Applied Foundation Testing

October 9, 2008

Final Report of Axial STAT*NAMIC* Load Testing
Drilled Shaft Load Test Program
I-80 Bridge Project
(Broadway Bridge Viaduct)
Council Bluffs,
Pottawattamie County, Iowa
NHS-080-1(318)0-11-78
AFT Project No.: 108026

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INTRODUCTION

This report is provided to summarize the procedures and results of the Statnamic load testing performed as part of the subject drilled shaft load test project. The Statnamic load testing was performed in accordance with ASTM D7383-08, which is the newly approved standard for this test method. Objectives of the load test program included evaluation of conventional drilled shafts and post grouted drilled shafts for use in the future design and construction of the Broadway Bridge Viaduct in Council Bluffs, Iowa. The test site was located adjacent to the future bridge at the intersection of 12th Street and West Broadway. Please refer to the contract documents for a site plan of the actual location of the test site. The project entailed construction of three strain instrumented test drilled shafts. Two shafts included sleeve port and plate base grouting apparatus and were post grouted. The third shaft was un-grouted and served as a control shaft for comparison of the un-grouted end bearing. All shafts were 60 inches in diameter and had lengths of approximately 55 and 65 feet. The control shaft was 55 feet. Post grouted shafts were 55 feet and 65 feet. The testing sequence included performing SPT borings at each test shaft location prior to construction, performing integrity testing using the crosshole sonic logging method on the test shafts prior to post grouting. Post grouting was then performed. All embedded strain gages were monitored during the post grouting process as well. Finally, Statnamic load testing was performed on all three shafts. Each test method performed was documented in separate reports. This report only contains the Statnamic load test results.

As an innovative means to cost effectively obtain load test data, the load test program was included in the construction contract for the nearby I-80 Bridge over the Missouri River project, which also utilized drilled shafts. The I-80 Bridge general contractor was Jensen Construction Company. Longfellow Drilling performed the land based drilled shafts for the I-80 bridge and Jensen Construction performed the marine based drilled shafts. Longfellow Drilling managed all load test program activities including: site preparation, shaft installation and all required support to carry out the testing. Applied Foundation Testing was contracted through Longfellow Drilling on behalf of the Iowa DOT to perform the instrumentation, CSL, post grouting, Statnamic load testing and associated engineering reports for this load test program only. Construction inspection and oversight was provided by Iowa DOT and FHWA representatives. The Geotechnical engineer of record was CH2MHILL. Additional soil borings at each test shaft location along with concrete QC testing for the test program was performed by Geotechnical Services, Inc.

GENERALIZED SOIL CONDITIONS

A geotechnical investigation was initially performed by CH2MHILL. Additional SPT borings were required as part of the load test contract at each of the three test shaft locations. These borings were performed by Geotechnical Services, Inc. and are attached with this report. We have provided a brief description of the soil conditions for general use while reviewing this report. For detailed information on the soil conditions, please refer to the attached borings.

The borings were drilled to a depth of 70 feet below the ground surface at each specific test shaft location. The ground surface elevation at the site was between +988.5 and +990.5 feet. In general, the upper 20 feet in all three borings consisted of soft to stiff silty clay (CL) and (CL-CH). SPT N values in this upper silty clay ranged from 3 to 14. The water table was around 16 to 17 feet in depth below ground surface. The upper silty clays were underlain by fine sand and silty fine sand (SP-SM) to depths of 55 to 60 feet. SPT N values in the sands ranged from 6 to 42. the fine silty sands gave way to a more coarse grained sand which extended to the boring bottom of 70 feet. In the TS-1 boring a soft clay layer was encountered between the bottom of the fine sand at 55 feet and the coarse sand at 60 feet.

FOUNDATION DESCRIPTION AND CONSTRUCTION

The three test shafts designated as TS-1, TS-2 and TS-3 were all 60 inch in diameter. TS-1 which was a 65 foot post grouted shaft was tipped in the coarse sand while TS-2 (55 ft post grouted shaft) and TS-3 (ungrouted) were tipped in the fine grained sandy soils. These medium dense granular soils were very conducive to improvement by base grouting. It is important to note that the two post grouted shafts, TS-1 and TS-2 were tipped in very similar relative density sands with SPT N values of 11 and 12, respectively. The un-grouted control shaft, TS-3, was tipped in a more competent sand with an SPT N of 24. This is an important consideration when evaluating the improvement from grouting. The test shafts were designed and constructed with planned ultimate capacities such that they could be reached within the constrains of the 2,000 ton Statnamic device.

Table 1. Summary of Test Shafts Properties.

Shaft Number	Shaft Diameter (inches)	Shaft Length (feet)	f'c at Time of Statnamic Load Test (psi)	Shaft Type
TS-1	60	66.3	5,780	Post Grouted
TS-2	60	55.4	5,580	Post Grouted
TS-3	60	55.9	5,770	Non-Grouted

Shaft reinforcement was the same for all three test shafts and consisted of 23 - #10 longitudinal bars with #5 shear hoops on 12 inch centers. A 3 inch clear spacing governed the steel hoop diameter. Sisterbar strain gages were installed at four vertical levels in all three test shafts as shown in the schematic drawings in Appendix B.

Shaft construction was performed using the “wet method”. The general procedure included placing a short upper permanent casing 4 to 5 feet in length to both stabilize the near surface soils and provide a clean and level surface for load testing. A Watson track mount drill rig was used with various drilling tools, auger and cleanout bucket to excavate the soils. Water and polymer slurry admixture drilling fluid was introduced early on to stabilize the walls of the excavation. Once the required depths were attained, the shaft bottoms were mechanically cleaned using a standard clean-out bucket and inspected for cleanliness by sounding with a weighted tape. The upper 4 to 5 feet of each shaft was constructed 66 inches in diameter using a permanent casing that extended approximately one foot above grade. The nominal shaft diameter for the remaining depth was 60 inches. A pump truck slick line was used to place the concrete while maintaining the discharge 10 to 15 feet below the rising concrete head at all times. Concrete quality assurance testing was performed by Geotechncial Services, Inc. The concrete compressive strength data is the only QA QC results included in this report. Drilled shaft construction was completed between August 20 and August 22, 2008. For more information on the shaft construction and QAQC please see the drilled shaft construction records included in Appendix B or contact the Iowa DOT.

For the purpose of analysis of the load test measurements, shaft properties provided in Table 1 were used in our calculations. It is noted that the concrete strengths were based on tests performed within a close time frame of the Statnamic load testing.

AXIAL STATNAMIC TEST SETUP AND INSTRUMENTATION

Preparation for the load test included leveling of the ground surrounding the test shafts and assembly of the Statnamic equipment. Foundation preparation prior to testing included pouring a layer of high strength grout on the top of the test shaft. This was to ensure a full and uniform contact surface area for the load transfer plate. Load testing was accomplished in a single load cycle to each foundation utilizing a 2,000 ton Statnamic device equipped with a mechanical catch system. Various key dates related to the Statnamic testing sequence are summarized in Table 2.

Table 2. Statnamic Load Testing Key Dates Summary

Shaft Number	Date Instrumented	Date Constructed	Statnamic Load Test Date
TS-1	8/22/2008	8/22/2008	9/15/2008
TS-2	8/21/2008	8/21/2008	9/12/2008
TS-3	8/20/2008	8/20/2008	9/13/2008

During the Statnamic load tests, load, acceleration, and strain were monitored to obtain the foundation response. A description of the instrumentation used during the Statnamic tests is given below. Calibration data is included in Appendix C.

- Statnamic Device and Load Cell - This device uses a controlled burn of fuel to generate gas pressure inside a cylinder and ram (analogous to a gas actuated jack). As the pressure builds, it reacts against a heavy mass above the shaft. The pressure eventually builds high enough to propel the reaction mass upward, in turn a downward load is simultaneously applied to the shaft top which is many times greater than the weight of the reaction mass. The Statnamic device produces a time dependent load on the order of 1/2 second or less. The load produced is not an impact, which makes the Statnamic analysis very simplified and more reliable than dynamic techniques. The applied Statnamic load is measured with a ring type electronic resistance load cell, located between the pile top and the Statnamic piston. The load cell has a working capacity of 2,000 tons and is calibrated full scale. The manufacturer George Kelk Corporation recommends calibration every 10,000 cycles or 5 years which ever occurs first.

- Displacement - Three capacitive accelerometers were arranged across the top of the pile approximately 120 degrees apart during Statnamic testing. The capacitive accelerometers were manufactured by PCB Piezotronics, Inc. From the measured accelerations, pile displacements at each accelerometer location were calculated by double integration. This provides very reliable and highly accurate displacement data.

- Strain Gages- Schematic drawing of the quarter bridge electronic resistance sister bar strain gage location are provided in the Appendix B. These gages were manufactured at AFT's lab in Tampa, Florida by our engineers using Micro-Measurements gauge type CEA-06-125UW-350 gages.

- Data Acquisition System - MEGADAC Data Acquisition System, manufactured by Optim, Inc. This system monitored the load cell, and accelerometers and recorded data at 5,000 samples per second for each sensor. This is more than ample to fully define the load and displacement response of the shaft during the test.

AXIAL STATNAMIC LOAD TEST RESULTS

The derived static capacities from the Statnamic tests were determined using the Segmental Unloading Point Method (SUP). A detailed explanation of the Segmental Unloading Point is presented in Appendix D. The Segmental Unloading Point Method (SUP), discretizes a foundation into segments. The number of segments and their lengths are defined by the locations of the embedded strain gages. This allows the standard or Modified Unloading Point Method (UPM) to be applied to each segment. Then the total derived static response is calculated as the sum of the derived static response from the individual segments. Rate Effect Factors (REF) were used in conjunction with the SUP method as suggested in the national Cooperative Highway Research Program (NCHRP) Project: NCHRP 21-08.

Based on our interpretation of the soil borings, we used the published REF of 0.91 for the sands and 0.65 for the clayey soils in our analysis. From the NCHRP data sets, the combined values of all soil types show that the REF corrected SLD capacity analysis method performs very well with an average bias of 1.017 and a standard deviation of 9.7 %. Therefore, after application of the REF, the Statnamic UPM capacity tended to under estimate the static capacity by approximately 2%. Based on several additional comparisons that are not published in the NCHRP report we have found similar results.

At this site we would expect the static capacities presented in this report to also be slightly on the conservative side by approximately a couple percent.

Although a limit state of 1.0 inch of movement at the shaft top was specified as the failure criteria for this project, we have included the following interpretation methods for determination of the shaft ultimate capacity.

A. Limit State = 1 inch, which is defined in the project specifications as the required method.
B. Slope and Tangent Method (Butler and Hoy, 1977).
C. Offset Limit (Davisson, 1972) modified by FHWA for large diameter piles.
D. Maximum achieved during load test.

Load transfer during testing was determined using embedded strain instrumentation placed at four vertical levels as shown in the attached test shaft schematic drawings. Strain data processing included minor offset corrections for the static load applied from the reaction mass prior to testing. In addition, the measured strains at each level were averaged. Load at each gage level was then calculated by multiplying the average strain by the respective cross-sectional area and composite modulus of elasticity. The in-situ modulus was determined by manually varying the modulus term until the calculated load from the upper most strain gage level generally matched the measured load from the load cell. Other drilled shaft structural properties summarized in Table 1 were used to calculate load from the measured strains within the shaft.

Unit side shear was determined by subtracting the derived static loads at each level and dividing by the respective segment surface area. The segment unit side shear is presented in the form of a Tz curve or soil response curve. In the Tz curve, the displacement shown is average of the segment or at the midpoint of the segment. The midpoint displacement is calculated by subtracting the cumulative elastic shortening of the length of shaft above from the measured top displacement. Examination of the Tz curves shows that the maximum values in each soil layer develop at slightly different displacements depending on the layer characteristics. Residual stresses sometimes make the unloading portion of the Tz curve appear very unusual in shape. This is typically manifest in the upper segments.

The load at the lowest strain gage level is also plotted versus tip displacement. The tip displacement is determined as discussed above. This is known as a Qz curve and is not the true end bearing since it also consists of a small zone of side shear resistance. Therefore, an estimated side shear resistance has to be subtracted to obtain the end bearing. The unit values provided in the succeeding tables include this correction.

TS-1 Load Test Results

The derived static load versus displacement response of TS-1 is presented in Figure 1. On this figure we have graphically shown the various criteria for determination of ultimate capacity which were outlined above.

Table 2. Summary of Load and Displacement for TS-1.

Method of Interpretation	Ultimate Capacity (kips)	Maximum Displacement (inches)	Permanent Displacement (inches)
Slope and Tangent	2,350		
1.0" Limit State	**2,465**		
FHWA	2,530		
Maximum	2,530	1.74	1.10

Examining the three methods of defining the ultimate capacity showed a 180 kip difference in the interpreted highest and lowest which is about 7 percent variation. However, the 1.0 inch Limit State method specified in the contract documents shows an ultimate of 2,465 kips. It is also noted that this shaft did not experience sufficient movement to strictly meet the FHWA criteria so AFT has estimated this value. The maximum measured displacement during the test was 1.74 inches and the measured permanent displacement upon complete unloading was 1.10 inches. Table 2 also summarizes the load and displacement response during testing.

Table 3 presents the maximum unit side shear values calculated from the measured strains within the shaft. Figures 2 through 4 present the side shear versus displacement (Tz) curves for each segment. The ultimate unit side shear values ranged from 1.7 ksf in the silty fine sand to 2.4 ksf in the lower fine sand layer. The end segment load versus displacement response Qz curve is presented in Figure 5. The ultimate unit end bearing was 32 ksf after accounting for the plug shear contribution.

Table 3. Load Transfer Summary for TS-1.

Segment Boundaries	Maximum Mobilized Unit Side Shear (ksf)
990.04 ft to 969.04 ft (Segment 1)	1.8
969.04 ft to 957.29 ft (Segment 2)	1.7
957.29 ft to 926.29 ft (Segment 3)	2.4
926.29 ft to 923.79 ft (Toe Segment)	2.4*
Maximum Unit End Bearing (ksf)	32

*Unit side shear assumed from segment above.

TS-2 Load Test Results

The derived static load versus displacement response of TS-2 is presented in Figure 6 with the various criteria for determination of ultimate capacity graphically displayed.

They are summarized in Table 4. A difference of 122 kips in the interpreted highest and lowest which is about 5 percent variation. The 1.0 inch Limit State method specified in the contract documents shows an ultimate of 2,362 kips. The maximum displacement during testing showed 2.34 inches with the measured permanent displacement upon complete unloading being 1.70 inches.

Table 4. Summary of Load and Displacement for TS-2.

Method of Interpretation	Ultimate Capacity (kips)	Maximum Displacement (inches)	Permanent Displacement (inches)
Slope and Tangent	2,247		
1.0" Limit State	**2,362**		
FHWA	2,285		
Maximum	2,369	2.34	1.70

Table 5 presents the ultimate unit side shear values calculated from the measured strains within the shaft. Figures 7 through 9 present the side shear versus displacement (Tz curves) for each segment. The ultimate unit side shear values ranged from 1.8 ksf in the silty clay to 2.8 ksf in the lower fine sand layer. The end segment load versus displacement response or Qz curve is presented in Figure 10. The ultimate unit end bearing was 34 ksf after accounting for the plug shear contribution.

Table 5. Load Transfer Summary for TS-2.

Segment Boundaries	Maximum Mobilized Unit Side Shear (ksf)
988.72 ft to 966.97 ft (Segment 1)	1.8
966.97 ft to 954.47 ft (Segment 2)	1.9
954.47 ft to 935.22 ft (Segment 3)	2.8
935.22 ft to 933.30 ft (Toe Segment)	2.8*
Maximum Unit End Bearing (ksf)	34

*Unit side shear assumed from segment above.

TS-3 Load Test Results

The derived maximum static load versus displacement response of TS-3 presented in Figure 11 shows an ultimate load of 2,033 kips with a total displacement of 2.57 inches. The measured permanent displacement upon complete unloading was 1.67 inches. We have also graphically displayed the various criteria for determination of ultimate capacity on Figure 11 and they are summarized in Table 6. A 108 kip difference in the interpreted highest and lowest values was shown, which is about 5 percent variation.

Table 6. Summary of Load and Displacement for TS-3.

Method of Interpretation	Ultimate Capacity (kips)	Maximum Displacement (inches)	Permanent Displacement (inches)
Slope and Tangent	1,925		
1.0" Limit State	**2,033**		
FHWA	1,950		
Maximum	2,033	2.57	1.67

Table 7 presents the ultimate unit side shear values calculated from the measured strains within the shaft. Figures 12 through 14 present the side shear versus displacement or Tz curves for each segment. The ultimate unit side shear values ranged from 1.7 ksf in the silty clay to 2.4 ksf in the lower fine sand layer. The end segment load versus displacement response known as a Qz curve is presented in Figure 15. The measured unit end bearing was 16 ksf after accounting for the plug shear contribution.

Table 7. Load Transfer Summary for TS-3.

Segment Boundaries	Maximum Mobilized Unit Side Shear (ksf)
990.47 ft to 970.52 ft (Segment 1)	1.7
970.52 ft to 960.52 ft (Segment 2)	1.9
960.52 ft to 937.77 ft (Segment 3)	2.4
937.77 ft to 935.69 ft (Toe Segment)	2.4*
Maximum Unit End Bearing (ksf)	16

*Unit side shear assumed from segment above.

SUMMARY AND CONCLUSIONS

A design phase load test program was successfully carried out for the future construction of the Broadway Bridge Viaduct in Council Bluffs, Iowa. Objectives of the load test program included evaluation of conventional drilled shafts and post grouted drilled shafts. Three drilled shafts designated as TS-1, TS-2, and TS-3 were load tested to evaluate axial capacity and load transfer characteristics at this site using the Statnamic load test method in accordance with ASTM D7383-08. Two shafts TS-1 and TS-2 included base grouting apparatus and were post grouted prior to load testing as described in a separate report. The third shaft TS-1 was un-grouted and served as a control shaft for comparison of the un-grouted end bearing. All shafts were 60 inches in diameter and had lengths of approximately 55 and 65 feet. The control shaft TS-1 was 55 feet. Post grouted shafts TS-1 and TS-2 were 65 feet and 55 feet, respectively. The testing sequence included performing SPT borings at each test shaft location prior to construction, performing integrity testing using the crosshole sonic logging method on the test shafts prior to post grouting. Post grouting was then performed. All embedded strain gages were monitored during the post grouting process as well. Finally, Statnamic load testing was performed on all three shafts. Each test method performed was documented in separate reports. This report only contains the Statnamic load test results.

The Statnamic load tests were performed and the ultimate shaft capacities were obtained for each shaft. The project specifications define the ultimate capacity as the capacity at 1 inch of movement at the top of the drilled shaft. The ultimate capacity of each shaft according to this criteria are summarized in Table 8 below. We have also provided a composite plot of the load versus displacement of all three shafts in Figure 16. This figure provides a relative comparison of each shafts' load and displacement responses.

Unit side shear values range from 1.7 ksf to 2.8 ksf at the site with values comparing well in segments with similar elevation boundaries. The difference in the end bearing response for each shaft as a result of post grouting can be seen in Figure 17. After accounting for the bottom plug shear, ultimate unit end bearing values in the two post-grouted shafts, TS-1 and TS-2, were 32 ksf and 34 ksf, respectively as compared to 16 ksf for the un-grouted shaft TS-3. This is approximately 100 to 110 percent greater than the un-grouted end bearing. It also needs to be considered that TS-3 was tipped in a more competent material than the other two shafts (SPT N value of 24 as opposed to 11 and 12) so the increase in end bearing resistance due to the post-grouting is likely higher considering baseline soil conditions were not equal.

Table 8. Summary of Ultimate Capacities for Test Shafts.

Shaft Number	Shaft Diameter (inches)	Shaft Length (feet)	Shaft Type	Ultimate Capacity @ 1 " Limit State (kips)	Ultimate End Segment Capacity (kips)
TS-1	60	66.3	Post Grouted	2,465	712
TS-2	60	55.4	Post Grouted	2,362	749
TS-3	60	55.9	Non-Grouted	2,033	390

CLOSURE

It has been a pleasure working with you and we appreciate all your support in the field to successfully complete the Statnamic load testing. If you have any questions relating to this information please contact us at your convenience.

LIMITATIONS

This report presents test measurements made by AFT. Interpretations were made based upon the measurements made by AFT with the latest techniques available and currently accepted standards of care recognized by Geotechnical Engineering professionals. AFT is an independent agency and is not the Geotechnical Engineer of Record. The Geotechnical Engineer of Record should ultimately make final recommendations for foundation design and construction.

Instrumented rebar cages.

Drilling test shafts.

Sounding bottom of excavation.

Placing instrumented cage.

Pouring concrete with pump truck slick line.

Assembling the Statnamic load test device.

Statnamic device fully assembled and ready for load testing.

