Auburn University

DRILLED SHAFT FOUNDATION TESTING AND DESIGN RECOMMENDATIONS I-65 OVER MULBERRY FORK OF THE WARRIOR RIVER BLOUNT/CULLMAN COUNTIES, ALABAMA

by

Dan A. Brown, P.E., Ph.D.
Robert Thompson, E.I.T.

sponsored by

The State of Alabama Highway Department
Montgomery, Alabama

Highway Research Center
Harbert Engineering Center
Auburn University, Alabama 36849-5337

January 1994

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College of Engineering

Department of Civil Engineering
Harbert Engineering Center

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Jan. 6, 1994

Morris-Shea Bridge Company, Inc.
c/o Christy Cobb Consulting Engineers
2320 Highland Ave., Suite 100
Birmingham, Alabama 35205

Att: Mr. Len Cobb

Re: Drilled Shaft Foundation Testing and Design Recommendations
I-65 over Mulberry Fork of the Warrier River
Blount/Cullman Counties, Alabama

Gentlemen:

As a part of our research effort into the behavior of drilled shafts in Alabama, we have agreed to assist Mr. Dick Shea and Morris-Shea Bridge Company in the load testing program and development of drilled shaft foundation design recommendations for the subject project. We understand that this effort is part of a value engineering alternative design for the bridge substructure.

We feel that this testing program has provided much needed information on the performance of drilled shafts socketed into relatively weak rock and will be of great benefit to our research and to the economical and reliable design of drilled shafts on future Alabama DOT bridges in similar geomaterials. We trust that the design recommendations developed from these data will assist you in the completion of a successful and profitable project. Our report is attached which summarizes the load test data, analyses, and recommendations for design.

If you have any questions or comments concerning this information, please do not hesitate to call; my direct line at Auburn University is (205) 844-6283.

Sincerely,

Dan A. Brown

Dan A. Brown, P.E., Ph.D.

Robert Thompson

Robert Thompson, E.I.T.

Drilled Shaft Foundation Testing and Design Recommendations

I-65 over Mulberry Fork of the Warrior River

Blount/Cullman Counties, Alabama

1. Introduction and Background Information. As a part of the Auburn University research effort into the behavior of drilled shafts in Alabama, the writers have agreed to assist Mr. Dick Shea and Morris-Shea Bridge Company in the load testing program and development of drilled shaft foundation design recommendations for the subject project. This effort is understood to be part of a value engineering alternative design for the bridge substructure.

The central portion of the alternative design for the bridge is planned to consist of two-column bents in lieu of the originally designed "hammerhead" bents. The two proposed columns would be designed with a single drilled shaft foundation under each. Estimated maximum column loads for the two-column bents are anticipated to be 1570kips axial compression, 52kips shear, with 2006 kip-ft. applied overturning moments.

A program of static load tests to measure axial and lateral performance of drilled shafts socketed into this shale was performed, with additional geotechnical data to better characterize the shale within the load test regions. A lateral load test was performed at a shale outcrop near Bent 2, SBL by horizontally jacking two 30 inch diameter by 15 foot deep reinforced concrete drilled shafts. An axial load test was performed within the shale near Bent 5, SBL using the downhole "Osterberg Cell" jack at the base of a 30 inch diameter shaft which was approximately 60 feet deep and concreted to a level 12 feet above the base (i.e. a 12 foot socket over a length between 47 and 59 feet below existing grade). Results of the additional geotechnical investigation and the load testing program are summarized below and provided in detail in Appendices A, B, and C. Results of analyses of lateral load performance for test shafts and proposed design shafts are provided in Appendix D.

2. Geotechnical Data. Initial geotechnical characterization of the site was provided by Ala. DOT borings at approximately each bent location. These indicated the founding stratum for drilled shafts to consist of a medium hard gray shale, with up to 40 feet of loose, sandy overburden soil in the alluvial

valley of the stream. The majority of the overburden is expected to be removing during a maximum scour event, leaving the shale to be largely exposed. Beneath a few feet of relatively weathered shale, Ala. DOT borings penetrated by auguring to depths of typically 25 to 40 feet before meeting auger refusal. No cores were made, and attempts to perform standard penetration tests achieved no penetration below the top few feet of shale.

Additional borings were made by Ground Engineering and Testing Service, Inc. of Birmingham, at two locations chosen to correlate with the load testing program. Boring B-1 was made to a depth of 29.5 feet at a location near Bent No. 2 on the south bound lane (west) side of the project near the location of lateral load tests. At this location the shale is quite shallow, and a bench was cut into the hillside so as to allow lateral load testing on a shale outcrop. Boring B-1 indicated about 9.5 feet of weathered shale and sandstone layers, underlain by medium hard shale with numerous joints and fissures and sandstone stringers and layers. This boring was cored between the 9.5 and 29.5 foot depths. Unconfined compression test data from cores in shale were 2700 to 5700 psi, with a compressive strength of approximately 15,000 psi in a core from a sandstone layer. Splitting tensile strengths were 430 to 480 psi in the shale, 1200 psi on one core from the sandstone. Pressuremeter tests were also performed in this boring at levels 11 to 17 feet below the surface. Pressuremeter moduli were in the range of about 6,000 to 13,000 psi.

Boring B-2 was made to a depth of 69 feet at a location near Bent No. 5 on the south bound lane (west) side of the project near the location of axial load tests. At this location the shale is fairly deep, covered by approximately 30 feet of loose silt and sand. Boring B-2 indicated about 8.5 feet of weathered shale and sandstone layers, underlain by medium hard shale with numerous fine fractures. Sandstone stringers and layers were somewhat less common than at Boring B-1. This boring was cored between the 38.6 and 69 foot depths. Unconfined compression test data from cores in shale ranged from 817 to 3183 psi, with the larger values within the upper 10 feet of coring (between 38 and 47 feet below existing grade). The axial load test shaft was concreted within the weaker portion of the shale between 47 and 60 feet below grade; compressive strength tests within this zone ranged from 817 to 975 psi. Splitting tensile strengths were 135 to 709 psi, and ranged from 135 to 410 psi within the 48 to 60 foot depth range.

Detailed boring logs and tabular results of the core testing are provided in Appendix A. Pressuremeter test data are summarized along with a brief description of the lateral load test in Appendix C.

3. Axial Load Test. A detailed report of the axial load test of a drilled shaft socketed into the shale at Bent 5, SBL is provided in Appendix B. This test was conducted using the Osterberg load cell, which is placed at the base of a shaft excavation and used to jack the socket upward (testing the socket side friction) against the base resistance downward. Mr. Jeff Goodwin of Loadtest, Inc. installed the cell and conducted the test and has prepared the detailed report of Appendix B.

The test shaft was a 32 inch diameter by 12 foot long socket which was installed with the base at a depth of approximately 60 feet. A schematic is provided on Figure 1.2 of Appendix B. Allowing for the thickness of the cell, etc., the socket tested covered the zone from 47 to 59 feet below the existing ground surface. The medium hard shale began at a depth of about 35 feet at this location; however, the core test data suggest that the 47 to 59 foot zone tested represents the relatively weaker material.

The original intent was to fail the 12 foot socket during testing and then place additional concrete to complete a longer socket and attempt to achieve base failure. However, the unanticipated high capacities exceeded the capacity of the Osterberg jack for even the 12 foot socket length. The cell was inflated to the maximum capacity of 1160 tons during testing, and only very minor upward movements were observed in the socket. In fact, the strain gauges which were present within the socket suggest that the shaft friction was not even close to failure over the length of the socket. Just conservatively considering the load divided by the surface area of the socket (1160 tons / (π x dia. x 12 ft.)) yields an average **mobilized unit socket friction of over 11.5 tons/sq.ft.** The strain gauge data suggest that much higher socket friction is available, but the 11.5 tsf value can be reliably justified as an ultimate value for use in design.

More significant downward movement was observed of the base of the cell during testing; however, the load - displacement relationship for the base of the shaft suggest that some and perhaps most of this movement is likely attributed to crushing of incomplete grout below the base of the shaft or possibly the presence of some debris which may have been knocked into the base of the excavation during placement of the cell. The load - displacement data for the base of the shaft clearly indicate that the **over 200 tsf unit base pressure** was not approaching a limiting value (or else the load - movement curve on Figure 1.4 of Appendix B would be curving down).

4. Lateral Load Testing. A bench was cut into the hillside near Bent 2, SBL to allow lateral load testing of the shale formation at a shallow depth. Geologic data and Ala. DOT borings from the earlier investigation suggest that the shale at Bent 2 is of the same formation as will be present at other bents. Two 32 inch diameter by 15 to 18 foot deep shafts were installed about 30 feet apart and jacked toward each other using Dywidag bars as tension connectors. Load was measured with a calibrated jack, and displacements were monitored with both dial gauges and using an inclinometer in each shaft to measure the lateral displacement as a function of depth. A description of the test setup and presentation of results is provided in Appendix C.

These two shafts were intentionally installed into slightly different materials so as to provide test data for a range of conditions. The shale at this location is such that there is about 5 feet of weathered and highly fractured shale overlying the medium hard material. The bench was cut and the shafts installed so that the east shaft would have this weathered and highly fractured shale in the upper 5 feet below grade and the west shaft would be installed in a location where this weathered and highly fractured material has been removed. Thus, the east shaft will represent a test under conservative conditions for design while the west shaft may be more representative. Results from the core strength tests suggest that the shallow hard rock had significant amounts of sandstone layers and may be harder than the shale at greater depths and at other locations. Therefore, the lateral test in the relatively poorer rock of the east shaft has been used for design.

As can be observed in the figures of Appendix C, these shafts were loaded to 300 kips lateral load with lateral movements just above grade of well under 1/2 inch. The west shaft was over twice as stiff as the east shaft. 300 kips was considered the safe limit of the jacking system. Inclinometer data indicate that virtually all of the movement (and therefore load transfer to the shale) occurred within the upper 8 feet below grade.

5. Recommendations for Design. Given the maximum column loads of 1570 kips axial compression, 52 kips shear, and 2006 kip-ft. overturning moment (at the top of the shale formation), the following

minimum shaft dimensions are proposed:

minimum shaft diameter:	60 inches
minimum socket length:	15 feet
full length shaft reinforcement	
designed for bending moments of at least:	2116 kip-ft.

The socket length is considered to be a length into medium hard shale and can conservatively be taken as the length below the bottom of casing where the casing has been seated into the shale to seal off the water. Other diameters could be considered; however, discussions have indicated that the 60 inch diameter shaft would work well with the structure planned.

6. Analyses for Lateral Loads. Analyses indicate that lateral and overturning forces govern the shaft dimensions for this site. The recommendations cited above have been developed using the computer code COM624, with p-y curves derived from the load test measurements. All soils above the shale have been neglected to account for scour, and any severely weathered shale which is easily penetrated by the casing is also neglected. Backward fitting of the lateral load test data to provide p-y curves for a reasonable match was achieved with the properties outlined below.

Best fit using p-y criteria of Matlock & Reese for "soft clay" with the following properties:

depth below top of shale	cohesion (psi)	ε_{50} value
0 ft.	100	.009
4 ft.	400	.007
6+ ft.	500	.007

The p-y curves generated using the properties shown above yielded computed displacements for the load test conditions as indicated below:

Displacements, inches

load, kips	computed	meas'd - east	meas'd - west
100	.059	.074	.033
200	.189	.198	.079
300	.377	.357	.141

As is evident from the data shown above, the p-y curves generated using these "best fit" properties provide reasonable agreement with the more conservative load test conditions. The computed displacements below grade, although not shown, provided a reasonably similar distribution to the load test data.

Analyses of the proposed design have been performed using the p-y curves derived from the load test data (although adjusted for diameter according to the Matlock & Reese criteria). In order to account for potential variability across the site, these analyses have been performed for both the best estimated p-y curves as well as for p-y curves using cohesion values which are taken as 1/2 the back derived values. In addition to the design loads, calculations have also been performed for 2 and 3 times the design load shear and moment combination. This evaluation of overload condition is simply made to verify that the bearing formation has more than adequate reserve capacity; it is probable that the shaft and/or column would fail in bending and form a plastic hinge before 3 times the design shear and moment could be achieved. Results are summarized below:

Deflection at top of Shale, inches

DLF*	Case I**	Case II**
1	.08	.12
2	.21	.34
3	.37	.69

* DLF stands for "design load factor" and represents multiples of the design shear and moment loads of 52 kips and 2006 k-ft., respectively.

** Case I represents analyses using the p-y parameters derived from the load test data as indicated above; Case II represents analyses using similar p-y parameters but with cohesion values multiplied by 1/2 to account for possible variability across the site.

Maximum moments in the shaft at the design loads were computed to be 2033 and 2116 k-ft. for Case I and Case II, respectively.

Details of COM624 analyses are provided in the computer printouts attached in Appendix D. Note that the cover sheet describes the code as "LPILE1"; this is the commercial version of the FHWA distribution of COM624 and is identical software.

The data presented above suggest that the design as proposed would proved adequate reserve capacity for lateral loads and would be subject to very small lateral deflections under the design load conditions including scour.

7. Analysis for Axial Loads. For a 60 inch diameter by 15 foot long socket into shale subject to an axial compression load of 1570 kips, the average computed socket friction (completely ignoring end bearing) would be 6.66 kips/sq.ft. or 3.33 tsf. With a measured socket friction of at least 11.5 tsf, this computed socket friction would be supported with a factor of safety of at least 3.5 without any reliance upon end bearing. Thus, the proposed shaft design has more than adequate reserve capacity for axial loads and would be expected to support the design axial loads with only nominal displacements (a few hundredths of an inch).

8. Construction Recommendations. A few noteworthy points with respect to construction of the proposed design are summarized below.

* The required socket length should generally be taken from the base of the casing which has been seated into the shale, thus neglecting any severely fractured and/or weather zones.
* The shale did not appear to be unusually prone to deterioration in the presence of water unless allowed to dry out. However, it is recommended that a socket be completed and concrete placed in the same day that the socket excavation is commenced so as to take every precaution to preserve the integrity of the bearing formation.
* The shale formation is expected to contain numerous fractures and sandstone seams; it is not necessary that the shaft be formed in unfractured shale material.

* Because of the fractures and seams, it may also be the case that a watertight seal is not possible in some instances. A moderate amount of seepage is no cause for concern with respect to the integrity of the bearing formation, but if the shaft cannot be dewatered so that no more than one or two inches of water remains then the concrete should be placed using a tremie. For dry or nearly dry shafts, free fall placement of concrete is considered acceptable (pending Ala. DOT approval, of course).

Appendix A

Additional Soil Borings

and

Core Test Data

GROUND ENGINEERING & TESTING SERVICE, INC.
4764 FIRST AVENUE NORTH - BIRMINGHAM, AL 35222 TELEPHONE (205) 591-4340 FAX (205) 591-6028

LETTER OF TRANSMITTAL

TO: DAN BROWN
AUBURN UNIVERSITY
DEPT. OF CIVIL ENGINEERING
HARBERT ENGINEERING CENTER
AUBURN UNIVERSITY, ALABAMA
36841-3501

DATE 1-6-94 JOB NO. 7315-001

RE: BORING LOGS AND LABORATORY TEST RESULTS - I-65 BRIDGE

TRANSMITTAL METHOD:
- ☑ Fax
- ☐ Mail
- ☐ Hand Deliver
- ☐ UPS
- ☐ Other

TRANSMITTED THE FOLLOWING:
- ☐ Report
- ☐ Proposal
- ☐ Plans/Drawings
- ☐ Copy of Letter
- ☑ Other RESULTS

DESCRIPTION:

☑ **GEOTECHNICAL**
- ☑ Boring Logs
- ☐ Specifications
- ☐ Other*

☐ **ENVIRONMENTAL**
- ☐ Site Assessment
- ☐ UST Closure Assessment
- ☐ Phase II
- ☐ Sampling Results
- ☐ Other*

☑ **TESTING**
- ☐ Field Density
- ☐ Concrete
- ☐ Soil Compaction
- ☐ Asphalt
- ☑ Other*

☐ **INSPECTION**
- ☐ Site
- ☐ Footings
- ☐ Caissons
- ☐ Driven Piles
- ☐ Other*

* ROCK CORE TESTING

COPIES	DATE	NUMBER	DESCRIPTION
1	1-6-94		BORING LOGS
1	1-6-94		TEST RESULTS

THESE ARE TRANSMITTED AS CHECKED BELOW:

- ☐ For approval
- ☑ For your use
- ☐ As requested
- ☐ For review and comment
- ☐ FOR BIDS DUE ____________, 19____
- ☐ Approved as submitted
- ☐ Approved as noted
- ☐ Returned for corrections
- ☐ ____________
- ☐ Resubmit ______ copies for approval
- ☐ Submit ______ copies for distribution
- ☐ Return ______ corrected prints
- ☐ RETURN OF LOANED PRINTS

REMARKS

IF ADDITIONAL INFORMATION IS NEEDED, PLEASE ADVISE.

