	Out	
		11:04	In	DUMP BED
		11:11	Out	SAND w/ COBBLES
		11:12	In	CLAY w/ GRAVEL & SAND
		11:19	Out	PH 6.8 V=45 S=TRACE D=64
✓		11:46	In	CHANGE TO NEW AUGER 40° PITCH.
		11:54	Out	FULL LOAD ~~[illegible]~~ SHALE
		12:33	In	1/2 FULL
		12:40	Out	GREY SHALE w/ SAND
		12:46	In	
		12:54	Out	GREY SHALE STICKING TO AUGER
102.1		12:59	In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	
			In	
			Out	

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DRILLED SHAFT ROCK EXCAVATION LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE   Page ___ of ___
Project No. KBD-00101-002-000   Pier No. ______
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY   Shaft No. ______
Inspected By ____________ Date _/_/_   Station ______
Approved By ____________ Date _/_/_   Offset ______

Rock Auger Diameter ______   Reference Elev. ______
Core Tool Diameter ______   Msd. Top of Rock Elev. ______
Overream Tool Diameter ______   Msd. Shaft Bottom Elev. ______
Notes ______

Depth	Elev.	Rpm	Time	Rock Description & Notes
102.1			12:59	SHALE 2/3 FULL PLUMBED KELLY
			1:13	
			1:18	
			1:28	
			1:30	GREY SHALE W/ SAND
			1:44	STUCK TO AUGER
			1:51	CLEANED AUGER
105.5			1:59	PH=7 S=.16% V=.46 D=64
			2:10	
			2:18	DARK GREY SHALE
			2:35	STUCK TO AUGER
			2:39	GREY SHALE
			2:55	
			3:00	1/3 FULL GREY SHALE
			3:12	AUGER CLEANED ITSELF
			3:14	1/2 FULL GREY SHALE, DRY.
108.4			3:29	DIFFICULT CLEANING.
			3:39	
			3:55	GREY SHALE, DRY
			3:57	PH=7 S=1/4% V=47 D=64PCF
110.9			4:17	
			4:35	CHANGE TO 6' D.H. AUGER W/ ROCK TEETH
			4:49	
			5:04	DK GREY SHALE 1/2 FULL
			5:15	
			5:17	DK GREY SHALE W/ COAL
			5:26	

DRILLED SHAFT ROCK EXCAVATION LOG

Project Name ______________________ Page ___ of ___
Project No. ______________________ Pier No. ______
Contractor ______________________ Shaft No. ______
Inspected By ______________ Date _/_/_ Station ______
Approved By ______________ Date _/_/_ Offset ______

Depth	Elev.	Rpm	Time	Rock Description & Notes
			5:27	1/4 FULL AUGER
116.66			5:36	GREY SHALE NO COAL
			5:39	1/2 FULL
			5:49	LT. GREY SHALE
			5:51	1/2 FULL
			6:07	LT. GREY SHALE
			6:08	1/2 FULL
119.5			6:29	LT. GREY SHALE
			6:32	2/3 FULL
			6:48	LT. GREY SHALE
			7:16 pm	1/2 Full
			7:26 pm	LT. Gray shale
121.4'				V=47 Den=65 Sand= .75% PH=6.7 ADD SLURRY
				Change Augers (New Auger w/ Flat teeth)
			8:17	Full Auger
			8:33	LT. Gray Shale
			8:37	1/2 Full
			8:45	LT. Gray Shale
			8:49	ADD Slurry
			8:58	1/2 Full LT. Gray shale
			9:00	
			9:11	1/2 Full Lt. Gray Shale
123.0'			9:15	Add Slurry
			9:30	1/2 to 3/4 gray shale
125.4			9:32	
			9:43	1/2 Full gray shale
			9:47	Rig overheated
			10:32	3/4 to Full Auger LT. Gray shale Add Slur
127.7			10:40	Replace 3 teeth
			10:51	3/4 Full LT. Gray shale
			10:55	
			11:05	1/4 Full Lt. Gray Shale
			11:19	Rig Overheated
			11:52	1/2 to 3/4 Full lite gray shale
129.0				V:48 DenS=65 Sand= .75% PH= 6.8
				ADD Slurry

Shift change 121.4'

DRILLED SHAFT SOIL EXCAVATION LOG

Depth	Elev.	Time		Soil Description & Notes
129.0		12:07 AM	In	
		12:28	Out	1/4 Full Gray Shale
		12:31	In	Cable Problems
		12:55	Out	1/4 to 1/2 Full Gray Shale
			In	Change Augers to 4 lift Rock Auger
			Out	Buck + Roy worked on Drill Rig (Lunch)
		3:13 AM	In	
		3:38	Out	1/4 Full Gray Shale
		3:42	In	
		3:55	Out	1/4 Full Gray Shale
130.8		4:05	In	Add Slurry
		4:13	Out	Gray Shale
		4:20	In	
		4:30	Out	Gray Shale
		4:32	In	
		4:47	Out	1/2 Full Gray Shale
		4:52	In	ADD Slurry
		5:13	Out	1/4 to 1/2 Full Gray Shale
135.0		5:20	In	Replace Teeth on Auger.
		5:35	Out	1/4 Full Gray Shale
		5:39	In	ADD Slurry
		5:50	Out	1/4 Full Gray Shale
		5:55	In	
		6:05	Out	1/4 Full Gray Shale
136.1		6:10	In	ADD Slurry
		6:24	Out	1/4 Full Gray Shale
		6:30	In	
		6:45	Out	Less than 1/4 Full Gray Shale
		6:48	In	
		7:26	Out	LESS THAN 1/4 FULL DK GREY SHALE
		7:33	In	SWITCH TO CORE BARREL.
		8:54	Out	BROUGHT OUT VERY LITTLE
		8:55	In	PLACING CABLE INSIDE CORE BARREL
			Out	
		9:03	In	CONTINUING W/ ADAPTED CORE BARREL.
		9:15	Out	NOTHING ON CORE BARREL
		9:30	In	
			Out	
		9:47	In	BREAK UP LIMESTONE
		10:00	Out	

DRILLED SHAFT ROCK EXCAVATION LOG

Project Name ______________________ Page ___ of ___
Project No. ______________________ Pier No. ______
Contractor ______________________ Shaft No. ______
Inspected By ______________________ Date __/__/__ Station ______
Approved By ______________________ Date __/__/__ Offset ______

WATER ELEV. = 359²

Depth	Elev.	Rpm	Time	Rock Description & Notes
			10:07	11:00 SAMPLED OFF TOP OF CAISSON V= 48 D=65
			11:18	
138.9'			11:30	TOOK OFF 48" AUGER / BROKE UP ROCK WITH KELLY BAR
			11:46	TOOK COMPOSITE SAMPLE V=49. D=64 PH=6.6 SAND=.25%
			11:51	PUT 6' "BULLET TOOTH" ROCK AUGER (BROKE THROUGH LIMESTONE
			12:27	BROKEN LIMESTONE PIECES MIXED WITH GREY SHALE
140.5'			12:28	BROKEN LIMESTONE MIXED WITH GREY SHALE
			12:49	CLEAN AUGER – SPOILS STUCK TO IT
			12:55	AIR SLURRY – PLUMBED KELLY – CLEANED AUGER
			1:14	BROKEN LIMESTONE MIXED WITH GREY SHALE & SAND AUGER 2/3 FULL
142.8			1:15	AIR SLURRY – 1/5 LOAD LIMESTONE FRAG./SHALE/SAND COMPOSITE TEST
			1:26	V=49 D=65 PH:6.8 SAND .5%
			1:37	1 HELIX FULL
142.8			2:22	GREY SHALE, LIGHT.
144.8			2:25	GREY SHALE
			2:50	
			2:56	CLEAN AUGER
146.3			3:38	FULL AUGER LT. GREY SHALE
			3:40	1/2 FULL AUGER
147.7			4:05	
~~[illegible]~~			4:15	CHANGE TO 1-EYE
148.25			4:23	
			4:28	1-EYE
148.33			4:35	V=48
			4:46	SWITCH TO [illegible] ROCK AUGER
148.83			504	2/3 FULL LT. GREY SHALE
			5:13	SWITCH TO 1-EYE
149.75			5:20	
			5:25	BACKSCRATCHER
			5:39	
			5:52	RECLEAN W/ 1-EYE.
			5:54	STATE SAYS "NOT CLEAN"
			6:00	V=52 P=6.8 D=64.5 S.25%
			6:25	BEGAN CALIPER

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DRILLED SHAFT ROCK EXCAVATION LOG

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Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
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Pier No. ______
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Rock Auger Diameter ______
Core Tool Diameter ______
Overream Tool Diameter ______
Notes ______

Reference Elev.
Msd. Top of Rock Elev. ______
Msd. Shaft Bottom Elev. ______

Depth	Elev.	Rpm	Time	Rock Description & Notes
			6:26 PM	CON'T CALIPER
			6:45	OUT OF HOLE W/ CALIPER
				CALIPER IS
			6:49	[redacted] OFF 4100
			6:56	BACK IN W/ 1-EYE PER KDOT
NO CHANGE			7:02	Replace Slurry's
			10:05	Back in w/ 1-Eye. per KDOT
			10:18	
149.7				V=52 Sec Den=65 Sand-Trace
				PH = 6.8

RESTEEL PLACEMENT @ 1:00

DRILLED SHAFT ROCK EXCAVATION LOG

Project Name ____________________ Page ___ of ___

Project No. ____________________ Pier No. ______

Contractor ____________________ Shaft No. ______

Inspected By ____________________ Date _/_/_ Station ______

Approved By ____________________ Date _/_/_ Offset ______

Depth	Elev.	Rpm	Time	Rock Description & Notes
			7:20	GROUT PUMP ATTACHED
			7:37	BEGIN PUMPING GROUT.
			8:35	GROUT PUMP BROKE DOWN
			8:45	BEGAN POURING GROUT WITH 5 gal. BUCKETS
			9:44	FINISHED MANUAL GROUTING
				90 BAGS ± PER JEFF OF "LOADTEST"
			9:50	HOOK UP/PLACE TREMIE TEMPLATE
			10:15	SET TREMIE AS PER LOADTEST'S REP. DIRECTION
			11:05	PULL OUT TREMIE - HITTING OBSTRUCTIONS IN HOLE WHERE 'LOADTEST'S' REP TOLD US TO PLACE IT.
			11:05	ASK LOADTEST'S REP IF LOCATION OF TREMIE COULD BE ELSEWHERE IN CAISSON
				HE INDICATED TO US THAT WHAT THEY REQ'D AND IF WE COULD PLACE IT ELSEWHERE
				& MEET REQ'MTS IT WOULD BE O.K.
			11:15	PLACE TREMIE IN LOCATION WE FELT WOULD BEST WORK IN CURRENT CONDITIONS - O.K. WITH "LOADTEST'S" REP
			11:45	FINISHED INSTALLING TREMIE - NO OBSTRUCTIONS MET DURING THIS INSTALLATION.
				SEE CONC. PLACEMENT SHEETS
			5:25pm	BEGIN TREMIE ASSEMBLY
			5:50 pm	BEGIN AIRLIFT SETTING
			5:55	MOVE MAT'L w/ AIRLIFT
			7:00	FINISH AIRLIFT

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DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
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Approved By ______ Date ______

Page 1 of 2
Pier No. 9
Shaft No. 42
Station ______
Offset ______

Placement Method ___ Tremie
X Pumped
Deairing Method ___ Relief Valve
___ Tremie Plug
X Tremie Cap

Volume in Lines	#	ID	Length	Volume
Total Volume in Lines				

Reference Elev. 370.2 = 2'4" ABOVE CASING
Shaft Top Elev. 367.9
Top of Rock Elev. 273.6
Shaft bottom Elev. 251.3

Depth to Water Inside ___ ID Casing at Start ___
Rebar Cage Top Elev. at Start 328.8 at Finish ___

Bucket No.	CY Concrete Volume	Conc in Arrival Bucket Time	Start Time	Finish Time IN PUMP	Tremie Depth	Depth to Concrete	Notes
	1	9:48PM			117.0	117.0	GROUT TO PRIME PUMP
1	4	9:54PM		10:01PM			SLUMP 7", TEMP. 69, AIR 8.0
2	8	10:02PM		10:08PM			
3	12	10:09PM		10:15PM			
4	16	10:16PM		10:23PM	114.0	107.6	RAISED TREMIE 3', AIR 5.8
5	20	10:24PM		10:33PM			AIR 7.3, TEMP 75, SLUMP 6.75
6	24	10:34PM		10:42PM	110.8	100.2	RAISED TREMIE 3.2'
7	28	10:43PM		10:52PM			
8	32	10:53PM		11:01PM	105.0	89.5	RAISED TREMIE 5'.6
9	36	11:02PM		11:10PM			
10	40	11:11PM		11:19PM			
11	44	11:20PM		11:32PM	91.5	79.0	PROBLEM WITH SLURRY, PUMP DOWN 4 MINUTES
12	48	11:33PM		11:44PM			AIR 7.0, TEMP 65, SLUMP 7.5"

48 Concrete Volume Delivered

Placement Time (Casing Removed) NO

	ID	Top Elev.	Bot. Elev.	Start	Finish
Casing		367.9	273.6		
Removal					

Rebar Cage Centered STANDEE
Concrete Finished ______

Notes: #5 - 02-06-AM.A #12 - 02-006-AM.BCD EARLY BREAK CYLINDER #18 - 02-006-AM.E

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

FINAL DEPTH 50.7'

DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-6-93
Approved By ______ Date ______

Page 2 of 2
Pier No. 9
Shaft No. 42
Station ______
Offset ______

Placement Method ____ Tremie
X Pumped
Deairing Method ____ Relief Valve
____ Tremie Plug
X Tremie Cap