Appendix A

Test Shaft 1

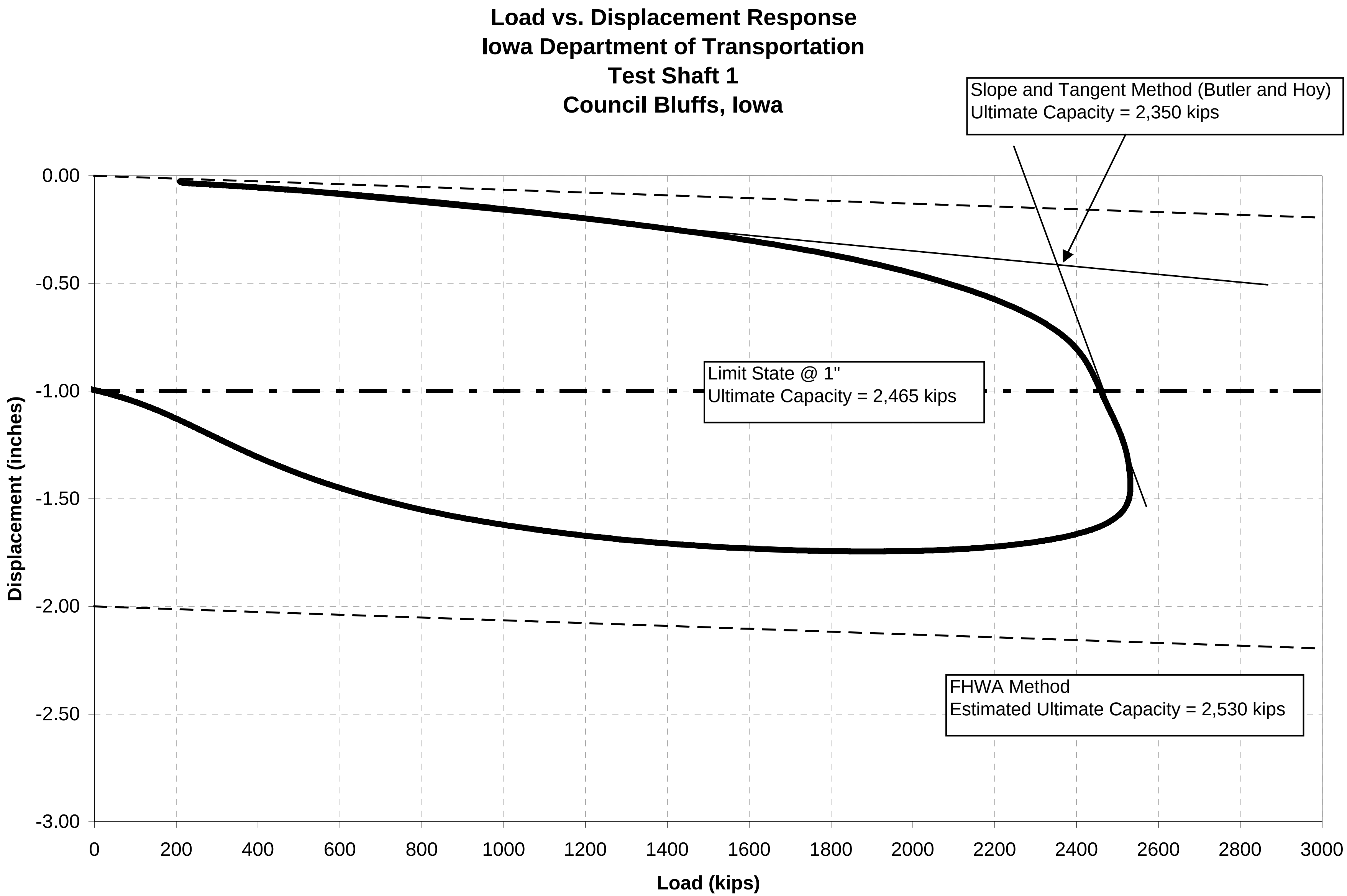


Figure 1

Applied Foundation Testing, Inc.

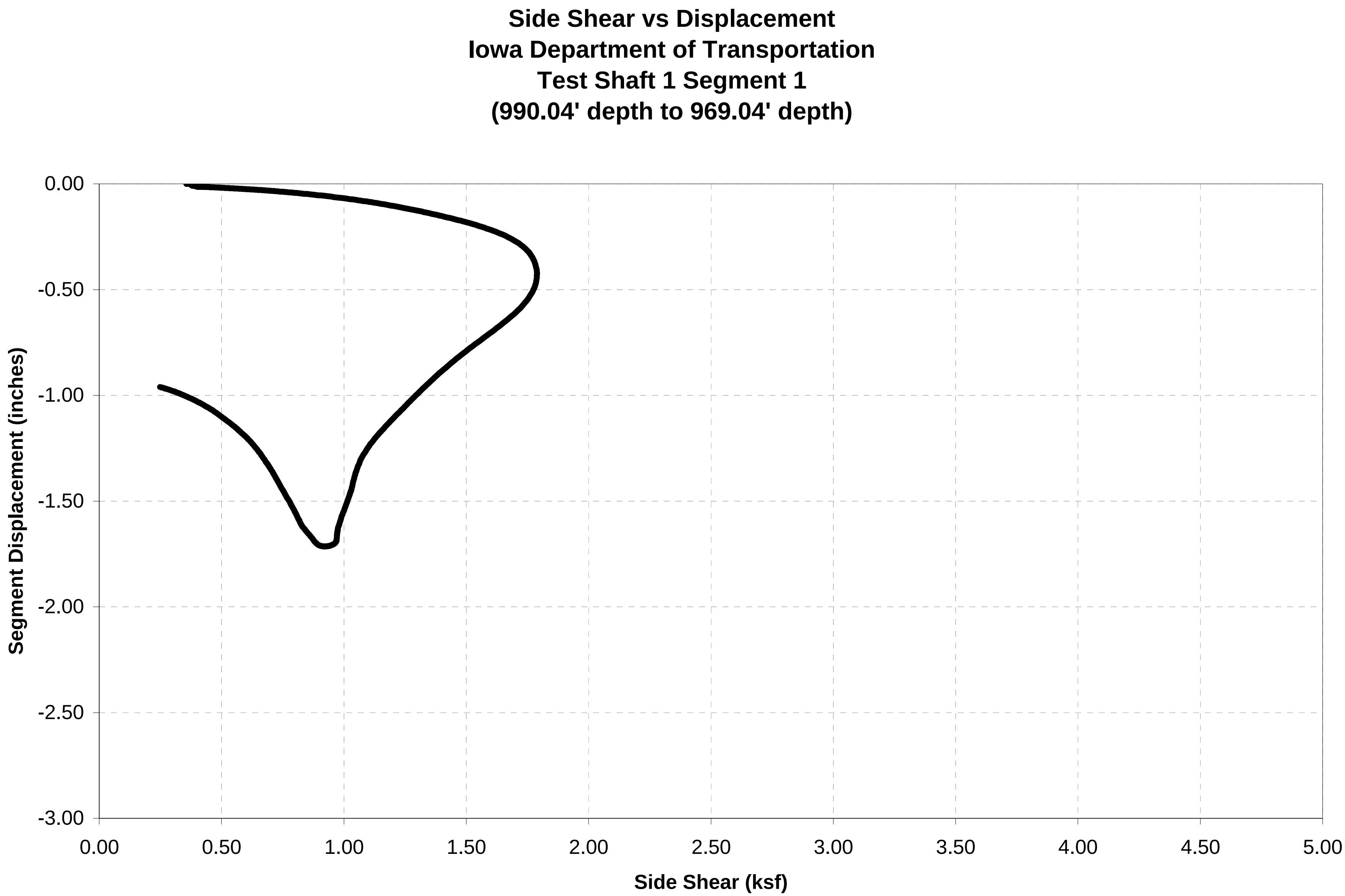
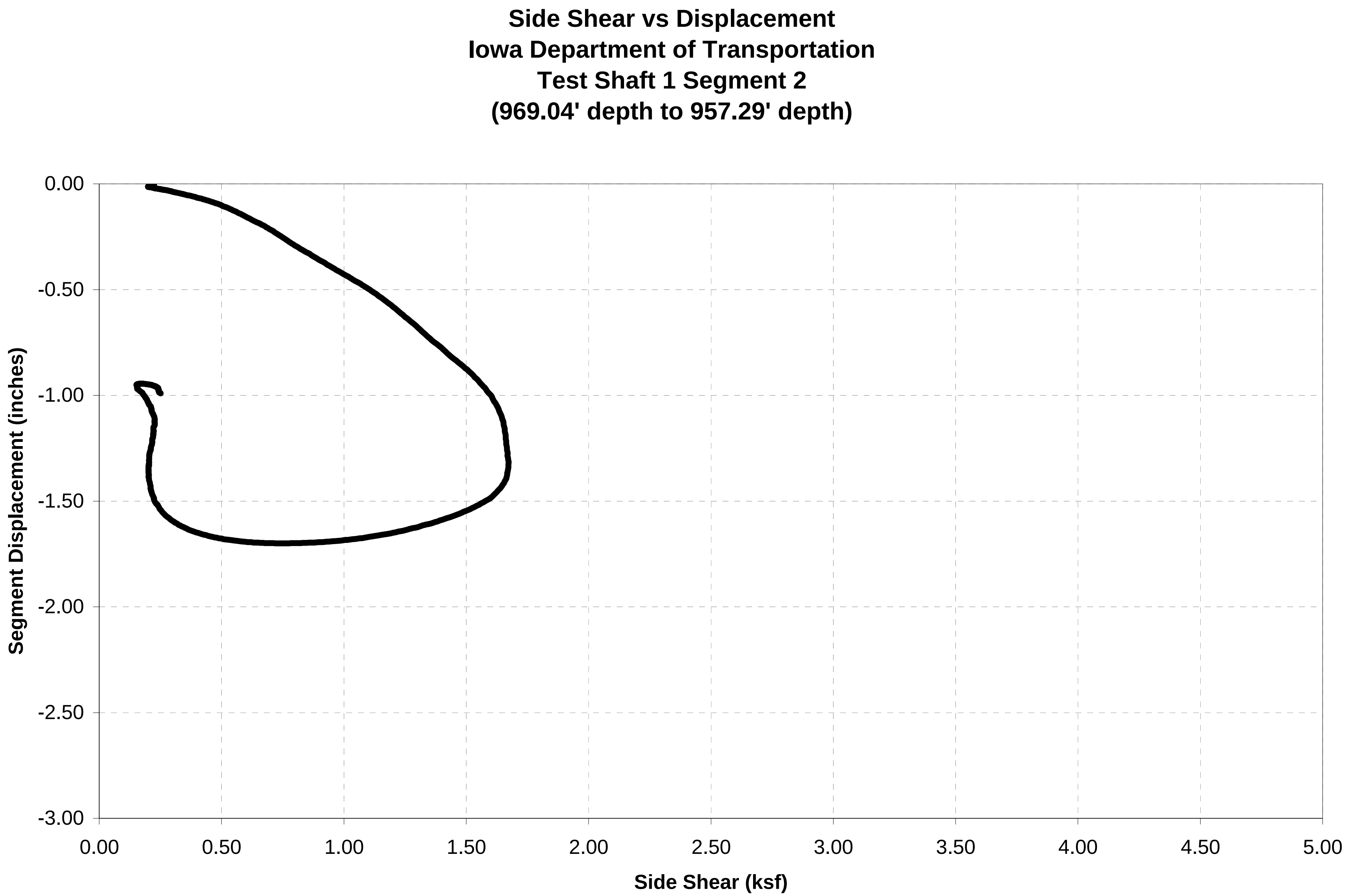
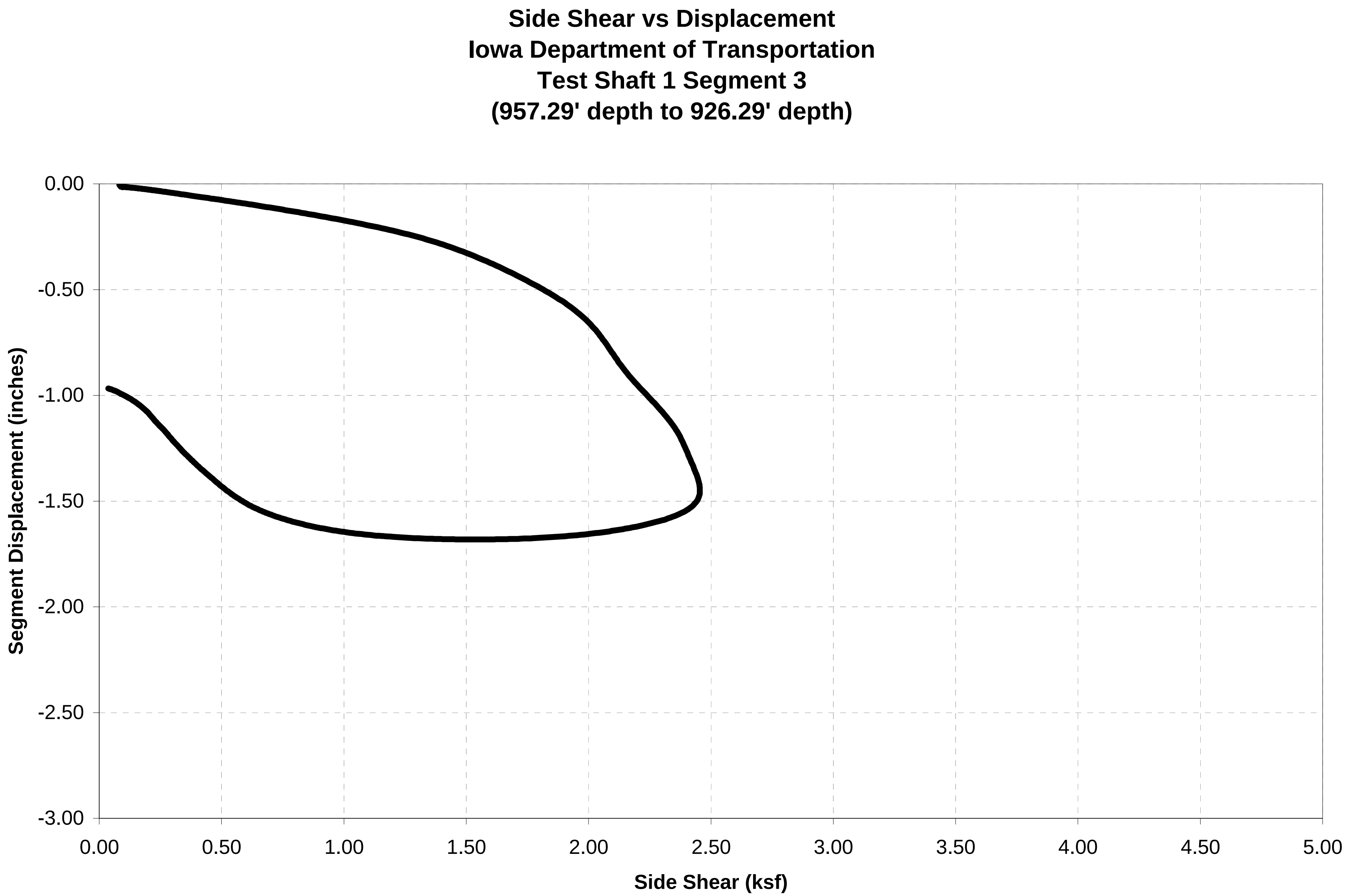


Figure 2

Applied Foundation Testing, Inc.


Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 1 Segment 2
(969.04' depth to 957.29' depth)
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)


Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 1 Segment 3
(957.29' depth to 926.29' depth)
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

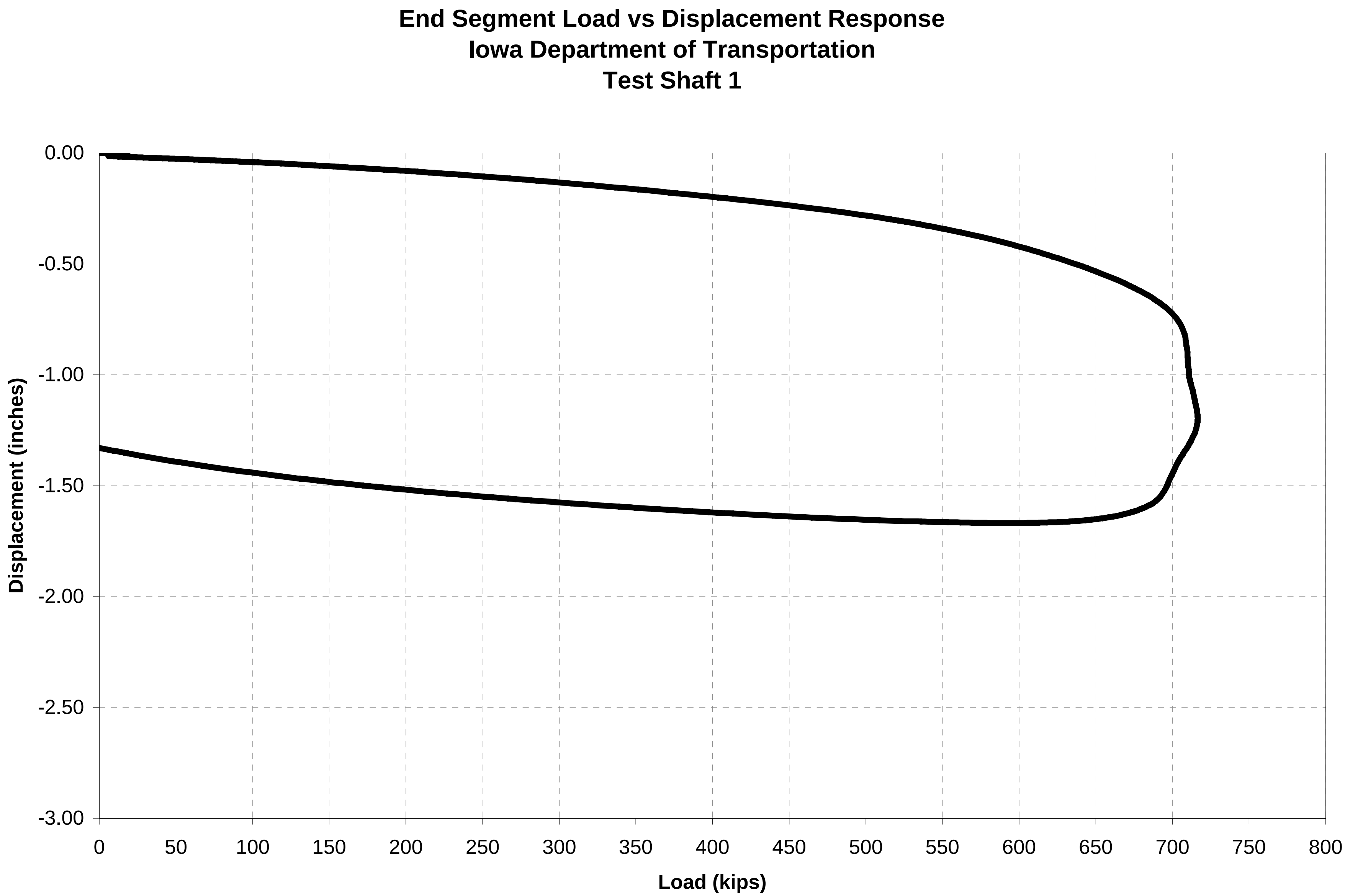
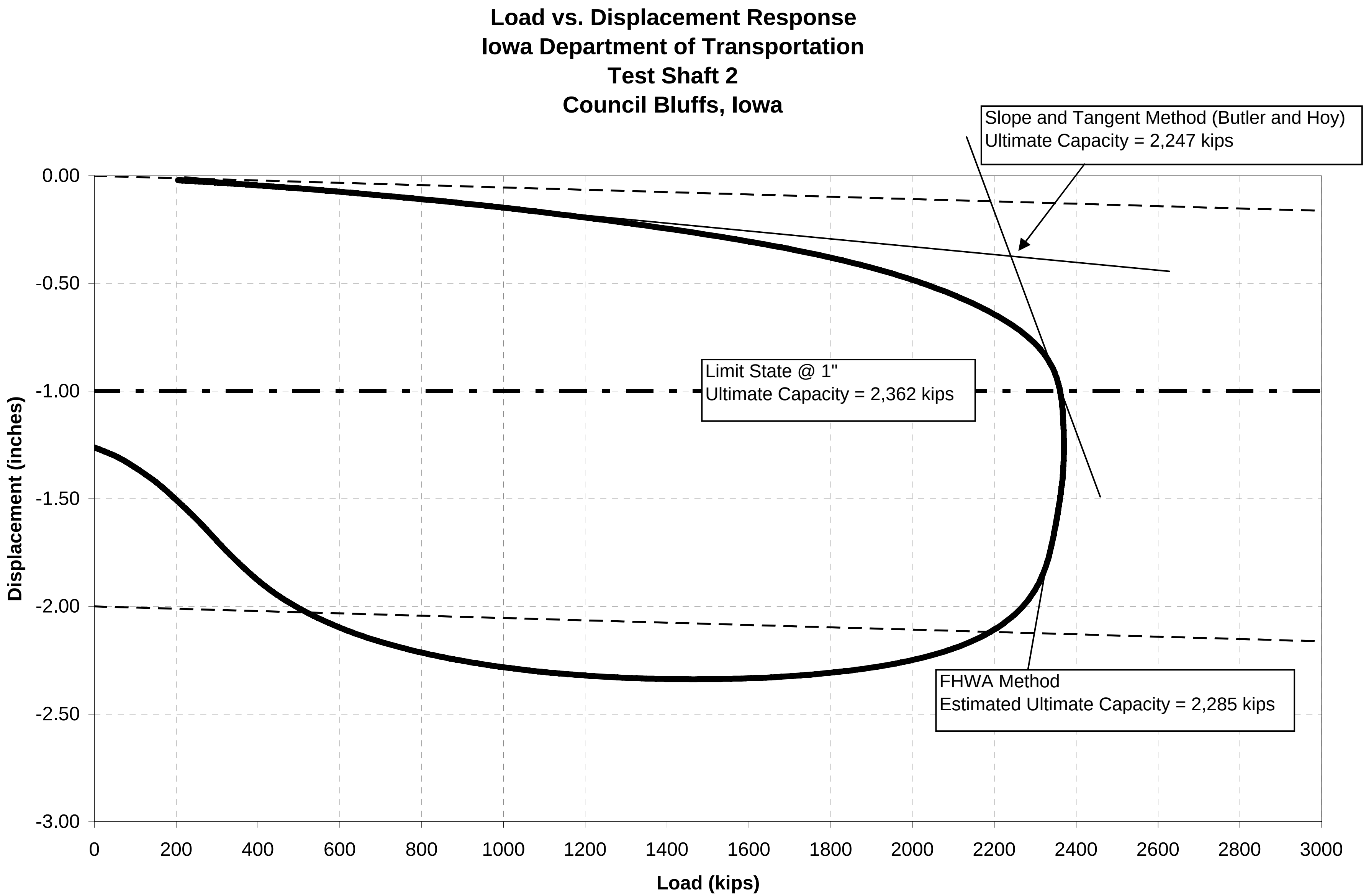