KEN

GROUND ENGINEERING AND TESTING SERVICE, INC.

ROCK CORE UNCONFINED COMPRESSION
ASTM D2938-86

CLIENT / PROJECT INFORMATION / TEST DATES

CLIENT NAME: DRILL SOUTH, INC.

JOB NUMBER: B7315-001

PROJECT NAME: I-65 BRIDGE

DATE CORED: 12/20/93

CORES SOAKED in WATER PRIOR to TEST

DATE TESTED: 01/05/94

ROCK CORE IDENTIFICATION / TEST RESULTS

TEST NUMBER	CORE ID AND LOCATION	DIAMETER INCHES	LENGTH ORIG.	INCHES CUT	LOAD POUNDS	L/D	CORR. FACTOR	COMPRESSIVE STRENGTH PSI
1	B1/11-12	2.05	0	3.938	49710	1.921	.995	14986
2	B1/13-14.5	2.049	0	3.688	18970	1.8	.987	5678
3	B1/16.5-17.5	2.037	0	2.125	9840	1.043	.901	2721
4	B2/42.8-43.5	2.037	0	2.813	10930	1.381	.949	3183
5	B2/45.8-46.5	2.036	0	3.438	5090	1.689	.978	1529
6	B2/48.7-50	2.036	0	2.438	3100	1.197	.926	882
7	B2/52.7-53.8	2.036	0	3.5	2710	1.719	.981	817
8	B2/57-58.4	2.036	0	3.125	3290	1.535	.965	975

NOTE: NATURAL ROCK CORES

RESPECTFULLY SUBMITTED

GROUND ENGINEERING AND TESTING SERVICE, INC.

ROCK CORE SPLITTING TENSILE STRENGTH
ASTM D3967-86

CLIENT / PROJECT INFORMATION / TEST DATES

CLIENT NAME: DRILL SOUTH, INC.

JOB NUMBER: B7315-001

PROJECT NAME: I-65 BLOUNT/CULLMAN CO. BRIDGE

DATE CORED: 12/20/93

CORES SOAKED in WATER PRIOR to TEST

DATE TESTED: 01/05/94

ROCK CORE IDENTIFICATION / TEST RESULTS

TEST NUMBER	CORE ID AND LOCATION	DIAMETER INCHES	LENGTH ORIG.	INCHES CUT	LOAD POUNDS	SPLITTING TENSILE, PSI
1	B1 @11-12	2.057	3.188	3.188	12430	1207
2	B1 @13-14.5	2.054	3.375	3.375	5250	482
3	B1 @16.5-17.5	2.042	3.907	3.907	5390	430
4	B2 @42.8-43.5	2.021	1.75	1.75	3940	709
5	B2 @45.8-46.5	2.041	3.29	3.29	5000	474
6	B2 @48.7-50	2.044	3.063	3.063	1330	135
7	B2 @52.7-53.8	2.033	1.875	1.875	1320	220
8	B2 @57-58.4	2.044	4.5	4.5	5930	410

NOTE: NATURAL ROCK CORE

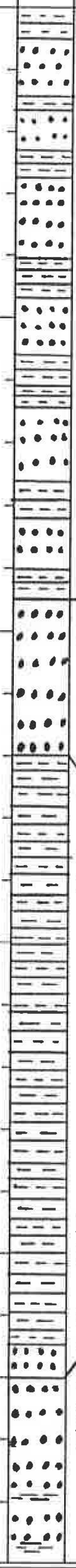
RESPECTFULLY SUBMITTED

BORING RECORD

JOB NO: D2065-001 **PROJECT:** I-65 BRIDGE **BORING NO.** B-1

DATE: 12-9-93 **LOCATION:** BENT # 2 **PAGE** 1 **OF** 2

SURFACE ELEVATION: N/A **FT.** **CASING LENGTH:** 10 FT. **METHOD:** ROCK CORE



DEPTH FT.	Sy	SOIL-ROCK DESCRIPTION Surface:	Qupp/ Core	REMARKS
		SHALE AND SANDSTONE, highly weathered, gray-brown		
5				
			-----	9.5 ft. (Coring Initiated)
10		Auger refusal at ~ 9.5 ft.		
		SANDSTONE, fine to medium grained, brown, becoming gray at ~10.5 ft.		
		SHALE with fine sand, gray contains irregular sandstone stringers, medium hard, contains numerous joints and fractures		
15			98 NQ 41	Recovery: 118 inches
		Moderately fractured below 19.5 ft.	-----	19.5 ft.
20				
		6 inches brown sandstone at ~20.7 ft.		
		SANDSTONE with shale stringers, gray, hard, moderately thick beds, fine grained sandstone	100 NQ 61	Recovery: 120 inches
25				

See Legend for Symbols

DRILL SOUTH, INC.

BORING RECORD

JOB NO: D2065-001	**PROJECT:** I-65 BRIDGE	**BORING NO.** B-1
DATE: 12-9-93	**LOCATION:** BENT # 2	**PAGE** 2 **OF** 2
SURFACE ELEVATION: N/A **FT.**	**CASING LENGTH:** 10 FT.	**METHOD:** ROCK CORE

DEPTH FT.	Sy	SOIL-ROCK DESCRIPTION Surface:	Qupp/ Core	REMARKS
		SANDSTONE continued		
		High angle joint at ~ 29 ft.	-----	29.5 ft.
30		Coring terminated at 29.5 ft.		

See Legend for Symbols

DRILL SOUTH, INC.

BORING RECORD

JOB NO: D2065-001 **PROJECT:** I-65 BRIDGE		**BORING NO.** B-2
DATE: 12-9-93 **LOCATION:** BENT # 5		**PAGE** 1 **OF** 3
SURFACE ELEVATION: N/A **FT.**	**CASING LENGTH:** 38 FT.	**METHOD:** ROCK CORE

DEPTH FT.	Sy	SOIL-ROCK DESCRIPTION Surface:	Qupp/ Core	REMARKS
		Fine **SANDY SILT - SILTY SAND**, brown to orange brown, firm and very moist (SM/ML)		
5				
10				
15				
20				
25				

See Legend for Symbols **DRILL SOUTH, INC.**

BORING RECORD

JOB NO: D2065-001	**PROJECT:** I-65 BRIDGE	**BORING NO.** B-2
DATE: 12-9-93	**LOCATION:** BENT # 5	**PAGE** 2 **OF** 3
SURFACE ELEVATION: N/A **FT.**	**CASING LENGTH:** 38 FT.	**METHOD:** ROCK CORE

DEPTH FT.	Sy	SOIL-ROCK DESCRIPTION Surface:	Qupp/ Core	REMARKS
30		Becomes wet at ~ 30 ft. Weathered, sloping rock face encountered at ~ 30 ft.		
35				
			-----	38.6 ft. (Coring Initiated)
40		Coring initiated at 38.6 ft. **SHALE** with fine sand, highly weathered, gray, moderately hard, contains numerous fine fractures		
			95 NQ 27	Recovery: 114.5 inches
45		Broken shale layer at ~ 45 ft.		
		Broken shale layer at ~47.8 ft. Becomes slightly less weathered below 48 ft.	-----	48.6 ft.
50				

See Legend for Symbols

DRILL SOUTH, INC.

BORING RECORD

JOB NO: D2065-001 **PROJECT:** I-65 BRIDGE **BORING NO.** B-2

DATE: 12-9-93 **LOCATION:** BENT # 5 **PAGE** 3 **OF** 3

SURFACE ELEVATION: N/A **FT.** **CASING LENGTH:** 38 FT. **METHOD:** ROCK CORE

DEPTH FT.	Sy	SOIL-ROCK DESCRIPTION Surface:	Qupp/ Core	REMARKS
		Apparent bedding plane at ~51.3 ft.		
			97 NQ 82	Recovery: 121 inches
		Broken shale layer/weathered joint between 54 and 55 ft.		
55				
		Irregular coal inclusions in shale matrix at ~57.5 ft.		
			-----	59 ft.
60				
			98 NQ 65	Recovery: 118 inches
65				
66.1		7 inch coal seam 66.1 to 66.7 ft.		
		SANDSTONE, fine to medium grained, light gray, hard		
69			-----	69 ft.
		Coring terminated at 69 ft.		
70				

See Legend for Symbols

DRILL SOUTH, INC.

Appendix B

Axial Load Test Data

REPORT ON OSTERBERG CELL LOAD TESTING

I-65 BRIDGE REPLACEMENT
BLOUNT COUNTY, ALABAMA

PREPARED FOR R.R.DAWSON BRIDGE CO.

LOADTEST PROJECT No. LT-177

LOADTEST, INC. REPORT No. LT-177, JANUARY 6, 1994

I- 65 BRIDGE OVER THE BLACK WARRIOR RIVER
BLOUNT COUNTY, ALABAMA

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1. INTRODUCTION:

1.1 PROJECT INFORMATION: An Osterberg Cell load test was performed by **Loadtest, Inc.**, on a test shaft for the I-65 Bridge over the Black Warrior River, in Blount County, Decatur, Alabama, at the request of the foundation contractor, **Morris-Shea Bridge Company**. The shaft was constructed by **Morris-Shea Bridge Company** under the direction of superintendent, Mr. Richard Shea, on December 3, 1993. Following a 3 day curing period, **Loadtest, Inc.** completed the test on December 6, 1993. **Loadtest, Inc.** engineer Mr. Jeffrey Goodwin, P.E., supervised the installation of the Osterberg Cell and load testing.

The purpose of the load test was to assess and verify the drilled shaft design parameters at the site, leading to adjustments in the drilled shaft dimensions where feasible.

1.2 SITE CONDITIONS: Figure 1.1 of this report shows the general layout of the construction site for the I-65 Bridge. The immediate area of the test shaft had been cleared to accommodate the construction of the bridge foundation.

1.3 GENERALIZED SITE GEOLOGY: The generalized site stratigraphy consists of approximately 35 ft. of fill, sand, and clay overlying shale. Groundwater was encountered at a depth of approximately 40 feet. The geotechnical data compiled by the **State of Alabama Highway Department** should be reviewed for more detailed geologic information.

1.4 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. **Loadtest, Inc.** can assist with your obtaining one or more of these references if requested.

1.5 LIMITATIONS: Others provided services with respect to geotechnical engineering design, preparation of project specifications, means and methods, and observation of the test shaft construction. This report contains no interpretations of the data obtained with respect to their application to the design of the project under study.

2. INSTALLATION OF THE TEST SHAFT AND OSTERBERG CELL:

2.1 SHAFT CONSTRUCTION: Drawings showing the test shaft location, dimensions and elevations are included as Figure Nos. 1.1 & 1.2 at the end of this report. Photographs of the test shaft construction and test setup are included in Appendix B. The Contractor began construction of the test shaft on December 2, 1993. The shaft was installed using a track mounted drill rig. The shaft was drilled using continuous casing methods using a 32 in. auger and core barrel.

After reaching the shale at a depth of 35 feet, for the final bearing socket, the socket was advanced 24 feet using an the auger and core barrel. The shaft bottom was cleaned with a bottom cleanout bucket.

When the shaft was approved for concreting, the O-cell was lowered into the shaft after a 1.5-foot thick pad of concrete was placed at the base of the shaft as a seating layer. The O-cell, related piping and instrumentation were then lowered into the shaft and seated into the concrete base layer. A twelve-foot socket length was then concreted by free fall methods to a final cutoff elevation approximately 46.5 ft. below the ground surface.

Osterberg Cell number 92-391-3 was installed in the shaft. The O-cell was 12" high and 21" in diameter and had 26" O.D. top and bottom steel bearing plates. Sixty-five feet of piping were installed from the top of the cell to above the ground surface. The pipe consisted of 2" O.D. schedule 80 steel pressure piping and a 3/4" O.D. schedule 80 steel pipe to be used as a telltale nested inside the 2" pipe.

Calibration data for the Osterberg Cell is included in Appendix A at the end of this report. Note that the cell was calibrated to 600 tons and then extrapolated linearly to 1200 tons.

3. LOAD TEST PROCEDURES:

3.1 GENERAL INFORMATION: The Osterberg technique loads the shaft in two directions by hydraulically pressurizing the O-cell. The O-cell load was determined by measuring the applied hydraulic pressure at the pump and comparing this pressure with the O-cell's load calibration curve. Calibrated, high pressure Bourdon gages were used to read pressure. Pressurizing the O-cell simultaneously loaded the rock beneath the shaft in end bearing and the rock 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 (Osterberg, 1989a).

The O-cell load test was performed in general accordance with ASTM D-1143 Quick Load Test Procedures (ASTM, 1993). Load was applied through a hydraulic pressure system via a hydraulic pump driven by a regulated air compressor system, in increments of approximately 60 to 120 tons for loading, and 600 tons for unloading. Pressure was held at each loading increment for a total of 4 minutes, and for 1 minute per increment during unloading. Gages were read at 1, 2, and 4 minutes after the load was applied, with an average of 1 minute required to increase the jack pressure to the next load increment.

3.2 FIELD MEASUREMENTS: As illustrated in Figure No. 1.3, movement of the O-cell was indicated by an axial "telltale" pipe nested within the vertical hydraulic pressure pipe. From a connector on the bottom plate, the 2" telltale pipe projected upward through the cell where it was held inside 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 downward vertical movement of the bottom telltale relative to the reference beam was measured using Ames' dial gage Nos. 2 & 3 having a travel of 4.000" and read to the nearest 0.001" division. The dial stems of the gages were plumbed to ensure accurate vertical measurement. The raw movement data from the inner telltale was corrected for Poisson's ratio effects on the rod caused by the surrounding hydraulic fluid pressure. This mathematical correction was **added** to the measured movement and depended upon the piping material's inner and outer diameters, length, and the applied hydraulic pressure.

Movement of the top of the shaft was measured using Ames' dial gage No. 1, attached to the outer 4" I.D. hydraulic piping. The upward vertical movement was read relative to the reference beam system, using an Ames' dial gage read to the nearest 0.001" division. The stem of the gage rested on the reference beam.

A 1" I.D steel telltale casing extended from the top of the O-cell, running parallel to the hydraulic pressure piping, up to the ground surface. However, due to bending of this casing during installation, the 3/8" diameter stainless steel telltale rod could not pass through it and, hence, it was not used.

Four Geokon model VCE-4200 embedment strain gages were installed within the socket. The gages were installed 1.0, 3.0, 5.0, and 7.0 ft. above the O-cell. During the load test the strain gages were monitored using an electronic data logging device. The purpose of the strain gages was to determine the load transfer through the rock socket.