Volume in Lines	#	ID	Length	Volume
Total Volume in Lines				

Reference Elev. 370.2
Shaft Top Elev. 367.9
Top of Rock Elev. 273.6
Shaft bottom Elev. 251.3

Depth to Water Inside ____ ID Casing at Start ____
Rebar Cage Top Elev. at Start 326.8 at Finish ____

Bucket No.	Concrete Volume	Conc in Arrival Bucket Time	Start Time	Finish Time	Tremie Depth	Depth to Concrete	Notes
13	52	11:45PM		11:52PM			
14	56	11:53PM		11:59PM			
15	60	12:01AM	2-7-93	12:07AM			CLAMP AT DUMP CONNECTION LOOSE, DOWN 44 MINUTE:
		12:51AM		12:56AM	80.0	65.5	
16	64	12:57AM		1:02AM			MIX HELD IN BIN 45 MINUTES
17	68	1:05AM		1:11AM			MIX HELD IN MIXER 45 MINUTES
18	72	1:12AM		1:19AM	70.5	55.5	SLUMP 7", TEMP 74, AIR 6.3
19	76	1:25AM		1:32AM	67.5	50.7	REMOVING TREMIE PIPE
							2:50 PLACING JET IN SHAFT
						3:05AM	PUMPING OUT MATERIALS
						3:20AM	HOLE GOOD

76 Concrete Volume Delivered

Placement Time (Casing Removed) ____

	ID	Top Elev.	Bot. Elev.	Start	Finish
Casing		367.9	273.6		
Removal					

Rebar Cage Centered STANDEE
Concrete Finished ____

Notes:

502 685 3215 KY DEPT OF TRAN 01

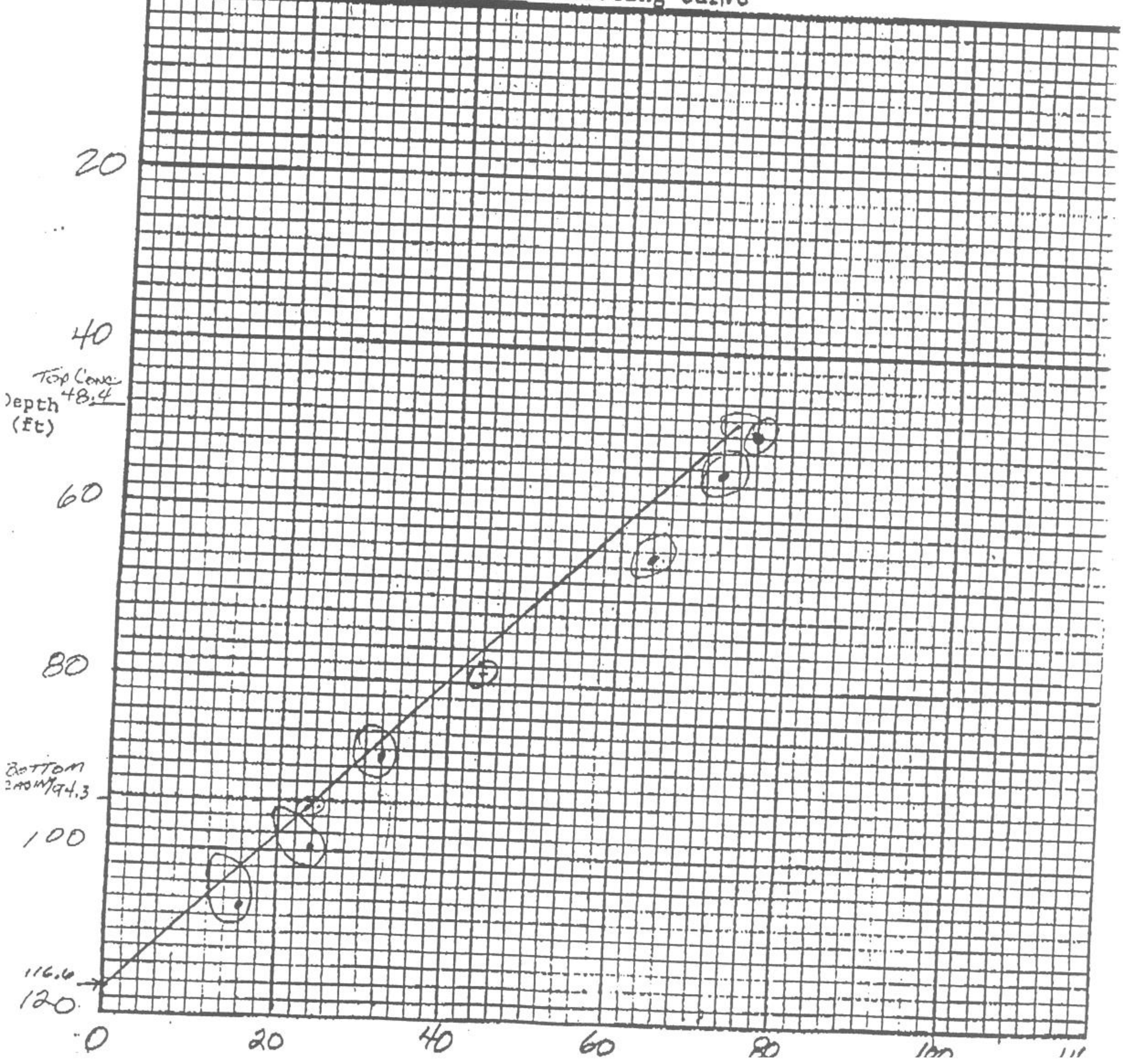
KENTUCKY TRANSPORTATION CABINET
Department of Highways
Division of Construction

Top of overpour Conc = 48.4 Below Top casing

FIELD & THEORETICAL CONCRETING CURVES

Project Name	OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE	Pier No.	9
Project No.	KDH-00101-002-000	Shaft No.	42
Contractor	NATIONAL ENGINEERING & CONTRACTING COMPANY	Station	
Inspected By		Date 2/06/93	Offset
Approved By		Date / /	

Concreting Curve



KENTUCKY TRANSPORTATION CABINET

Department of Highways

Division of Construction

DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-5-93
Approved By ______ Date ______

Page 1 of 3
Pier No. 6
Shaft No. 43
Station ______
Offset ______

Placement Method ___ Tremie
X Pumped
Deairing Method ___ Relief Valve
___ Tremie Plug
___ Tremie Cap

Volume in Lines	#	ID	Length	Volume

Total Volume in Lines ______

Reference Elev. ______
Shaft Top Elev. ______
Top of Rock Elev. 268.2
Shaft bottom Elev. 218.3

Depth to Water Inside ___ ID Casing at Start ___
Rebar Cage Top Elev. at Start 320 at Finish ___

Bucket No.	Concrete Volume	Conc in Bucket	Arrival Time	Start Time	Finish Time	Tremie Depth	Depth to Concrete	Notes
1	4			12:19PM	12:24PM			
2	8			12:26PM	12:29PM		142.9	
3	12			12:32PM	12:38PM			
4	16			12:40PM	12:45PM			
5	20			12:47PM	12:51PM		130.9	
6	24			12:55PM	1:00PM			STOPPED CONCRETING AT 1:02PM, WORKING ON TREMIE,
7	28			1:02PM	1:19PM	134	123.5	BACK CONCRETING 1:15PM
8	32			1:20PM	1:28PM			
9	36			1:30PM	1:35PM			
10	40			1:36PM	1:41PM	128	112.4	
11	44			1:43PM	1:47PM			
12	48			1:50PM	1:56PM	117	105.7	
13	52			2:00PM	2:05PM			

___ Concrete Volume Delivered

Placement Time (Casing Removed) ___

	ID	Top Elev.	Bot. Elev.	Start	Finish
Casing		368.2	268.2		
Removal					

Rebar Cage Centered STANDEE
Concrete Finished ___

Notes: ______

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-5-93
Approved By ______ Date ______

Page 2 of 3
Pier No. 8
Shaft No. 43
Station ______
Offset ______

Placement Method ___ Tremie
X Pumped
Deairing Method ___ Relief Valve
___ Tremie Plug
___ Tremie Cap

Volume in Lines	#	ID	Length	Volume
Total Volume in Lines				

Reference Elev. ______
Shaft Top Elev. ______
Top of Rock Elev. 268.2
Shaft bottom Elev. 218.3

Depth to Water Inside ___ ID Casing at Start ___
Rebar Cage Top Elev. at Start ___ at Finish ___

Bucket No.	Concrete Volume	Conc in Arrival Bucket Time	Start Time	Finish Time	Tremie Depth	Depth to Concrete	Notes
14	56		2:07PM	2:13PM			
15	60		2:15PM	2:20PM	114	94.8	
16	64		2:23PM	2:28PM			
17	68		2:31PM	2:36PM	110	87.9	
18	72		2:37PM	2:42PM			
19	76		2:45PM	2:50PM			
20	80		2:53PM	2:59PM	98	77.3	
21	84		3:00PM	3:04PM			
22	88		3:06PM	3:10PM			
23	92		3:12PM	3:16PM	90	67.3	
24	96		3:21PM	3:25PM			
25	100		3:27PM	4:09PM			REMOVING PART OF TREMIE PIPE
26	104		4:11PM	4:17PM	75	56.3	

___ Concrete Volume Delivered

Placement Time (Casing Removed) ___

	ID	Top Elev.	Bot. Elev.	Start	Finish
Casing Removal		368.2	268.2		

Rebar Cage Centered STANDEE
Concrete Finished ___

Notes: ______

KENTUCKY TRANSPORTATION CABINET

Department of Highways
Division of Construction

DRILLED SHAFT CONCRETE PLACEMENT LOG

Project Name OWENSBORO-INDIANA STATE LINE ROAD (US231) BRIDGE
Project No. KBD-00101-002-000
Contractor NATIONAL ENGINEERING & CONTRACTING COMPANY
Inspected By ______ Date 2-25-93
Approved By ______ Date ______

Page 3 of 3
Pier No. 8
Shaft No. 43
Station ______
Offset ______

Placement Method ___ Tremie
X Pumped
Deairing Method ___ Relief Valve
___ Tremie Plug
___ Tremie Cap

Volume in Lines	#	ID	Length	Volume

Total Volume in Lines ______

Reference Elev. ______
Shaft Top Elev. ______
Top of Rock Elev. 268.2
Shaft bottom Elev. 218.3

Depth to Water Inside ___ ID Casing at Start ___
Rebar Cage Top Elev. at Start ___ at Finish ___

Bucket No.	Concrete Volume	Conc in Arrival Bucket Time	Start Time	Finish Time	Tremie Depth	Depth to Concrete	Notes
27	108		4:19PM	4:24PM			
28	112		4:25PM	4:31PM	68	49.3	
29	114		4:43PM	4:46PM		47.1	
			5:49PM				PLACED AIR LIFT IN SHAFT
			5:57PM				STARTED AIR LIFT OVER POUR OUT
			6:45PM				STOPPED AIR LIFTING, APPROX. 54.7'
			6:48PM				PULLING AIR LIFT OUT OF SHAFT
			6:50PM				SHAFT COMPLETE

___ Concrete Volume Delivered

Placement Time (Casing Removed) ___

	ID	Top Elev.	Bot. Elev.	Start	Finish
Casing		368.2	268.2		
Removal					

Rebar Cage Centered STANDEE
Concrete Finished ___

Notes: ______

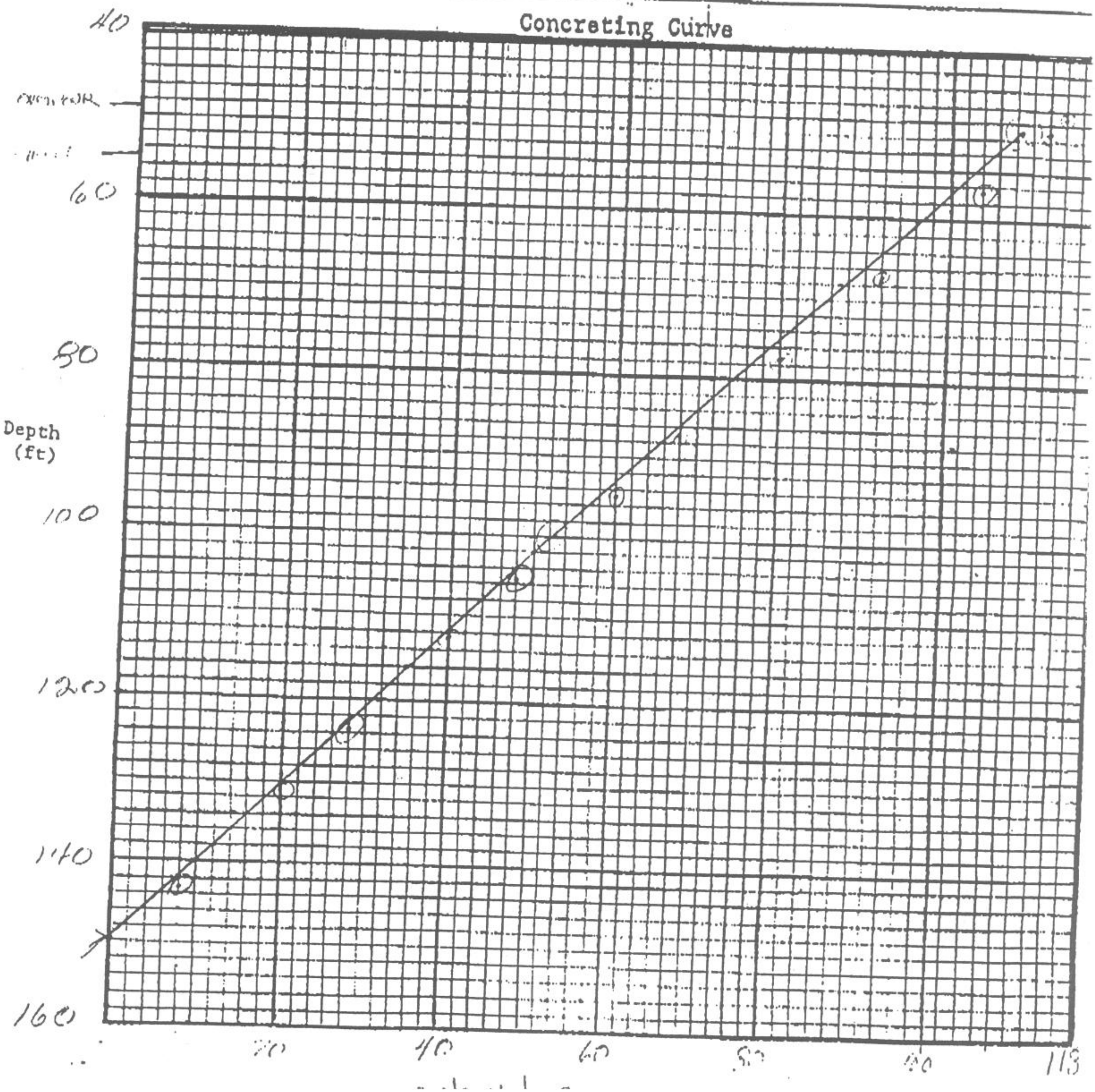
KENTUCKY TRANSPORTATION CABINET
Department of Highways
Division of Construction