End Segment Load vs Displacement Response
Iowa Department of Transportation
Test Shaft 1
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Displacement (inches)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
Load (kips)

Figure 5

Test Shaft 2



Load vs. Displacement Response
Iowa Department of Transportation
Test Shaft 2
Council Bluffs, Iowa
Slope and Tangent Method (Butler and Hoy)
Ultimate Capacity = 2,247 kips
Limit State @ 1"
Ultimate Capacity = 2,362 kips
FHWA Method
Estimated Ultimate Capacity = 2,285 kips
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Displacement (inches)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
Load (kips)

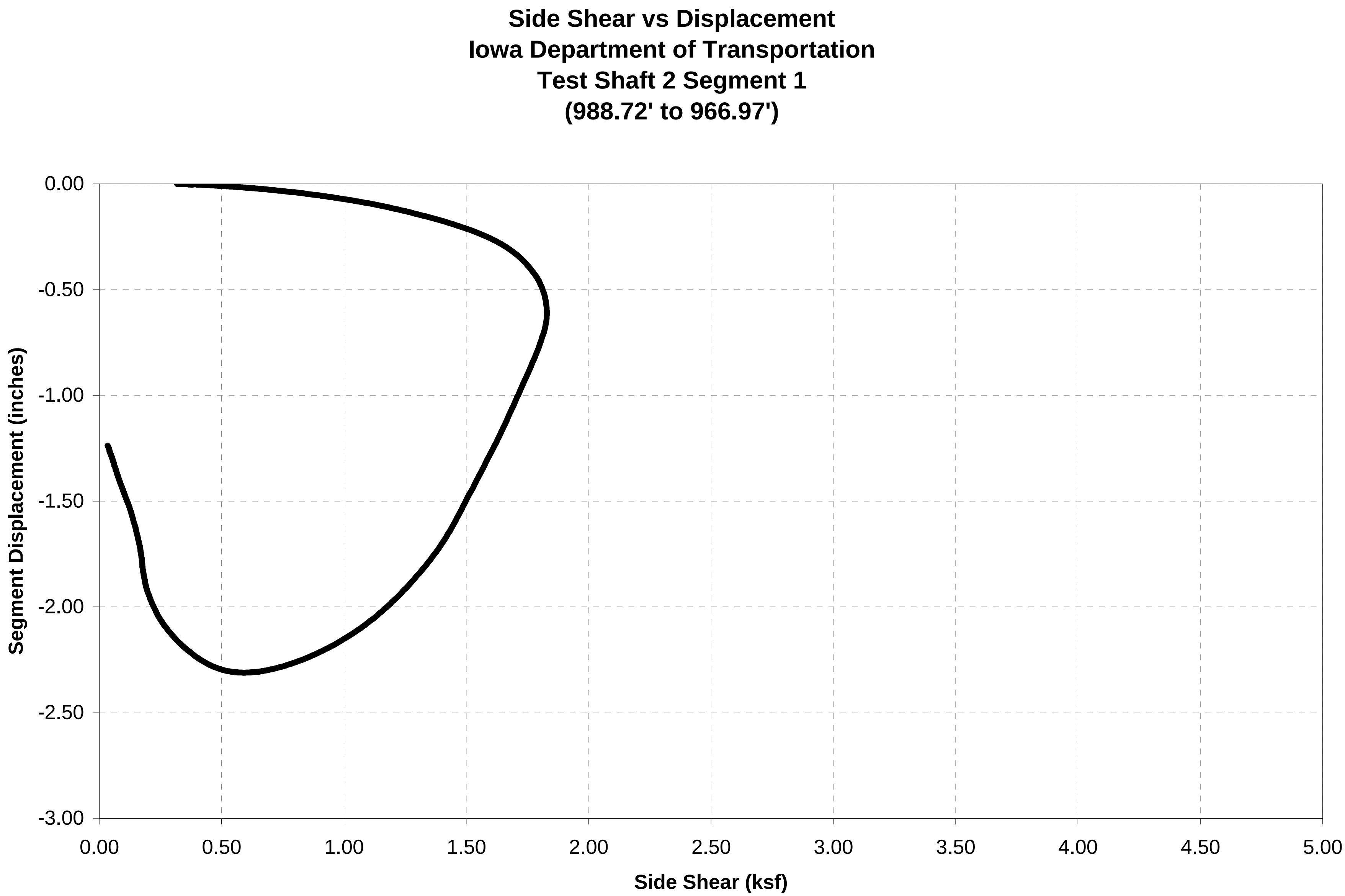
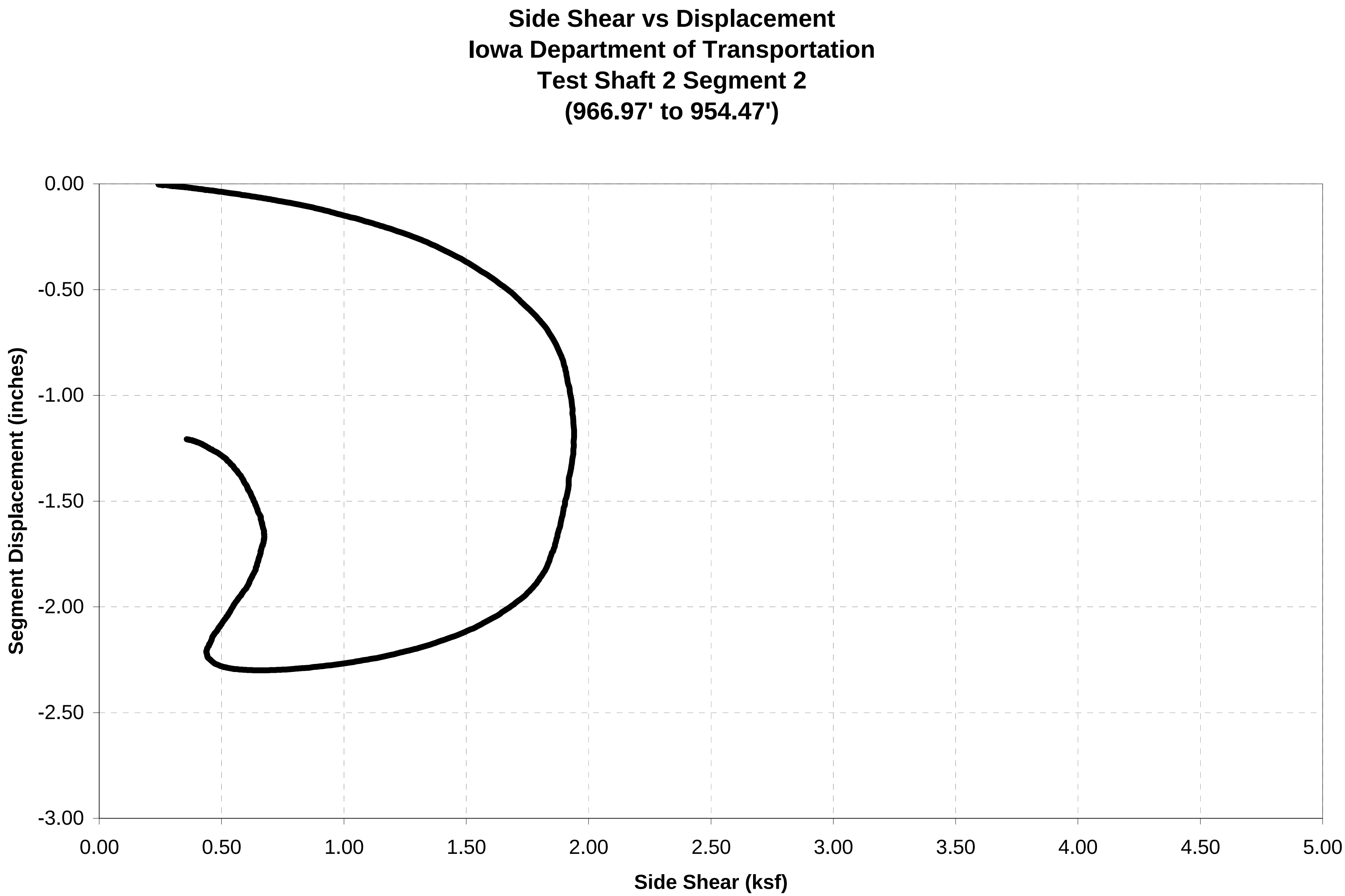
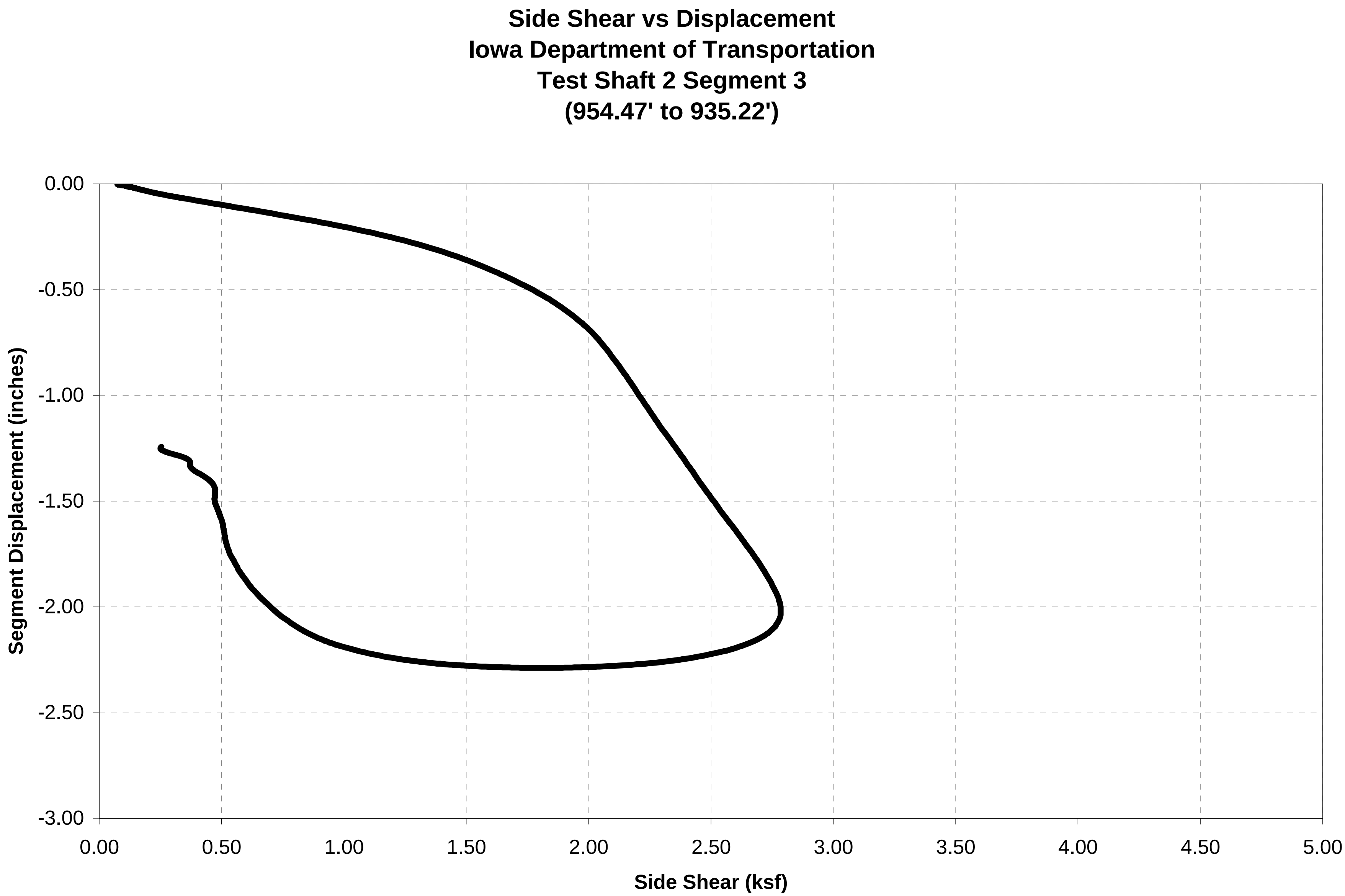

Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 2 Segment 1
(988.72' to 966.97')
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

Figure 7



Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 2 Segment 2
(966.97' to 954.47')
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

Figure 8



Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 2 Segment 3
(954.47' to 935.22')
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

Figure 9

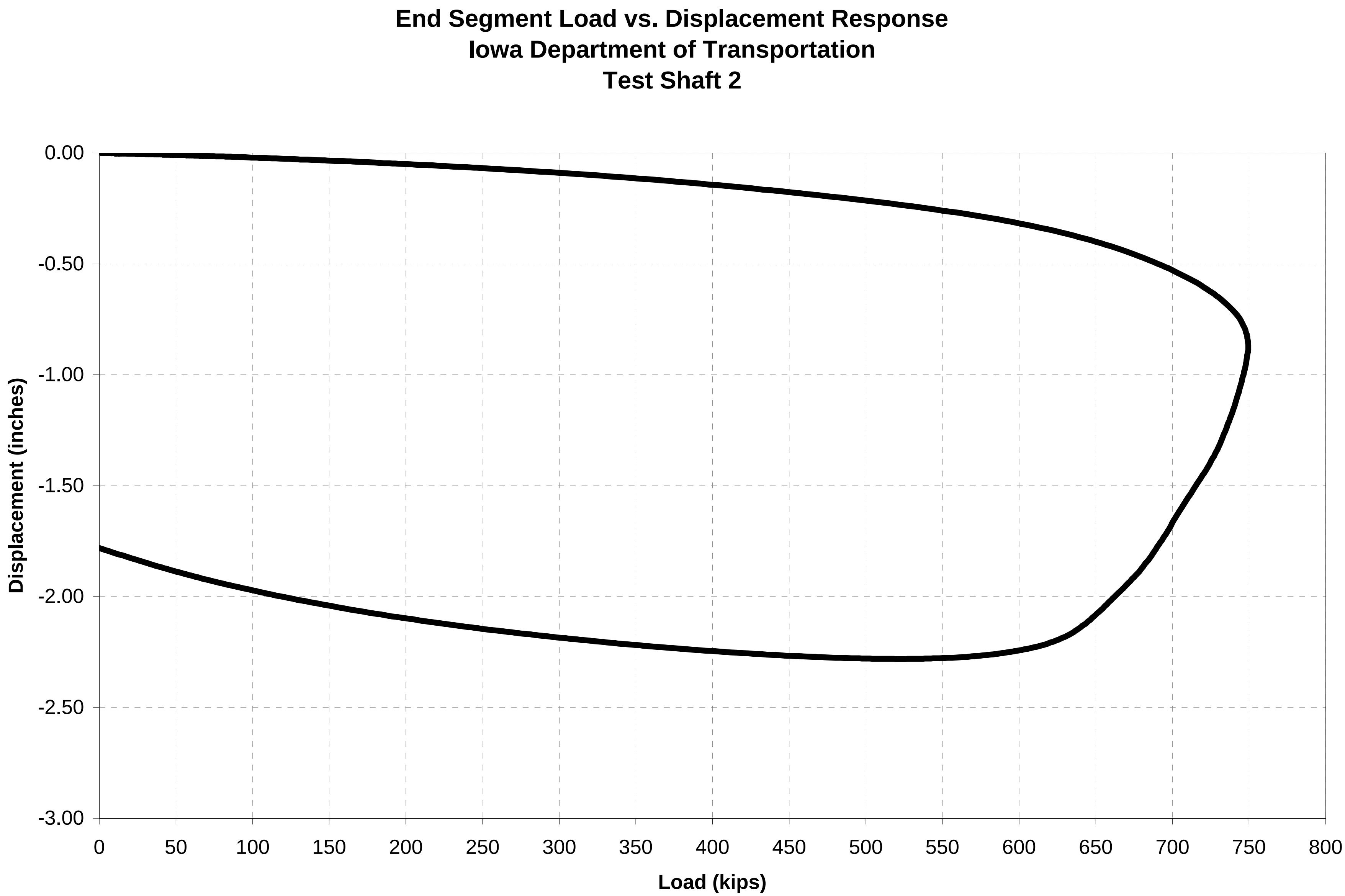
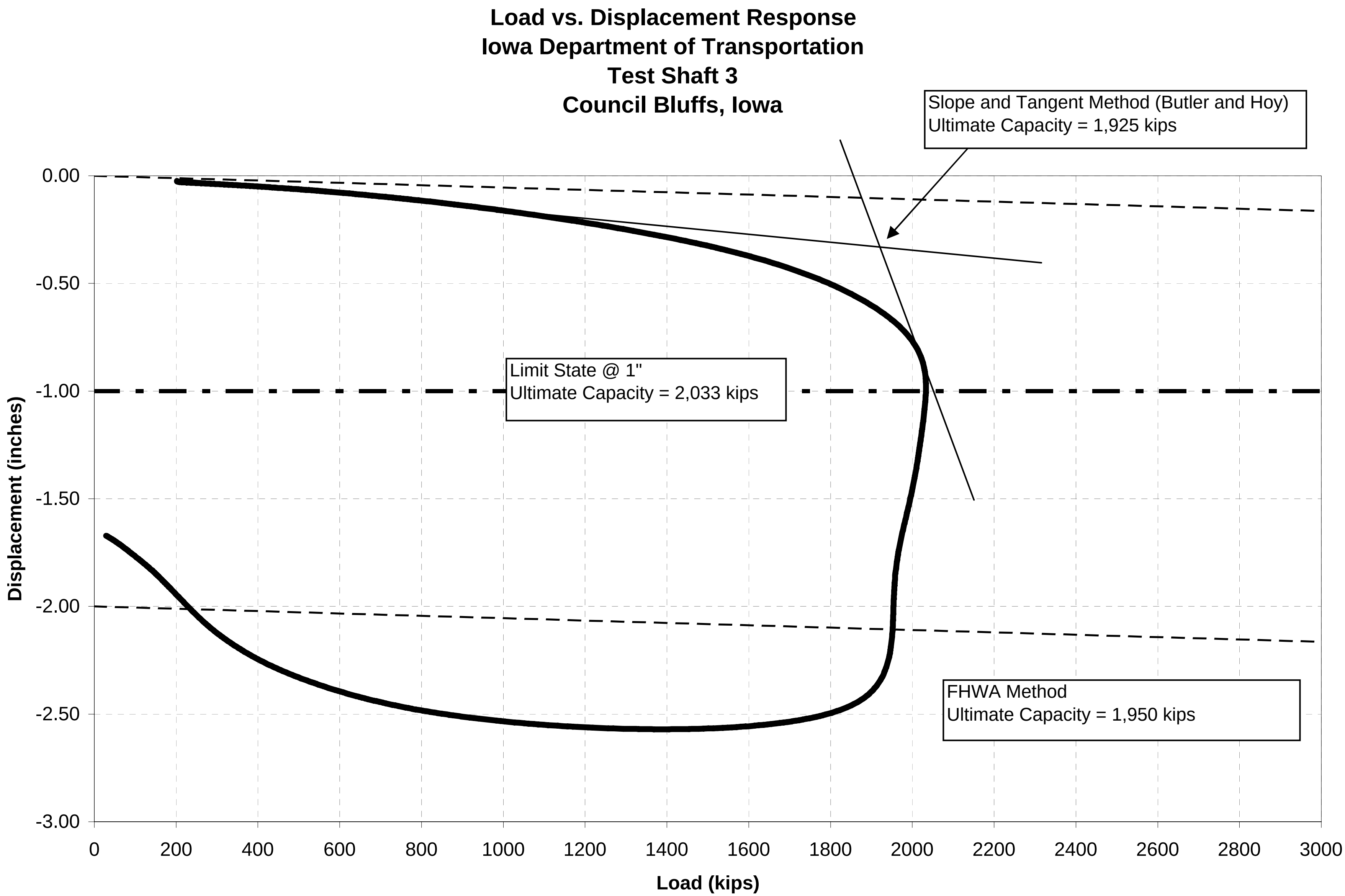

End Segment Load vs. Displacement Response
Iowa Department of Transportation
Test Shaft 2
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Displacement (inches)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
Load (kips)