4. LOAD TEST RESULTS:

4.1 GENERAL INFORMATION: The load test was begun on Monday December 6, 1993 at 10:40 AM, approximately 3 days after concrete placement. At the time of testing, concrete cylinders formed during the construction of the shafts were tested by others and found to have an average compressive strength of approximately 4200 psi. We obtained an approximate concrete modulus value of **E = 3.69E6 psi** by using the standard ACI formula for determining the concrete modulus of **E = 57000*√(fc)**, where fc is the concrete compressive strength in psi (ACI, 1989). Reinforcing steel accounted for approximately 0.4% of the shaft cross-sectional area, yielding a combined shaft moduli of **E = 3.8E6 psi**.

4.2 TOTAL MOVEMENTS: The O-cell was installed in a fully closed condition. The load was incrementally applied until a maximum cell load of 1162 tons was applied to the shaft resulting in a vertical upward movement of 0.069 inches at the top of the shaft as measured by dial gage No. 1. The average downward movement of the base of the O-cell at the maximum applied load was 1.492 inches as measured by dial gage Nos. 2 & 3. After the maximum test load was reached, the load was then incrementally released to zero. Due to Poisson's Ratio effects, a maximum correction of 0.236 inches was **added** to the field end bearing movement to give a corrected maximum downward movement of 1.727 inches.

In calculating the upward load applied, the buoyant weight of the shaft above the O-cell (4 tons) was subtracted from the load at the O-cell. A tabulation of all field data, including Poisson's Ratio corrections, is included as Appendix C.

4.3 LOAD MOVEMENT CURVES: The Engineer obtains various load-movement curves from an Osterberg Cell load test. The load-movement curve for the bottom of the device moving downward and the top of the shaft moving upward are obtained directly. One can also reconstruct an equivalent load settlement curve simulating the load being applied at the top of the shaft, as in a conventional load test. Also, one can construct creep movement curves to make an estimate of the creep load limit. Each of these is discussed below:

4.4 CORRECTED LOAD MOVEMENT CURVES: Figure No. 1.4 shows the downward movement of the base of the device (max 1.727 inches) and upward movement of the top of the shaft (max 0.069 inches) during the test. As explained previously in section **4.2**, the measured downward movement data was corrected for Poisson's ratio effects on the center-pipe telltale, and the loads for the upward movements are the net loads after subtracting the buoyant weight of the shaft (max 1162 tons, net 1158 tons).

4.5 COMPUTED TOP LOADING CURVE: Figure No. 1.5 presents our computed, equivalent load-settlement curve for the top of the shaft. This has been computed using the following assumptions: (Schmertmann, 1993)

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.

2. The shear-movement curve for the upward movement in an Osterberg cell test is the same as from the downward movement in a conventional test. We believe that this is generally a conservative assumption.

3. The compression of the shaft is considered negligible and the shaft is assumed rigid.

Using these assumptions allows the engineer to select a given top 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 settlement in a conventional test. This can be done up to the maximum movement in the Osterberg cell test. The heavy solid line in Figure No 1.5 shows this reconstruction over the range where the measured movement data was available from both components. The dashed line shows this movement continuing, based on the measured bottom-of-cell movements plus a graphically estimated extrapolation of the top-of-shaft movements. For this load test, the top of shaft movements were too small for our usual least squares, hyperbolic curve fit and extrapolation of the top-of-shaft movements.

The estimated equivalent top load exceeds 3000 tons for a settlement of 0.5 inches, based on the analysis illustrated in Figure 1.5.

4.6 CREEP LIMITS: The surface dial gages were set up and read so that creep was calculated separately for the friction and bearing components. Figure Nos. 1.6 & 1.7 show the amount of movement measured over the 1 to 4 minute load interval vs. the load applied, for top-of-shaft and bottom-of-cell, 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 3 minute time interval, denotes a creep limit that indicates a load below which creep movement of the shaft is unlikely to occur. No creep limits were observed within the test data.

4.7 STRAIN GAGE RESULTS: The strain gages indicated the point strain at the level of monitoring. Differences from the zero load reading were taken and corrected to the true strain by the formula:

$$\text{Strain} = (R1-R0)+12.2(T1-T0)$$

where R0 = Initial Zero Load Strain Reading
R1 = Strain Reading at Load Increment
T0 = Temperature at Zero Load (degrees C)
T1 = Temperature at Load Increment

Strain gage No. 4, located 1 foot above the top of the Osterberg Cell failed during testing after load increment 1L3, probably due to the installation and test induced strains encountered exceeding the unit's operating range. As shown in Figure 1.8, over 1000 tons shear load, as derived from the strain gage data was carried by the lower 3 ft. of the rock socket.

5. CONCLUSIONS:

5.1 The load test shaft was successfully tested to an equivalent surface load of 2320 tons, greatly exceeding the required 400 ton load test capacity.

5.2 Based on the loads and movements measured in the tests, and our extrapolations of the load test friction data, we conclude that an equivalent conventional load test would have given a total settlement of 0.5 inches at a load of over 3000 tons.

5.3 The telltale and strain gage data indicates that over 1000 tons shear load applied by the Osterberg Cell was carried by the lower 3 ft. of the socket.

5.4 The bottom-of-cell downward movement appears to be consolidation of material below the O-cell and does not appear to be true end bearing deflection of the shale formation.

5.5 No definable creep limit was reached during this load test.

END OF REPORT
Respectfully submitted by:

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John A. Hayes, P.Eng., DIC
for **LOADTEST, Inc.**

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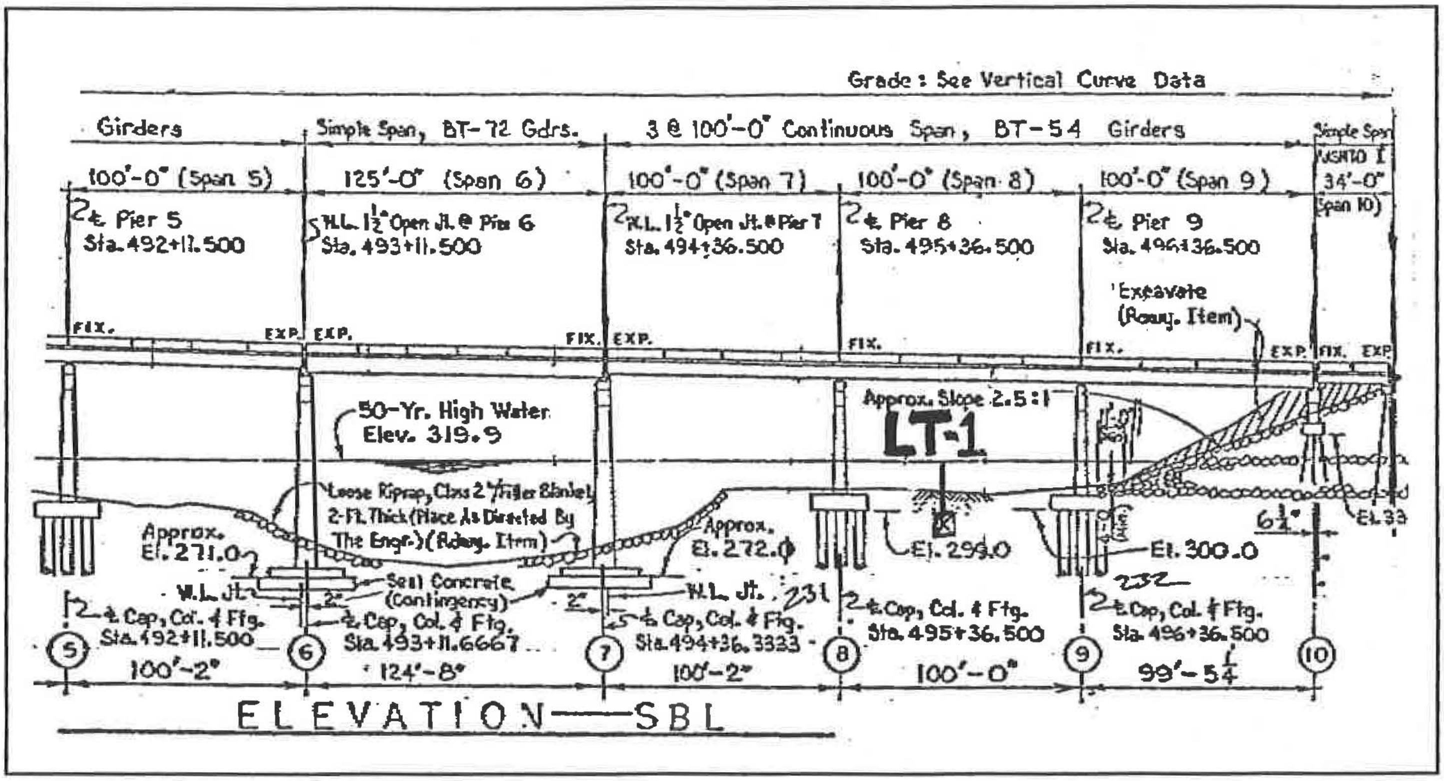
FIGURES

FIGURE 1.1

LOCATION PLAN: LT-1
(NOT TO SCALE)

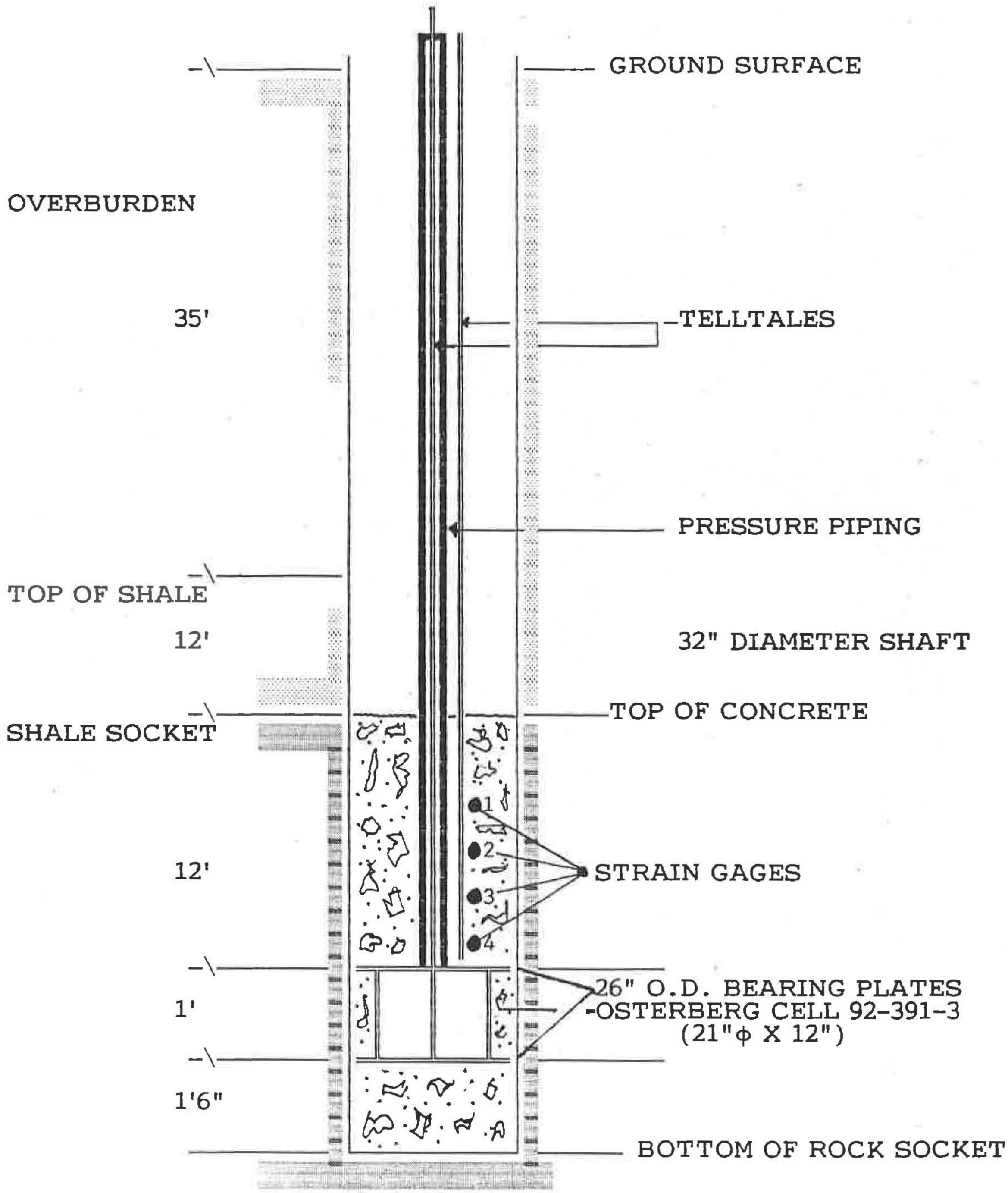


I-65 BRIDGE REPLACEMENT
BLACK WARRIOR RIVER
BLOUNT COUNTY, ALABAMA
LOADTEST PROJECT No. LT-177

FIGURE 1.2

TEST SHAFT INSTALLATION SCHEMATIC: LT-1

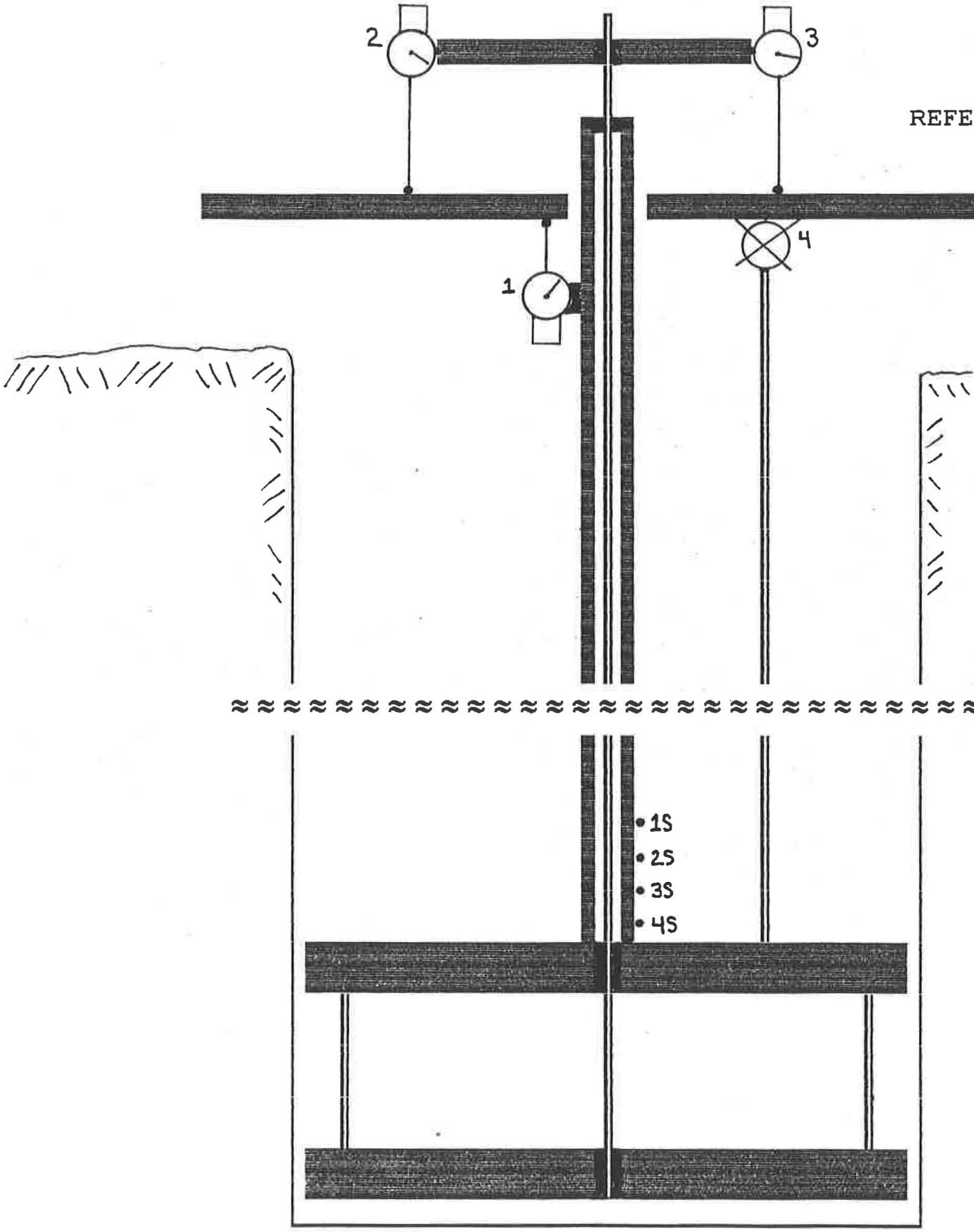
(NOT TO SCALE)



I-65 BRIDGE REPLACEMENT
BLACK WARRIOR RIVER
BLOUNT COUNTY, ALABAMA
LOADTEST PROJECT No. LT-177

Figure 1.3

REFERENCE INSTRUMENTATION SCHEMATIC: LT-1
(NOT TO SCALE)



REFERENCE BEAMS

1.) 0.001" Division dial gage measuring upward movement of the top-of-shaft.