Finish Conc. Grade

This includes 6' overpour

FIELD & THEORETICAL CONCRETING CURVES

Project Name	OWENSBORO-INDIANA STATE LINE ROAD (US 231) BRIDGE	Pier No.	
Project No.	KDH-00101-002-000	Shaft No.	
Contractor	NATIONAL ENGINEERING & CONTRACTING COMPANY	Station	
Inspected By	Date	Offset	
Approved By	Date / /		



Western Atlas International

ATLAS WIRELINE SERVICES

4-ARM CALIPER LOG

IN MAKING INTERPRETATIONS OF LOGS OUR EMPLOYEES WILL GIVE CUSTOMER THE BENEFIT OF THEIR BEST JUDGEMENT, BUT SINCE ALL INTERPRETATIONS ARE OPINIONS BASED ON INFERENCES FROM ELECTRICAL OR OTHER MEASUREMENTS, WE CANNOT, AND WE DO NOT GUARANTEE THE ACCURACY OR CORRECTNESS OF ANY INTERPRETATION. WE SHALL NOT BE LIABLE OR RESPONSIBLE FOR ANY LOSS, COST, DAMAGES, OR EXPENSES WHATSOEVER INCURRED OR SUSTAINED BY THE CUSTOMER RESULTING FROM ANY INTERPRETATION MADE BY ANY OF OUR EMPLOYEES.

FILE NO.

API NO.

FIELD PRINT

COMPANY PSC ASSOCIATES, INC.

WELL PIER NO.8, SHAFT NO.43

FIELD OHIO RIVER, OWENSBORO BRIDGE

COUNTY DAVIESS STATE KY

LOCATION:

SEC ____ TWP ____ RGE ____

OTHER SERVICES
NONE

PERMANENT DATUM MEAN SEA LEVEL ELEV. 368.2

LOGGING MEASURED FROM CASING 0 FT. ABOVE P.D.

DRILLING MEASURED FROM TOP OF CASING.

ELEVATIONS
KB N/A
DF N/A
GL N/A

DATE	4-FEB-93				
RUN	1				
SERVICE ORDER	113272				
DEPTH-DRILLER	149.8				
DEPTH-LOGGER	149.8				
BOTTOM LOGGED INTERVAL	148				
TOP LOGGED INTERVAL	80				
CASING - DRILLER	74	@ 99'	@	@	@
CASING - LOGGER	98'				
BIT SIZE	72"				
TYPE FLUID IN HOLE	POLYMER SLURRY				
DENSITY / VISCOSITY	N/A	N/A			
PH / FLUID LOSS	N/A	N/A			
SOURCE OF SAMPLE	N/A				
RM AT MEAS. TEMP.	N/A	@ N/A	@	@	@
RMF AT MEAS. TEMP.	N/A	@ N/A	@	@	@
RMC AT MEAS. TEMP.	N/A	@ N/A	@	@	@
SOURCE OF RMF / RMC	N/A	N/A			
RM AT BHT	N/A	@ N/A	@	@	@
TIME SINCE CIRCULATION	N/A				

HERE

RUN	TRIP	TOOL	SERIAL NO.	SERIES NO.	POSITION
1	1	4CAL	361178	4216	OPEN
1	1	GR	NOT RUN		

SECOND PASS - 8 SAMPLES/FT

FILE: 2

PARAMETERS

NAME	DEPTH INTERVAL	VALUE	UNITS
O.D.	148 TO 71	0.000	INCH

DISPLAY SCALE CHANGES

*** NONE ***

COMPANY: PSC ASSOCIATES, INC.