Figure 10

Test Shaft 3


Load vs. Displacement Response
Iowa Department of Transportation
Test Shaft 3
Council Bluffs, Iowa
Slope and Tangent Method (Butler and Hoy)
Ultimate Capacity = 1,925 kips
Limit State @ 1"
Ultimate Capacity = 2,033 kips
FHWA Method
Ultimate Capacity = 1,950 kips
Displacement (inches)
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
Load (kips)

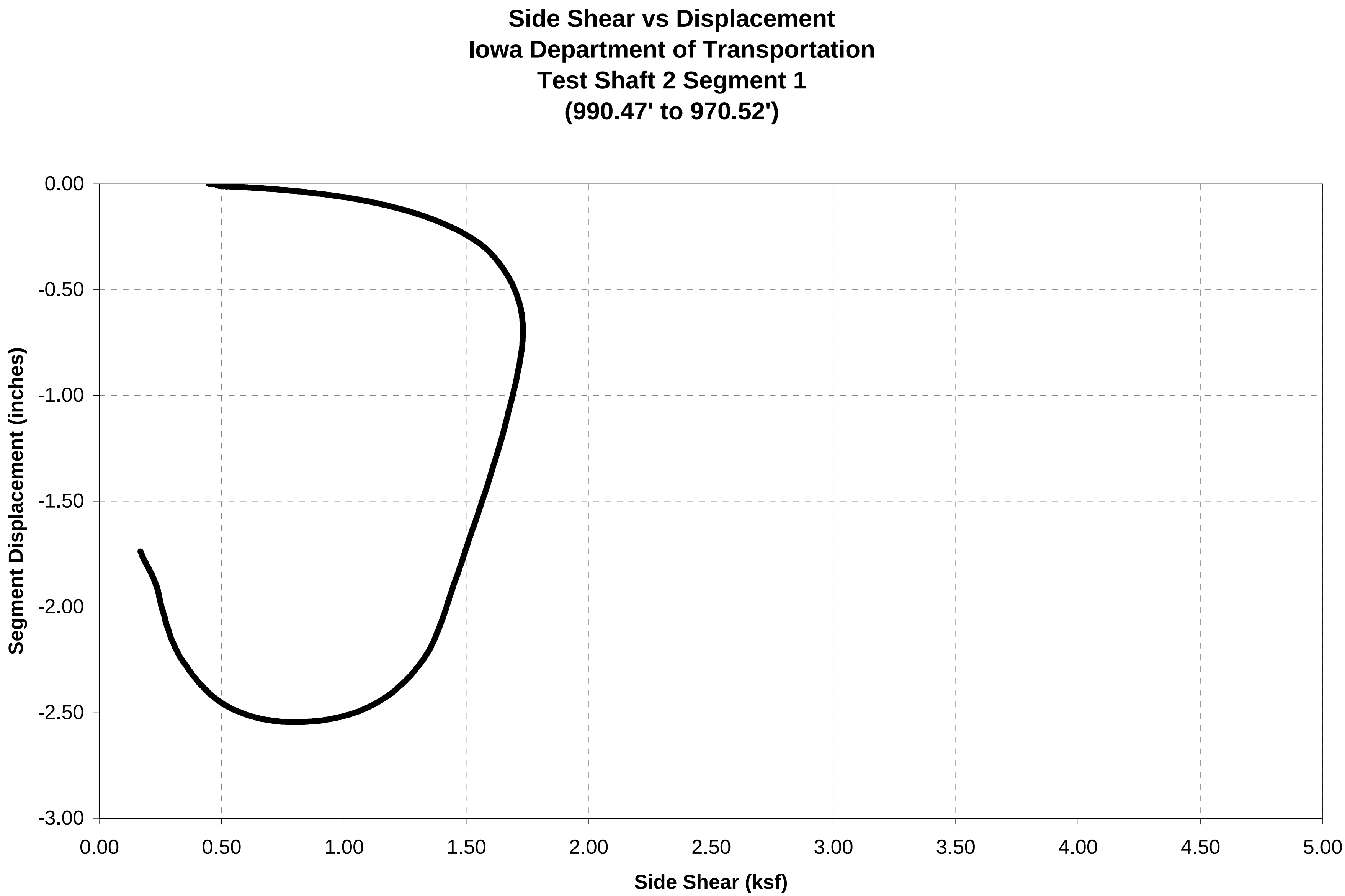


Figure 12

Applied Foundation Testing, Inc.

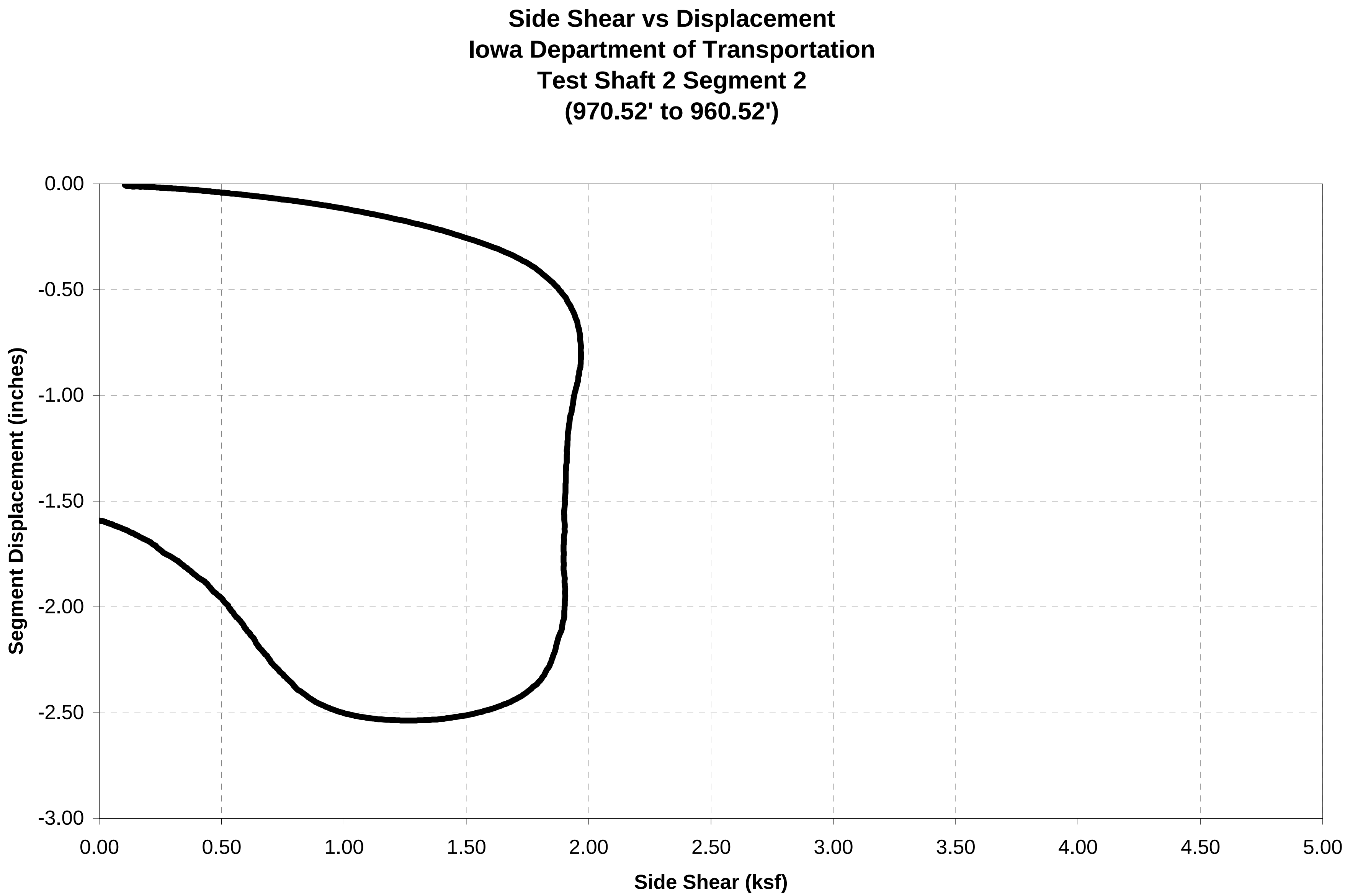
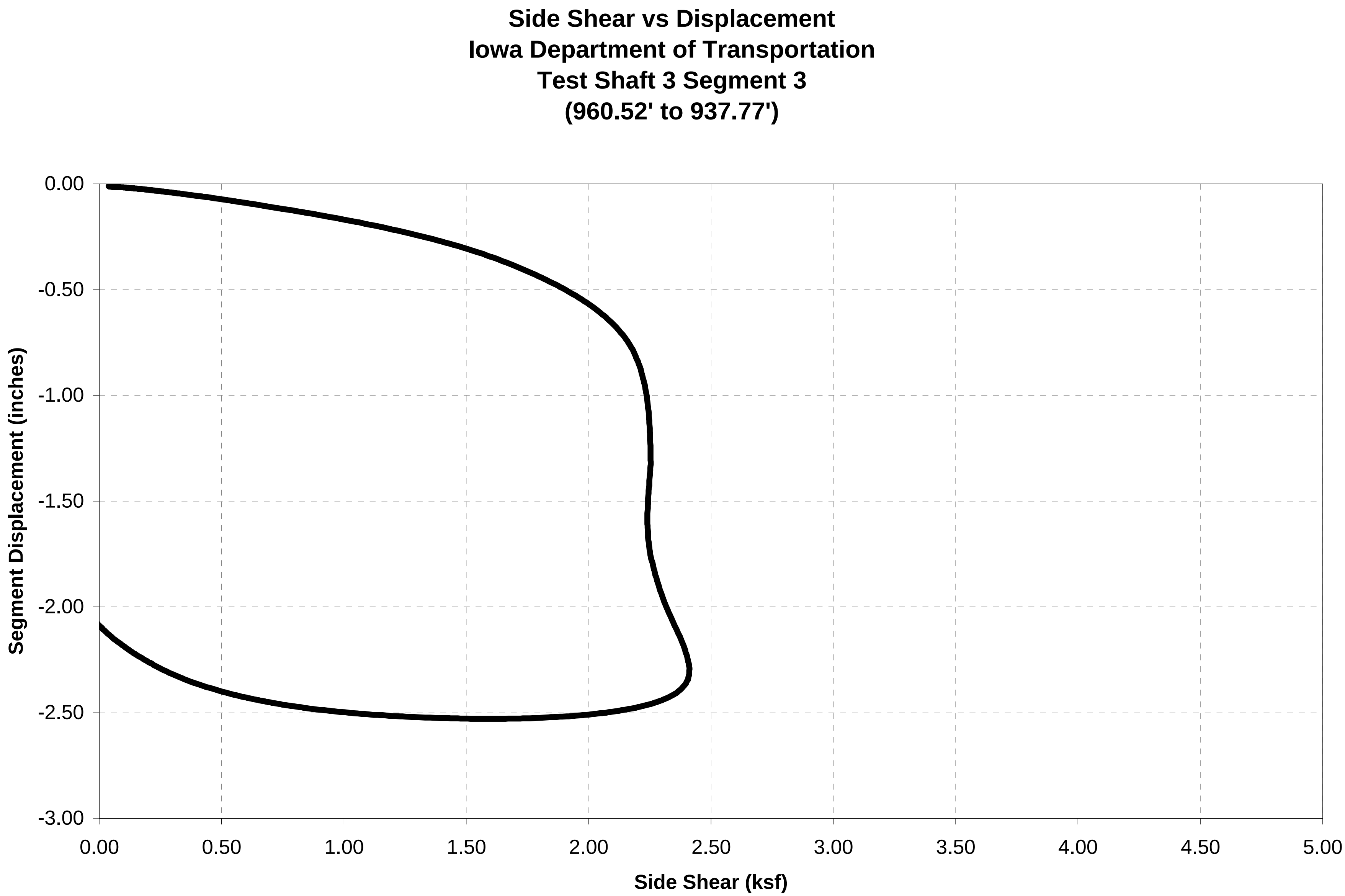

Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 2 Segment 2
(970.52' to 960.52')
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Segment Displacement (inches)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

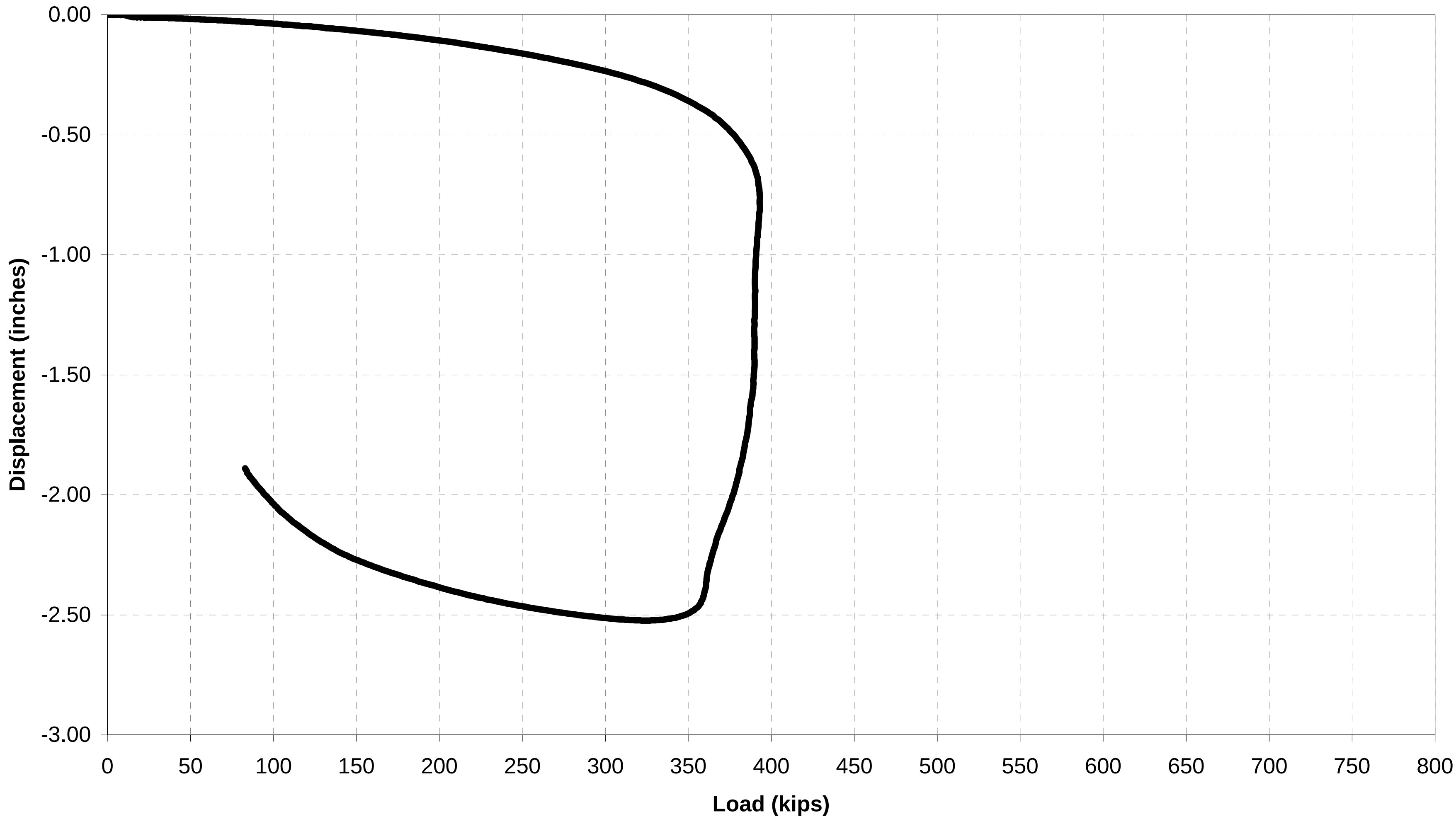
Figure 13



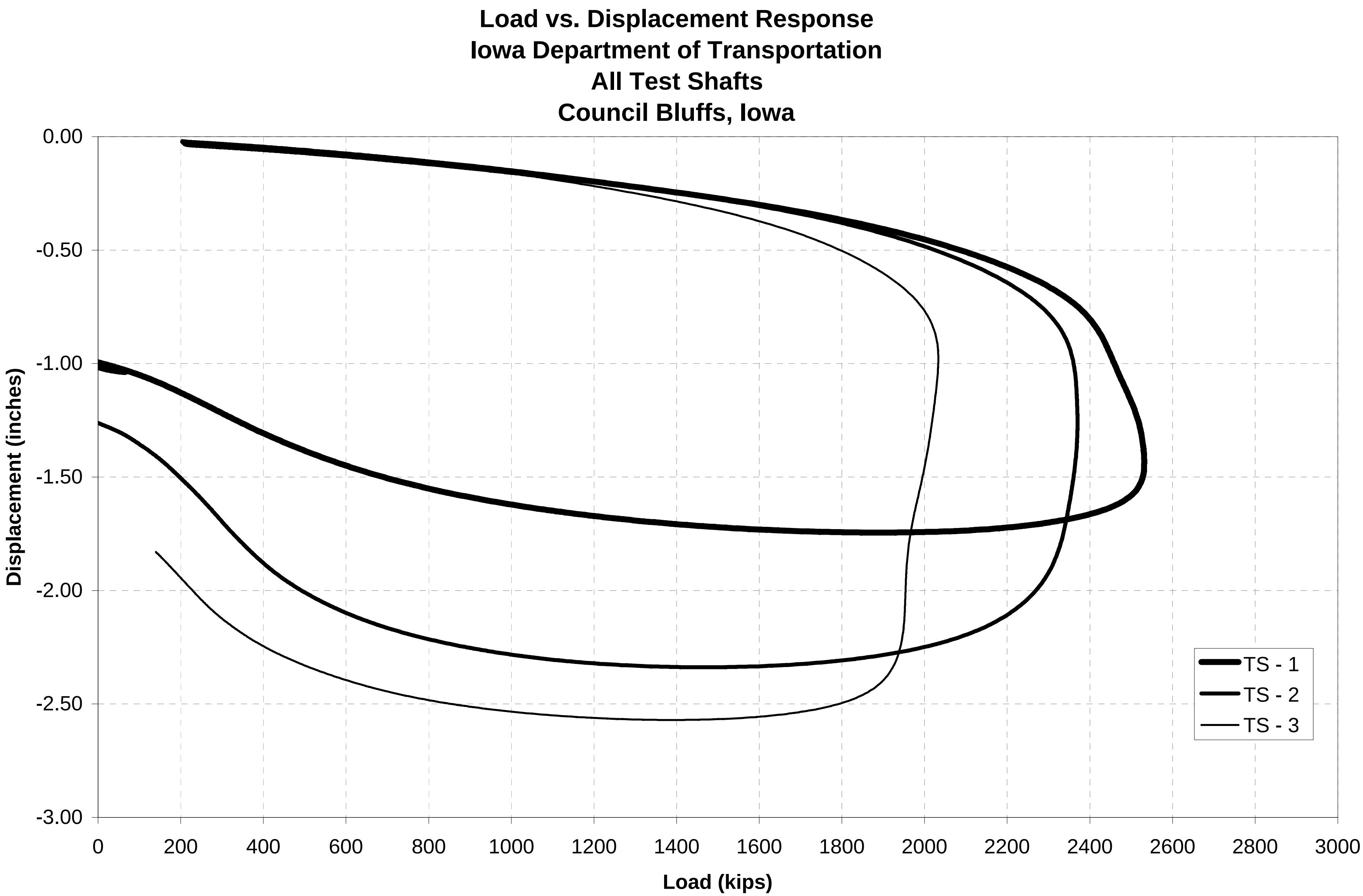
Side Shear vs Displacement
Iowa Department of Transportation
Test Shaft 3 Segment 3
(960.52' to 937.77')
Segment Displacement (inches)
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Side Shear (ksf)