2 & 3.) 0.001" Division dial gages measuring downward movement of the bottom-of-cell.

4.) 0.0001" Division dial gage for telltale measuring the shaft compression from the top-of-cell to the top-of-shaft. (Not Used)

1S - 4S) Vibrating wire strain gage instrumentation

I-65 BRIDGE REPLACEMENT
BLOUNT COUNTY, ALABAMA
LOADTEST PROJECT No. LT-177

LOADTEST INC

DEEP FOUNDATION TESTING, EQUIPMENT & SERVICES • SPECIALIZING IN OSTERBERG CELL TECHNOLOGY

OSTERBERG CELL LOAD-MOVEMENT CURVE

I-65 BRIDGE: BLOUNT COUNTY, ALABAMA

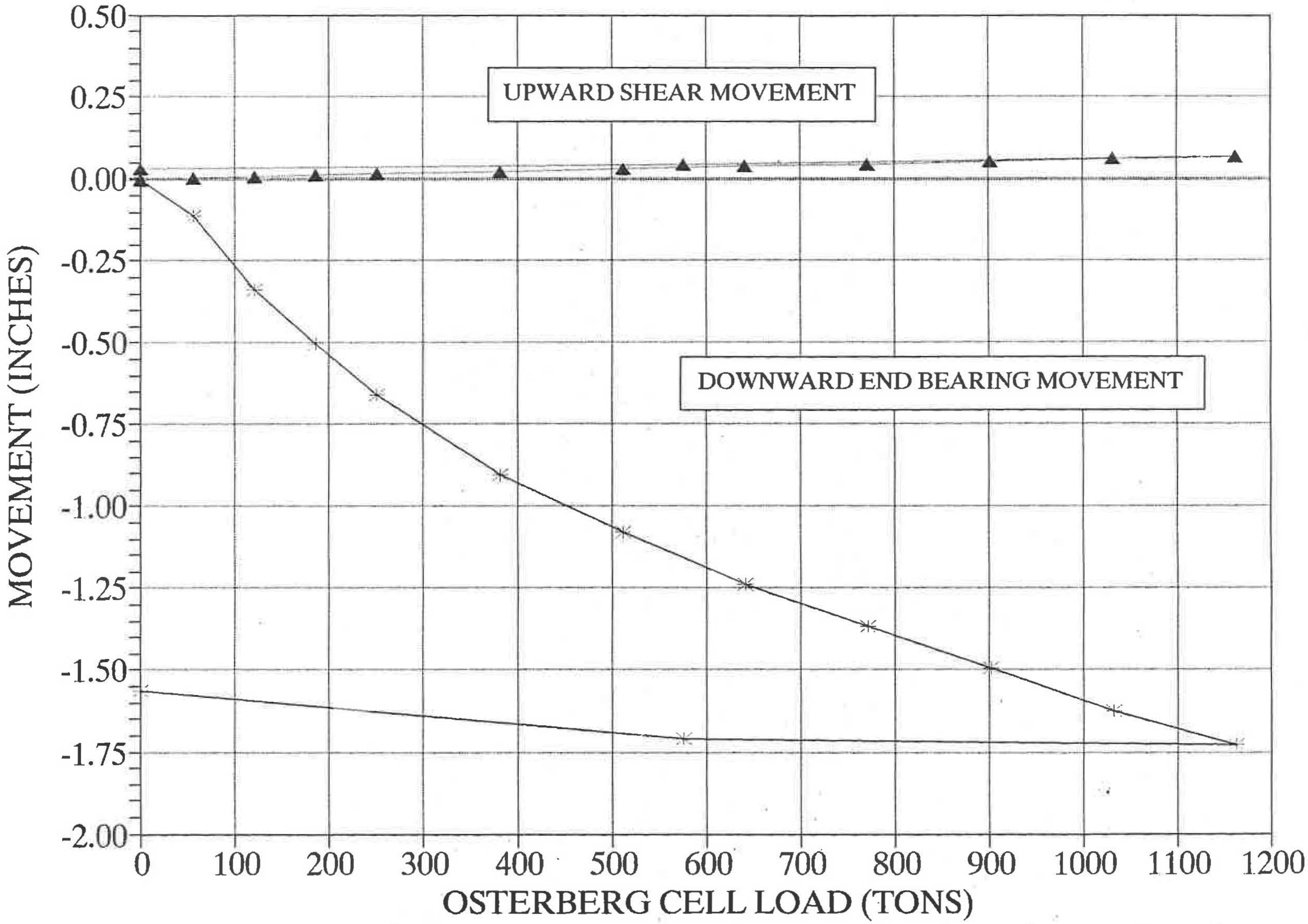


FIGURE 1.4

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EQUIVALENT TOP LOAD-SETTLEMENT CURVE
OBTAINED FROM OSTERBERG CELL LOAD TEST

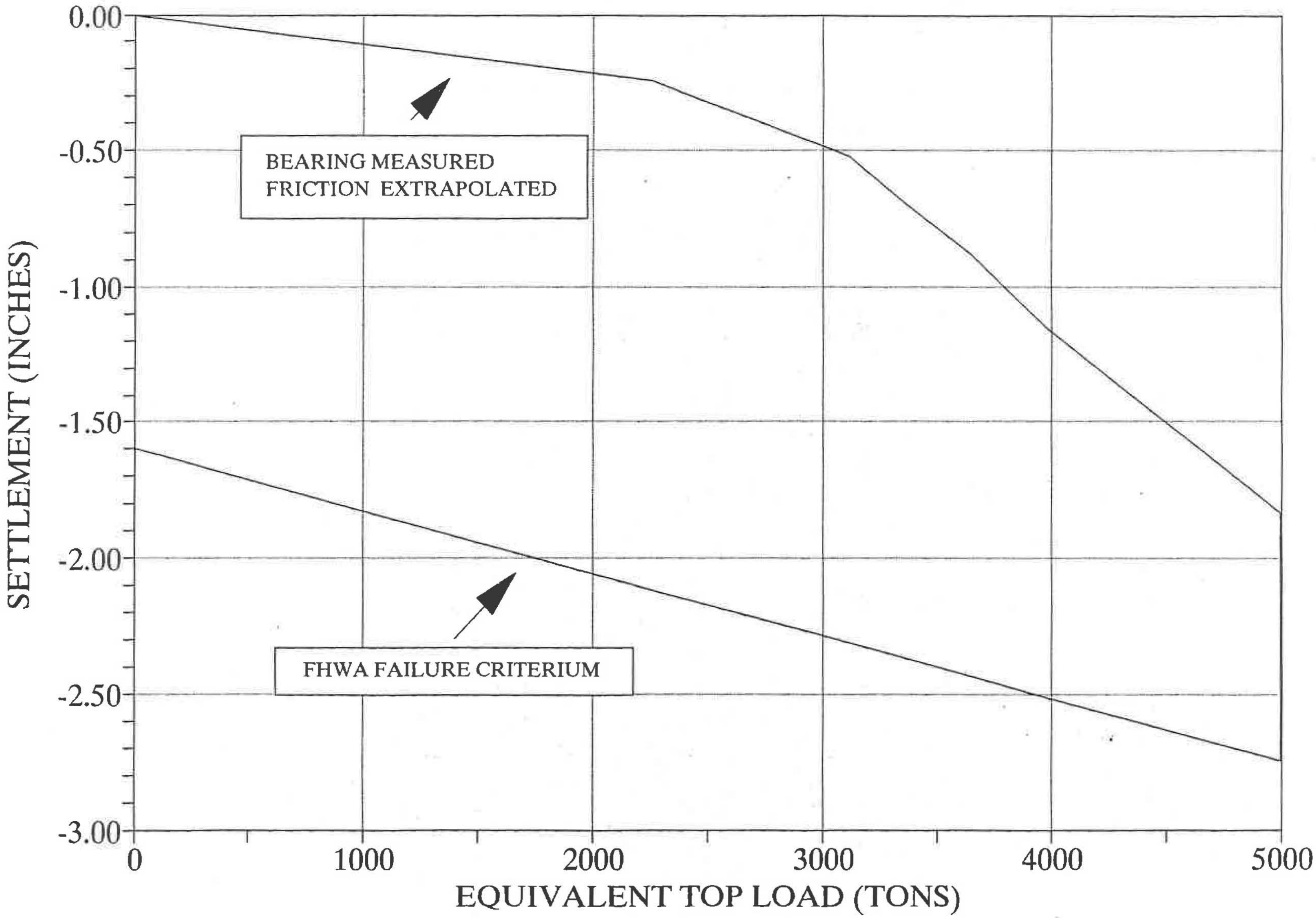

FIGURE 1.5

APPENDIX A

CALIBRATION OF THE OSTERBERG CELL

CALIBRATION OF OSTERBERG CELL NUMBER 92-391-3 DATE OF CALIBRATION 09/11/93					
1" STROKE		3" STROKE		5" STROKE	
Pressure (psi)	Load (tons)	Pressure (psi)	Load (tons)	Pressure (psi)	Load (tons)
0	0	0	0	0	0
500	62	500	59	500	58
1000	125	1000	122	1000	121
1500	189	1500	185	1500	185
2000	256	2000	252	2000	250
2500	321	2500	316	2500	315
3000	385	3000	382	3000	379
3500	451	3500	446	3500	444
4000	516	4000	512	4000	509
4500	582	4500	578	4500	574
4800	621	4800	616	4800	613

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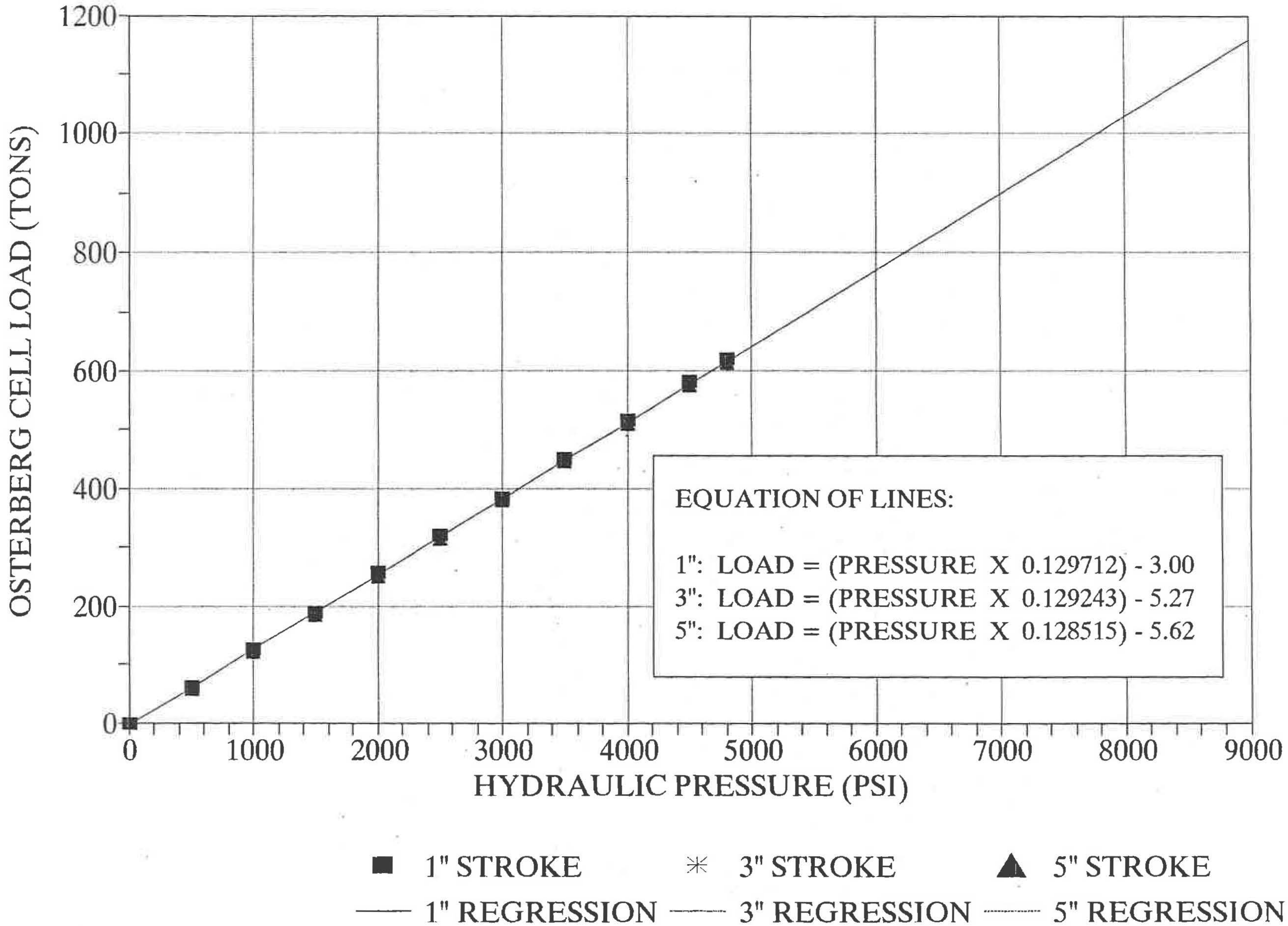
OSTERBERG CELL CALIBRATION

O-CELL NUMBER 92-391-3



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TOP-OF-SHAFT CREEP LIMIT CURVE