WELL NAME: PIER NO.8, SHAFT NO.43

SERVICE: S 202A FILE: 2

REVISION: FSYS256 REV:0002 VER:2.0

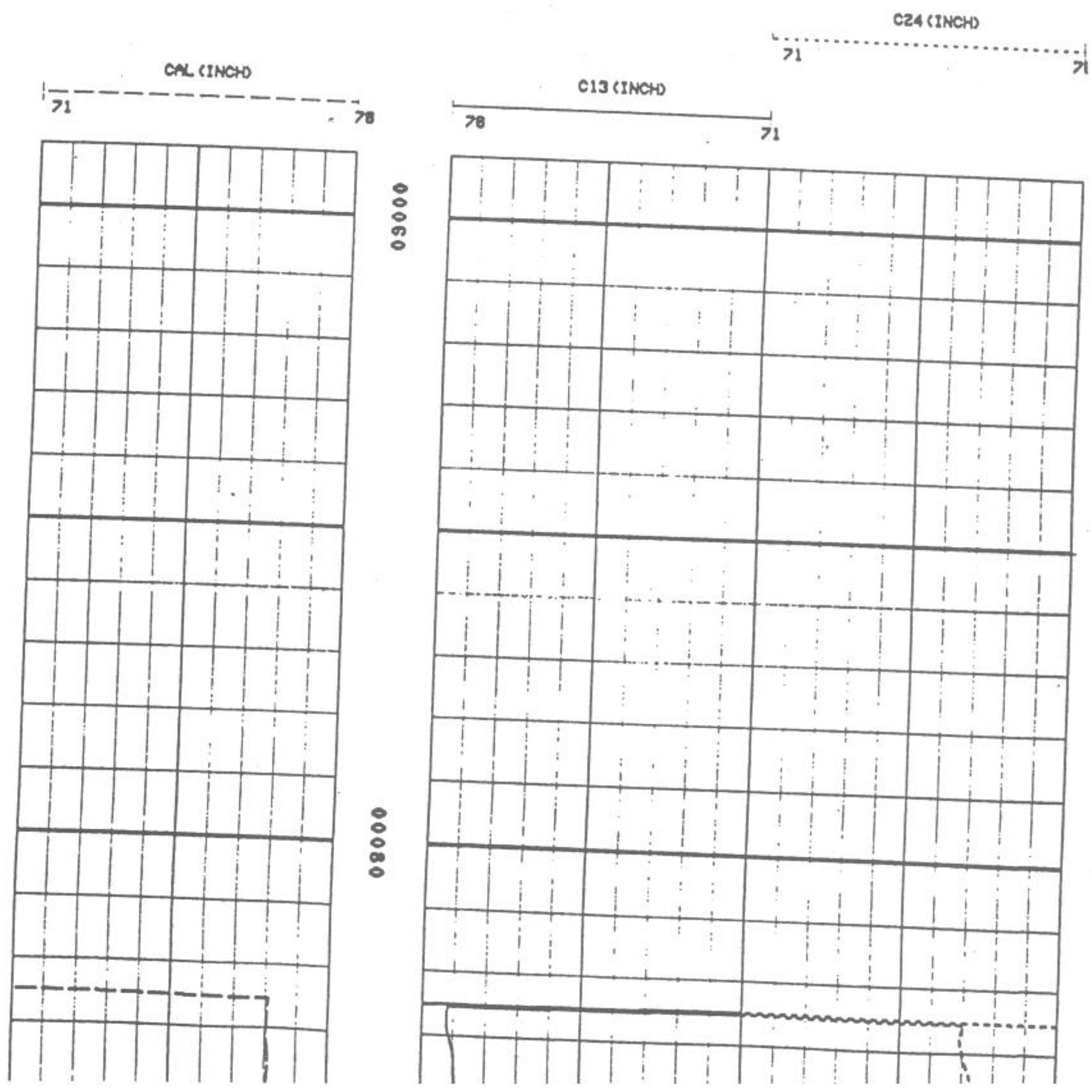
DATE: 02/04/93

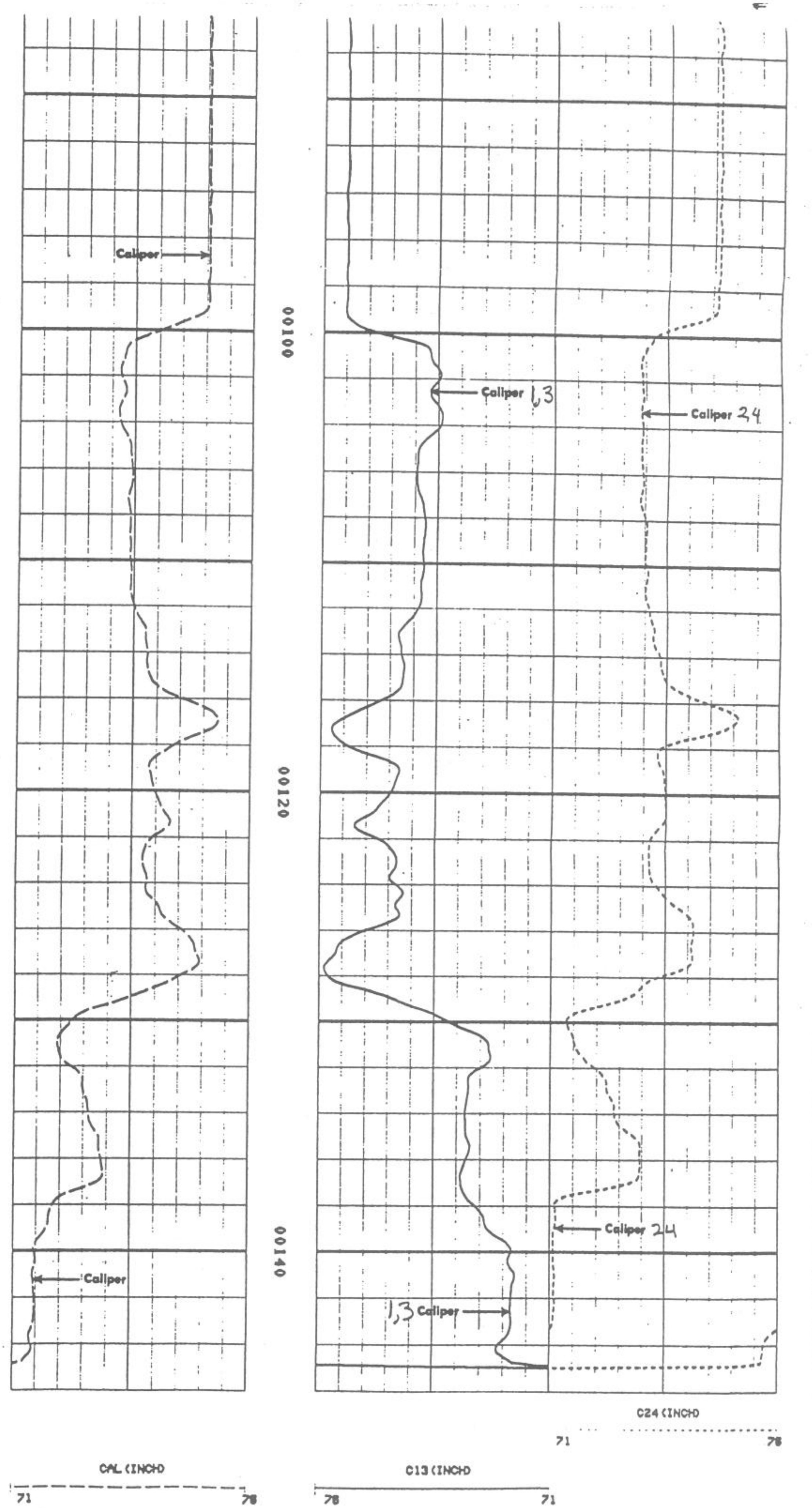
RUN: 1

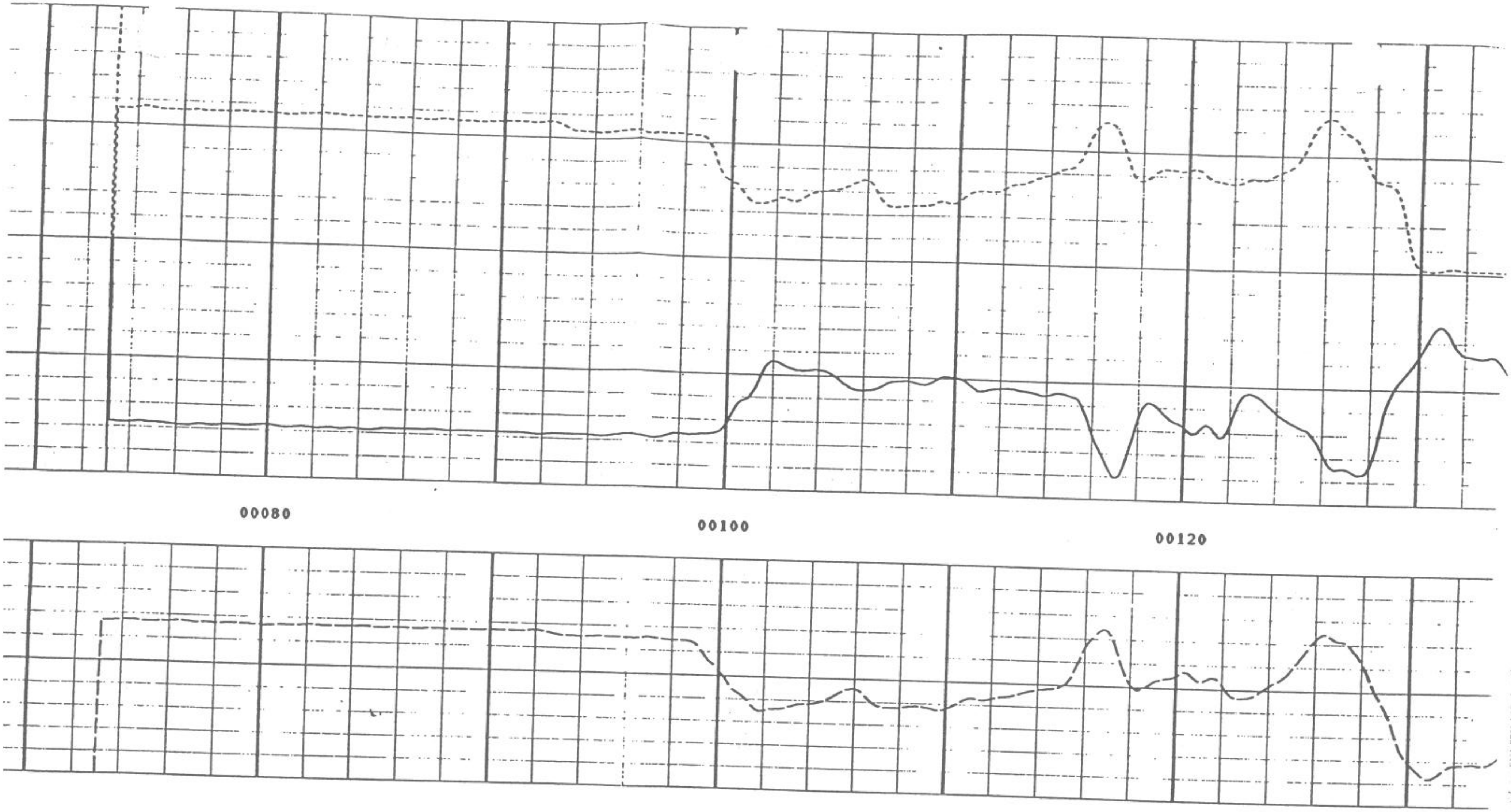
TRIP: 1

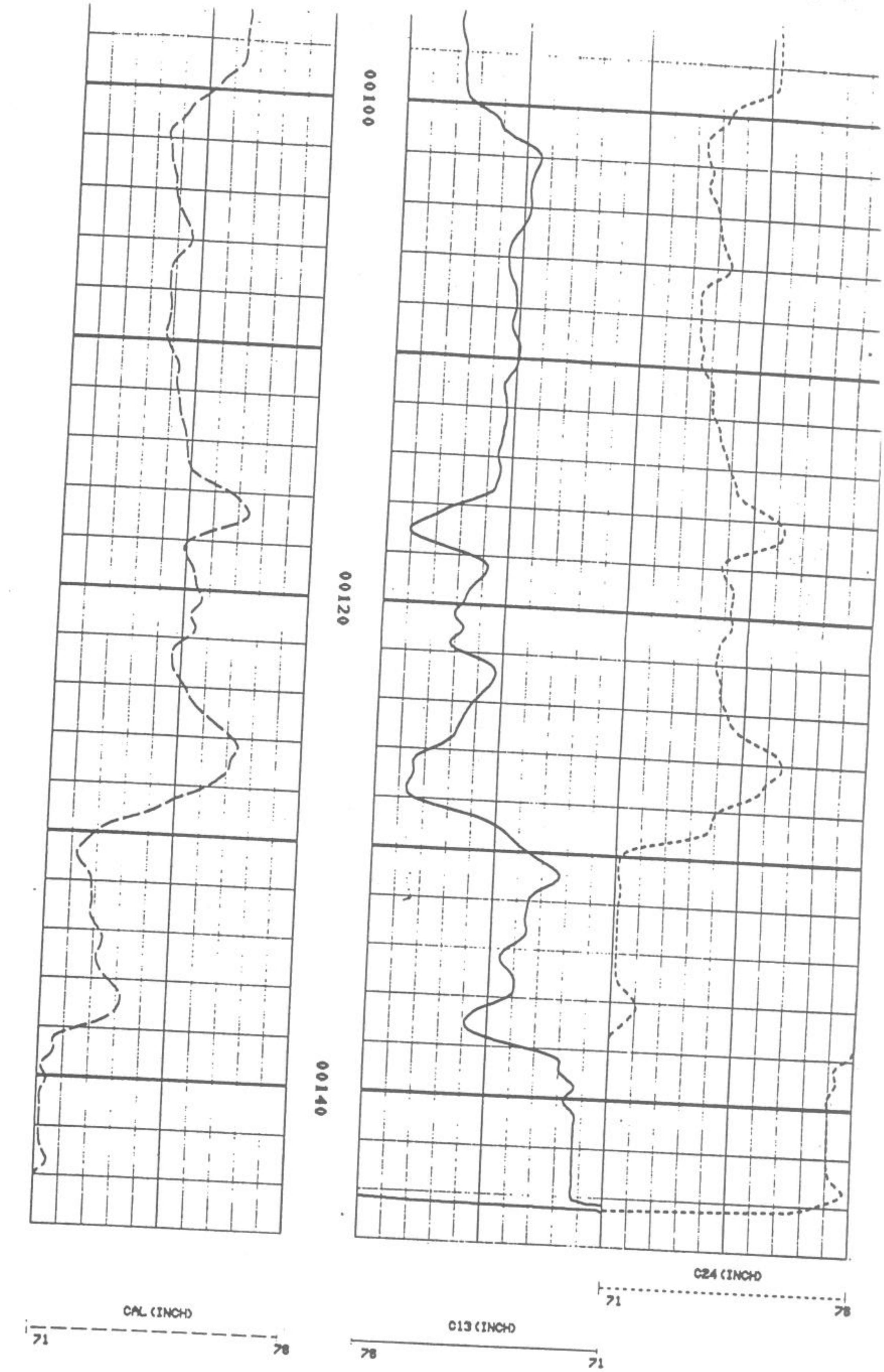
TIME: 18:43:48

MODE: PLAYBACK




Caliper
00100
Caliper 1,3
Caliper 2,4
00120
00140
Caliper 24
Caliper
1,3 Caliper
C24 (INCH)
71
78
CAL (INCH)
71
78
C13 (INCH)
78
71


00080
00100
00120



FILE: 1

NUMERICAL DATA

DEP	C13	C24	CAL
146.00	85.0	85.000	85.0
145.88	85.0	85.000	85.0
145.75	85.0	85.000	85.0
145.63	85.0	85.000	85.0
145.50	85.0	85.000	85.0
145.38	85.0	85.000	85.0
145.25	85.0	85.000	85.0
145.12	85.0	85.000	85.0
145.00	85.0	85.000	85.0
144.88	71.3	70.227	70.8
144.75	71.8	70.844	71.2
144.63	71.9	70.852	71.3
144.50	72.0	70.885	71.3
144.38	72.1	70.878	71.4
144.25	72.1	70.885	71.4
144.12	72.1	70.885	71.4
144.00	72.1	70.887	71.4
143.88	72.0	70.703	71.4
143.75	72.0	70.744	71.4
143.63	71.9	70.810	71.4
143.50	71.9	70.889	71.4
143.38	71.8	70.965	71.4
143.25	71.8	71.024	71.4
143.12	71.8	71.060	71.4
143.00	71.8	71.078	71.5
142.88	71.8	71.080	71.5
142.75	71.8	71.082	71.5
142.63	71.8	71.087	71.5
142.50	71.8	71.094	71.5
142.38	71.8	71.098	71.5
142.25	71.8	71.100	71.5
142.12	71.8	71.098	71.5
142.00	71.8	71.094	71.5
141.88	71.8	71.087	71.5
141.75	71.8	71.083	71.5
141.63	71.8	71.083	71.4
141.50	71.8	71.087	71.4
141.38	71.8	71.094	71.4
141.25	71.8	71.099	71.4
141.13	71.8	71.100	71.4
141.00	71.8	71.099	71.4
140.88	71.8	71.094	71.4
140.75	71.8	71.087	71.5
140.63	71.9	71.082	71.5
140.50	71.9	71.080	71.5
140.38	71.9	71.079	71.5
140.25	71.9	71.079	71.5
140.13	71.8	71.079	71.5
140.00	71.8	71.079	71.4
139.88	71.8	71.079	71.5
139.75	71.9	71.079	71.5
139.63	71.9	71.078	71.5
139.50	72.0	71.078	71.5
139.38	72.1	71.075	71.6
139.25	72.2	71.077	71.6
139.13	72.3	71.088	71.7
139.00	72.4	71.100	71.7
138.88	72.4	71.115	71.8
138.75	72.4	71.128	71.8
138.63	72.4	71.129	71.8
138.50	72.4	71.128	71.8
138.38	72.5	71.119	71.8
138.25	72.5	71.111	71.8
138.13	72.6	71.105	71.8
138.00	72.6	71.099	71.9
137.88	72.7	71.105	71.9
137.75	72.7	71.156	72.0
137.63	72.8	71.290	72.0
137.50	72.8	71.525	72.2
137.38	72.9	71.842	72.4
137.25	72.9	72.189	72.5
137.13	72.9	72.503	72.7
137.00	72.9	72.737	72.8
136.88	72.9	72.873	72.9
136.75	72.9	72.930	72.9
136.63	72.9	72.945	72.9
136.50	72.9	72.949	72.9
136.38	72.9	72.952	72.9
136.25	72.8	72.952	72.9
136.13	72.8	72.952	72.9
136.00	72.8	72.950	72.9
135.88	72.8	72.948	72.9
135.75	72.7	72.949	72.8
135.63	72.7	72.952	72.8
135.50	72.7	72.955	72.8
135.38	72.7	72.951	72.8
135.25	72.7	72.932	72.8
135.13	72.8	72.890	72.8
135.00	72.8	72.821	72.8
134.88	72.8	72.734	72.8
134.75	72.8	72.642	72.7
134.62	72.8	72.558	72.7
134.50	72.8	72.491	72.7
134.37	72.8	72.44[illegible]	72.[illegible]

134.88	72.6	72.734	72.8
134.75	72.6	72.642	72.7
134.62	72.6	72.558	72.7
134.50	72.6	72.491	72.7
134.37	72.6	72.443	72.6
134.25	72.6	72.410	72.6
134.13	72.6	72.390	72.6
134.00	72.6	72.380	72.6
133.88	72.6	72.378	72.6
133.75	72.6	72.382	72.6
133.62	72.6	72.379	72.6
133.50	72.6	72.381	72.6
133.37	72.6	72.328	72.6
133.25	72.6	72.284	72.5
133.13	72.6	72.245	72.5
133.00	72.6	72.220	72.5
132.88	72.7	72.210	72.5
132.75	72.6	72.210	72.5
132.62	72.6	72.209	72.5
132.50	72.6	72.194	72.5
132.37	72.7	72.159	72.4
132.25	72.7	72.103	72.4
132.13	72.6	72.031	72.3
132.00	72.5	71.950	72.2
131.88	72.4	71.889	72.1
131.75	72.3	71.795	72.1
131.62	72.3	71.732	72.0
131.50	72.3	71.677	72.0
131.37	72.3	71.628	72.0
131.25	72.3	71.584	72.0
131.13	72.3	71.547	71.9
131.00	72.3	71.522	71.9
130.88	72.4	71.508	71.9
130.75	72.4	71.484	72.0
130.62	72.5	71.478	72.0
130.50	72.6	71.454	72.0
130.37	72.8	71.422	72.1
130.25	72.9	71.388	72.2
130.13	73.1	71.358	72.2
130.00	73.2	71.342	72.3
129.88	73.3	71.353	72.3
129.75	73.4	71.409	72.4
129.62	73.6	71.524	72.5
129.50	73.7	71.699	72.7
129.37	73.9	71.918	72.9
129.25	74.1	72.158	73.1
129.13	74.3	72.383	73.3
129.00	74.4	72.578	73.5
128.88	74.6	72.725	73.6
128.75	74.8	72.831	73.8
128.63	75.0	72.908	73.9
128.50	75.2	72.961	74.1
128.37	75.5	73.009	74.2
128.25	75.6	73.074	74.4
128.13	75.7	73.180	74.5
128.00	75.8	73.338	74.6
127.88	75.8	73.528	74.7
127.75	75.9	73.725	74.8
127.63	75.9	73.898	74.9
127.50	75.9	74.018	74.9
127.37	75.8	74.083	74.9
127.25	75.7	74.104	74.9
127.13	75.7	74.101	74.9
127.00	75.6	74.082	74.9
126.88	75.6	74.083	74.8
126.75	75.6	74.077	74.8
126.63	75.6	74.079	74.8
126.50	75.5	74.088	74.8
126.37	75.4	74.101	74.8
126.25	75.3	74.113	74.7
126.13	75.1	74.120	74.6
126.00	75.0	74.121	74.6
125.88	74.8	74.117	74.5
125.75	74.6	74.112	74.4
125.63	74.4	74.101	74.3
125.50	74.3	74.080	74.2
125.37	74.3	74.040	74.1
125.25	74.3	73.979	74.1
125.13	74.3	73.901	74.1
125.00	74.3	73.811	74.1
124.88	74.3	73.718	74.0
124.75	74.3	73.634	74.0
124.63	74.3	73.581	73.9
124.50	74.2	73.500	73.8
124.37	74.2	73.448	73.8
124.25	74.2	73.400	73.8
124.13	74.3	73.349	73.8
124.00	74.4	73.294	73.8
123.88	74.4	73.240	73.8
123.75	74.5	73.194	73.8
123.63	74.5	73.164	73.8
123.50	74.4	73.151	73.8
123.37	74.4	73.147	73.8
123.25	74.3	73.147	73.7
123.13	74.3	73.145	73.7
123.00	74.3	73.143	73.7
122.88	74.3	73.143	73.7
122.75	74.3	73.143	73.7
122.63	74.4	73.145	73.8
122.50	74.4	73.147	73.8
122.38	74.5	73.147	73.8
122.25	74.5	73.148	73.8
122.13	74.6	73.148	73.9
122.00	74.7	[illegible]	[illegible]