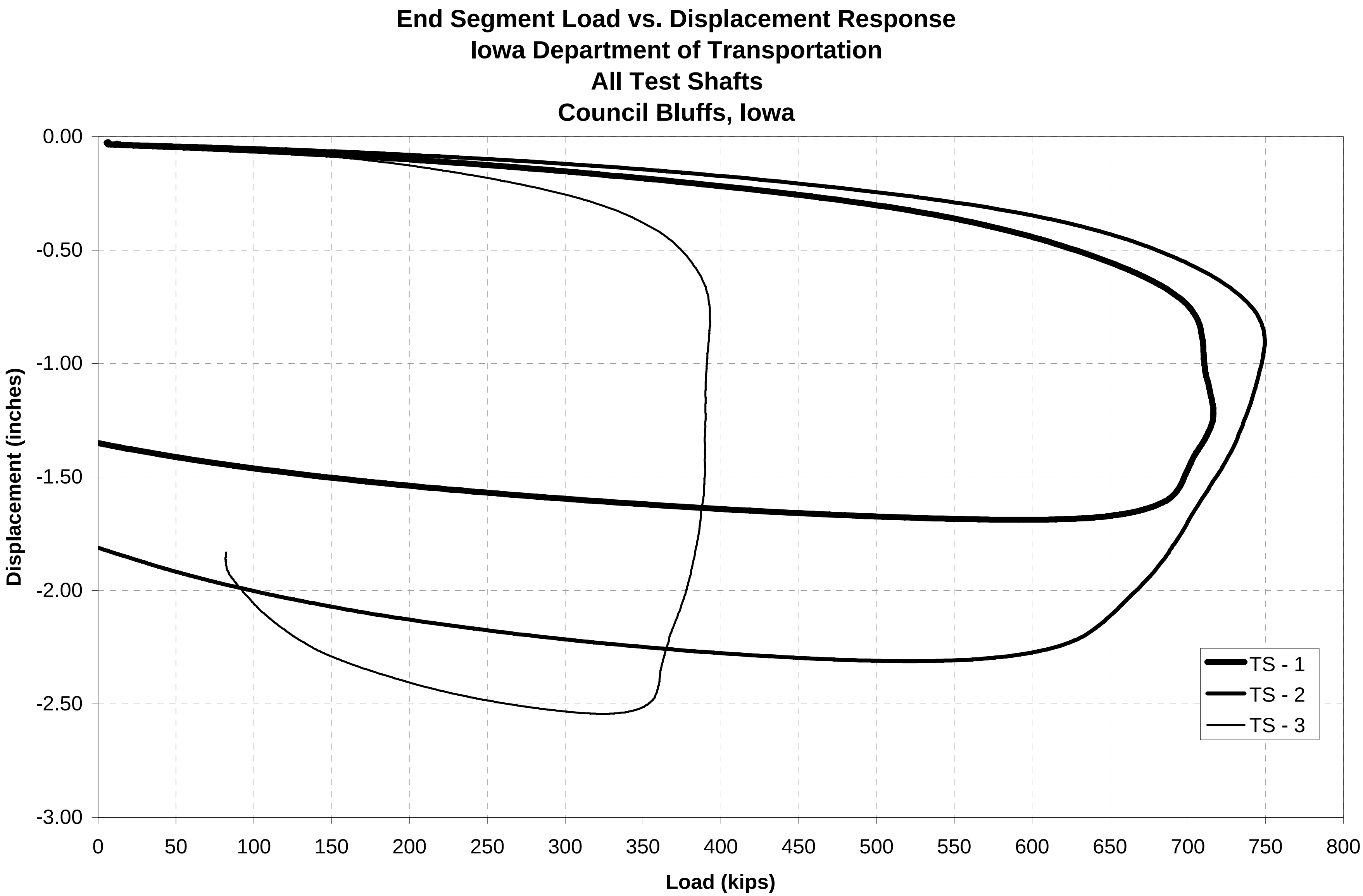
Figure 14

End Segment Load vs. Displacement Response
Iowa Department of Transportation
Test Shaft 3



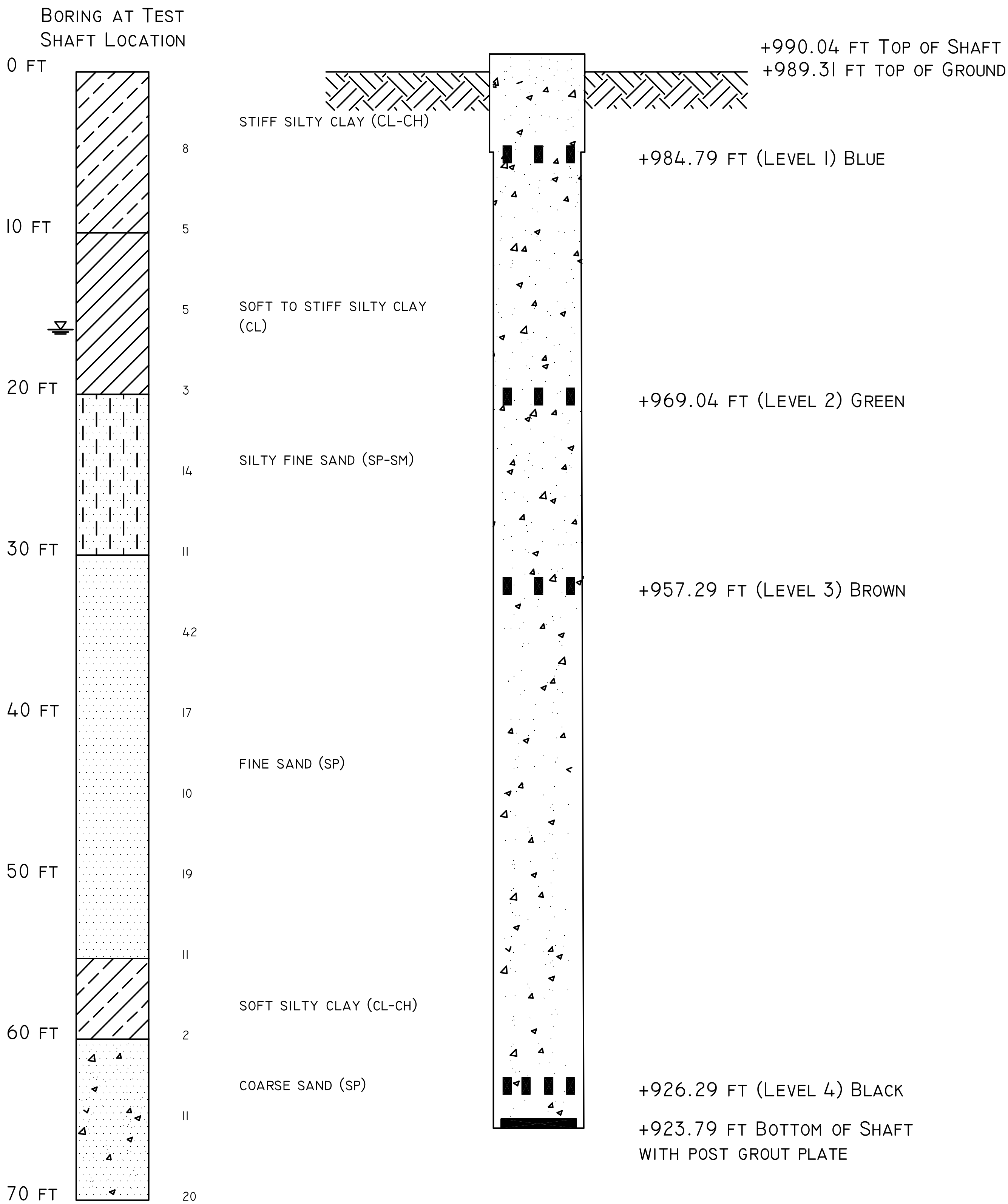
Comparison Figures


Load vs. Displacement Response
Iowa Department of Transportation
All Test Shafts
Council Bluffs, Iowa
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Displacement (inches)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
Load (kips)
TS - 1
TS - 2
TS - 3


End Segment Load vs. Displacement Response
Iowa Department of Transportation
All Test Shafts
Council Bluffs, Iowa
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Displacement (inches)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
Load (kips)
TS - 1
TS - 2
TS - 3

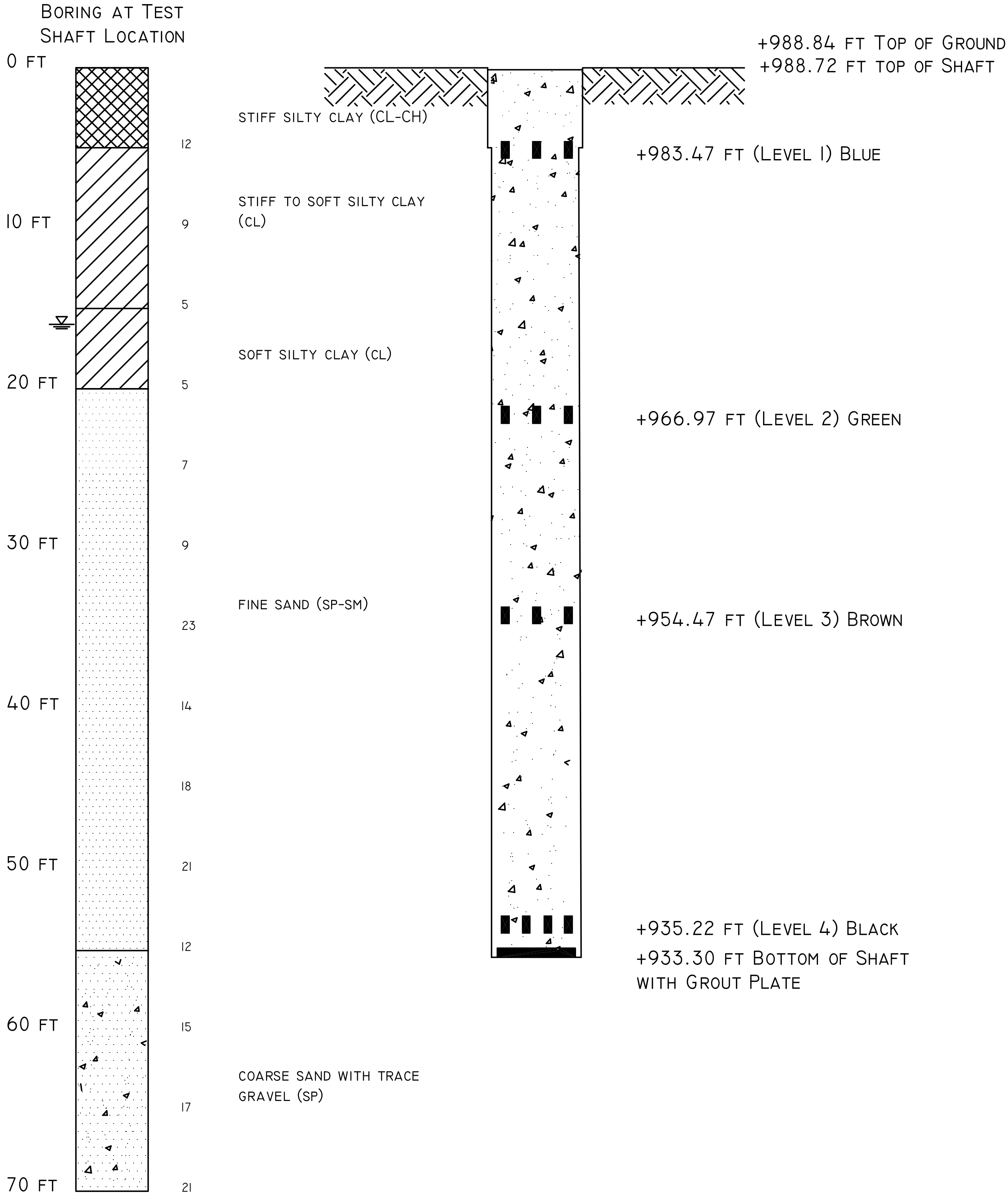
Appendix B

Iowa DOT Project TS-1
Schematic Drawing
Axial Statnamic Load Test



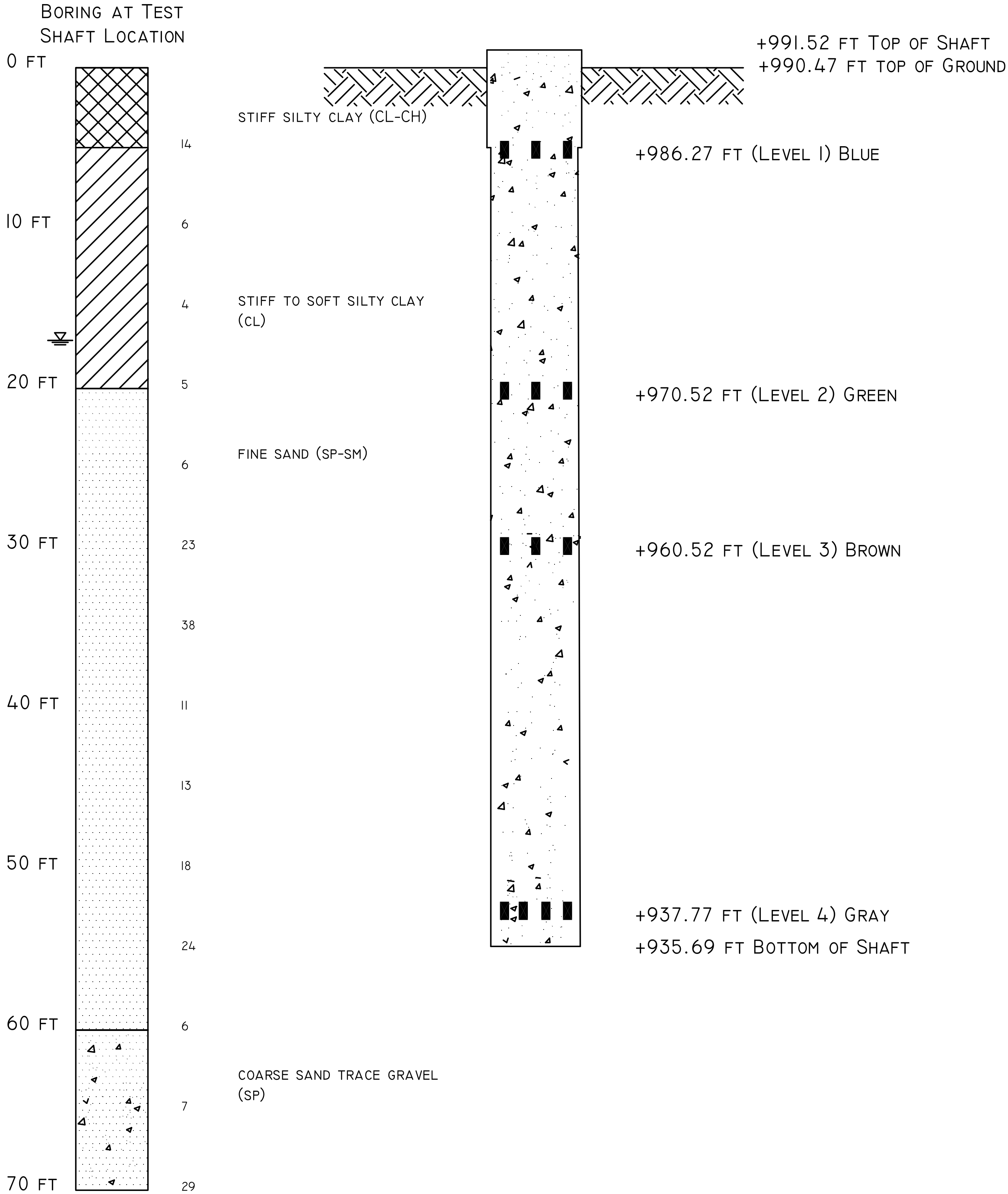
AFT	Applied Foundation Testing, Inc.	Iowa DOT TS-1 Council Bluffs, Iowa

Iowa DOT Project TS-2
Schematic Drawing
Axial Statnamic Load Test



AFT	Applied Foundation Testing, Inc.	Iowa DOT TS-2 Council Bluffs, Iowa

Iowa DOT Project TS-3
Schematic Drawing
Axial Statnamic Load Test

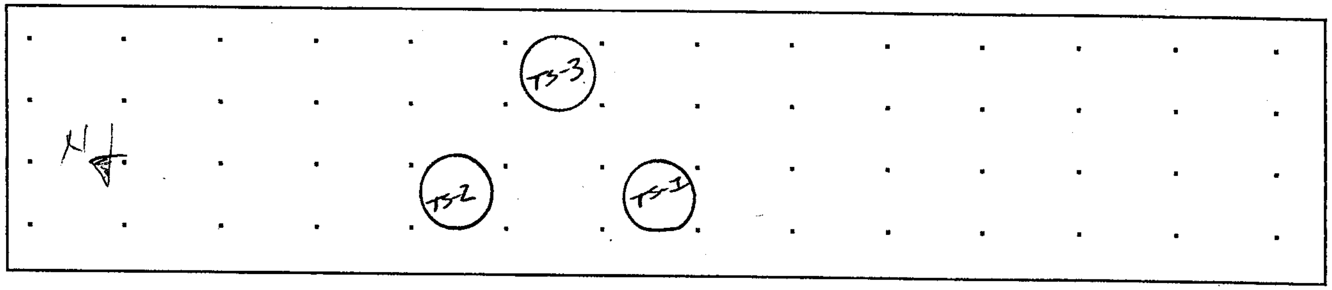


Applied Foundation Testing, Inc.

Iowa DOT TS-3
Council Bluffs, Iowa

DRILLED SHAFT FIELD REPORT

County POTTAWATTAMIE
Project # NHS-080-1(318)0--11-78
Design #
Prime Contractor
JENSEN
Drilled Shaft Contractor
LONGFELLOW

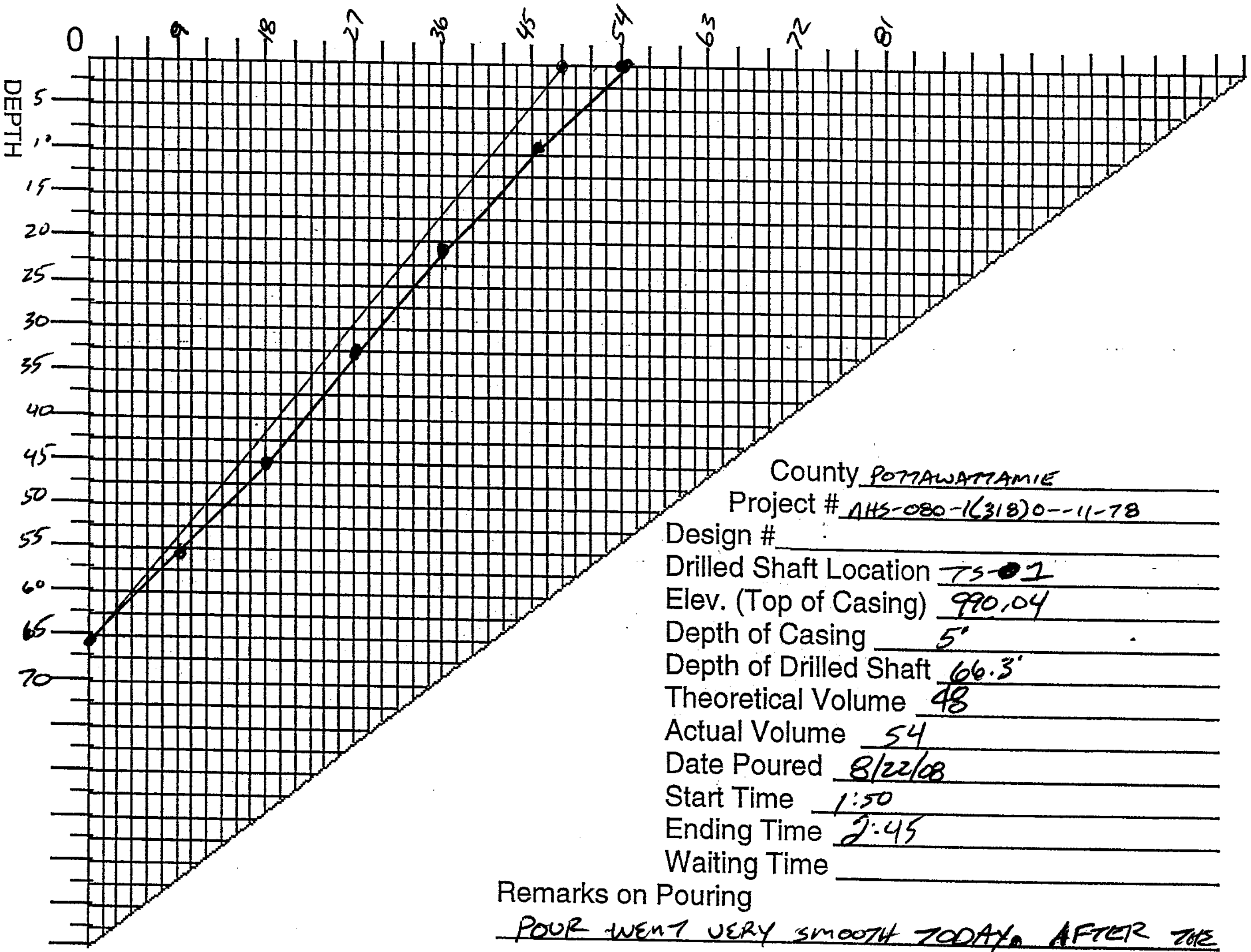


	Drilled Shaft Location	TS-3	TS-2	TS-1						
	Pouring Date	8/20/08	8/21/08	8/22/08						
DESIGN	Shaft Diameter	60"	60"	60"						
	Casing Diameter	— 60"	— 60"	— 60"						
	(Socket/Bell) Diameter	—	—	—						
	Top Elevation	990.47	989.84	990.31						
	Bottom Elevation	933.00	933.00	923.00						
AS BUILT	Cap Diameter									
	Shaft Diameter	60"	60"	60"						
	Casing Diameter	66"	66"	66"						
	Casing Length	5'	~~5~~ 4'	5'						
	(Socket/Bell) Diameter	60"	60"	60"						
	Top Elevation	55.9 991.52	55.4 988.72	990.04 66.3						
	Bottom Elevation	935.62	933.32	923.74						
	(Sand/Rock) Elevation	—	—	—						
	(Sand/Rock) Elevation	—	—	—						
	Bottom Casing Elev.	986.52	984.72	985.04						

PREPARED BY JASON KOHL APPROVED BY

DRILLED SHAFT CONCRETE REPORT

DEPTH	CYDS
66.3	0
~~56~~	4 1/4
~~45~~	18 1/4
32.5	27 1/2
~~21~~	36 3/4
9	46
0	54



County POTTAWATTAMIE
Project # NHS-080-1(318)0--11-78
Design # ____
Drilled Shaft Location TS-1
Elev. (Top of Casing) 990.04
Depth of Casing 5'
Depth of Drilled Shaft 66.3'
Theoretical Volume 48
Actual Volume 54
Date Poured 8/22/08
Start Time 1:50
Ending Time 2:45
Waiting Time ____