I-65 BRIDGE: BLOUNT COUNTY, ALABAMA

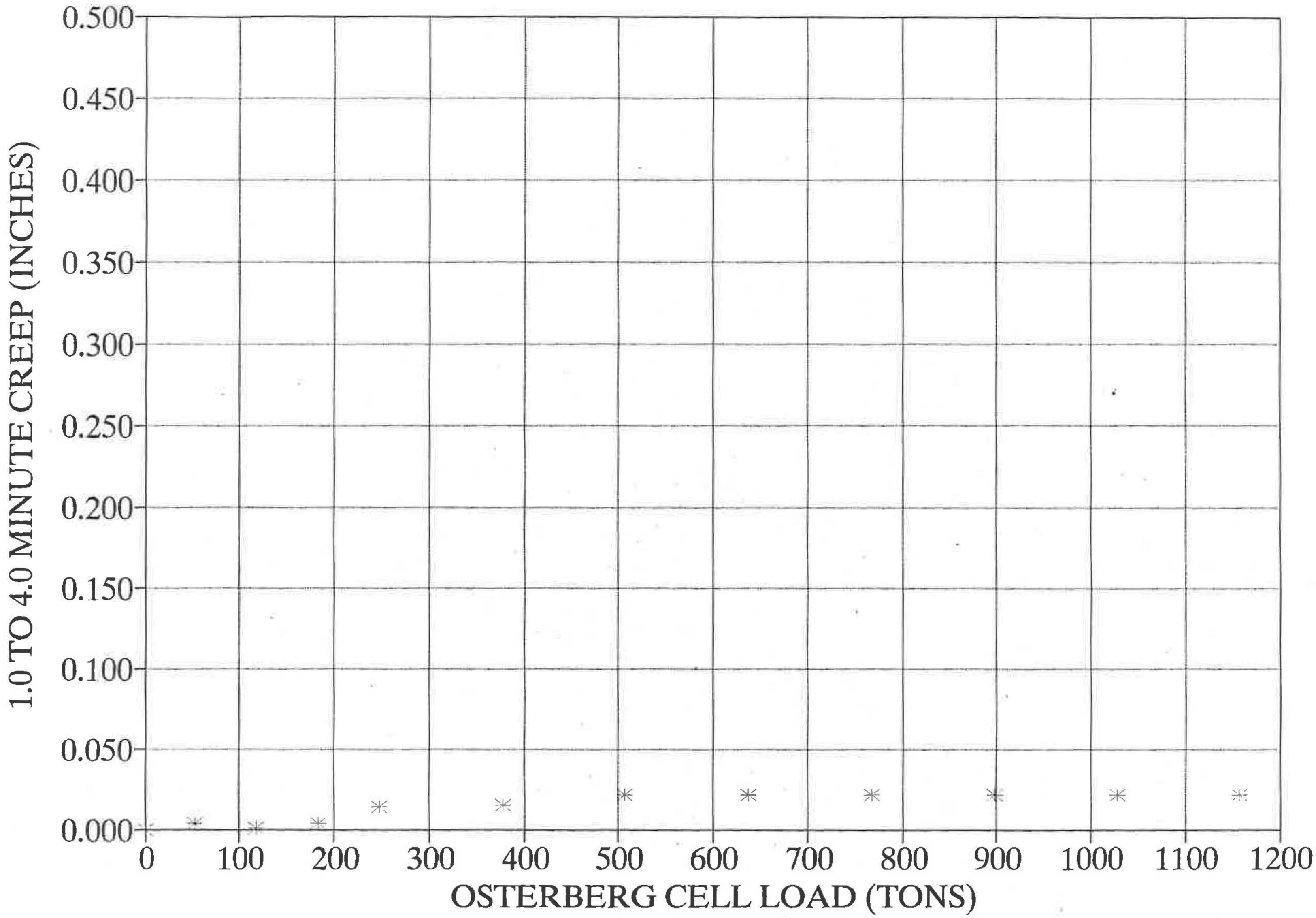

FIGURE 1.6

BOTTOM-OF-CELL CREEP LIMIT CURVE
I-65 BRIDGE: BLOUNT COUNTY, ALABAMA

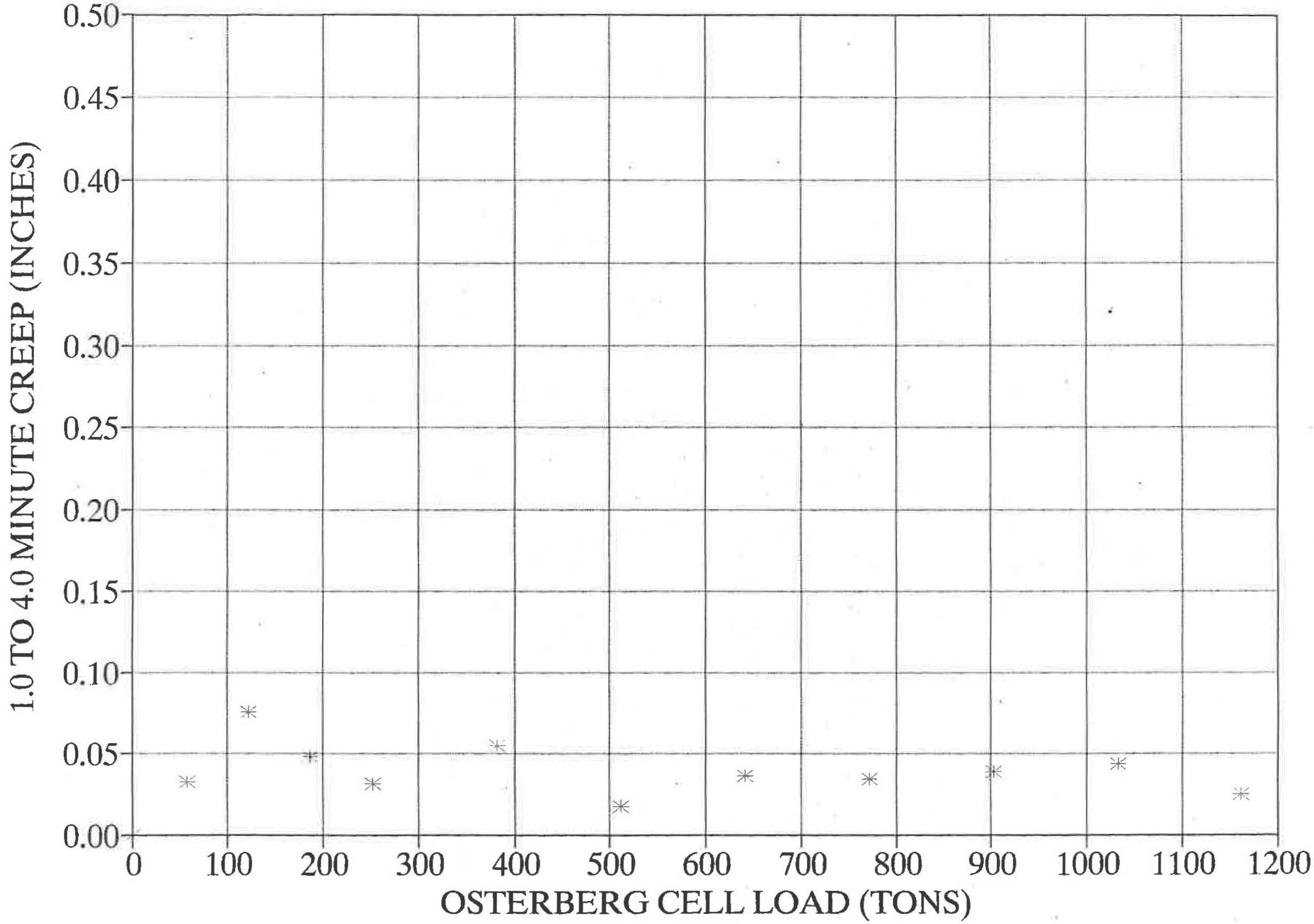

FIGURE 1.7

SHEAR LOAD TRANSFER FROM STRAIN GAGE DATA

I-65 BRIDGE: BLOUNT COUNTY, ALABAMA

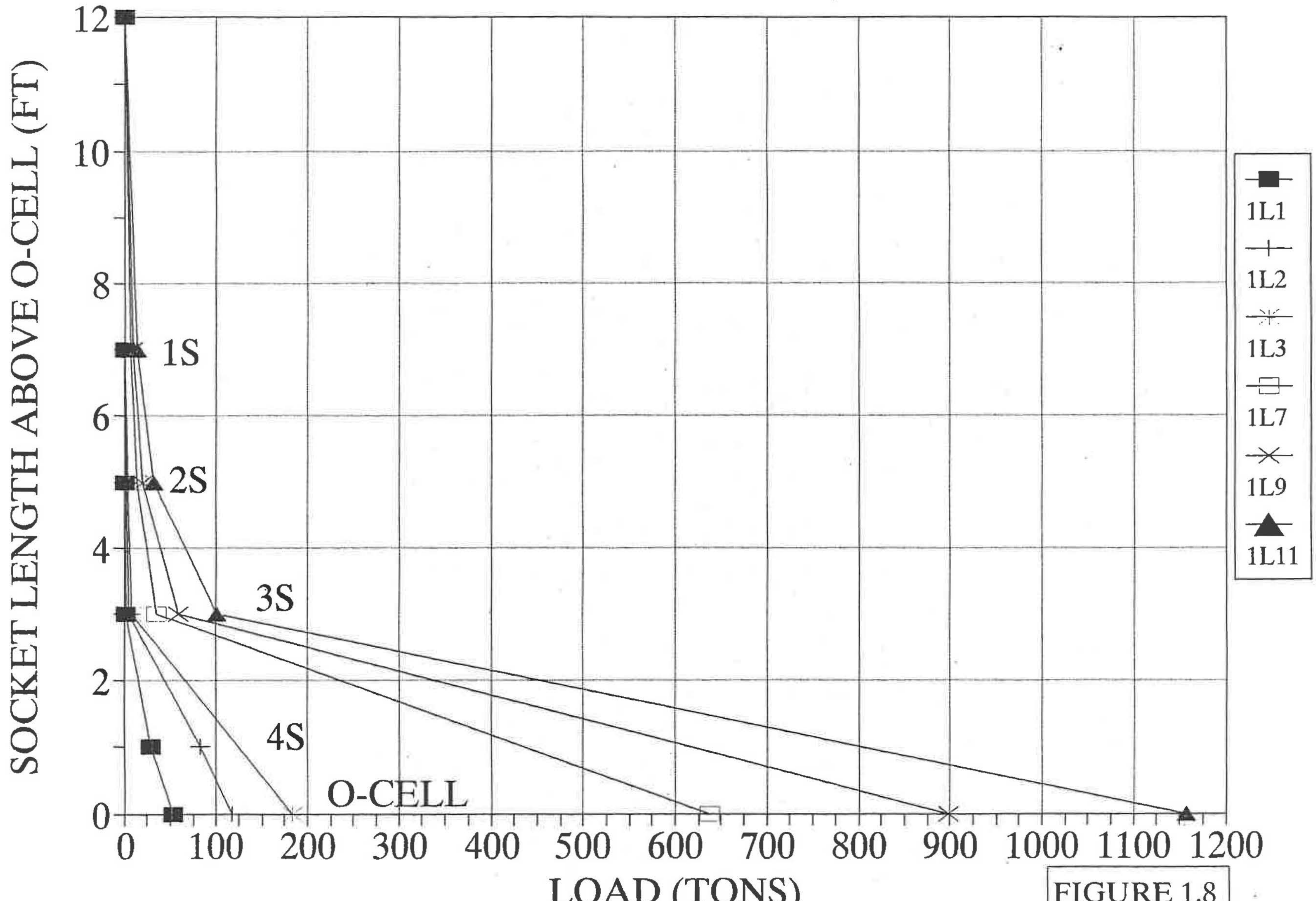

FIGURE 1.8

APPENDICES

APPENDIX B

DETAILED TABULATIONS OF LOAD TEST DATA

I-65 BRIDGE OVER THE WARRIOR RIVER, WARRIOR, ALABAMA
OSTERBERG CELL LOAD TEST, DECEMBER 6, 1993
WEIGHT OF SHAFT ABOVE O-CELL 4 TONS

LOAD INTERVAL	GAGE PRESSURE (PSI)	O-CELL LOAD (TONS)	CORR. UPWARD LOAD (TONS)	TIME	TOP-OF-SHAFT GAGE 1 MOVEMENT (UP) (INCHES) 1.0 MINUTES	BOTTOM-OF-CELL GAGE 2 MOVEMENT (DOWN) (INCHES) 1.0 MINUTES	BOTTOM-OF-CELL GAGE 3 MOVEMENT (DOWN) (INCHES) 1.0 MINUTES	AVERAGE MOVEMENT (DOWN) GAGES 2 & 3 (INCHES) 1.0 MINUTES	POISSON'S RATIO CORRECTION (ADD) (INCHES) 1.0 MINUTES	CORRECTED AVG BOTTOM MOVEMENT (DOWN) (INCHES) 1.0 MINUTES
L0	0	0	0	10:40	0.000	0.000	0.000	0.000	0.000	0.000
1L1	500	57	53	10:41	0.003	0.069	0.059	0.064	0.013	-0.077
1L2	1000	122	118	10:46	0.008	0.240	0.230	0.235	0.026	-0.261
1L3	1500	187	183	10:51	0.012	0.435	0.400	0.418	0.039	-0.457
1L4	2000	252	248	10:56	0.016	0.595	0.550	0.573	0.052	-0.625
1L5	3000	382	378	11:01	0.022	0.797	0.742	0.770	0.079	-0.848
1L6	4000	512	508	11:08	0.030	1.004	0.913	0.959	0.105	-1.063
1L7	5000	642	638	11:14	0.037	1.131	1.015	1.073	0.131	-1.204
1L8	6000	772	768	11:20	0.044	1.243	1.110	1.177	0.157	-1.334
1L9	7000	902	898	11:25	0.052	1.363	1.180	1.272	0.183	-1.455
1L10	8000	1032	1028	11:30	0.060	1.483	1.260	1.372	0.209	-1.581
1L11	9000	1162	1158	11:37	0.067	1.595	1.340	1.468	0.236	-1.703
1U1	4500	577	573	11:42	0.047	1.650	1.535	1.593	0.118	-1.710
1U2	0	0	0	11:47	0.033	1.575	1.557	1.566	0.000	-1.566

1

LOADTEST

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I-65 BRIDGE OVER THE WARRIOR RIVER, WARRIOR, ALABAMA
OSTERBERG CELL LOAD TEST, DECEMBER 6, 1993
WEIGHT OF SHAFT ABOVE O-CELL

LOAD INTERVAL	GAGE PRESSURE (PSI)	O-CELL LOAD (TONS)	CORR. UPWARD LOAD (TONS)	TIME	TOP-OF-SHAFT GAGE 1 MOVEMENT (UP) (INCHES) 4 MINUTES	BOTTOM-OF-CELL GAGE 2 MOVEMENT (DOWN) (INCHES) 4 MINUTES	BOTTOM-OF-CELL GAGE 3 MOVEMENT (DOWN) (INCHES) 4 MINUTES	AVERAGE MOVEMENT (DOWN) GAGES 2 & 3 (INCHES) 4 MINUTES	POISSON'S RATIO CORRECTION (ADD) (INCHES) 4 MINUTES	CORRECTED AVG BOTTOM MOVEMENT (INCHES) 4 MINUTES
L0	0	0	0	10:40	0.000	0.000	0.000	0.000	0.000	0.000
1L1	500	57	53	10:41	0.003	0.101	0.090	0.096	0.013	-0.109
1L2	1000	122	118	10:46	0.008	0.323	0.297	0.310	0.026	-0.336
1L3	1500	187	183	10:51	0.012	0.483	0.447	0.465	0.039	-0.504
1L4	2000	252	248	10:56	0.016	0.628	0.579	0.604	0.052	-0.656
1L5	3000	382	378	11:01	0.023	0.858	0.789	0.824	0.079	-0.902
1L6	4000	512	508	11:08	0.031	1.021	0.929	0.975	0.105	-1.080
1L7	5000	642	638	11:14	0.040	1.169	1.047	1.108	0.131	-1.239
1L8	6000	772	768	11:20	0.046	1.287	1.133	1.210	0.157	-1.367
1L9	7000	902	898	11:25	0.053	1.405	1.215	1.310	0.183	-1.493
1L10	8000	1032	1028	11:30	0.062	1.527	1.302	1.415	0.209	-1.624
1L11	9000	1162	1158	11:37	0.069	1.621	1.362	1.492	0.236	-1.727
1U1	4500	577	573	11:42	0.045	1.650	1.532	1.591	0.118	-1.709
1U2	0	0	0	11:47	0.033	1.575	1.557	1.566	0.000	-1.566

2

I-65 BRIDGE OVER THE WARRIOR RIVER, WARRIOR, ALABAMA
OSTERBERG CELL LOAD TEST, DECEMBER 6, 1993
WEIGHT OF SHAFT ABOVE O-CELL - 4 TONS

LOAD INTERVAL	GAGE PRESSURE (PSI)	O-CELL LOAD (TONS)	CORR. UPWARD LOAD (TONS)	TIME	TOP-OF-SHAFT 1 TO 4 MIN. CREEP (INCHES)	BOTTOM-OF-CELL 1 TO 4 MIN. CREEP (INCHES)
L0	0	0	0	10:40	0.000	0.000
1L1	500	57	53	10:41	0.000	0.032
1L2	1000	122	118	10:46	0.000	0.075
1L3	1500	187	183	10:51	0.000	0.047
1L4	2000	252	248	10:56	0.000	0.031
1L5	3000	382	378	11:01	0.001	0.054
1L6	4000	512	508	11:08	0.001	0.016
1L7	5000	642	638	11:14	0.003	0.035
1L8	6000	772	768	11:20	0.002	0.033
1L9	7000	902	898	11:25	0.001	0.038
1L10	8000	1032	1028	11:30	0.002	0.043
1L11	9000	1162	1158	11:37	0.002	0.024
1U1	4500	577	573	11:42		
1U2	0	0	0	11:47		

Appendix C

Lateral Load Test Data

Appendix C - Lateral Load Test

Lateral load testing was conducted on two 30 inch diameter shafts drilled into the shale underlying the site. The tests were located near Bent 2 where the shale was near the ground surface. A shelf was cut into the embankment to remove the overburden soils from the test site. Shaft 2 East was excavated to a depth of 18 feet while Shaft 2 West was excavated to a depth of 15 feet. The slope of the original ground surface and the dip of the bedding planes resulted in the shale at 2 East being partially weathered and the shale at 2 West being sound and unweathered.