113.25	74.3	73.186	73.8
113.13	74.3	73.197	73.8
113.00	74.3	73.210	73.8
112.87	74.2	73.218	73.7
112.75	74.2	73.206	73.7
112.63	74.1	73.183	73.7
112.50	74.1	73.155	73.6
112.38	74.0	73.131	73.6
112.25	74.0	73.118	73.5
112.13	73.9	73.112	73.5
112.00	73.9	73.107	73.5
111.87	73.9	73.096	73.5
111.75	73.8	73.077	73.5
111.63	73.8	73.055	73.4
111.50	73.8	73.038	73.4
111.38	73.8	73.023	73.4
111.25	73.8	73.017	73.4
111.13	73.8	73.017	73.4
111.00	73.8	73.021	73.4
110.87	73.8	73.030	73.4
110.75	73.8	73.044	73.4
110.63	73.8	73.062	73.4
110.50	73.8	73.080	73.4
110.38	73.8	73.089	73.4
110.25	73.8	73.092	73.4
110.13	73.8	73.063	73.4
110.00	73.8	73.040	73.4
109.88	73.8	73.023	73.4
109.75	73.8	73.017	73.4
109.63	73.8	73.022	73.4
109.50	73.8	73.032	73.4
109.38	73.8	73.039	73.4
109.25	73.8	73.040	73.4
109.13	73.8	73.035	73.4
109.00	73.8	73.031	73.4
108.88	73.8	73.031	73.4
108.75	73.8	73.038	73.4
108.63	73.8	73.050	73.4
108.50	73.8	73.060	73.4
108.38	73.8	73.064	73.4
108.25	73.8	73.060	73.4
108.13	73.8	73.049	73.4
108.00	73.8	73.033	73.4
107.88	73.8	73.012	73.4
107.75	73.8	72.987	73.4
107.63	73.8	72.960	73.4
107.50	73.8	72.934	73.4
107.38	73.8	72.912	73.4
107.25	73.8	72.900	73.4
107.13	73.9	72.897	73.4
107.00	73.9	72.903	73.4
106.88	73.9	72.915	73.4
106.75	73.9	72.929	73.4
106.63	73.9	72.941	73.4
106.50	74.0	72.951	73.5
106.38	74.0	72.956	73.5
106.25	74.0	72.955	73.5
106.13	74.0	72.949	73.5
106.00	74.0	72.940	73.4
105.88	74.0	72.930	73.4
105.75	73.9	72.922	73.4
105.63	73.9	72.915	73.4
105.50	73.9	72.910	73.4
105.38	73.9	72.907	73.4
105.25	73.9	72.907	73.4
105.13	73.9	72.907	73.4
105.00	73.9	72.907	73.4
104.88	73.9	72.907	73.4
104.75	73.8	72.907	73.4
104.63	73.8	72.910	73.3
104.50	73.7	72.915	73.3
104.38	73.6	72.921	73.3
104.25	73.6	72.928	73.2
104.13	73.5	72.928	73.2
104.00	73.4	72.928	73.2
103.88	73.4	72.920	73.2
103.75	73.4	72.912	73.2
103.63	73.4	72.902	73.2
103.50	73.4	72.893	73.2
103.37	73.4	72.890	73.2
103.25	73.5	72.894	73.2
103.12	73.5	72.905	73.2
103.00	73.6	72.920	73.2
102.88	73.6	72.934	73.3
102.75	73.7	72.942	73.3
102.63	73.7	72.942	73.3
102.50	73.7	72.935	73.3
102.37	73.6	72.928	73.3
102.25	73.6	72.924	73.2
102.12	73.5	72.924	73.2
102.00	73.5	72.927	73.2
101.88	73.4	72.927	73.2
101.75	73.4	72.923	73.2
101.63	73.5	72.913	73.2
101.50	73.5	72.904	73.2
101.37	73.6	72.899	73.2
101.25	73.6	72.900	73.3
101.12	73.7	72.908	73.3
101.00	73.7	72.922	73.3
100.88	73.7	72.943	73.3
100.75	73.7	72.974	73.3
100.63	73.7	73.013	73.4
100.50	73.8	73.058	73.5

123.63	74.5	73.164	73.8
123.50	74.4	73.151	73.8
123.37	74.4	73.147	73.8
123.25	74.3	73.147	73.7
123.13	74.3	73.145	73.7
123.00	74.3	73.143	73.7
122.88	74.3	73.143	73.7
122.75	74.3	73.143	73.7
122.63	74.4	73.145	73.8
122.50	74.4	73.147	73.8
122.38	74.5	73.147	73.8
122.25	74.5	73.148	73.8
122.13	74.6	73.148	73.9
122.00	74.7	73.159	73.9
121.88	74.9	73.184	74.0
121.75	75.0	73.223	74.1
121.63	75.2	73.274	74.2
121.50	75.2	73.332	74.3
121.38	75.2	73.389	74.3
121.25	75.2	73.438	74.3
121.13	75.1	73.472	74.3
121.00	74.9	73.492	74.2
120.88	74.8	73.496	74.2
120.75	74.8	73.497	74.1
120.63	74.7	73.492	74.1
120.50	74.7	73.486	74.1
120.38	74.6	73.481	74.1
120.25	74.6	73.478	74.0
120.13	74.5	73.478	74.0
120.00	74.5	73.477	74.0
119.88	74.4	73.474	73.9
119.75	74.4	73.469	73.9
119.63	74.4	73.463	73.9
119.50	74.4	73.458	73.9
119.38	74.3	73.456	73.9
119.25	74.3	73.453	73.9
119.12	74.3	73.447	73.9
119.00	74.3	73.430	73.9
118.87	74.3	73.402	73.9
118.75	74.4	73.367	73.9
118.63	74.5	73.335	73.9
118.50	74.7	73.316	74.0
118.38	74.9	73.311	74.1
118.25	75.1	73.318	74.2
118.12	75.3	73.335	74.3
118.00	75.4	73.384	74.4
117.87	75.5	73.491	74.5
117.75	75.6	73.667	74.6
117.63	75.7	73.904	74.8
117.50	75.7	74.172	74.9
117.38	75.7	74.432	75.1
117.25	75.8	74.652	75.2
117.12	75.7	74.821	75.3
117.00	75.7	74.942	75.3
116.87	75.6	75.031	75.3
116.75	75.5	75.089	75.3
116.63	75.3	75.101	75.2
116.50	75.2	75.053	75.1
116.38	75.1	74.948	75.0
116.25	75.0	74.798	74.9
116.13	74.8	74.613	74.7
116.00	74.7	74.421	74.5
115.87	74.5	74.235	74.4
115.75	74.4	74.065	74.2
115.63	74.4	73.914	74.1
115.50	74.3	73.781	74.0
115.38	74.3	73.671	74.0
115.25	74.2	73.583	73.9
115.13	74.2	73.518	73.9
115.00	74.2	73.471	73.8
114.87	74.2	73.437	73.8
114.75	74.2	73.411	73.8
114.63	74.2	73.388	73.8
114.50	74.2	73.376	73.8
114.38	74.2	73.368	73.8
114.25	74.2	73.363	73.8
114.13	74.2	73.359	73.8
114.00	74.3	73.347	73.8
113.87	74.3	73.324	73.8
113.75	74.3	73.290	73.8
113.63	74.3	73.250	73.8
113.50	74.3	73.213	73.8
113.38	74.3	73.190	73.8
113.25	74.3	73.188	73.8
113.13	74.3	73.197	73.8
113.00	74.3	73.210	73.8
112.87	74.2	73.218	73.7
112.75	74.2	73.208	73.7
112.63	74.1	73.183	73.7
112.50	74.1	73.155	73.6
112.38	74.0	73.131	73.6
112.25	74.0	73.118	73.5
112.13	73.9	73.112	73.5
112.00	73.9	73.107	73.5
111.87	73.9	73.098	73.5
111.75	73.8	73.077	73.5
111.63	73.8	73.055	73.4
111.50	73.8	73.038	73.4
111.38	73.8	73.023	73.4
111.25	73.8	73.017	73.4
111.13	73.8	73.017	73.4
111.00	73.8	73.021	73.4
110.87	73.8	73.030	73.4

PSC ASSOCIATES, INC.
PIER NO.8, SHAFT NO.43]
+ 0

SERVICE TABLE ***3 202A*** FILE LABEL # 2

TAPE LEVEL SPACING : 0.12500 TAPE STARTING DEPTH : 146.00
PRINT LEVEL SPACING : 0.12500 TAPE ENDING DEPTH : 0.00

** UNITS OF MEASURE **

DEPTH	FT
ACOUSTIC	US/FT
CALIPER	IN
TENSION	LBS
TEMPERATURE	F
PRESSURE	PSI
VOLUME	FT3

*** END OF LIST ***

CALIBRATION/VERIFICATION SUMMARIES
DATE 02/04/93 TIME 19:01:23
RUN 1 TRIP 1

LOG NAME 4CAL ASSET NO. 381178 UNIT NO. HL-8399
CALIBRATION ENTERED ON 02/04/93 AT 17:42:14

CURVE	MILLIVOLTS LOW	MILLIVOLTS HIGH	SCALING FACTORS ADD.	SCALING FACTORS MULT.	ENGINEERING VALUES LOW	ENGINEERING VALUES HIGH	UNITS
DATE 01/28/93		CALIBRATION			TIME 13:45:54		
C13	2032.0	3011.0	-27.837	0.1196	72.000	120.000	INCH
C24	2298.5	3323.8	-35.806	0.1142	72.000	120.000	INCH
DATE 01/28/93		PRIMARY VERIFICATION			TIME 13:51:08		
C13	2028.8	3011.9	-27.837	0.1196	71.832	120.048	INCH
C24	2302.9	3322.3	-35.806	0.1142	72.208	118.831	INCH
DATE 02/04/93		BEFORE LOG VERIFICATION			TIME 18:33:24		
C13	2099.1	2099.8	-27.837	0.1196	75.290	75.314	INCH
C24	2311.2	2311.2	-35.806	0.1142	72.584	72.584	INCH
DATE 02/04/93		AFTER LOG VERIFICATION			TIME 18:48:48		
C13	2100.8	2100.8	-27.837	0.1196	75.374	75.374	INCH
C24	2337.8	2339.0	-35.806	0.1142	73.838	73.896	INCH

*** OR NOT CALIBRATED ***

ATLAS WIRELINE SERVICES

104.75	73.6	72.907	73.4
104.63	73.6	72.910	73.3
104.50	73.7	72.915	73.3
104.38	73.6	72.921	73.3
104.25	73.6	72.926	73.2
104.13	73.5	72.928	73.2
104.00	73.4	72.928	73.2
103.88	73.4	72.920	73.2
103.75	73.4	72.912	73.2
103.63	73.4	72.902	73.2
103.50	73.4	72.893	73.2
103.37	73.4	72.890	73.2
103.25	73.5	72.894	73.2
103.12	73.5	72.905	73.2
103.00	73.6	72.920	73.2
102.88	73.6	72.934	73.3
102.75	73.7	72.942	73.3
102.63	73.7	72.942	73.3
102.50	73.7	72.935	73.3
102.37	73.6	72.928	73.3
102.25	73.6	72.924	73.2
102.12	73.5	72.924	73.2
102.00	73.5	72.927	73.2
101.88	73.4	72.927	73.2
101.75	73.4	72.923	73.2
101.63	73.5	72.913	73.2
101.50	73.5	72.904	73.2
101.37	73.6	72.899	73.2
101.25	73.6	72.900	73.3
101.12	73.7	72.908	73.3
101.00	73.7	72.922	73.3
100.88	73.7	72.943	73.3
100.75	73.7	72.974	73.3
100.63	73.7	73.013	73.4
100.50	73.9	73.059	73.5
100.37	74.1	73.108	73.6
100.25	74.3	73.154	73.7
100.12	74.6	73.199	73.9
100.00	74.9	73.256	74.1
99.88	75.1	73.348	74.2
99.75	75.2	73.491	74.4
99.63	75.3	73.688	74.5
99.50	75.4	73.918	74.7
99.38	75.5	74.150	74.8
99.25	75.5	74.348	74.9
99.13	75.5	74.489	75.0
99.00	75.5	74.570	75.0
98.87	75.5	74.606	75.1
98.75	75.5	74.618	75.1
98.63	75.5	74.619	75.1
98.50	75.5	74.616	75.1
98.37	75.5	74.611	75.0
98.25	75.5	74.605	75.0
98.13	75.5	74.600	75.0
98.00	75.5	74.598	75.0