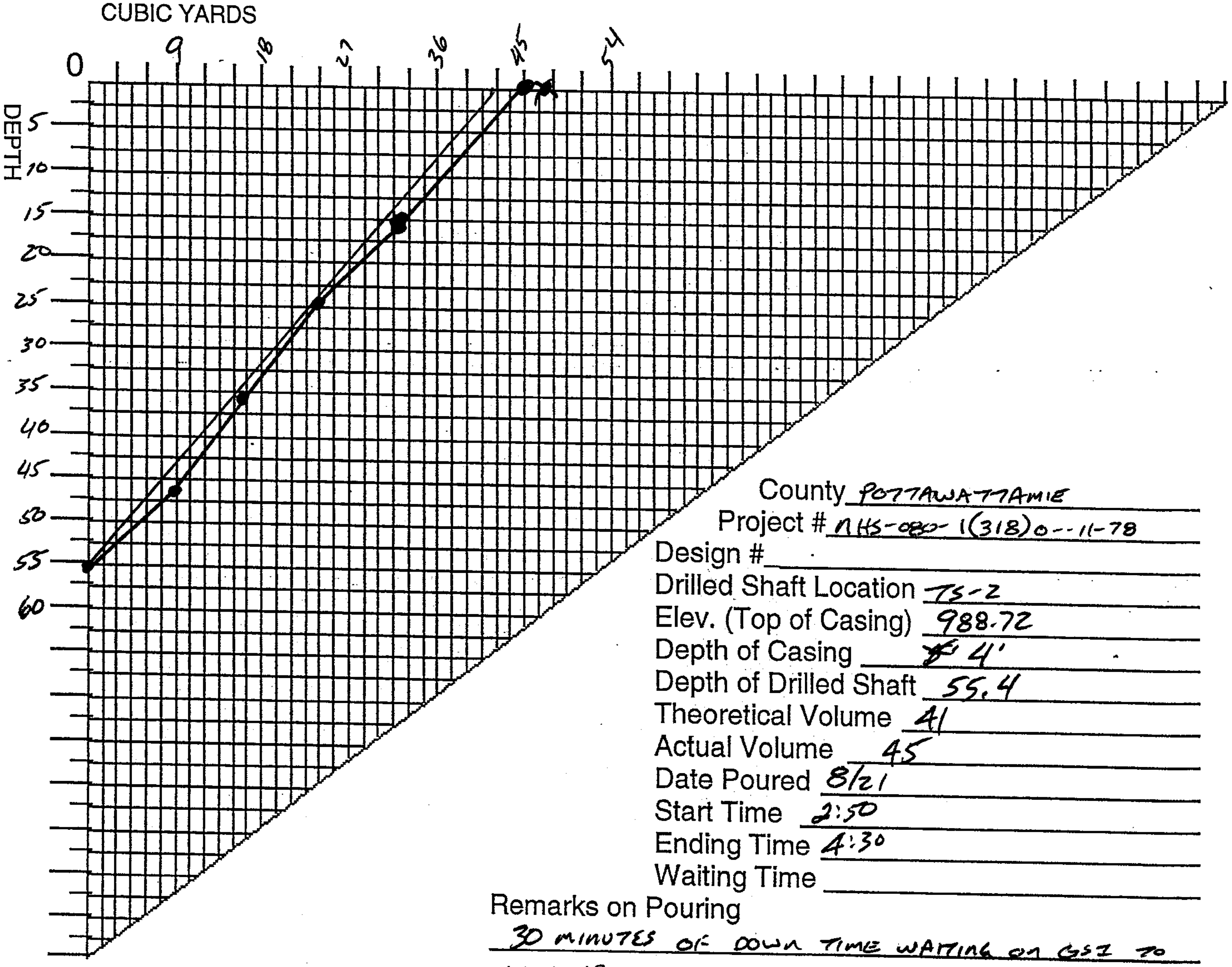
Remarks on Pouring

POUR WENT VERY SMOOTH TODAY. AFTER THE CONCRETE WAS PLACED JOHN w/ AFT DIRECTED LONGFELLOW TO CLEAN AND PRESSURIZE THE GROUT TUBES WHEN THE WATER ~~CLEANED~~ CLEARED UP. HAVING PRESSURE ON THE TUBES CAUSED WATER TO SEEP UP THRU THE SHAFT. WATER WAS IMMEDIATELY SHUT OFF WHEN THIS WAS NOTICED.

Prepared by JASON KOHL
Checked by ____

DRILLED SHAFT CONCRETE REPORT

DEPTH	CYDS
55.4	0
46.5	8
35	16
24	24
~~8~~16	32
~~40~~	40
0	~~8~~45



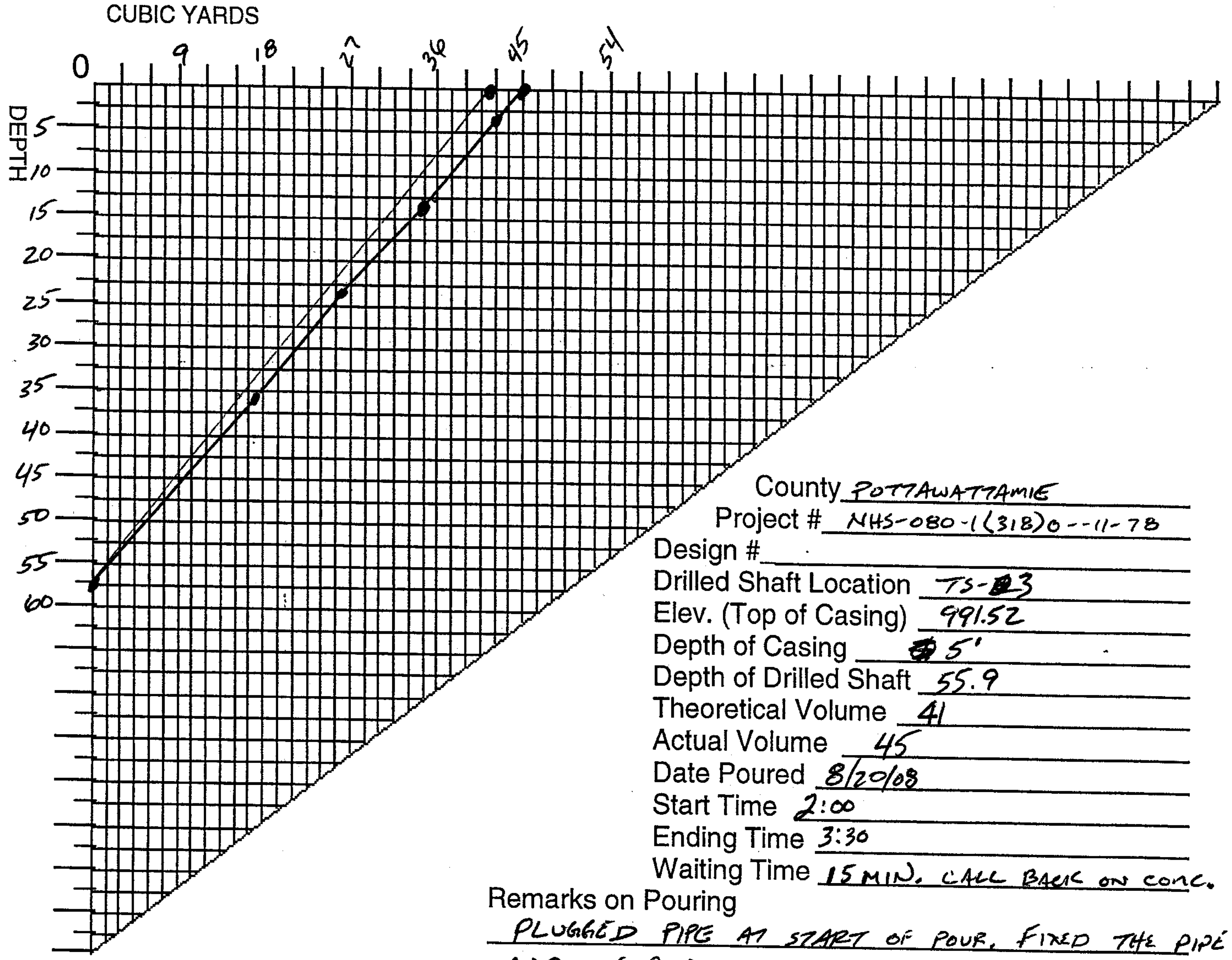
County POTTAWATTAMIE
Project # NHS-080-1(318)0--11-78
Design #
Drilled Shaft Location TS-2
Elev. (Top of Casing) 988.72
Depth of Casing ~~5~~ 4'
Depth of Drilled Shaft 55.4
Theoretical Volume 41
Actual Volume 45
Date Poured 8/21
Start Time 2:50
Ending Time 4:30
Waiting Time

Remarks on Pouring
30 MINUTES OF DOWN TIME WAITING ON GSI TO SHOW UP ON SITE TO TAKE TEST CYLINDERS. 10MIN. DOWN TO TAKE STRAPS OFF OF CABLE. AT THE START OF THE POUR THEY SHUT THE PUMP TRUCK OFF WHEN THEY NOTICED THE SLURRY PUMP WASN'T RUNNING. THERE WAS ONLY A SMALL AMOUNT OF CONCRETE IN THE HOLE AT THAT TIME, AND THE PUMP PIPE COULDN'T HAVE HAD MUCH EMBEDMENT IN THE CONC.

Prepared by JASON KOHL
Checked by

DRILLED SHAFT CONCRETE REPORT

DEPTH	CYDS
55.9	0
	9
36	17.5
23.5	26.0
13	34.5
3.3	43.0
0	45



County POTTAWATTAMIE
Project # NHS-080-1(318)0--11-78
Design #
Drilled Shaft Location TS-3
Elev. (Top of Casing) 991.52
Depth of Casing 5'
Depth of Drilled Shaft 55.9
Theoretical Volume 41
Actual Volume 45
Date Poured 8/20/08
Start Time 2:00
Ending Time 3:30
Waiting Time 15 MIN. CALL BACK ON CONC.
Remarks on Pouring
PLUGGED PIPE AT START OF POUR. FIXED THE PIPE AND THE POUR WENT VERY SMOOTH

Prepared by JASON KOHL
Checked by

Drilled Shaft Post Grout Field Record

Project Name: Broadway Bridge Load Test Project	
	Shaft Designation: TS-1 (Test Shaft)
Contractor: Longfellow Drilling	Post Grout Date: 8/29/08
Post Grouting By: Applied Foundation Testing, Inc.	AFT Grout Technician: Jason Frederic
	AFT Data Acquisition Engineer: Mike Muchard, P.E.

Post Grouting Information

Grout Plant Type: HANY IC 310	Grout Type: Type I Portland Cement
Pump Type: Single Stage Piston	Starting Water / Cement Ratio: 0.5 (+/- 0.05)
Mixer Type: Colloidal Mixing w/Agitator Holding	Yield: 36 Liters per bag (1.256 ft^3) @ 0.5 w/c ratio

Post Grouting Criteria

Maximum Permissible Displacement: 0.25 inch	Maximum Required Grout Pressure: 600 psi (41 bar)
Grout Volume Reset Value: Field Determined	

Post Grouting Data / Comments

starting w/c ratio = 0.5

Time	Grout Pressure		Upward Shaft Displacement		Grout Volume Tank Reading			Notes
	bar	psi	mm	inches	Inches			
12:47			131.0		10.0			Start pumping grout
12:53	8	118	131.00		20.5	to	8.00	Grout return - lock opposing valve
12:58	11	162	131.50		18.0	to	6.50	
1:02	12	176	131.75		14.5			
1:09	19	279	132.00		24.0	to	11.00	
1:13	21	309	132.00		22.0	to	10.00	
1:18	25	368	132.50		19.0	to	7.00	
1:23	30	441	132.50		18.0	to	6.00	
1:29	35	515	132.75		17.0	to	6.00	
1:31	30	441	132.75		12.0			
1:33	35	515	133.00		14.5			
1:38	38	559	133.25		20.5	to	9.00	
1:42	41	603	133.25		19.0	to	7.00	
1:46	45	662	133.25		11.0			Complete - DID NOT LOCK IN
								PRESSURE
	bar	psi	mm	inches	L		ft^3	
Maximums	**45**	**662**	**2.25**	**0.089**	**542.5**		**19.2**	

Conversions : Grout plant holding tank volume calibration = 1 inch in tank = .177 cubic feet grout

(1 cubic meter = 1.307 cubic yards = 35.3 cubic feet) (1 Bar = 100 kPa = 14.7 psi)

(28.32 Liters = 1 cubic foot) (3.785 Liters = 1 Gallon) (25.4 mm = 1 inch) (1 kg = 2.21 lbs)

Drilled Shaft Post Grout Field Record

Project Name: Broadway Bridge Load Test Project	
	Shaft Designation: TS-2 (Test Shaft)
Contractor: Longfellow Drilling	Post Grout Date: 8/29/08
Post Grouting By: Applied Foundation Testing, Inc.	AFT Grout Technician: Jason Frederic
	AFT Data Acquisition Engineer: Mike Muchard, P.E.

Post Grouting Information

Grout Plant Type: HANY IC 310	Grout Type: Type I Portland Cement
Pump Type: Single Stage Piston	Starting Water / Cement Ratio: 0.5 (+/- 0.05)
Mixer Type: Colloidal Mixing w/Agitator Holding	Yield: 36 Liters per bag (1.256 ft^3) @ 0.5 w/c ratio

Post Grouting Criteria

Maximum Permissible Displacement: 0.25 inch	Maximum Required Grout Pressure: 600 psi (41 bar)
Grout Volume Reset Value: Field Determined	

Post Grouting Data / Comments

starting w/c ratio = 0.5

	Grout Pressure		Upward Shaft Displacement		Grout Volume Tank Reading			
Time	bar	psi	mm	inches	Inches			Notes
9:48			45.0		10.5			Start pumping grout
9:53					19.0	to	7.50	Grout return - lock opposing valve
9:55	8	118	45.25		11.0			
9:58	12	176	45.50		15.5			
10:02	13	191	45.75		21.0	to	9.00	
10:05	14	206	46.00		14.5			
10:08	15	221	46.00		19.0	to	6.00	
10:12	15	221	46.00		13.0			
10:16	16	235	46.00		19.0	to	6.00	
10:20	18	265	46.25		10.5			
10:25	19	279	46.25		23.5	to	11.00	
10:29	21	309	46.50		19.0	to	6.00	Reduce w/c ratio to 0.48
10:36	24	353	46.50		16.0			
10:41	23	338	46.75		25.0	to	13.50	
10:46	26	382	47.00		22.0	to	9.50	
10:51	27	397	47.00		16.5			
10:54	32	470	47.00		22.0	to	11.00	Reduce w/c ratio to 0.45
11:00	37	544	47.25		23.0	to	11.00	
11:05	45	662	47.25		18.0			Complete - lock off ball valve
	bar	psi	mm	inches	L		ft^3	
Maximums	**45**	**662**	**2.25**	**0.089**	**647.5**		**22.9**	

Conversions : Grout plant holding tank volume calibration = 1 inch in tank = .177 cubic feet grout

(1 cubic meter = 1.307 cubic yards = 35.3 cubic feet) (1 Bar = 100 kPa = 14.7 psi)

(28.32 Liters = 1 cubic foot) (3.785 Liters = 1 Gallon) (25.4 mm = 1 inch) (1 kg = 2.21 lbs)

BORING LOG No. TS 1

BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 1	TS1 Stake	987.65	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	16			3 1/4" Hollow Stem Auger with Drilling Mud	70'

DEP. FT.	SAMPLE DATA: SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY / GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	LABORATORY DATA: % MC	DRY DENS. pcf	Qu psf	DEP. FT.
				Gray brown, Moist, STIFF SILTY CLAY					
5	S-1	8	90		CL-CH	29.2			5
10	S-2	5	90	ALLUVIUM 10.0'		13.1			10
				Light gray, Very moist, SOFT to STIFF SILTY CLAY					
15	S-3	5	95		CL	32.3			15
20	S-4	3	85	ALLUVIUM 20.0'		29.9			20
				Gray brown, Wet, SILTY FINE SAND					
25	S-5	14	85		SP-SM	24.0			25
30	S-6	11	90	ALLUVIUM 30.0'		23.3			30
				Gray, Wet, FINE SAND					
35	S-7	42	90			22.1			35
	S-8	17	85			24.7			

GSI Geotechnical Services, Inc.
2853 99th Street, Des Moines, IA 50322
(515) 270-6542 FAX (515) 270-1911

PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-23-08

BORING LOG No. TS 1

BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 1	TS1 Stake	987.65	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	16			3 1/4" Hollow Stem Auger with Drilling Mud	70'

DEP. FT.	SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY; GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	% MC	DRY DENS. pcf	Qu psf	DEP. FT.
					SP				
45	S-9	10	90			28.4			45
50	S-10	19	85			29.5			50
55	S-11	11	88	ALLUVIUM —55.0'		26.2			55
				Light gray, Very moist, SOFT SILTY CLAY	CL-CH				
60	S-12	2	90	ALLUVIUM —60.0'		52.9			60
				Dark gray, Wet, COARSE SAND					
65	S-13	11	85		SP				65
70	S-14	20	90	ALLUVIUM —70.0'		14.9			70
				Bottom of Boring @ 70'					
75									75

GSI Geotechnical Services, Inc.
2853 99th Street, Des Moines, IA 50322
(515) 270-6542 FAX (515) 270-1911

PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-23-08

BORING LOG No. TS 2

BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 2	TS2 Stake	987.42	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	16			3 1/4" Hollow Stem Auger with Drilling Mud	70'

DEP. FT.	SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY / GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	% MC	DRY DENS. pcf	Qu psf	DEP FT.
5	S-1	12	90	Dark brown and gray mixed, Moist, STIFF SILTY CLAY traces of brick and crushed rock; FILL 5.0'	CL-CH	22.3			5
10	S-2	9	95	Dark brown to gray, Very moist STIFF to SOFT SILTY CLAY	CL	34.8			10
15	S-3	5	90	ALLUVIUM 15.0'		27.8			15
20	S-4	5	90	Gray to light gray, Very moist, SOFT SILTY CLAY trace sand; ALLUVIUM 20.0'	CL	35.1			20
25	S-5	7	85	Gray brown to gray, Wet, FINE SAND		22.3			25
30	S-6	9	95			24.4			30
35	S-7	23	90		SP-SM	25.4			35
	S-8	14	90			24.9			

GSI Geotechnical Services, Inc.
2853 99th Street, Des Moines, IA 50322
(515) 270-6542 FAX (515) 270-1911

PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-24-08

BORING LOG No. TS 2

BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 2	TS2 Stake	987.42	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	18			3 1/4" Hollow Stem Auger with Drilling Mud	70'

DEP. FT.	SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY / GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	% MC	DRY DENS. pcf	Qu psf	DEP. FT.
45	S-9	18	90			26.0			45
50	S-10	21	95			20.9			50
55	S-11	12	85	ALLUVIUM 55.0'		24.5			55
				Dark gray, Wet, COARSE SAND trace gravel					
60	S-12	15	85			25.9			60
					SP				
65	S-13	17	85			19.9			65
70	S-14	21	80	ALLUVIUM 70.0'		20.8			70
				Bottom of Boring @ 70'					
75									75

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PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-24-08

BORING LOG No. TS 3

BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 3	TS2 Stake	987.84	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	17			3 1/4" Hollow Stem Auger with Drilling Mud	80'

DEP. FT.	SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY; GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	% MC	DRY DENS. pcf	Qu psf	DEP. FT.
				Dark brown and gray mixed, Moist, STIFF SILTY CLAY traces of brick and crushed rock	CL-CH				
	S-1	14	85	FILL		18.5			
5.5				5.0'					5.5
				Gray brown to gray, Very moist STIFF to SOFT SILTY CLAY					
	S-2	6	90			26.7			
11					CL				11
	S-3	4	85			34.7			
16.5									16.5
	S-4	5	85	ALLUVIUM		38.5			
				20.0'					
				Gray brown to gray, Wet, FINE SAND					
22									22
	S-5	6	90			24.4			
27.5									27.5
	S-6	23	95			24.0			
33									33
	S-7	38	85			25.7			
38.5									38.5
	S-8	11	90		SP-SM	29.1			
				ALLUVIUM					

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(515) 270-6542 FAX (515) 270-1911

PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-24-08

BORING LOG No. TS 3

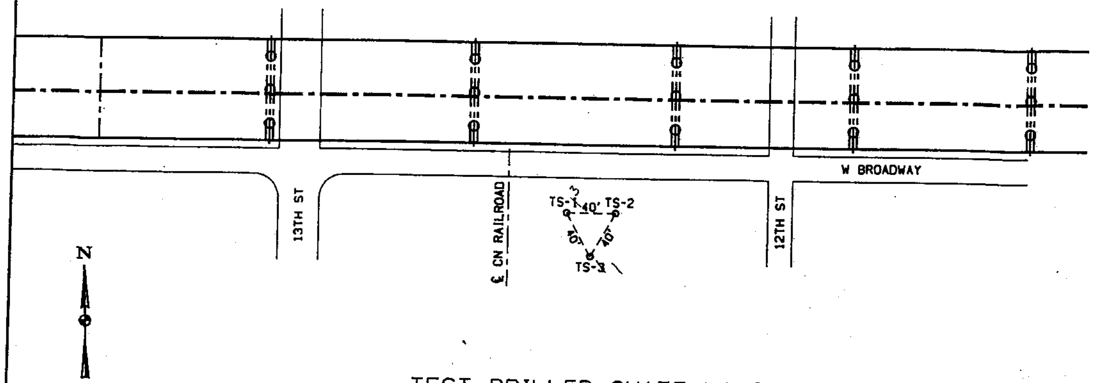
BORING NO.	LOCATION OF BORING	ELEVATION	DATUM	DRILLER	LOGGER
TS 3	TS2 Stake	987.84	IDOT Plan Sheet	DAH	JLW