Both shafts were drilled and concreted on December 1, 1993. A reinforcing cage of 12 #10 deformed bars was included in each shaft. An inclinometer tube was tied to each cage prior to installation with the grooves oriented in the direction of loading. The shafts were pulled together using a calibrated hydraulic jack, steel plates, and dywidag bars. The jack was placed against shaft 2 West with the center of loading at a distance of 1.09 feet above grade. Dial gages measured deflection of the shaft at load increments of 25 kips. The gage on shaft 2 East was located 1.54 feet above grade and the gage on shaft 2 West was located 1.61 feet above grade. Inclinometer readings were taken at 80 kips, 120 kips, 150 kips, and intervals of 50 kips thereafter. Figures 1 and 2 show the load-deflection curves for shafts 2 East and 2 West, respectively. Figures 3 and 4 give the depth-deflection relationships for 2 East and 2 West, respectively. Loading was begun on December 9, 1993. Upon reaching 120 kips, the jack malfunctioned and required replacing. A new jack was acquired and the test completed on December 13, 1993.

In conjunction with the lateral testing, Texam pressuremeter tests were conducted in a borehole near shaft 2 East. Tests were conducted at depths of 11, 14, and 17 feet below the ground surface. Table 1 gives the pressuremeter moduli determined by these tests. Due to the dipping of the shale beds, these values are believed to be representative of the partially weathered shale in which shaft 2 East was constructed. The modulus is expected to increase with depth, but the variable nature of the bedded shale could result in weak layers interbedded with stronger layers. This could explain the lower modulus measured at 17 feet.

Table 1 - Pressuremeter Modulus

Depth (ft)	Modulus (psi)
11	10,180
14	13,280
17	5890

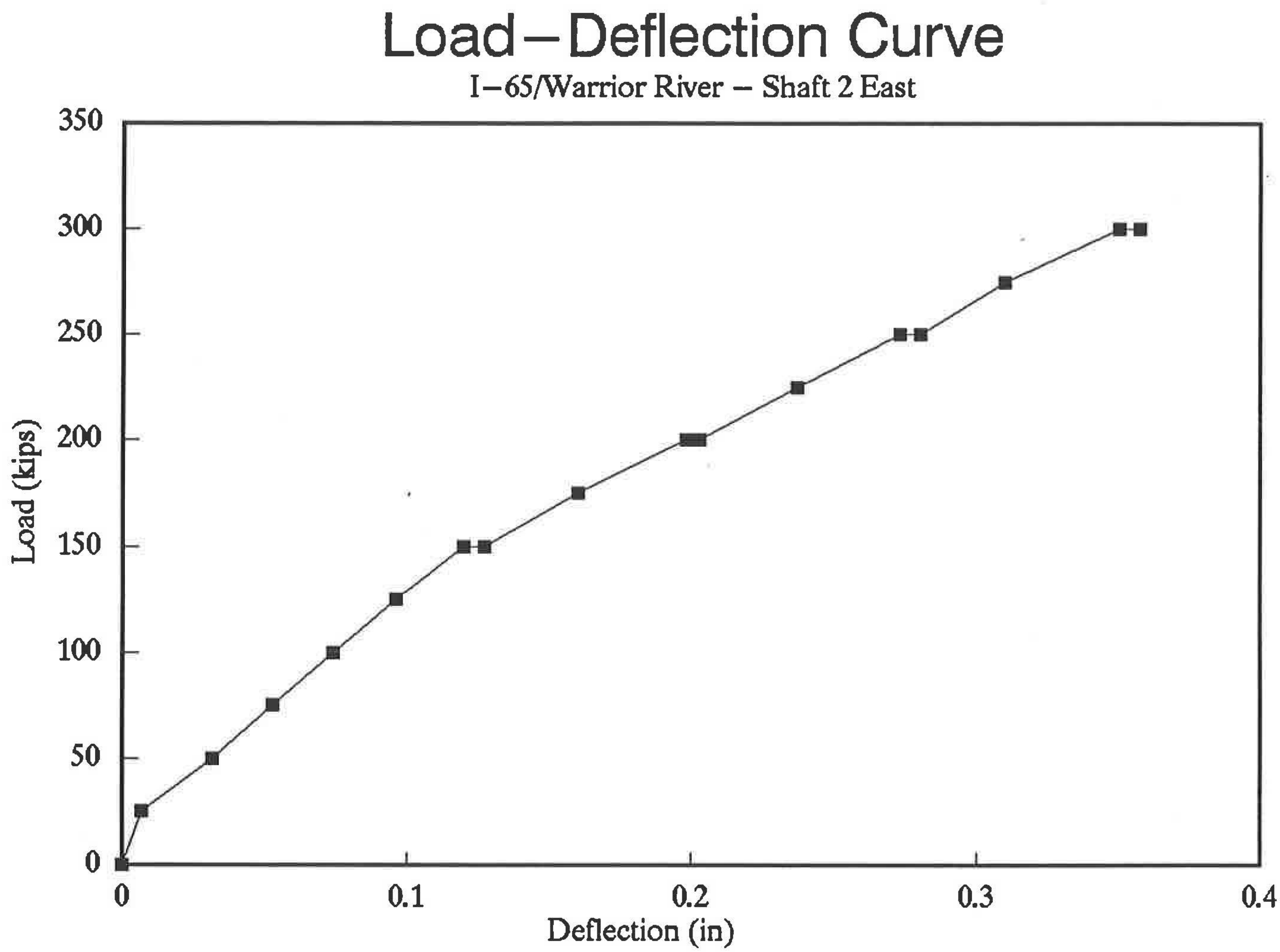

Figure 1 - Load Deflection Curve (Shaft 2 East)

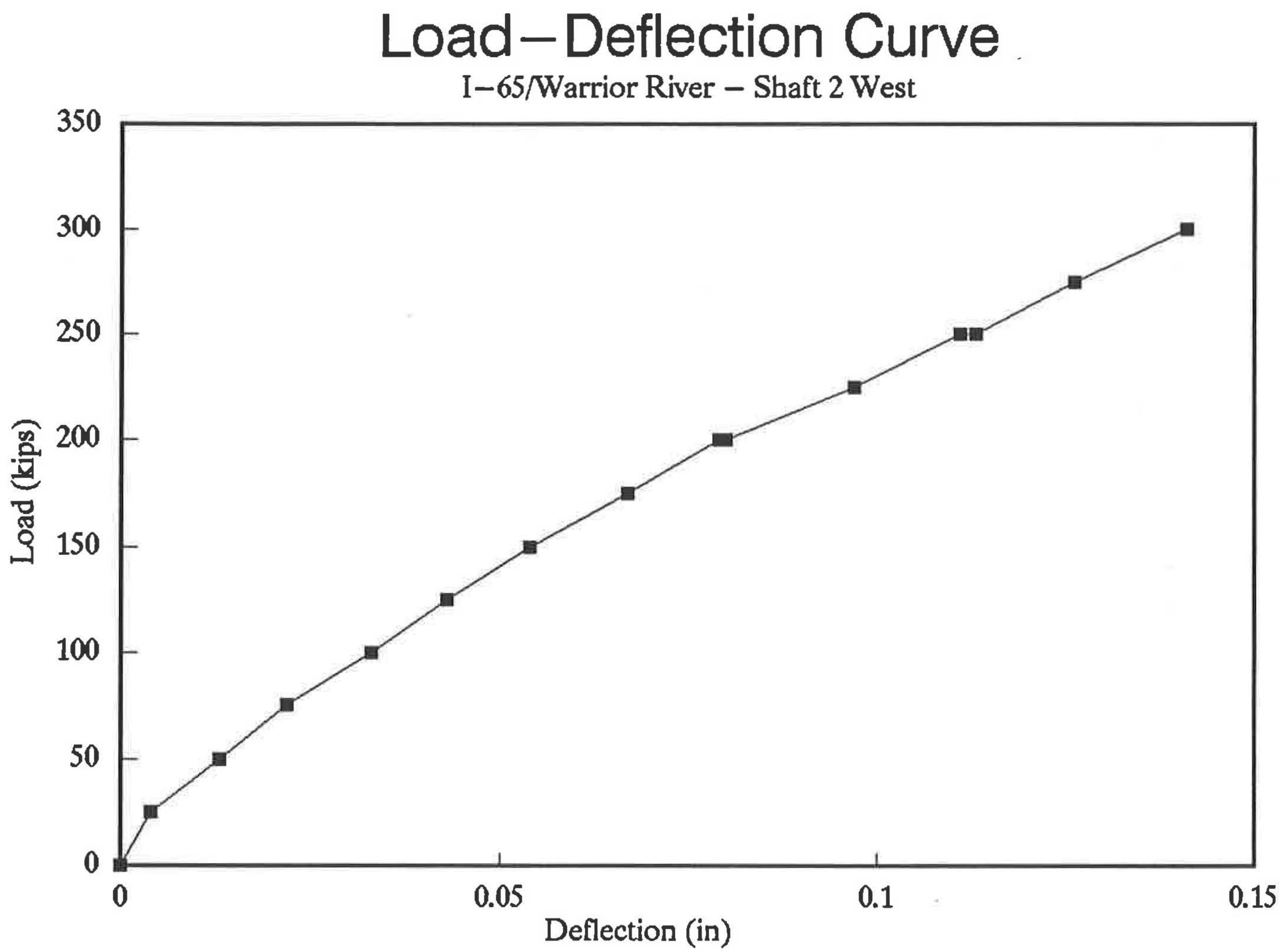

Figure 2 - Load Deflection Curve (Shaft 2 West)

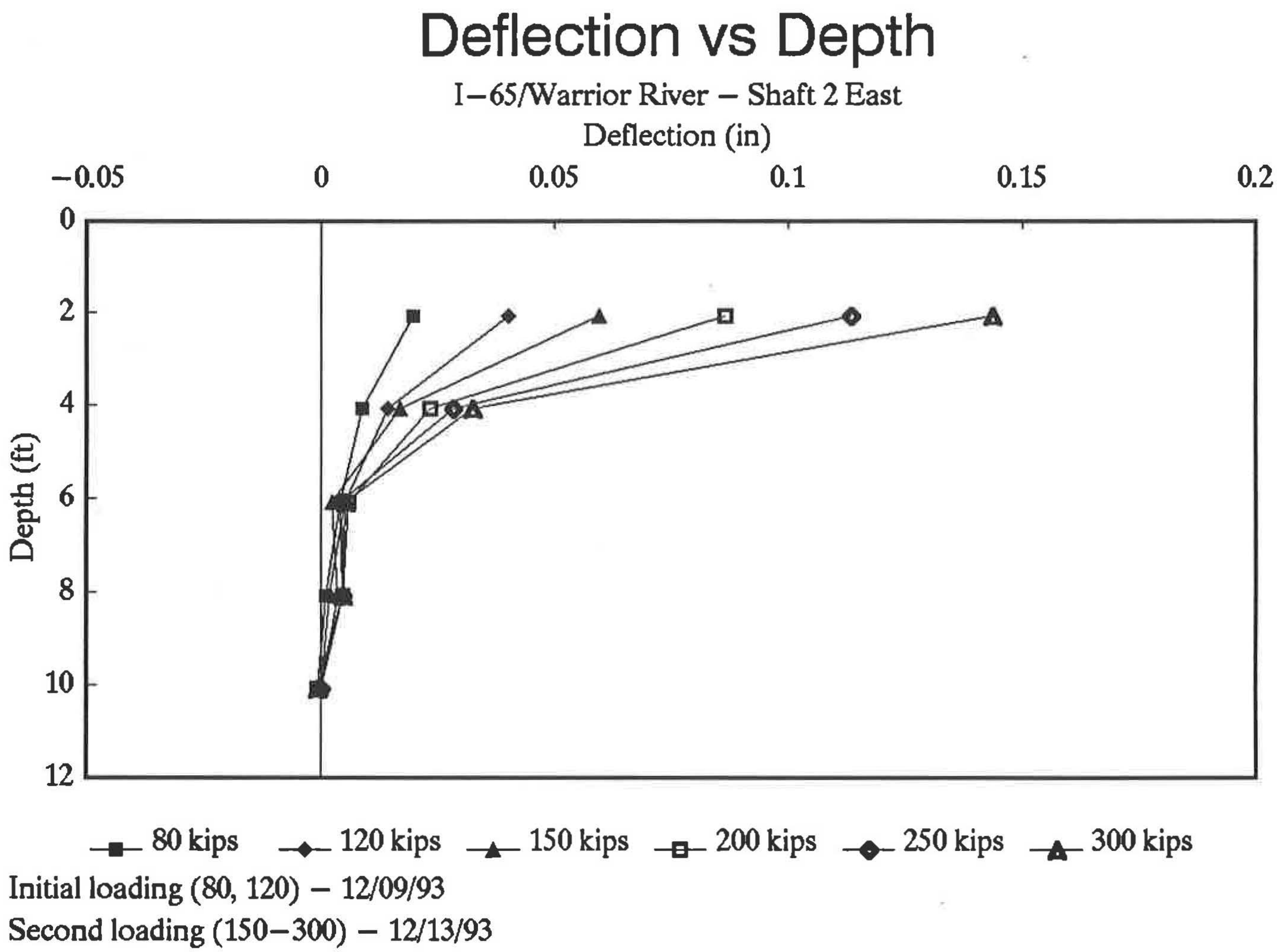

Figure 3 - Depth vs Deflection (Shaft 2 East)