** INTERVAL AVERAGES --

0.00	73.5	72.928	73.1

PSC ASSOCIATES, INC.
PIER NO.8, SHAFT NO.43

SERVICE TABLE ***3 282A*** FILE LABEL # 2

TAPE LEVEL SPACING : 0.12500 TAPE STARTING DEPTH : 148.00
PRINT LEVEL SPACING : 0.12500 TAPE ENDING DEPTH : 0.00

** UNITS OF MEASURE **

DEPTH	FT
ACOUSTIC	US/FT
CALIPER	IN
TENSION	LBS
TEMPERATURE	F
PRESSURE	PSI
VOLUME	FT3

*** END OF LIST ***

CAL (INCH) 71 — 78

C13 (INCH) 78 — 71

FILE: 2

REPEAT SECTION - 8 SAMPLES/FT

FILE: 1

PARAMETERS

NAME	DEPTH INTERVAL	VALUE	UNITS
O.D.	143 TO 58	0.000	INCH

DISPLAY SCALE CHANGES

*** NONE ***

COMPANY: PSC ASSOCIATES, INC.
WELL NAME: PIER NO.8, SHAFT NO.43
SERVICE: S 202A FILE: 1
REVISION: FSYS258 REV:0002 VER:2.0

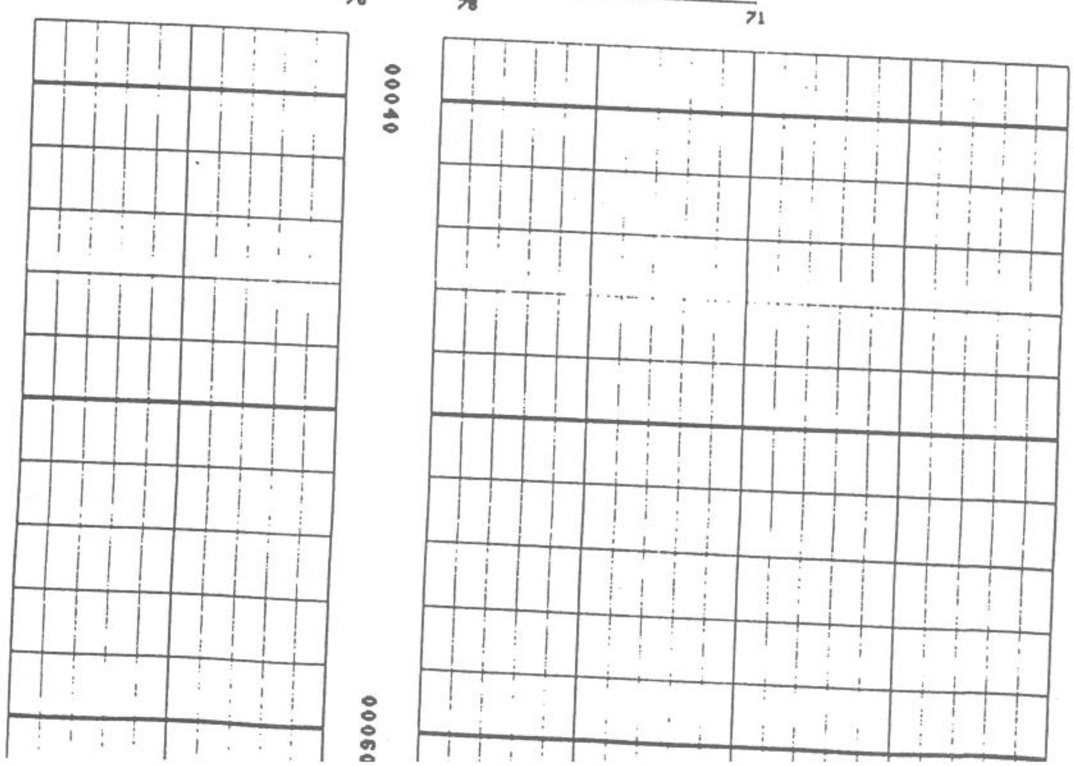
DATE: 02/04/93

RUN: 1
TRIP: 1
TIME: 18:38:30
MODE: PLAYBACK

C24 (INCH) 71 — 78

CAL (INCH) 71 — 78

C13 (INCH) 78 — 71



n-wky

M E M O R A N D U M

TO: Jim Lile, P.E.
Division of Construction

FROM: Henry Mathis, P.E.
Geotechnical Engineering
Branch Manager
Division of Materials

BY: Daryl J. Greer, P.E. DJG ✓
Geotechnical Branch

DATE: April 27, 1993

SUBJECT: Daviess County
KBD 10-1(2); FSP 030 0231 017-018 C
New Ohio River Bridge at Owensboro
Drilled Shaft Tip Elevations

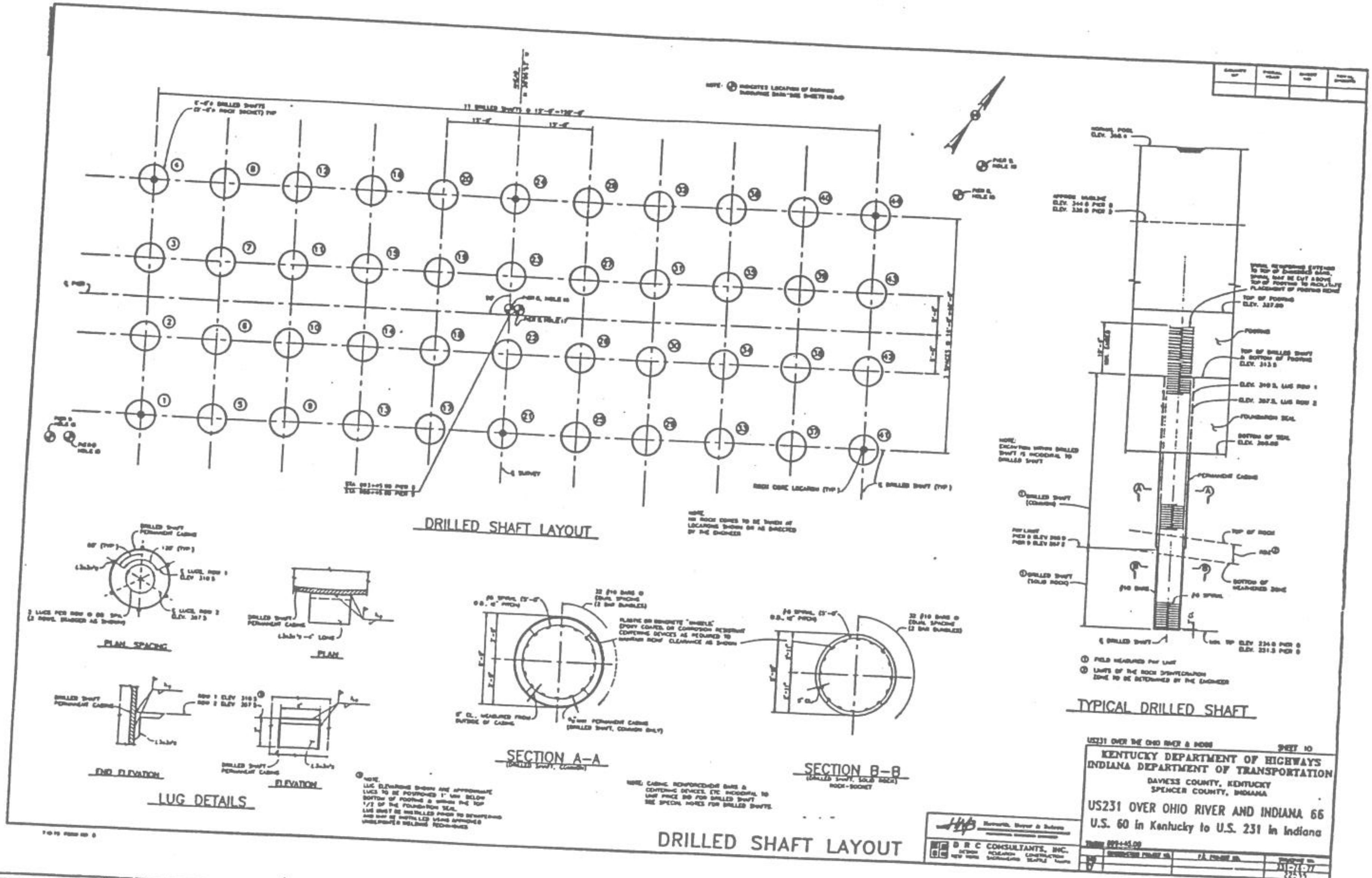
The Geotechnical Branch reviewed the information provided by Loadtest for the Osterberg testing at the subject pier. This office discussed the results of the test with Dr. Crapps of Loadtest and Dr. Michael O'Neill of the University of Houston. Based on the results of the load test and discussions, the recommended tip elevation for the shafts at Pier 8 is Elevation. 225. This elevation should provide sufficient axial and lateral capacity for the expected loadings. This elevation should also limit settlement of the shaft group to within tolerable levels (<3").

The shaft tip elevations at Pier 9 should remain at the plan elevation. While this elevation should provide more than sufficient axial capacity, this elevation should remain at plan elevation to provide sufficient lateral capacity.

At your request this office will review the interior shaft loadings to see if these shafts can shortened further. Please contact this office if you have further questions.

HM:DJG

cc: J. E. Arnold
W. H. McKinney
W. H. Phillips
R. Greer
HMB
FMSM


DRILLED SHAFT LAYOUT
LUG DETAILS
PLAN SPACING
PLAN
END ELEVATION
ELEVATION
SECTION A-A
SECTION B-B
TYPICAL DRILLED SHAFT
US231 OVER THE OHIO RIVER & BRIDGE
SHEET 10
KENTUCKY DEPARTMENT OF HIGHWAYS
INDIANA DEPARTMENT OF TRANSPORTATION
DAVIESS COUNTY, KENTUCKY
SPENCER COUNTY, INDIANA
US231 OVER OHIO RIVER AND INDIANA 66
U.S. 60 in Kentucky to U.S. 231 in Indiana
D R C CONSULTANTS, INC.
DRILLED SHAFT LAYOUT

REVISED: 11-14-91
27 Sheets

SPECIAL NOTE FOR DRILLED SHAFTS

1.0 DESCRIPTION

This work shall consist of furnishing all materials and labor necessary for constructing drilled shafts in accordance with details shown on the plans or as directed by the Engineer. The intent of these plans is to provide reinforced concrete shafts cast in cylindrically excavated holes which extend sufficiently into sound rock to adequately support the structure and all externally applied loads for which it was designed. The shafts shall be constructed to the lines and dimensions shown on the plans, or as directed by the Engineer. The Kentucky Standard Specifications for Road and Bridge Construction, current edition, shall govern unless otherwise specified in this special note.

2.0 PRECONSTRUCTION SUBMITTALS

2.1 At the time of bid, the contractor shall submit both a list containing at least three (3) projects on which the contractor has installed drilled shafts of a diameter and length similar to those shown on the plans, and a signed statement that the contractor has inspected both the project site and all the subsurface information including any soil or rock samples available. The list of projects shall contain names and phone numbers of owner's representatives who can verify the contractor's participation on those projects.

2.2 [redacted] This plan shall provide information on the following:

a) Name and experience record of the drilled shaft superintendent in charge of drilled shaft operations for this project.

b) List of proposed equipment to be used including barges, cranes, drill rigs, drills, augers, bailing buckets, final cleaning equipment, slurry desanding equipment, slurry pumps, core sampling equipment, tremies or concrete pumps, casing, etc.

c) Details of overall construction operation sequence and the sequence of shaft construction in the bents or groups.

d) Details of shaft excavation.

e) Details of casing to be used, including calculations showing ability of casing to withstand anticipated hydraulic and earth pressures and to withstand stresses due to installation without undue deformation. Casing designs shall not contain casings less than 5/8" nominal thickness or the minimum plan thickness.

f) Details of the methods to mix, circulate and desand slurry, when the use of slurry is anticipated.

g) Details of methods to clean the shaft excavation.

h) Details of reinforcing cage fabrication and placement including support of the reinforcement cage during handling, after installation, and during concrete placement and methods/devices to be used to center the reinforcement cage in the drilled shaft.

i) Details of concrete placement including proposed operational procedures for free fall, tremie or pumping methods.

The Engineer will evaluate the Drilled Shaft Installation Plan for conformance with the plans, Specifications and this Special Note. Within 14 days after receipt of the plan, the engineer will notify the contractor of any additional information required and/or changes necessary to meet the contract requirements. All procedural approvals given by the engineer shall be subject to trial in the field and shall not relieve the contractor of the responsibility to satisfactorily complete the work as detailed in the plans and specifications.

nice clause

3.0 MATERIALS

3.1 <u>CONCRETE</u>

Portland cement concrete deposited in the excavated hole shall be Class A Modified conforming to Section 601 of the Standard Specifications, except that the [redacted] and the coarse aggregate shall be size 67, 68, 78, 8 or 9M. Water reducing and retarding admixtures shall be used. The use of superplasticizers, or fly ash w[redacted] be permitted.

The concrete mix design shall demonstrate gradual slump loss behavior which will result in a slump loss of approximately 4 in. or less occurring 4 hours after batching.

3.2 STEEL REINFORCEMENT

Steel Reinforcement shall be Grade 60 deformed bars. Placement shall conform to Section 602 of the Standard Specifications and this Special Note. [redacted]

3.3 CASINGS

Casings shall be steel, smooth, clean, watertight, true and straight, and of ample strength to withstand both handling and installation 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 shaft. No extra compensation will be allowed for concrete required to fill an oversized casing or oversized excavation.