WATER LEVEL OBSERVATIONS				TYPE OF SURFACE	DRILL RIG
WHILE DRILLING	END OF DRILLING	24 HOURS AFTER DRILLING		Grass and Weeds	B-57
				DRILLING METHOD	TOTAL DEPTH
20	17			3 1/4" Hollow Stem Auger with Drilling Mud	80'

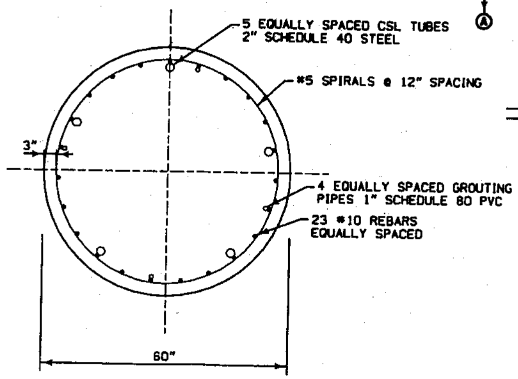
DEP. FT.	SAMPLE NO. & TYPE	"N" BLOWS (FT)	% REC.	SOIL DESCRIPTION: COLOR, MOISTURE, CONSISTENCY / GEOLOGIC DESCRIPTION & OTHER REMARKS	USCS CLASS.	% MC	DRY DENS. pcf	Qu psf	DEP. FT.
	S-9	13	90	Gray brown to gray, Wet, FINE SAND		30.1			
49.5	S-10	18	90			26.7			49.5
55	S-11	24	90			29.6			55
	S-12	6	85	ALLUVIUM		33.8			
60.5				60.0' — Dark gray, Wet, COARSE SAND trace gravel					60.5
	S-13	7	85			21.2			
66									66
	S-14	29	85		SP	13.3			
71.5									71.5
	S-15	10	85			20.6			
77									77
	S-16	14	85	ALLUVIUM — 80.0'		16.6			
				Bottom of Boring @ 80'					
82.5									82.5

GSI Geotechnical Services, Inc.
2853 99th Street, Des Moines, IA 50322
(515) 270-6542 FAX (515) 270-1911

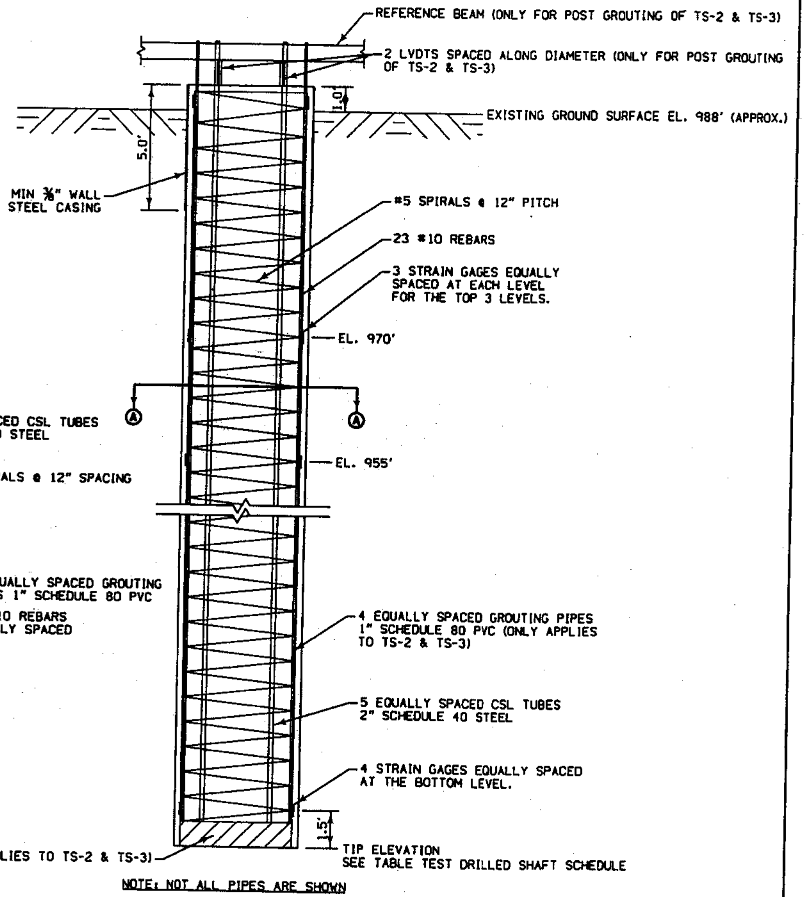
PROJECT: Drilled Shaft Load Tests
LOCATION: 12th Street & Broadway, Council Bluffs, IA
JOB NO.: 086103
DATE: 6-24-08



TEST DRILLED SHAFT LAYOUT



SECTION A



NOTE: NOT ALL PIPES ARE SHOWN

TEST DRILLED SHAFT DETAIL (NTS)

TEST DRILLED SHAFT SCHEDULE

TEST SHAFT NO.	DIA. (FT)	TIP ELEVATION (FT) (APPROX.)	POST GROUTING	NORTHING	EASTING	SURFACE ELEVATION (FT)
TS-1	5	933	NO	468477.17	991002.80	987.65
TS-2	5	933	END GROUTING	468475.75	991042.78	987.42
TS-3	5	923	END GROUTING	468441.84	991021.56	987.84

990.47

CONSTRUCTION SEQUENCE:

1.
THE CONTRACTOR SHALL PHYSICALLY LOCATE ALL UNDERGROUND UTILITIES INCLUDING FIBER OPTIC LINES PRIOR TO STARTING WORK. IF THERE IS A CONFLICT BETWEEN A TEST DRILLED SHAFT LOCATION AND AN EXISTING UTILITY (IES), RELOCATE THE TEST SHAFT AS DIRECTED BY THE ENGINEER. THE MINIMUM CENTER TO CENTER DISTANCE BETWEEN TEST SHAFTS SHALL BE 40 FEET. THE CONTRACTOR SHALL CONDUCT THE WORK IN SUCH A MANNER TO ENSURE THAT UTILITIES WILL NOT REQUIRE RELOCATION OR WILL NOT BE DISTURBED .

2.
CONFIRMATION BORINGS SHALL BE COMPLETED AT THE TEST SHAFT LOCATIONS TO A DEPTH 15 FEET BELOW THE TIP OF THE THREE TEST SHAFTS PRIOR TO TEST SHAFT INSTALLATION. THE DRILLED SHAFT TIP ELEVATION MAY BE ADJUSTED BY ENGINEER BASED ON THE CONFIRMATION BORING LOGS.

3.
INSTALL DRILLED SHAFT WITH INSTRUMENTATIONS AND GROUT CELL UNDER THE INSTRUCTIONS OF POST GROUTING SPECIALTY CONTRACTOR AND STATNAMIC LOAD TEST SPECIALTY CONTRACTOR. THE TEST DRILLED SHAFT SHALL BE INSTALLED ACCORDING TO THE IOWA DOT SPECIAL PROVISION FOR TEST DRILLED SHAFT.

4.
PERFORM CROSSHOLE SONIC LOGGING (CSL) TESTS ON ALL THREE TEST SHAFTS PRIOR TO POST PRESSURE GROUTING.

5.
PERFORM POST GROUTING OF TEST DRILLED SHAFTS TS-2 AND TS-3. POST GROUTING SHALL BE PERFORMED BY THE SPECIFIED POST GROUTING SPECIALTY CONTRACTOR.

6.
PREPARE THE DRILLED SHAFT TOP SURFACE AS REQUIRED FOR STATNAMIC LOAD TEST AND PERFORM STATNAMIC LOAD TEST. STATNAMIC LOAD TEST SHALL BE PERFORMED BY THE SPECIFIED STATNAMIC LOAD TEST CONTRACTOR.

LAP LENGTH TABLE	
BAR	REQUIRED LAP LENGTH
#5	2'-6"
#10	4'-10"

* The contractor may substitute mechanical rebar splicers for lap splices. No additional compensation resulting from this substitution will be provided. Mechanical splicers shall be in accordance with IM 451, Appendix E.

DESIGN FOR
DRILLED SHAFT LOAD TEST

TEST DRILLED SHAFT

SEPTEMBER 2007

POTTAWATTAMIE COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION
DESIGN SHEET NO. 2 OF 2 FILE NO. 30168 DESIGN NO. 808

DESIGN TEAM IOWA DOT\CH2M HILL

11-OCT-2007 11:51 nbucking C:\workdir - Broadway Viaduct-185034\Test Drilled Shaft.dgn

POTTAWATTAMIE COUNTY | PROJECT NUMBER | NHS-80-K3180--11-78 | SHEET NUMBER DSLT.02

ESTIMATED QUANTITIES

ITEM NO.	ITEM CODE	ITEM	UNIT	TOTAL	AS BUILT QUANTITY
1	2404-7775000	REINFORCING STEEL	LB	22,000	
2	2533-4980005	MOBILIZATION	LS	1	
3	2599-9999005	POST PRESSURE GROUTING	EACH	2	
4	2599-9999005	STATNAMIC LOAD TEST	EACH	3	
5	2599-9999009	TEST DRILLED SHAFT	LF	178	

ITEM NO. ESTIMATE REFERENCE INFORMATION

2. INCLUDES THE COST ASSOCIATED WITH THE MOBILIZATION AND DEMOBILIZATION OF ANY EQUIPMENTS NECESSARY FOR INSTALLING THE TEST DRILLED SHAFTS.

3. INCLUDES THE COST OF ALL LABOR, MATERIALS, AND EQUIPMENT ASSOCIATED WITH POST GROUTING AND FINAL REPORTS IN ACCORDANCE WITH SPECIAL PROVISION, POST PRESSURE GROUTING. INCLUDES THE COST OF MOBILIZATION AND DEMOBILIZATION OF EQUIPMENTS FOR POST PRESSURE GROUTING.

4. INCLUDES THE COST OF ALL LABOR, MATERIALS, AND EQUIPMENT ASSOCIATED WITH THE INSTRUMENTATION OF THE TEST DRILLED SHAFTS AND PERFORMING AND REPORTING THE STATNAMIC LOAD TEST IN ACCORDANCE WITH SPECIAL PROVISION, STATNAMIC LOAD TEST. INCLUDES THE COST OF MOBILIZATION AND DEMOBILIZATION OF EQUIPMENTS FOR STATNAMIC LOAD TEST.

5. INCLUDES ALL LABOR, MATERIALS, AND EQUIPMENT NECESSARY TO INSTALL DRILLED SHAFTS IN ACCORDANCE WITH SPECIAL PROVISION, TEST DRILLED SHAFT AND SUPPLEMENTAL SPECIFICATION 01032, CONCRETE DRILLED SHAFT. INCLUDES THE COST OF ALL LABOR, MATERIALS, AND EQUIPMENT TO RESTORE LOAD TEST SITE TO ITS ORIGINAL CONDITION.

SPECIFICATIONS:

CONSTRUCTION: IOWA DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR HIGHWAY AND BRIDGE CONSTRUCTION, SERIES OF 2001, PLUS APPLICABLE SUPPLEMENTAL SPECIFICATIONS AND SPECIAL PROVISIONS FOR, TEST DRILLED SHAFT, POST PRESSURE GROUTING AND STATNAMIC LOAD TEST.

DESIGN STRESSES:

DESIGN STRESSES FOR THE FOLLOWING MATERIALS ARE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGE, 17TH EDITION, 2002.

CONCRETE, FC' = 4000 PSI.

REINFORCING STEEL, ASTM A615 GRADE 60.

STEEL CASING, ASTM A252, GRADE 2.

GEOTECHNICAL DESIGN

LICENSED PROFESSIONAL ENGINEER Emad Farouz 15897 IOWA

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of Iowa.

Emad Farouz 10/29/07
Signature Date

Emad Farouz
Printed or Typed Name

My license renewal date is December 31, 20 08

Pages or sheets covered by this seal: DSLT.01-DSLT.02

DESIGN FOR
DRILLED SHAFT LOAD TEST

QUANTITIES

SEPTEMBER 2007

POTTAWATTAMIE COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION

DESIGN SHEET NO. 1 OF 2 FILE NO. 30168 DESIGN NO. 806

DESIGN TEAM IOWA DOT\CH2M HILL | POTTAWATTAMIE COUNTY | PROJECT NUMBER NHS-80-K3(81)--1I-78 | SHEET NUMBER DSLT.01

29-OCT-2007 16:45 nbucking C:\workdir - Broadway Viaduct-185034\Test_Drilled_Shaft.dgn

Appendix C

GEORGE KELK CORPORATION

LOAD CELL CALIBRATION REPORT

Model No.:	C3929		Work Order No:	JR8883
Serial No.:	15	Bridge 1 of 1	Test date:	01-10-2008
Capacity:	1800000	kg	Test by:	WP
Calibration facility:	8000000	lb		

*************************** PRELOAD CYCLE ***************************

Preload	Load %	Load [kg]	Load cell output [mV/V]
1	150.3	2704605.3	2.0936
2	150.3	2704526.0	2.1003
3	150.1	2701455.0	2.1003

************************* CALIBRATION CYCLE *************************

Test No.	Coefficients of straight line fit: OUTPUT = A+B*LOAD		Theoretical output at full load B*CAPACITY	Maximum linearity	Maximum hysteresis	Number of test
	A [mV/V]	B [mV/V/kg]	[mV/V]	error [%]	error [%]	samples:
1	-0.0082	7.8177E-07	1.4072	0.297	0.348	670
2	-0.0081	7.8175E-07	1.4072	0.293	0.340	678
3	-0.0082	7.8172E-07	1.4071	0.303	0.333	677

Load values were based on the output from master calibration load cells,

C1880_12, C1880_13, C1880_14, C1880_15, C1880_16, C1880_17, C1880_18, C1880_19

calibrated in accordance with ASTM specification E 74-91 against standard traceable to the National Institute of Standards and Technology in Washington D.C. USA. Based on the test data from 3 load cycles, the load cell output vs. applied load is represented by the straight line equation of the form: OUTPUT=A+B*LOAD, where the coefficients A and B are as follows:

A = -0.0082 mV/V

B = 7.81748E-07 mV/V/kg

Optimized full load sensitivity=B*CAPACITY = 1.4071 mV/V

Standard deviation of least squares fit: s = 0.0015 mV/V

Uncertainty (ASTM E74-91) = 2.4*s = 0.0036 mV/V

or 4633.6 kg

or 0.26 % of full scale output

Output with shunt resistor put across BLACK and WHITE leads:

	Shunt Res. [K-ohm]	**OUTPUT(shunt)** [mV/V]	SIMULATED OUTPUT %	[N]	[kg]	[lb]
=>	50	1.2057	85.69	15125303.8	1542317.7	3400302.8
->	100	0.6038	42.91	7574408.9	772357.7	1702794.5
->	150	0.4028	28.62	5052598.4	515210.3	1135869.1
	200	0.3021	21.47	3790286.9	386493.2	852090.2
	250	0.2418	17.18	3033017.8	309274.9	681849.4
	300	0.2015	14.32	2528083.2	257787.1	568335.6
	350	0.1728	12.28	2167129.3	220980.8	487189.9
	400	0.1512	10.74	1896347.3	193369.3	426315.7
	500	0.1210	8.60	1517683.5	154757.2	341188.7
	600	0.1008	7.17	1264841.3	128975.1	284347.6
	700	0.0865	6.15	1084787.3	110615.1	243869.8
	800	0.0756	5.37	948562.3	96724.3	213245.2

Maximum test linearity error 0.303 % of full scale output

Maximum test hysteresis error 0.348 % of full scale output

Maximum test repeatability error 0.013 % of full scale output

GEORGE KELK CORPORATION

LOAD CELL CALIBRATION REPORT

Model No.:	C3929		Work Order No:	JR8883
Serial No.:	15	Bridge 1 of 1	Test date:	01-10-2008
Capacity:	1800000	kg	Test by:	WP
Calibration facility:	8000000	lb		

Test No.1			Test No.2			Test No.3		
Load Step %	Applied Load [kg]	Load Cell Output [mV/V]	Load Step %	Applied Load [kg]	Load Cell Output [mV/V]	Load Step %	Applied Load [kg]	Load Cell Output [mV/V]
10.03	180485.0	0.1370	10.04	180757.2	0.1373	10.06	181168.2	0.1376
15.08	271424.9	0.2066	15.09	271647.1	0.2068	14.86	267464.0	0.2035
20.10	361845.1	0.2766	19.96	359317.4	0.2746	20.07	361170.8	0.2758
25.13	452304.3	0.3470	25.01	450166.5	0.3453	24.97	449399.8	0.3445
30.02	540363.9	0.4156	30.02	540440.2	0.4157	29.87	537619.4	0.4133
34.88	627791.0	0.4839	35.05	630818.3	0.4863	35.10	631796.9	0.4869
39.86	717518.4	0.5540	40.10	721728.3	0.5573	39.89	717983.3	0.5542
45.12	812184.9	0.6281	45.10	811796.9	0.6277	44.95	809100.9	0.6255
49.90	898207.3	0.6953	50.13	902302.7	0.6984	50.05	900879.2	0.6972
54.91	988332.9	0.7657	55.05	990983.6	0.7677	55.12	992080.0	0.7684
59.89	1078024.9	0.8358	60.00	1079919.5	0.8372	59.91	1078359.0	0.8359
64.87	1167659.1	0.9058	65.02	1170385.8	0.9079	64.92	1168623.6	0.9064
70.05	1260959.6	0.9786	69.94	1259006.0	0.9770	69.94	1258940.6	0.9769
74.98	1349673.8	1.0478	75.11	1351902.6	1.0495	74.91	1348332.3	1.0466
80.10	1441841.6	1.1196	79.99	1439772.3	1.1180	80.08	1441399.1	1.1192
84.88	1527811.4	1.1867	85.04	1530737.8	1.1890	84.96	1529232.5	1.1876
89.89	1617998.1	1.2569	90.01	1620222.8	1.2588	89.97	1619467.4	1.2580
95.11	1711933.3	1.3301	94.97	1709486.3	1.3282	95.07	1711177.6	1.3294
100.12	1802239.6	1.4005	99.90	1798162.3	1.3974	99.98	1799554.5	1.3983
94.92	1708535.5	1.3272	94.97	1709489.9	1.3281	94.95	1709029.8	1.3275
89.88	1617918.0	1.2565	89.93	1618807.5	1.2572	90.12	1622150.6	1.2596
85.01	1530233.9	1.1879	84.90	1528247.9	1.1865	84.89	1527996.5	1.1860
80.04	1440707.9	1.1180	79.88	1437867.0	1.1157	80.06	1441069.0	1.1181
74.96	1349273.6	1.0463	75.12	1352183.3	1.0487	74.93	1348690.4	1.0458
69.91	1258433.3	0.9752	70.08	1261514.1	0.9777	70.03	1260502.5	0.9767
64.90	1168242.9	0.9045	65.10	1171825.8	0.9074	64.99	1169880.8	0.9057
60.04	1080713.3	0.8359	60.00	1080058.5	0.8355	60.11	1081971.6	0.8368
55.02	990442.4	0.7650	54.97	989393.9	0.7643	55.09	991610.4	0.7658
49.98	899627.9	0.6936	49.98	899715.1	0.6937	50.01	900160.1	0.6940
44.94	808949.1	0.6223	44.96	809362.3	0.6228	44.95	809111.4	0.6224
39.98	719550.3	0.5520	40.13	722366.1	0.5543	39.92	718581.1	0.5511
35.12	632109.9	0.4831	34.99	629886.3	0.4816	35.09	631688.3	0.4828
29.99	539773.7	0.4106	29.92	538621.3	0.4098	29.93	538815.8	0.4097
24.91	448422.1	0.3391	25.10	451853.6	0.3420	25.09	451687.1	0.3418
19.93	358805.8	0.2700	19.99	359803.6	0.2710	19.98	359558.5	0.2708
15.10	271752.8	0.2035	14.99	269873.8	0.2023	15.00	269962.4	0.2024
10.10	181819.1	0.1354	10.03	180526.8	0.1347	10.00	180013.0	0.1345

LOAD CELL CALIBRATION CARD

MODEL No. C3929-1 SERIAL No. 15 BRIDGE 1

CAPACITY 1800000 Kg SENSITIVITY 1.4071 mV/V AT RATED LOAD*

FOR ROUTINE MILL **CALIBRATION** OF LOAD METERING SYSTEM

'CAL' BUTTON ON KELK AMPLIFIER SIMULATES APPLICATION OF 85.69 % OF RATED LOAD

OUTPUT WITH 50,000 OHMS SHUNT RESISTOR
APPLIED ACROSS BLACK AND WHITE LEADS 1.2057 mV/V*

INPUT RES. 261.0 OMHS AT 78° F (25° C)
GREEN (+), BLACK (–)

OUTPUT RES. 241.3 OMHS.
RED (+), WHITE (–)

DATE Jan 14 / 08

BY TR

*OPEN CIRCUIT MEASUREMENT (AMPLIFIER DISCONNECTED)

GEORGE KELK CORPORATION
48 LESMILL ROAD
DON MILLS, ONTARIO, CANADA

~ Calibration Certificate ~

Model Number: 3701G2FA50G

Serial Number: 7497

Description: DC Capacitive Accelerometer

Method: Back-to-Back Comparison Calibration

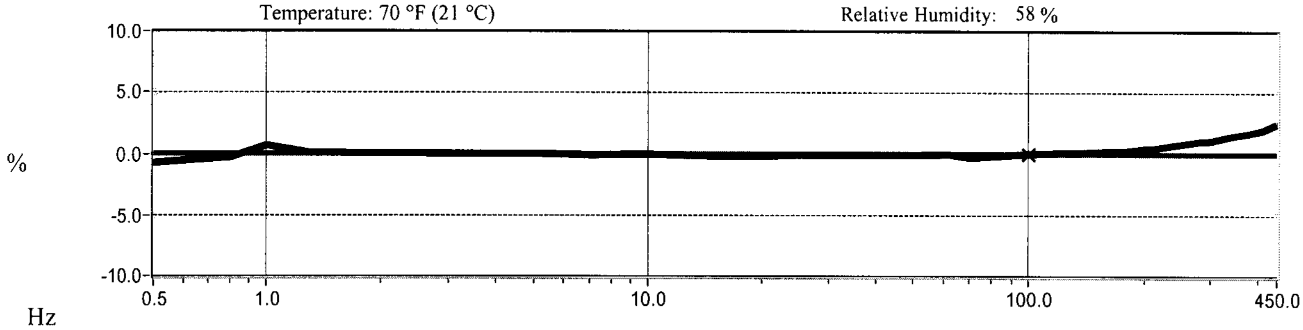
Manufacturer: PCB

ACS-11

Calibration Data

Sensitivity @ 100.0 Hz	**60.8 mV/g**	Offset Voltage (@ 0 g)	25.0 mVDC
	(6.20 mV/m/s²)	Resonant Frequency	1630.0 Hz

Sensitivity Plot



Data Points

Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)
0.5	-0.7	10.0	-0.0	70.0	-0.3
1.0	0.7	15.0	-0.2	REF. FREQ.	0.0
2.0	0.0	20.0	-0.2	200.0	0.4
5.0	0.0	30.0	-0.1	450.0	2.5
7.0	-0.1	50.0	-0.1		

Mounting Surface: Stainless Steel w/Silicone Grease Coating Fastener: Stud Mount Fixture Orientation: Vertical

Acceleration Level (rms)[1]: 1.00 g (9.81 m/s²)[2]

[1]The acceleration level may be limited by shaker displacement at low frequencies. If the listed level cannot be obtained, the calibration system uses the following formula to set the vibration amplitude: Acceleration Level (g) = 0.133 x (freq)².