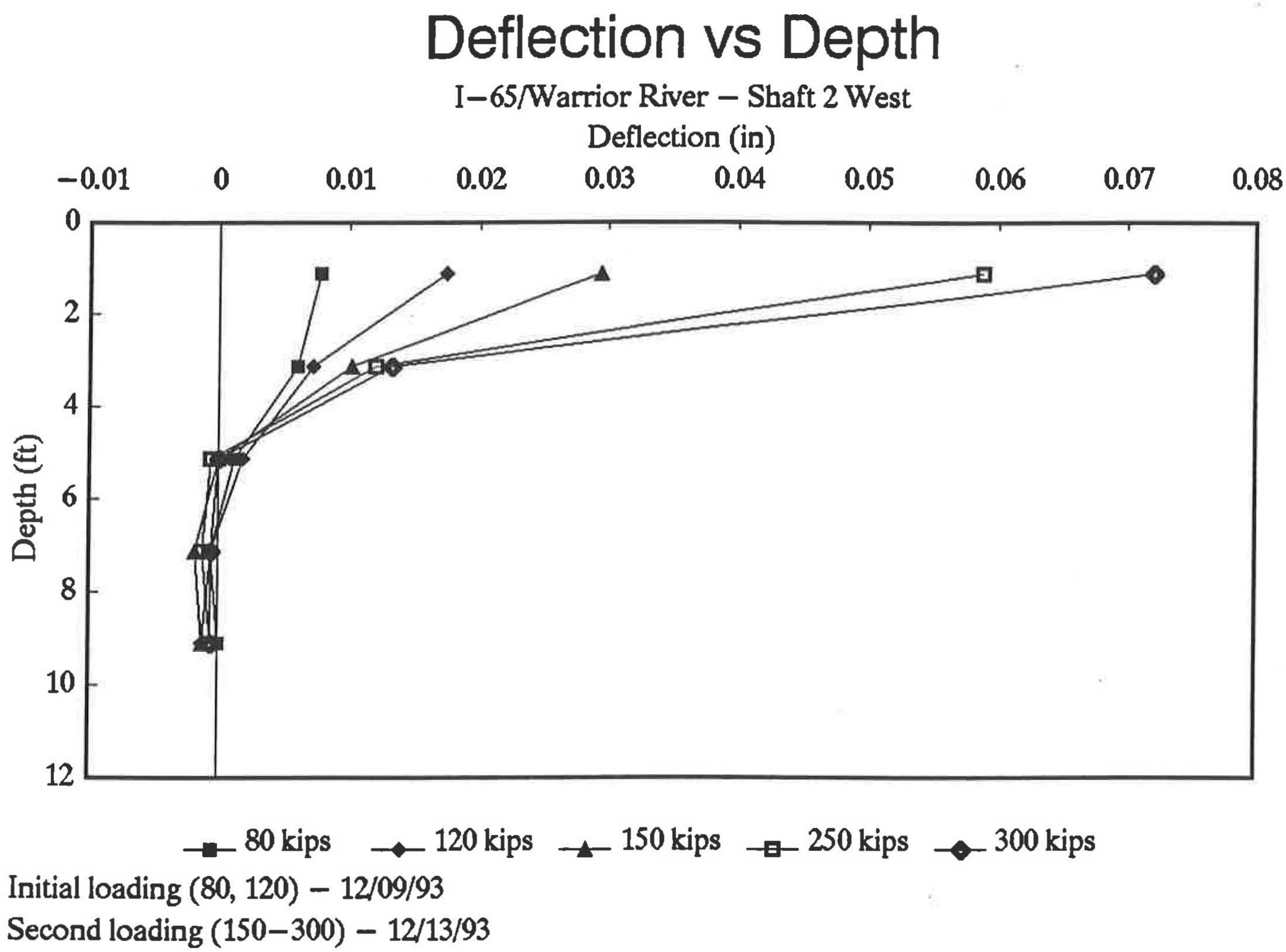

Figure 4 - Depth vs Deflection (Shaft 2 West)

Appendix D

Lateral Load Analyses

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I-65 over Mulberry Fork, Ld Test, 32" x 18', 1-7-94, soft clay criteria


UNITS--ENGLISH UNITS



                   I N P U T   I N F O R M A T I O N
                   *********************************


THE LOADING IS STATIC
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PILE GEOMETRY AND PROPERTIES
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    PILE LENGTH                          =      228.00 IN
     2 POINTS

        X          DIAMETER     MOMENT OF        AREA       MODULUS OF
                                 INERTIA                    ELASTICITY
        IN            IN          IN**4          IN**2       LBS/IN**2
        .00         32.000      .130D+05       .804D+03      .400D+07
    228.00          32.000      .130D+05       .804D+03      .400D+07


SOILS INFORMATION
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    X AT THE GROUND SURFACE              =       12.00 IN

    1 LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER            =       12.00 IN
    X AT THE BOTTOM OF THE LAYER         =      600.00 IN
    MODULUS OF SUBGRADE REACTION         =    .700D+03 LBS/IN**3


    DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                       4 POINTS
                X,IN      WEIGHT,LBS/IN**3
                 .00         .78D-01
              144.00         .78D-01
              145.00         .78D-01
              600.00         .78D-01

    DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                       4 POINTS
                X,IN       C,LBS/IN**2      PHI,DEGREES      E50
                 .00        .100D+03           .000        .900D-02
               48.00        .400D+03           .000        .700D-02
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72.00	.500D+03	.000	.700D-02
600.00	.500D+03	.000	.700D-02

BOUNDARY AND LOADING CONDITIONS

LOADING NUMBER 1

BOUNDARY-CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .100D+06 LBS
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+00 LBS

LOADING NUMBER 2

BOUNDARY-CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .200D+06 LBS
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+00 LBS

LOADING NUMBER 3

BOUNDARY-CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .300D+06 LBS
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+00 LBS

FINITE-DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 19
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE = .100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .30D+03 IN

OUTPUT CODES
KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .100D+06 LBS
MOMENT AT THE PILE HEAD = .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+00 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
.00	.593D-01	.000D+00	.100D+06	.000D+00	.622D-03	.520D+11
12.00	.382D-01	.120D+07	.807D+05	-.322D+04	.148D+04	.520D+11
24.00	.205D-01	.194D+07	.370D+05	-.406D+04	.238D+04	.520D+11
36.00	.813D-02	.209D+07	-.125D+05	-.419D+04	.257D+04	.520D+11
48.00	.154D-02	.164D+07	-.569D+05	-.321D+04	.201D+04	.520D+11
60.00	-.506D-03	.724D+06	-.627D+05	.224D+04	.891D+03	.520D+11
72.00	-.552D-03	.133D+06	-.321D+05	.285D+04	.164D+03	.520D+11
84.00	-.229D-03	-.476D+05	-.760D+04	.124D+04	.586D+02	.520D+11
96.00	-.387D-04	-.495D+05	.116D+04	.219D+03	.609D+02	.520D+11
108.00	.150D-04	-.198D+05	.194D+04	-.888D+02	.243D+02	.520D+11
120.00	.141D-04	-.287D+04	.889D+03	-.865D+02	.353D+01	.520D+11
132.00	.515D-05	.157D+04	.173D+03	-.329D+02	.194D+01	.520D+11
144.00	.578D-06	.128D+04	-.478D+02	-.384D+01	.157D+01	.520D+11
156.00	-.456D-06	.426D+03	-.520D+02	.315D+01	.524D+00	.520D+11

168.00	-.312D-06	.282D+02	-.197D+02	.223D+01	.354D-01	.520D+11
180.00	-.903D-07	-.478D+02	-.233D+01	.667D+00	.595D-01	.520D+11
192.00	-.675D-09	-.277D+02	.170D+01	.516D-02	.347D-01	.520D+11
204.00	.122D-07	-.690D+01	.116D+01	-.963D-01	.912D-02	.520D+11
216.00	.599D-08	.538D-01	.288D+00	-.487D-01	.688D-03	.520D+11
228.00	-.893D-10	.000D+00	.000D+00	.748D-03	.622D-03	.520D+11

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.744D-08 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.461D-09 LBS

OUTPUT SUMMARY

```
PILE-HEAD DEFLECTION          =  .593D-01 IN
COMPUTED SLOPE AT PILE HEAD   =  .175D-02
MAXIMUM BENDING MOMENT        =  .209D+07 LBS-IN
MAXIMUM SHEAR FORCE           =  .100D+06 LBS
NO. OF ITERATIONS             =        22
NO. OF ZERO DEFLECTION POINTS =         5
```

LOADING NUMBER 2

```
BOUNDARY CONDITION CODE              =        1
LATERAL LOAD AT THE PILE HEAD        =  .200D+06 LBS
MOMENT AT THE PILE HEAD              =  .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD          =  .500D+00 LBS
```

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.189D+00	-.100D-07	.200D+06	.000D+00	.622D-03	.520D+11
12.00	.132D+00	.240D+07	.171D+06	-.486D+04	.295D+04	.520D+11
24.00	.809D-01	.410D+07	.103D+06	-.641D+04	.505D+04	.520D+11
36.00	.414D-01	.488D+07	.214D+05	-.722D+04	.600D+04	.520D+11
48.00	.154D-01	.461D+07	-.633D+05	-.689D+04	.568D+04	.520D+11
60.00	.210D-02	.336D+07	-.130D+06	-.420D+04	.413D+04	.520D+11
72.00	-.187D-02	.150D+07	-.127D+06	.472D+04	.184D+04	.520D+11
84.00	-.169D-02	.315D+06	-.698D+05	.478D+04	.387D+03	.520D+11
96.00	-.638D-03	-.178D+06	-.194D+05	.361D+04	.219D+03	.520D+11
108.00	-.825D-04	-.151D+06	.517D+04	.487D+03	.186D+03	.520D+11
120.00	.545D-04	-.542D+05	.608D+04	-.335D+03	.667D+02	.520D+11
132.00	.413D-04	-.532D+04	.249D+04	-.264D+03	.655D+01	.520D+11
144.00	.133D-04	.556D+04	.376D+03	-.885D+02	.684D+01	.520D+11
156.00	.777D-06	.370D+04	-.187D+03	-.535D+01	.455D+01	.520D+11
168.00	-.152D-05	.107D+04	-.154D+03	.109D+02	.131D+01	.520D+11
180.00	-.862D-06	-.200D+01	-.508D+02	.637D+01	.308D-02	.520D+11
192.00	-.210D-06	-.153D+03	-.300D+01	.160D+01	.190D+00	.520D+11
204.00	.171D-07	-.740D+02	.581D+01	-.135D+00	.917D-01	.520D+11
216.00	.394D-07	-.139D+02	.308D+01	-.320D+00	.178D-01	.520D+11
228.00	.231D-07	.000D+00	.000D+00	-.194D+00	.622D-03	.520D+11

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .146D-07 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .168D-08 LBS

OUTPUT SUMMARY

```
PILE-HEAD DEFLECTION          =  .189D+00 IN
COMPUTED SLOPE AT PILE HEAD   =  .480D-02
MAXIMUM BENDING MOMENT        =  .488D+07 LBS-IN
MAXIMUM SHEAR FORCE           =  .200D+06 LBS
NO. OF ITERATIONS             =        27
NO. OF ZERO DEFLECTION POINTS =         4
```

LOADING NUMBER 3

```
BOUNDARY CONDITION CODE                    =         1
LATERAL LOAD AT THE PILE HEAD              =    .300D+06 LBS
MOMENT AT THE PILE HEAD                    =    .000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD                =    .500D+00 LBS
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X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.377D+00	-.401D-07	.300D+06	.000D+00	.622D-03	.520D+11
12.00	.273D+00	.360D+07	.263D+06	-.620D+04	.443D+04	.520D+11
24.00	.179D+00	.631D+07	.175D+06	-.836D+04	.776D+04	.520D+11
36.00	.103D+00	.781D+07	.667D+05	-.977D+04	.961D+04	.520D+11
48.00	.477D-01	.791D+07	-.522D+05	-.101D+05	.973D+04	.520D+11
60.00	.147D-01	.656D+07	-.161D+06	-.804D+04	.807D+04	.520D+11
72.00	-.220D-03	.405D+07	-.202D+06	.113D+04	.498D+04	.520D+11
84.00	-.388D-02	.170D+07	-.158D+06	.631D+04	.210D+04	.520D+11
96.00	-.283D-02	.267D+06	-.840D+05	.594D+04	.328D+03	.520D+11
108.00	-.104D-02	-.314D+06	-.217D+05	.444D+04	.386D+03	.520D+11
120.00	-.122D-03	-.255D+06	.943D+04	.748D+03	.313D+03	.520D+11
132.00	.939D-04	-.876D+05	.103D+05	-.601D+03	.108D+03	.520D+11
144.00	.668D-04	-.699D+04	.405D+04	-.444D+03	.861D+01	.520D+11
156.00	.204D-04	.963D+04	.540D+03	-.141D+03	.118D+02	.520D+11
168.00	.684D-06	.596D+04	-.334D+03	-.489D+01	.734D+01	.520D+11
180.00	-.254D-05	.160D+04	-.251D+03	.188D+02	.197D+01	.520D+11
192.00	-.134D-05	-.597D+02	-.769D+02	.102D+02	.741D-01	.520D+11
204.00	-.301D-06	-.247D+03	-.135D+01	.237D+01	.304D+00	.520D+11
216.00	.534D-07	-.922D+02	.103D+02	-.434D+00	.114D+00	.520D+11
228.00	.153D-06	.000D+00	.000D+00	-.128D+01	.622D-03	.520D+11

OUTPUT VERIFICATION

```
THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT  =   .119D-07 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT =   .147D-08 LBS
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OUTPUT SUMMARY

```
PILE-HEAD DEFLECTION           =   .377D+00 IN
COMPUTED SLOPE AT PILE HEAD    =   .867D-02
MAXIMUM BENDING MOMENT         =   .791D+07 LBS-IN
MAXIMUM SHEAR FORCE            =   .300D+06 LBS
NO. OF ITERATIONS              =        23
NO. OF ZERO DEFLECTION POINTS  =         4
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S U M M A R Y   T A B L E
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BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.1000D+06	.0000D+00	.5000D+00	.5928D-01	.2089D+07	.1000D+06
.2000D+06	.0000D+00	.5000D+00	.1894D+00	.4876D+07	.2000D+06
.3000D+06	.0000D+00	.5000D+00	.3772D+00	.7908D+07	.3000D+06

I-65 over Mulberry Fork, Ld Test, 60" x 15', 1-7-94, soft clay criteria

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 180.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
.00	60.000	.208D+06	.283D+04	.360D+07
180.00	60.000	.208D+06	.283D+04	.360D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = .00 IN

1 LAYER(S) OF SOIL

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LAYER  1
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER        =        .00 IN
X AT THE BOTTOM OF THE LAYER     =     600.00 IN
MODULUS OF SUBGRADE REACTION     =   .700D+03 LBS/IN**3
```

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
4 POINTS

X,IN	WEIGHT,LBS/IN**3
.00	.78D-01
144.00	.78D-01
145.00	.78D-01
600.00	.78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
4 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
.00	.100D+03	.000	.900D-02
48.00	.400D+03	.000	.700D-02

72.00	.500D+03	.000	.700D-02
600.00	.500D+03	.000	.700D-02

BOUNDARY AND LOADING CONDITIONS

LOADING NUMBER 1

BOUNDARY-CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.520D+05 LBS
MOMENT AT THE PILE HEAD	=	.240D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.500D+06 LBS

LOADING NUMBER 2

BOUNDARY-CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.104D+06 LBS
MOMENT AT THE PILE HEAD	=	.480D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.500D+06 LBS

LOADING NUMBER 3

BOUNDARY-CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.156D+06 LBS
MOMENT AT THE PILE HEAD	=	.720D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.500D+06 LBS

FINITE-DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS	=	15
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE	=	.100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS	=	100
MAXIMUM ALLOWABLE DEFLECTION	=	.30D+03 IN