Permanent casing shall be used for this project. The casing shall be continuous between top and bottom elevations shown in the plans, or determined in the field. The permanent casing shall be cut off at the prescribed elevation and concrete trimmed to within plan tolerances prior to acceptance of the completed drilled shaft. Permanent casing shall be stopped below top of rock as shown on the plans and shall be extended into the RDZ (Rock Disintegration Zone) sufficient to stabilize the shaft excavation.

A continuous 6 ft. dia. casing shall be installed from the work platform above the water surface down into the RDZ. The portion of casing above cut-off elevation 313.5 is considered temporary casing. The casing below elevation 313.5 is permanent casing. If multiple (telescoping) temporary casings are used, each casing shall extend up to the work platform. All temporary casing used shall be removed prior to final acceptance, but after completion of the load test.

All casing splices shall have full penetration butt welds with no exterior or interior splice plates, and produce true and straight casing. All welding shall be in accordance with ANSI/AASHTO/AWS D1.5-88, Bridge Welding Code.

The contractor shall submit details concerning the proposed casing design with the Drilled Shaft Installation Plan in accordance with Paragraph 2.2(e)

3.4 SLURRY

Drilling slurry shall be used only if detailed in the approved Drilled Shaft Installation Plan or approved in writing by the Engineer. Only mineral slurries shall be employed when slurry is used in the drilling process. The slurry shall have both a mineral grain size that will remain in suspension and sufficient viscosity and gel characteristics to transport excavated material to a suitable screening system. The percentage and specific gravity of the material used to make the suspension shall be sufficient to maintain the stability of the excavation and to allow proper concrete placement. During construction, the level of the slurry shall be maintained at a height sufficient to prevent caving of the hole. In the event of a sudden significant loss of slurry to the hole, the construction of that foundation shall be stopped until either methods to stop slurry loss or an alternate construction procedure has been approved by the engineer.

The mineral slurry shall be premixed thoroughly with clean fresh water and adequate time (as prescribed by the mineral manufacturer) allotted for hydration prior to introduction into the shaft excavation. Slurry tanks of adequate capacity will be required for slurry circulation, storage, and treatment. No excavated slurry pits will be allowed in lieu of slurry tanks without the written premission of the engineer. Desanding equipment shall be provided by the contractor as necessary to control slurry sand content to less than 4 percent by volume at any point in the borehole. Desanding will not be required for setting temporary casing, sign posts, or lighting mast foundations unless shown in the plans or contract documents. The contractor shall take all steps necessary to prevent the slurry from "setting up" in the shaft. Such methods may include but are not limited to; agitation, circulation, and/or adjusting the properties of the slurry. Disposal of all slurry shall be done offsite in suitable areas.

Control tests using suitable apparatus shall be carried out on the mineral slurry by the contractor to determine density, viscosity, and pH. An acceptable range of values for those physical properties is shown in Table I.

Tests to determine density, viscosity, and pH value shall be done during the shaft excavation to establish a consistent working pattern. [redacted]

The contractor shall insure that heavily contaminated slurry suspension, which could impair the free flow of concrete, has not accumulated in the bottom of the shaft. Prior to placing concrete in any shaft excavation, the contractor shall take slurry samples using a sampling tool similar to that shown in Figure 1. Slurry samples shall be extracted from the base of the shaft and at intervals not exceeding 10 feet up the shaft, until two consecutive samples produce acceptable values for density, viscosity, pH, and sand content.

When any slurry samples are found to be unacceptable, the contractor shall take whatever action is necessary to bring the mineral slurry within specification requirements. Concrete shall not be poured until resampling and testing results produce acceptable values.

Reports of all tests required above, signed by an authorized representative of the contractor, shall be furnished to the Engineer on completion of each drilled shaft.

During construction, the level of mineral slurry in the shaft excavation shall be maintained at a level not less than 4 feet above the highest expected piezometric pressure head along the depth of the shaft. If at any time, the slurry construction method fails, in the opinion of the Engineer, to produce the desired final results, then the contractor shall both discontinue this method and propose an alternate method for approval of the Engineer.

3.5 Contrary to the Standard Specifications, no direct payment will be made for any of the above items used in the drilled shafts, including portions of the steel reinforcement extending above the top of the shafts. These items shall be included in the unit price bid for drilled shafts.

4.0 CONSTRUCTION METHODS AND EQUIPMENT

4.1 The contractor shall demonstrate the adequacy of his methods and equipment on the first production shaft by successfully constructing a reinforced concrete shaft in accordance with this Special Note. The shaft shall be drilled to the maximum depths as shown on the plans or as directed by the engineer. Failure by the contractor to demonstrate to the Engineer the adequacy of methods and equipment shall be reason for the engineer to require alterations in equipment and/or method by the contractor to eliminate unsatisfactory results. Any additional trial holes required to demonstrate the adequacy of altered methods or construction equipment shall be at the contractor's sole expense. Once approval has been given to construct production shafts, no changes will be permitted in the methods or equipment used to construct the satisfactory shaft without written approval of the Engineer.

4.2 All drilled shafts shall extend from the top elevation, as shown on the plans, to a specified depth into sound rock. The lengths shown in the estimate of quantities are based on information obtained during the subsurface investigation and are not necessarily the lengths needed to achieve the required rock socket. The contractor will be responsible for installing concrete and reinforcement to the required depth determined by the top of sound rock uncovered during excavation and in accordance with ~~paragraph 5.0 of this Special~~ Note. The geotechnical engineer's report, drill logs, and rock cores are available for review at the Division of Materials, Wilkinson Boulevard, Frankfort, Kentucky.

4.3 The casing method shall be used to prevent hole caving or excessive deformation of the hole. In this method the casing may be either placed in a predilled hole or advanced through the ground by twisting, driving or vibrating before being cleaned out.

Where drilled shafts are located in open water areas, casings shall be extended from above the water elevation into the ground to protect the shaft concrete from water action during placement and curing of the concrete. The casing shall be advanced into the RDZ a distance sufficient to stablize the shaft excavation.

4.4 The wet construction method, inside casing, may be used at sites where a dry excavation cannot be maintained for placement of the shaft concrete. This method consists of using water or mineral slurry to maintain stability of the hole perimeter while advancing the excavation to final depth, placing the reinforcing cage, and concreting the shaft under a balanced hydrostatic head at all times.

4.5 The excavation and drilling equipment shall have adequate capacity including power, torque and down thrust to excavate a hole of both the maximum diameter and to a depth of 20 percent greater than the longest shaft.

The excavation tools shall be of adequate design, size and strength to perform the work shown in the plans or described herein. When the material encountered cannot be drilled using conventional earth augers with soil or rock teeth, and/or drill buckets, the contractor shall provide special drilling equipment including but not limited to: rock core barrels, rock tools, air tools, and other equipment as necessary to construct the shaft excavation to the size and depth required. Blasting excavation methods shall not be permitted.

5.0 SUBSURFACE EXPLORATION

[redacted] shown on the plans or as directed by the Engineer to verify the character of material directly below the anticipated shaft tip elevation. Rock Coring shall be completed prior to beginning excavation for any drilled shaft in a group. The rock cores shall be cut with an approved double or triple tube NX size core barrel to a depth [redacted] excavation as shown in the plans. At the direction of the Engineer, the contractor may be required to perform additional rock corings prior to and/or during the course of the drilled shaft excavations. Rock cores shall be measured, visually identified, and described on the geologist's log. The Engineer shall be on-site during the rock coring process to evaluate the rock core samples. Based on his evaluation, the Engineer shall determine the need to extend the cores to depths greater than 20' below the anticipated tip elevations of the drilled shafts. The samples shall be placed in standard rock core boxes, identified by shaft location, elevation, project number, and delivered with the geologist's field log to the Engineer within 24-hours of completion of rock coring. The Engineer will inspect the cores and determine the final depth of required excavation (final drilled shaft tip elevation) based on evaluation of the material's suitability. Final tip elevations for shaft locations other than those for which rock coring has been performed, will be established by the Engineer based on the results of the rock coring. For Pier 8, the tip of Drilled Shaft shall be at or below el. 234. For Pier 9, the tip of Drilled Shaft shall be at or below el. 251.5 Rock coring shall be in accordance with the Division of Materials Geotechnical Guidance Manual. The unit price bid for 'Rock Coring' shall be full compensation for all mobilization, equipment, labor, incidental items, and operations necessary to complete the coring operations in accordance with the Geotechnical Guidance Manual. Rock coring performed at the direction of the Engineer in excess of the above described 20', shall be measured and paid for at the flat rate of $30 per linear foot. 20' Rock coring performed in addition to locations shown on the plans shall be paid for at the unit price bid for 'Rock Coring', each.

Two copies of the typed final contractor's log shall be furnished to the Engineer for review and approval one(1) week prior to beginning any shaft excavation.

6.0 EXCAVATIONS

Shaft excavations shall be made at the locations and to the dimensions shown in the contract documents. The drilled shaft shall be excavated to the elevation shown in the contract documents, or as determined in Item 5.0 *Subsurface Exploration*, or as directed by the Engineer when the material encountered during excavation is unsuitable and/or differs from that anticipated in the design of the drilled shaft.

Excavation of the rock socket is to proceed only if the contractor can complete the excavation, excavation inspection, reinforcement placement and concreting as a continuous operation. No two adjacent rock sockets can be excavated and left open at the same time.

The contractor shall maintain a construction method log during shaft excavation. The log shall contain information including but not limited to: the description and approximate top and bottom elevation of each soil or rock material, seepage or groundwater, and remarks.

The Contractor shall provide the Department with the following records:

1) Report Form
2) Single Shaft Record Report
3) Drilling Procedures and Results
4) Casings
5) Concreting
6) Drilling Slurry (when used)

Examples of these forms are included.

Excavated materials which are removed from shaft excavations shall be disposed of by the contractor in accordance with the Standard Specifications or the contract documents.

On projects with cofferdams, the contractor shall provide a qualified diver to inspect the cofferdam conditions when a seal is required for construction. Prior to concrete seal placement the diver shall inspect the cofferdam interior periphery including each sheeting indentation and around each drilled shaft to insure no layers of mud or undesirable material remain above the planned bottom elevation of seal.

The contractor shall not permit workmen to enter the shaft excavation for any reason unless: both a suitable casing has been installed and adequate safety equipment and procedures have been provided to workmen entering the excavation.

7.0 OBSTRUCTIONS

Subsurface obstructions at drilled shaft 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 augers fitted with soil teeth, drilling buckets and/or underreaming tools. Such special procedures or tools may include but are not limited to rock augers, core barrels, air tools, hand excavation, temporary casing, and increasing the hole diameter. Blasting shall not be permitted unless specifically approved in writing by the engineer. Obstruction removal will be paid for at the unit price bid for *70" Drilled Shaft, Solid Rock.*

Drilling tools which are lost in the excavation shall not be considered obstructions and shall be promptly removed by the contractor without compensation. All costs due to tool removal shall be at the sole expense of the contractor including but not limited to, costs associated with hole degradation due to removal operations or the time the hole remains open.

8.0 INSPECTION

The contractor shall provide and maintain suitable means for access and electric lighting to enable the Engineer to inspect the rock sockets and check dimensions, locations, and alignment of the shafts. Mechanical equipment placed in the excavation for pumping or drilling shall be operated by compressed air or electricity to keep the shaft free from toxic fumes. The Contractor shall provide equipment for checking the dimensions and alignment of each permanent shaft excavation. The dimensions and alignment shall be determined by the contractor under the direction of the engineer. Final shaft depths shall be measured with a suitable weighted tape or other approved methods after final cleaning. Unless otherwise stated in the plans, the base of each shaft shall have less than 1/2 inch of sediment at the time of placement of the concrete. In addition, for dry excavations, the maximum depth of water shall not exceed 3 inches prior to concrete pour for tremie or pump methods of concrete placement or 1 inch prior to concrete placement for free fall methods. Shaft cleanliness will be verified by the engineer using direct visual inspection or other methods, including the use of video camera inspection, underwater inspection procedures, or other procedures deemed appropriate by the engineer for wet or dry shafts. The sides surfaces of 'Drilled Shaft, Solid Rock' shall be inspected. The surfaces shall in general be rough and of such condition as to ensure bond between the shaft concrete and the rock. The contractor shall mechanically roughen smooth surfaces found in the inspection, at the direction of the Engineer. The contractor shall provide equipment necessary to verify shaft cleanliness for the method of inspection selected by the Engineer.

9.0 CONSTRUCTION TOLERANCES

The following construction tolerances apply to drilled shafts unless otherwise stated in the contract documents:

a) The drilled shaft shall be within 3 inches of plan position in the horizontal plane at the plan elevation for the top of the shaft.

b) The vertical alignment of a vertical shaft excavation shall not vary from the plan alignment by more than 1/4 inch per foot depth.

c) After all the concrete is placed, the top of the reinforcing steel cage shall be no more than 6 inches above and no more than 3 inches below plan position.

d) All casing diameters shown on the plans refer to O.D. (outside diameter) dimensions. The dimensions of casings are subject to American Pipe Institute tolerances applicable to regular steel pipe. When approved, the contractor may elect to provide a casing larger in diameter than shown in the plans.

e) The top elevation of the shaft shall have a tolerance of plus 3 inch or minus 3 inches from the plan top of shaft elevation, as measured after excess shaft concrete has been removed.

f) Excavation equipment and methods shall be designed so that the completed shaft excavation will have a planar bottom. The cutting edges of excavation equipment shall be normal to the vertical axis of the equipment within a tolerance of $\pm$ 3/8 inch per foot of diameter.

Drilled shaft excavations and completed shafts not constructed within the required tolerances are unacceptable. The contractor shall be responsible for correcting all unacceptable shaft excavations and completed shafts to the satisfaction of the engineer. Materials and work necessary, including engineering analysis and redesign, to complete corrections for out of tolerance drilled shaft excavations shall be furnished without either cost to the State or an extension of the contract time.

When a shaft excavation is completed with unacceptable tolerances, the contractor shall be responsible for proposing, developing, and after approval by the Engineer, implementing corrective treatment. Typical corrective treatments include:

a) Overdrilling the shaft excavation to a larger diameter to permit accurate placement of the reinforcing steel cage with the required minimum concrete cover.

b) Increasing the number and/or size of the steel reinforcement bars.

c) Drilling out the green concrete and reforming the hole.

The approval of correction procedures is dependent on analysis of the effect of the degree of misalignment and improper positioning. Redesign drawings and computations submitted by the contractor shall be signed by a Professional Engineer registered to practice in Kentucky.

10.0 REINFORCING STEEL CAGE FABRICATION AND PLACEMENT

The reinforcing steel cage, consisting of longitudinal bars, ties, spirals, cage stiffener bars, spacers, centering devices, and other necessary appurtenances, shall be completed, assembled and placed as a prefabricated unit immediately after the shaft excavation is inspected and accepted, and just prior to concrete placement.

The reinforcing steel in the shaft shall be 100% tied and supported so that the reinforcing steel will remain within allowable tolerances for position. Splices shall only be allowed in the lower half of the drilled shafts. No more than 50% of the longitudinal reinforcing shall be spliced within 2 lap splice lengths of any location or within 3' of the splice location if mechanical connectors (developing at least 125% of yield strength of the reinforcing bar) are used). Bands, temporary cross ties, etc. shall be used as required to provide a reinforcement cage of sufficient rigidity to prevent racking, permanent deformations, etc. during installation. Concrete spacers or other approved noncorrosive spacing devices shall be used at the sufficient intervals (near the bottom and at intervals not exceeding 10 feet along the shaft) to insure concentric spacing for the entire cage length. Concrete spacers, if used, shall be constructed of material equal in quality and durability to the concrete specified for the shaft. The spacers shall be of adequate dimension to maintain the specified annular space between the outside of the reinforcing cage and the side of the excavated hole. Approved cylindrical concrete feet (bottom supports) shall be provided to insure that the bottom of the cage is maintained a minimum of 3" clear above the bottom of the drilled shaft excavation. In the event that the shaft has been excavated below the anticipated tip elevation, the reinforcing cage may be extended at the tip (low) end by lap splices, mechanical connectors, or welded splices in conformance with the Standard Specifications. In this instance, splices need not be staggered and 100% of the reinforcing bars may be spliced at a given location.

During concrete placement, the reinforcing cage shall be temporarily supported at or near the top of shaft such that the bottom cage supports are positioned approx. 1" above the bottom of shaft excavation. At the completion of concrete placement, temporary supports shall be removed.

Prior to placing the reinforcement cage, the contractor shall demonstrate to the satisfaction of the Engineer that the fabrication and handling methods to be used will result in a reinforcing cage placed in the proper position, with the proper clearances, and without permanent bending or racking of the reinforcement cage.

The elevation of the top of the steel cage shall be checked before and after the concrete is placed. If the rebar cage is not maintained within the specified tolerances, corrections shall be made by the contractor to the satisfaction of the engineer. No additional shafts shall be constructed until the contractor has modified his rebar cage support in a manner satisfactory to the engineer.

11.0 CONCRETE PLACEMENT

11.1 GENERAL

Concrete placement shall be performed in accordance with applicable portions of the Standard Specifications and with the requirements setforth herein.

The provisions of the *Special Note for Structural Mass Concrete* shall not apply to concrete within Drilled Shafts.

Concrete shall be placed as soon as practicable after reinforcing steel placement. Concrete placement shall be continuous from the bottom to above the top elevation of the shaft. Unless otherwise directed by the Engineer, concrete placement shall continue to approx. 1 shaft dia. (6 feet) above the top of shaft elevation to insure that sound concrete is present at the top of shaft elevation. Contaminated concrete and deleterious material accumulated above the top of shaft elevation may be removed to within 1' of plan top of shaft immediately after concrete placement has been completed. Deleterious material and contaminated concrete may be airlifted under a head of water or slurry, provided that the head is maintained at or near the exterior water surface elevation. Any concrete remaining above plan top of shaft shall be carefully removed after the cofferdam is dewatered. Concrete shall be placed either by free fall or through a tremie or concrete pump. The free fall placement method shall only be permitted in dry holes. The maximum height of free fall placement shall not exceed 5 feet. Concrete placed by free fall shall fall directly to the base without contacting either the rebar cage or hole sidewall. Drop chutes may be used to direct concrete to the base during free fall placement.

The elapsed time from the beginning of concrete placement in the shaft to the completion of the placement shall not exceed 2 hours. All admixtures, when approved for use, shall be adjusted for the conditions encountered on the job site so that the concrete remains in a workable plastic state throughout the 2 hour placement limit. Prior to concrete placement the contractor shall provide test results of both a trial mix and a slump loss test conducted by an approved testing laboratory using approved methods to demonstrate that the concrete meets this 2 hour requirement. The Contractor may request a longer placement time provided that a concrete mix, slump loss test demonstrates a gradual slump loss where at least 4 inches of slump remains at 4 hours after batching. Trial mix and slump loss tests shall be conducted using conditions and ambient temperatures approximating site conditions.

11.2 TREMIE PLACEMENT

Tremies may be used for concrete placement in either wet or dry holes. Tremies used to place concrete shall consist of a tube of sufficient length, weight, and diameter to discharge concrete at the shaft base elevation. The tremie shall not contain aluminum parts which will be in contact with the concrete. The tremie inside diameter shall be at least 6 times the maximum size coarse aggregate to be used in the concrete mix but shall not be less than 10 inches. The inside and outside surfaces of the tremie shall be clean and smooth to permit both flow of concrete and unimpeded withdrawal during concreting. The wall thickness of the tremie shall be adequate to prevent crimping or sharp bends which restrict concrete placement.

The tremie used for wet excavation concrete placement shall be watertight. Underwater placement shall not begin until the tremie is placed to the shaft base elevation. Valves, bottom plates or plugs may be used only if concrete discharge can begin within approx. 2" above the excavation bottom. Plugs shall either be removed from the excavation or be of a material, approved by the Engineer which will not cause defects in the drilled shaft if not removed. The discharge end of the tremie shall be constructed to permit the free radial flow of concrete during placement operations. The tremie discharge end shall remain at or near the bottom of excavation as long as practicable during concrete placement. The tremie discharge end shall remain immersed as deep as practicable in the concrete, consistent with the contractor's construction methods, and shall be immersed at least 10 feet in concrete at all times after starting the flow of concrete. The flow of the concrete shall be continuous. The concrete in the tremie shall be maintained at a positive pressure differential at all times to prevent water or slurry intrusion into the shaft concrete.

If at any time during the concrete pour, the tremie line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the entire drilled shaft shall be considered defective. In such case, the contractor shall remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer and repour the shaft. All costs of replacement of defective shafts shall be the responsibility of the Contractor and shall be at no cost to the Department.

11.3 PUMPED CONCRETE

Concrete pumps and lines may be used for concrete placement in either wet or dry excavations. All pump lines shall have a minimum diameter of 5 inches, and shall be constructed with watertight joints. Concrete placement shall not begin until the pump line discharge orifice is at the shaft base elevation.

For wet excavations, a plug or similar device shall be used to separate the concrete from the fluid in the hole until pumping begins. The plug shall either be removed from the excavation or be of a material, approved by the Engineer, which will not cause a defect in the shaft if the plug is not removed.

The discharge orifice shall remain at least 10 feet below the surface of the fluid concrete. When lifting the pump line during concreting, the Contractor shall temporarily reduce the line pressure until the orifice has been repositioned at a higher level in the excavation.

If at any time during the concrete pour, the pump line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the shaft shall be considered defective. In such case, the contractor shall remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer, and repour the shaft. Any and all costs associated with replacement of defective shafts shall be at the sole expense of the Contractor.

11.4 DROP CHUTES

Drop chutes may be used to direct placement of free fall concrete in excavations where the maximum depth of water does not exceed 1 inch. The free fall method of placement shall not be used in wet excavations. Drop Chutes shall be a smooth tube constructed either as a continuous one piece unit or as removable sections. Concrete may be placed through either a hopper at the top of the tube or side openings as the drop chute is retrieved during concrete placement. The drop chute shall be supported so that the free fall of the concrete measured from the bottom of the chute is less than 5 feet at all times. If the concrete placement causes the shaft excavation to cave or slough, of if the concrete strikes the rebar cage or sidewall, the contractor shall reduce the height of free fall and/or reduce the rate of concrete flow into the excavation. If placement cannot be satisfactorily accomplished by free fall in the opinion of the Engineer, the Contractor shall use either tremie or pumping to accomplish the pour.

12.0 METHOD OF MEASUREMENT

The accepted drilled shafts will be measured for payment to the nearest 0.1 foot of shaft in place.

For pay purposes, the length of any drilled shaft installed and accepted in Pier 8 above elev. 265.9 shall be measured and paid for at the unit price bid for 'Drilled Shaft, Common', irrespective of the character of the material actually encountered during excavation. Drilled shaft installed and accepted in Pier 8 below elevation 265.9 shall be measured and paid for at the unit price bid for 'Drilled Shaft, Solid Rock', irrespective of the character of the material actually encountered during excavation.

For pay purposes, the length of any drilled shaft installed and accepted in Pier 9 above elev. 267.2 shall be measured and paid for at the unit price bid for 'Drilled Shaft, Common', irrespective of the character of the material actually encountered during excavation. Drilled shaft installed and accepted in Pier 9 below elevation 267.2 shall be measured and paid for at the unit price bid for 'Drilled Shaft, Solid Rock', irrespective of the character of the material actually encountered during excavation.

Rock coring shown on the plans, and as specified in Section 5 of this Special Note, or as directed by the Engineer, shall be paid for at the unit price bid for 'Rock Coring', each. In the event that the Engineer directs rock coring more than 20' below anticipated tip of shaft elevation as shown on the plans, that portion of the rock coring in excess of 20' below the anticipated tip of shaft elevation as shown on the plans shall be measured and paid for at the flat rate of $30 per linear foot of excess.

13.0 BASIS OF PAYMENT

The accepted quantities of drilled shafts shall be paid for at the contract unit price bid per linear foot of drilled shaft of the size detailed on the plans. This shall constitute full compensation for all costs incurred during installation as described herein or in the contract documents including but not limited to subsurface exploration, excavation, sealing, dewatering, steel casings, reinforcing steel, concrete, submitting records, and all incidental labor, equipment and materials which may be necessary to complete the drilled shafts. No additional compensation will be permitted for shafts constructed larger in diameter than those shown on the plans.

Payments shall be made under:

Pay Item	Pay Unit
72" Drilled Shaft, Common	Linear Foot
70" Drilled Shaft, Solid Rock	Linear Foot
Rock Coring	Each

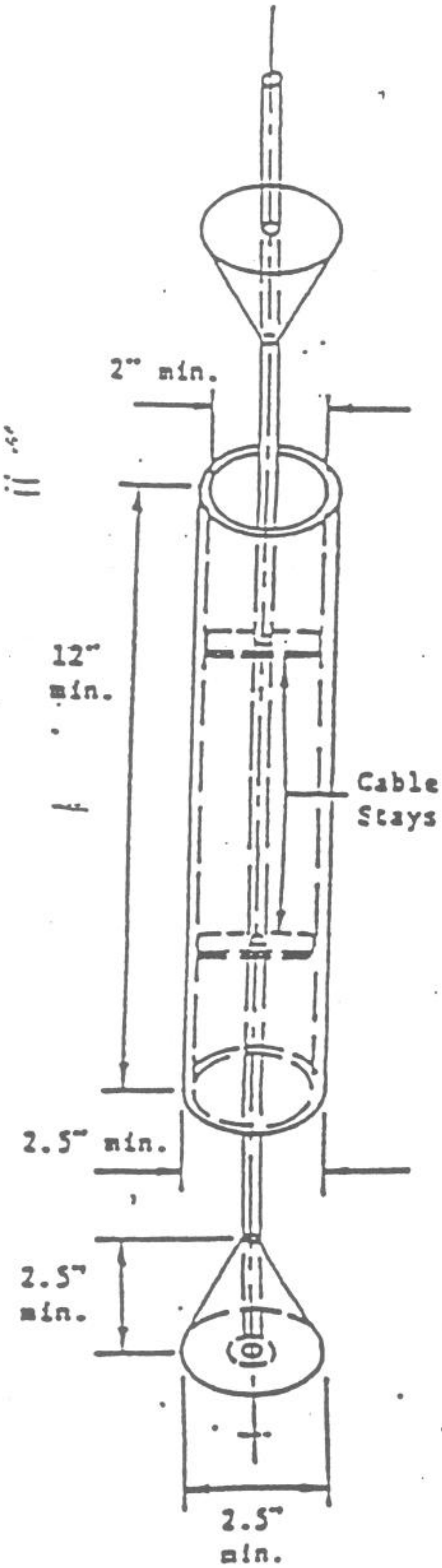


FIGURE 1 SLURRY SAMPLER

The sampler consists of three components:

1. Cable with weighted cone-shaped stopper
2. Cylindrical sampler center stayed for alignment.
3. Top stopper with hole drilled through the center for slipping onto cable.

SAMPLING IS ACCOMPLISHED BY:

1. Lower cable with stopper to desired sampling elevation.
2. Slide cable through aligning guides of sampler.
3. Let sampler drop down the cable and seat onto bottom cone-shaped stopper.
4. Slide cable through hole in top stopper and let drop to seat on top of sample.
5. Withdraw entire assembly from shaft.
6. Sample may be emptied into separate container and used as necessary to perform required testing.