OUTPUT CODES

KOUTPT = 1
KPYOP = 0
INC = 1

O U T P U T   I N F O R M A T I O N

LOADING NUMBER 1

BOUNDARY CONDITION CODE	=	1
LATERAL LOAD AT THE PILE HEAD	=	.520D+05 LBS
MOMENT AT THE PILE HEAD	=	.240D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD	=	.500D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
.00	.777D-01	.240D+08	.520D+05	-.347D+04	.364D+04	.749D+12
12.00	.537D-01	.244D+08	-.287D+04	-.567D+04	.369D+04	.749D+12
24.00	.344D-01	.240D+08	-.811D+05	-.736D+04	.363D+04	.749D+12
36.00	.197D-01	.225D+08	-.176D+06	-.838D+04	.342D+04	.749D+12
48.00	.936D-02	.198D+08	-.277D+06	-.848D+04	.303D+04	.749D+12
60.00	.280D-02	.158D+08	-.367D+06	-.657D+04	.246D+04	.749D+12
72.00	-.716D-03	.109D+08	-.386D+06	.338D+04	.176D+04	.749D+12
84.00	-.213D-02	.656D+07	-.324D+06	.705D+04	.112D+04	.749D+12
96.00	-.228D-02	.318D+07	-.237D+06	.741D+04	.636D+03	.749D+12
108.00	-.182D-02	.873D+06	-.150D+06	.705D+04	.303D+03	.749D+12
120.00	-.119D-02	-.421D+06	-.704D+05	.623D+04	.237D+03	.749D+12
132.00	-.638D-03	-.817D+06	-.124D+05	.343D+04	.295D+03	.749D+12
144.00	-.246D-03	-.719D+06	.163D+05	.136D+04	.280D+03	.749D+12
156.00	.758D-05	-.425D+06	.242D+05	-.428D+02	.238D+03	.749D+12

168.00	.179D-03	-.138D+06	.177D+05	-.104D+04	.197D+03	.749D+12
180.00	.324D-03	.000D+00	.000D+00	-.192D+04	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .144D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .932D-08 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .777D-01 IN
COMPUTED SLOPE AT PILE HEAD = .219D-02
MAXIMUM BENDING MOMENT = .244D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.386D+06 LBS
NO. OF ITERATIONS = 18
NO. OF ZERO DEFLECTION POINTS = 2

LOADING NUMBER 2

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .104D+06 LBS
MOMENT AT THE PILE HEAD = .480D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.210D+00	.480D+08	.104D+06	-.484D+04	.710D+04	.749D+12
12.00	.153D+00	.489D+08	.267D+05	-.805D+04	.723D+04	.749D+12
24.00	.106D+00	.487D+08	-.859D+05	-.107D+05	.720D+04	.749D+12
36.00	.679D-01	.469D+08	-.226D+06	-.127D+05	.694D+04	.749D+12
48.00	.389D-01	.433D+08	-.384D+06	-.136D+05	.642D+04	.749D+12
60.00	.183D-01	.377D+08	-.539D+06	-.123D+05	.562D+04	.749D+12
72.00	.486D-02	.304D+08	-.667D+06	-.903D+04	.456D+04	.749D+12
84.00	-.271D-02	.217D+08	-.676D+06	.764D+04	.331D+04	.749D+12
96.00	-.611D-02	.142D+08	-.568D+06	.103D+05	.222D+04	.749D+12
108.00	-.679D-02	.808D+07	-.441D+06	.109D+05	.134D+04	.749D+12
120.00	-.591D-02	.359D+07	-.311D+06	.107D+05	.694D+03	.749D+12
132.00	-.435D-02	.630D+06	-.187D+06	.993D+04	.268D+03	.749D+12
144.00	-.266D-02	-.896D+06	-.753D+05	.863D+04	.306D+03	.749D+12
156.00	-.114D-02	-.118D+07	.153D+05	.647D+04	.347D+03	.749D+12
168.00	.145D-03	-.529D+06	.491D+05	-.836D+03	.253D+03	.749D+12
180.00	.133D-02	.000D+00	.000D+00	-.735D+04	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.796D-07 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .513D-08 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .210D+00 IN
COMPUTED SLOPE AT PILE HEAD = .512D-02
MAXIMUM BENDING MOMENT = .489D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.676D+06 LBS
NO. OF ITERATIONS = 26
NO. OF ZERO DEFLECTION POINTS = 2

LOADING NUMBER 3

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .156D+06 LBS
MOMENT AT THE PILE HEAD = .720D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+06 LBS

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.376D+00	.720D+08	.156D+06	-.588D+04	.106D+05	.749D+12
12.00	.282D+00	.735D+08	.616D+05	-.986D+04	.108D+05	.749D+12
24.00	.202D+00	.736D+08	-.773D+05	-.133D+05	.108D+05	.749D+12
36.00	.136D+00	.717D+08	-.253D+06	-.160D+05	.105D+05	.749D+12
48.00	.843D-01	.676D+08	-.455D+06	-.176D+05	.992D+04	.749D+12
60.00	.453D-01	.608D+08	-.660D+06	-.166D+05	.895D+04	.749D+12
72.00	.179D-01	.517D+08	-.843D+06	-.139D+05	.764D+04	.749D+12
84.00	.506D-03	.406D+08	-.942D+06	-.246D+04	.604D+04	.749D+12
96.00	-.908D-02	.292D+08	-.886D+06	.117D+05	.438D+04	.749D+12
108.00	-.131D-01	.194D+08	-.734D+06	.136D+05	.297D+04	.749D+12
120.00	-.133D-01	.115D+08	-.568D+06	.141D+05	.184D+04	.749D+12
132.00	-.113D-01	.574D+07	-.402D+06	.137D+05	.100D+04	.749D+12
144.00	-.828D-02	.190D+07	-.244D+06	.126D+05	.451D+03	.749D+12
156.00	-.484D-02	-.123D+06	-.104D+06	.108D+05	.194D+03	.749D+12
168.00	-.143D-02	-.591D+06	.526D+04	.736D+04	.262D+03	.749D+12
180.00	.187D-02	.000D+00	.000D+00	-.823D+04	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .127D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.535D-07 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .376D+00 IN
COMPUTED SLOPE AT PILE HEAD = .842D-02
MAXIMUM BENDING MOMENT = .736D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.942D+06 LBS
NO. OF ITERATIONS = 25
NO. OF ZERO DEFLECTION POINTS = 2

S U M M A R Y   T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.5200D+05	.2400D+08	.5000D+06	.7765D-01	.2439D+08	-.3862D+06
.1040D+06	.4800D+08	.5000D+06	.2101D+00	.4893D+08	-.6756D+06
.1560D+06	.7200D+08	.5000D+06	.3763D+00	.7356D+08	-.9418D+06

```
***************************************************
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*   ----------------------------------------------*
*                                                 *
*                  Prepared for                   *
*                                                 *
*               PROFESSOR DAN BROWN               *
*            Dept. of Civil Engineering           *
*         Auburn University, Alabama 36849        *
*             License No. 230-082389              *
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*                                                 *
***************************************************
```

PROGRAM LPILE Version 3.0

I-65 over Mulberry Fork, Ld Test, 60" x 15', 1-7-94, .5x comp. c

UNITS--ENGLISH UNITS

I N P U T   I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 180.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
.00	60.000	.208D+06	.283D+04	.360D+07
180.00	60.000	.208D+06	.283D+04	.360D+07

SOILS INFORMATION

X AT THE GROUND SURFACE = .00 IN

1 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER = .00 IN
X AT THE BOTTOM OF THE LAYER = 600.00 IN
MODULUS OF SUBGRADE REACTION = .700D+03 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
4 POINTS

X,IN	WEIGHT,LBS/IN**3
.00	.78D-01
144.00	.78D-01
145.00	.78D-01
600.00	.78D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
4 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
.00	.500D+02	.000	.900D-02
48.00	.200D+03	.000	.700D-02

72.00	.250D+03	.000	.700D-02
600.00	.250D+03	.000	.700D-02

BOUNDARY AND LOADING CONDITIONS

LOADING NUMBER 1

```
BOUNDARY-CONDITION CODE             =        1
LATERAL LOAD AT THE PILE HEAD       =   .520D+05 LBS
MOMENT AT THE PILE HEAD             =   .240D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD         =   .500D+06 LBS
```

LOADING NUMBER 2

```
BOUNDARY-CONDITION CODE             =        1
LATERAL LOAD AT THE PILE HEAD       =   .104D+06 LBS
MOMENT AT THE PILE HEAD             =   .480D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD         =   .500D+06 LBS
```

LOADING NUMBER 3

```
BOUNDARY-CONDITION CODE             =        1
LATERAL LOAD AT THE PILE HEAD       =   .156D+06 LBS
MOMENT AT THE PILE HEAD             =   .720D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD         =   .500D+06 LBS
```

FINITE-DIFFERENCE PARAMETERS

```
NUMBER OF PILE INCREMENTS                                  =        15
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =   .100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS     =       100
MAXIMUM ALLOWABLE DEFLECTION                               =    .30D+03 IN
```

OUTPUT CODES

```
KOUTPT = 1
KPYOP  = 0
INC    = 1
```

O U T P U T   I N F O R M A T I O N
****

LOADING NUMBER 1

```
BOUNDARY CONDITION CODE             =        1
LATERAL LOAD AT THE PILE HEAD       =   .520D+05 LBS
MOMENT AT THE PILE HEAD             =   .240D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD         =   .500D+06 LBS
```

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.122D+00	.240D+08	.520D+05	-.202D+04	.364D+04	.749D+12
12.00	.915D-01	.245D+08	.195D+05	-.339D+04	.371D+04	.749D+12
24.00	.653D-01	.245D+08	-.283D+05	-.457D+04	.371D+04	.749D+12
36.00	.438D-01	.238D+08	-.885D+05	-.548D+04	.362D+04	.749D+12
48.00	.268D-01	.224D+08	-.158D+06	-.604D+04	.341D+04	.749D+12
60.00	.142D-01	.201D+08	-.228D+06	-.567D+04	.307D+04	.749D+12
72.00	.547D-02	.169D+08	-.290D+06	-.471D+04	.262D+04	.749D+12
84.00	-.273D-04	.131D+08	-.318D+06	.664D+02	.207D+04	.749D+12
96.00	-.300D-02	.930D+07	-.293D+06	.408D+04	.152D+04	.749D+12
108.00	-.419D-02	.608D+07	-.241D+06	.468D+04	.105D+04	.749D+12
120.00	-.420D-02	.353D+07	-.184D+06	.481D+04	.686D+03	.749D+12
132.00	-.354D-02	.167D+07	-.127D+06	.466D+04	.418D+03	.749D+12
144.00	-.256D-02	.484D+06	-.732D+05	.428D+04	.246D+03	.749D+12
156.00	-.148D-02	-.870D+05	-.256D+05	.366D+04	.189D+03	.749D+12

168.00	-.420D-03	-.131D+06	.367D+04	.122D+04	.196D+03	.749D+12
180.00	.615D-03	.000D+00	.000D+00	-.183D+04	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.345D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.185D-07 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .122D+00 IN
COMPUTED SLOPE AT PILE HEAD = .277D-02
MAXIMUM BENDING MOMENT = .245D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.318D+06 LBS
NO. OF ITERATIONS = 19
NO. OF ZERO DEFLECTION POINTS = 2

LOADING NUMBER 2

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .104D+06 LBS
MOMENT AT THE PILE HEAD = .480D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.339D+00	.480D+08	.104D+06	-.284D+04	.710D+04	.749D+12
12.00	.265D+00	.491D+08	.580D+05	-.484D+04	.726D+04	.749D+12
24.00	.200D+00	.495D+08	-.109D+05	-.664D+04	.731D+04	.749D+12
36.00	.145D+00	.489D+08	-.998D+05	-.817D+04	.723D+04	.749D+12
48.00	.997D-01	.471D+08	-.205D+06	-.936D+04	.697D+04	.749D+12
60.00	.631D-01	.440D+08	-.317D+06	-.931D+04	.652D+04	.749D+12
72.00	.350D-01	.395D+08	-.425D+06	-.875D+04	.588D+04	.749D+12
84.00	.146D-01	.338D+08	-.518D+06	-.671D+04	.505D+04	.749D+12
96.00	.590D-03	.271D+08	-.567D+06	-.148D+04	.409D+04	.749D+12
108.00	-.817D-02	.202D+08	-.541D+06	.585D+04	.309D+04	.749D+12
120.00	-.130D-01	.142D+08	-.464D+06	.701D+04	.222D+04	.749D+12
132.00	-.152D-01	.909D+07	-.376D+06	.757D+04	.149D+04	.749D+12
144.00	-.156D-01	.512D+07	-.284D+06	.782D+04	.915D+03	.749D+12
156.00	-.150D-01	.227D+07	-.190D+06	.791D+04	.505D+03	.749D+12
168.00	-.140D-01	.567D+06	-.947D+05	.791D+04	.258D+03	.749D+12
180.00	-.128D-01	.000D+00	.000D+00	.787D+04	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = .791D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .318D-07 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .339D+00 IN
COMPUTED SLOPE AT PILE HEAD = .655D-02
MAXIMUM BENDING MOMENT = .495D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.567D+06 LBS
NO. OF ITERATIONS = 31
NO. OF ZERO DEFLECTION POINTS = 1

LOADING NUMBER 3

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = .156D+06 LBS
MOMENT AT THE PILE HEAD = .720D+08 IN-LBS
AXIAL LOAD AT THE PILE HEAD = .500D+06 LBS

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.691D+00	.720D+08	.156D+06	-.360D+04	.106D+05	.749D+12
12.00	.559D+00	.737D+08	.972D+05	-.620D+04	.108D+05	.749D+12
24.00	.442D+00	.745D+08	.808D+04	-.864D+04	.109D+05	.749D+12
36.00	.340D+00	.740D+08	-.109D+06	-.108D+05	.108D+05	.749D+12
48.00	.251D+00	.719D+08	-.250D+06	-.127D+05	.106D+05	.749D+12
60.00	.177D+00	.681D+08	-.406D+06	-.131D+05	.999D+04	.749D+12
72.00	.115D+00	.623D+08	-.562D+06	-.130D+05	.916D+04	.749D+12
84.00	.656D-01	.546D+08	-.707D+06	-.111D+05	.805D+04	.749D+12
96.00	.266D-01	.454D+08	-.824D+06	-.843D+04	.672D+04	.749D+12
108.00	-.376D-02	.349D+08	-.847D+06	.451D+04	.521D+04	.749D+12
120.00	-.274D-01	.250D+08	-.767D+06	.898D+04	.379D+04	.749D+12
132.00	-.462D-01	.165D+08	-.647D+06	.110D+05	.256D+04	.749D+12
144.00	-.618D-01	.954D+07	-.507D+06	.124D+05	.155D+04	.749D+12
156.00	-.756D-01	.435D+07	-.351D+06	.136D+05	.804D+03	.749D+12
168.00	-.885D-01	.112D+07	-.182D+06	.146D+05	.338D+03	.749D+12
180.00	-.101D+00	.000D+00	.000D+00	.157D+05	.177D+03	.749D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.122D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.989D-07 LBS

OUTPUT SUMMARY

PILE-HEAD DEFLECTION = .691D+00 IN
COMPUTED SLOPE AT PILE HEAD = .115D-01
MAXIMUM BENDING MOMENT = .745D+08 LBS-IN
MAXIMUM SHEAR FORCE = -.847D+06 LBS
NO. OF ITERATIONS = 35
NO. OF ZERO DEFLECTION POINTS = 1

S U M M A R Y   T A B L E

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
.5200D+05	.2400D+08	.5000D+06	.1224D+00	.2450D+08	-.3180D+06
.1040D+06	.4800D+08	.5000D+06	.3387D+00	.4946D+08	-.5672D+06
.1560D+06	.7200D+08	.5000D+06	.6905D+00	.7446D+08	-.8475D+06