[2]The gravitational constant used for calculations by the calibration system is: 1 g = 9.80665 m/s².

Condition of Unit

As Found: In Tolerance, No Adjustment Necessary

As Left: In Tolerance

Notes

1. Calibration is traceable to one or more of the following report numbers; PTB 5399, PTB 5400 and NIST 822/271196.
2. This certificate shall not be reproduced, except in full, without written approval from PCB Piezotronics, Inc.
3. Calibration is performed in compliance with ISO 9001, ISO 10012-1, ANSI/NCSL Z540-1-1994 and ISO 17025.
4. See Manufacturer's Specification Sheet for a detailed listing of performance specifications.
5. Measurement uncertainty (95% confidence level with coverage factor of 2) for frequency ranges tested during calibration are as follows: 0.5-0.99 Hz; +/- 1.8%, 1-30 Hz; +/- 1.0%, 30.01-199 Hz; +/- 1.5%, 200-1 kHz; +/- 3.0%.

Technician: John Pattison Date: 04/04/08

PCB PIEZOTRONICS INC
VIBRATION DIVISION
3425 Walden Avenue · Depew, NY 14043
TEL: 888-684-0013 · FAX: 716-685-3886 · www.pcb.com

ACCREDITED
CALIBRATION CERT #1862.01

cal14 - 3290186199.85

~ Calibration Certificate - Phase ~

Model Number: 3701G2FA50G

Serial Number: 7497

Description: DC Capacitive Accelerometer

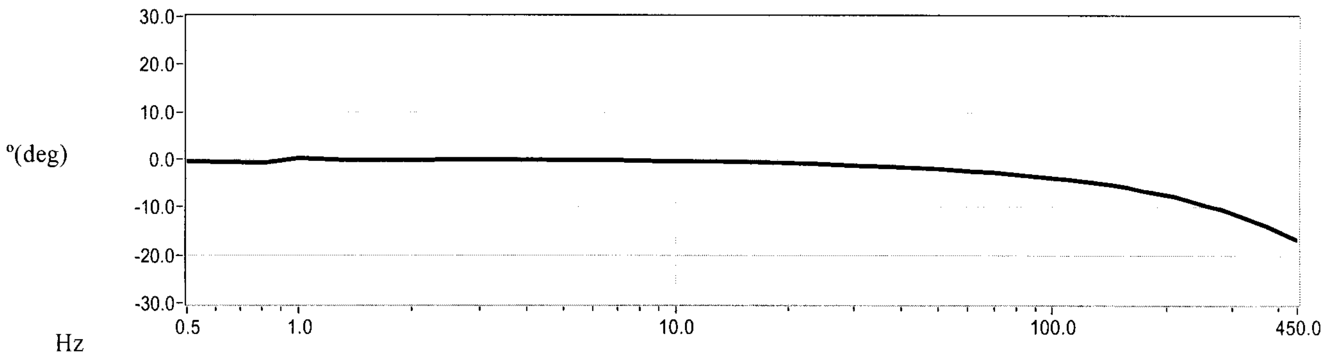
Method: Back-to-Back Comparison Calibration

Manufacturer: PCB

Calibration Data

[2]**Sensitivity @ 100.0 Hz** **60.8** **mV/g** **(6.20** **mV/m/s²)**

Phase Plot



Data Points

Frequency (Hz)	Phase (°)	Frequency (Hz)	Phase (°)
0.5	-0.5	30	-1.4
1.0	0.2	50	-2.0
2.0	-0.2	70	-2.7
5.0	-0.2	REF. FREQ.	-3.9
7.0	-0.2	200	-7.4
10	-0.4	450	-16.8
15	-0.6		
20	-0.8		

Notes

1. Calibration is traceable to one or more of the following report numbers; PTB 5399, PTB 5400 and NIST 822/271196.
2. This certificate shall not be reproduced, except in full, without written approval from PCB Piezotronics, Inc.
3. Calibration is performed in compliance with ISO 9001, ISO 10012-1, ANSI/NCSL Z540-1-1994 and ISO 17025.
4. See Manufacturer's Specification Sheet for a detailed listing of performance specifications.
5. Measurement uncertainty (95% confidence level with coverage factor of 2) for frequency ranges tested during calibration are as follows: 0.5-0.99 Hz; +/- 1.8%, 1-30 Hz; +/- 1.0%, 30.01-199 Hz; +/- 1.5%, 200-1 kHz; +/- 3.0%.

Technician: John Pattison

Date: 04/04/08

PCB PIEZOTRONICS INC.
VIBRATION DIVISION
3425 Walden Avenue · Depew, NY 14043
TEL: 888-684-0013 · FAX: 716-685-3886 · www.pcb.com

~ Calibration Certificate ~

Model Number: 3701G2FA50G

Serial Number: 7498

Description: DC Capacitive Accelerometer

Method: Back-to-Back Comparison Calibration

Manufacturer: PCB

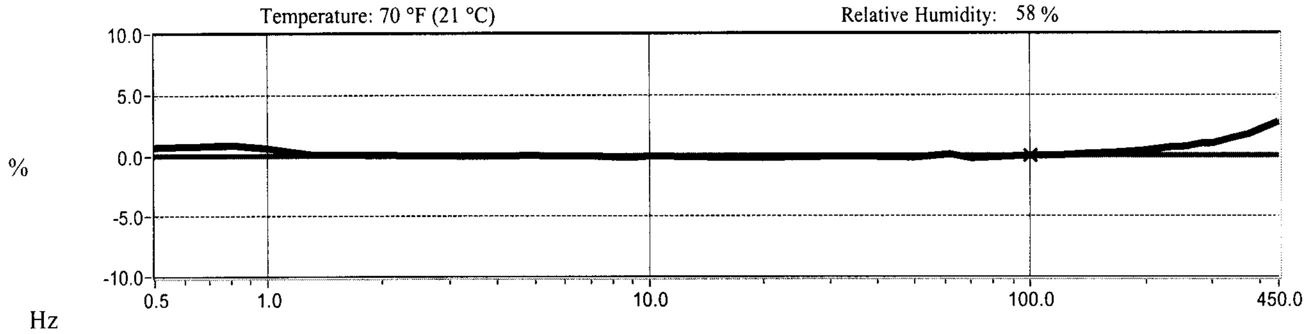
ACS-11

Calibration Data

Sensitivity @ 100.0 Hz	**59.8**	**mV/g**	Offset Voltage (@ 0 g)	4.5	mVDC
	(6.10	**mV/m/s²)**	Resonant Frequency	1650.0	Hz

Sensitivity Plot

Temperature: 70 °F (21 °C) Relative Humidity: 58 %



Data Points

Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)
0.5	0.7	10.0	-0.0	70.0	-0.2
1.0	0.6	15.0	-0.1	REF. FREQ.	0.0
2.0	0.1	20.0	-0.2	200.0	0.4
5.0	0.0	30.0	-0.1	450.0	2.8
7.0	-0.0	50.0	-0.2		

Mounting Surface: Stainless Steel w/Silicone Grease Coating Fastener: Stud Mount Fixture Orientation: Vertical

Acceleration Level (rms)[1]: 1.00 g (9.81 m/s²)[2]

[1]The acceleration level may be limited by shaker displacement at low frequencies. If the listed level cannot be obtained, the calibration system uses the following formula to set the vibration amplitude: Acceleration Level (g) = 0.133 x (freq)². [2]The gravitational constant used for calculations by the calibration system is; 1 g = 9.80665 m/s².

Condition of Unit

As Found: In Tolerance, No Adjustment Necessary

As Left: In Tolerance

Notes

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4. See Manufacturer's Specification Sheet for a detailed listing of performance specifications.
5. Measurement uncertainty (95% confidence level with coverage factor of 2) for frequency ranges tested during calibration are as follows: 0.5-0.99 Hz; +/- 1.8%, 1-30 Hz; +/- 1.0%, 30.01-199 Hz; +/- 1.5%, 200-1 kHz; +/- 3.0%.

Technician: John Pattison Date: 04/04/08

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cal14 - 3290187290.52

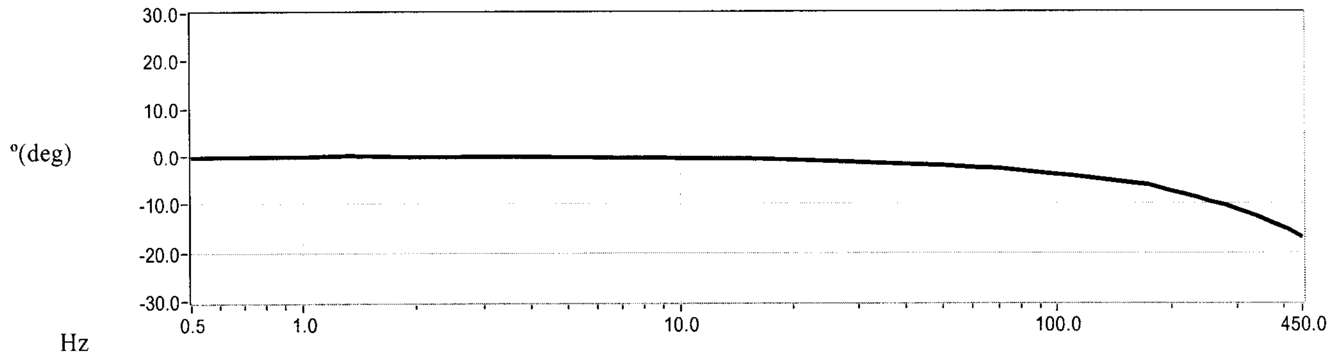
~ Calibration Certificate - Phase ~

Model Number:	**3701G2FA50G**		
Serial Number:	**7498**		
Description:	DC Capacitive Accelerometer	Method:	Back-to-Back Comparison Calibration
Manufacturer:	PCB		

Calibration Data

[2]Sensitivity @ 100.0 Hz 59.8 mV/g (6.10 mV/m/s²)

Phase Plot



Data Points

Frequency (Hz)	Phase (°)	Frequency (Hz)	Phase (°)
0.5	-0.3	30	-1.4
1.0	-0.2	50	-2.0
2.0	-0.1	70	-2.6
5.0	-0.3	REF. FREQ.	-3.9
7.0	-0.3	200	-7.3
10	-0.5	450	-16.9
15	-0.6		
20	-0.8		

Notes

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4. See Manufacturer's Specification Sheet for a detailed listing of performance specifications.
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Technician: John Pattison Date: 04/04/08

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~ *Calibration Certificate* ~

Model Number: 3701G2FA50G

Serial Number: 7499

Description: DC Capacitive Accelerometer

Method: Back-to-Back Comparison Calibration

Manufacturer: PCB

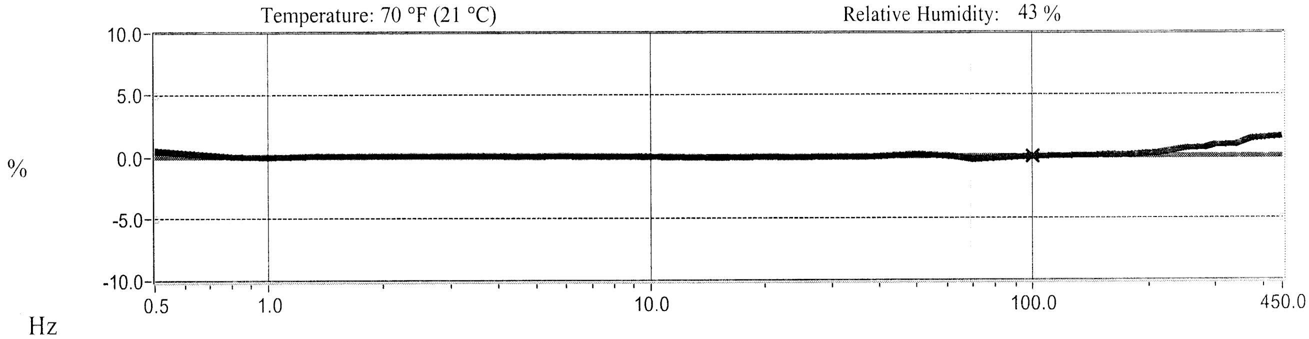
ACS-11

Calibration Data

Sensitivity @ 100.0 Hz	**59.1 mV/g**	Offset Voltage (@ 0 g)	6.5 mVDC
	(6.03 mV/m/s²)	Resonant Frequency	1700.0 Hz

Sensitivity Plot

Temperature: 70 °F (21 °C)   Relative Humidity: 43 %



Data Points

Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)	Frequency (Hz)	Dev. (%)
0.5	0.5	10.0	0.0	70.0	-0.3
1.0	-0.0	15.0	-0.1	REF. FREQ.	0.0
2.0	0.0	20.0	-0.0	200.0	0.2
5.0	0.0	30.0	-0.0	450.0	1.6
7.0	0.0	50.0	0.1		

Mounting Surface: Stainless Steel w/Silicone Grease Coating   Fastener: Stud Mount   Fixture Orientation: Vertical

Acceleration Level (rms)[1]: 1.00 g (9.81 m/s²)[2]

[1]The acceleration level may be limited by shaker displacement at low frequencies. If the listed level cannot be obtained, the calibration system uses the following formula to set the vibration amplitude; Acceleration Level (g) = 0.133 x (freq)².

[2]The gravitational constant used for calculations by the calibration system is; 1 g = 9.80665 m/s².

Condition of Unit

As Found: In Tolerance, No Adjustment Necessary

As Left: In Tolerance

Notes

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4. See Manufacturer's Specification Sheet for a detailed listing of performance specifications.
5. Measurement uncertainty (95% confidence level with coverage factor of 2) for frequency ranges tested during calibration are as follows: 0.5-0.99 Hz; +/- 1.8%, 1-30 Hz; +/- 1.0%, 30.01-199 Hz; +/- 1.5%, 200-1 kHz; +/- 3.0%.

Technician: John Pattison   Date: 02/08/08

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ENGINEERING DATA SHEET

THE INFORMATION APPEARING ON THIS SHEET HAS BEEN COMPILED SPECIFICALLY FOR THE GAGES CONTAINED IN THIS PACKAGE. THIS FORM IS PRODUCED WITH ADVANCED EQUIPMENT & PROCEDURES WHICH PERMIT COMPREHENSIVE QUALITY ASSURANCE VERIFICATION OF ALL DATA SUPPLIED HEREIN. SHOULD ANY QUESTIONS ARISE RELATIVE TO THESE GAGES, PLEASE MENTION GAGE TYPE, BATCH AND LOT NUMBER.

H001

Final QA	Check	Batch
DD	FD	S185908

Micro-Measurements Division
Made in USA

MEASUREMENTS GROUP, INC.
RALEIGH, NORTH CAROLINA

PRECISION STRAIN GAGES

F007

CEA-06-125UW-350	
TYPE	CEA-06-125UW-350
OPTION	
QUANTITY	5
LOT NUMBER	R-A58AD810
RESISTANCE IN OHMS AT 24°C	350.0±0.3%
GAGE FACTOR AT 24°C	2.095 ±0.5%
TRANSVERSE SENSITIVITY AT 24°C	(+0.5 ±0.2)%
CODE	091411-3166

GENERAL INFORMATION: CEA-SERIES STRAIN GAGES

GENERAL DESCRIPTION: CEA gages are a general-purpose family of constantan strain gages widely used in experimental stress analysis. The gages are supplied with a fully encapsulated grid and exposed copper-coated integral solder tabs.

TEMPERATURE RANGE: -100° to +400° F (-75° to +205° C) for continuous use in static measurements.

SELF-TEMPERATURE COMPENSATION: See data curve below.

STRAIN LIMITS: Approximately 5% for gage lengths 1/8 in. (3.2 mm) and larger; approximately 3% for gage lengths under 1/8 in. (3.2 mm).

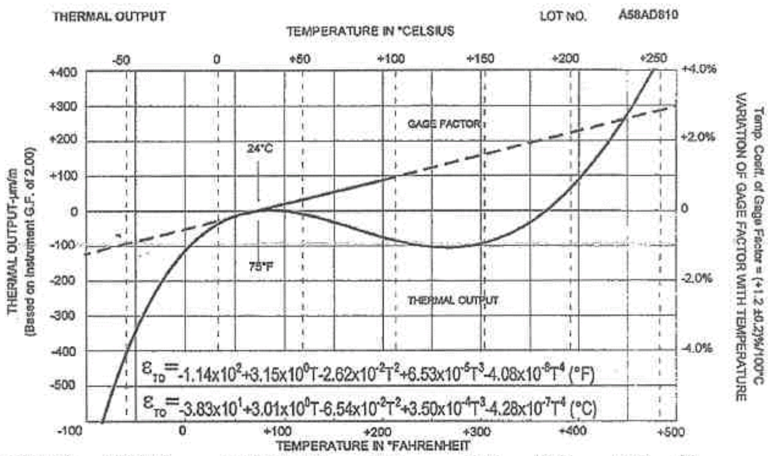
FATIGUE LIFE: Fatigue life is a marked function of solder joint formation. With 30-AWG leads directly attached to gage tabs, fatigue life will be 10^5 cycles at ±1500µin/in (µm/m) using M-Line 361A solder.

CEMENTS: Compatible with M-M Certified M-Bond 200, but it will normally not provide the greatest strain limit. Micro-Measurements M-Bond AE-10/15, M-Bond GA-2, M-Bond 600, and M-Bond 610 are excellent. M-Bond 610 is the best choice over the entire operating range. Refer to M-M Catalog A-110 for information on bonding agents, and Bulletins B-127, B-130, and B-137 for installation procedures.

SOLDER: If operating temperature will not exceed +300° F (+150° C), M-Line solder 361A (63-37) tin-lead solder may be used for lead attachment. M-Line solder (95-5) tin-antimony is satisfactory to +400° F (+205° C). Refer to M-M Catalog A-110 for further information on solders, and Tech Tip TT-609 for lead attachment techniques.

NOTE: The backing of CEA-Series gages has been specially treated for optimum bond formation with all appropriate strain gage adhesives. No further cleaning is necessary if contamination of the prepared surface is avoided during handling.

G045



TESTED ON: 1018 STEEL TEST PATTERN: 250BG CODE: 82718 ENG.: GU

TEST PROCEDURES USED BY MICRO-MEASUREMENTS

OPTICAL DEFECT ANALYSIS	M-M Procedures and Standards
GAGE RESISTANCE AT 24°C AND 50% RH	M-M Procedure, Direct NIST Traceability on Resistance Standards
GAGE FACTOR AT 24°C & 50% RH (UNIAXIAL STRESS FIELD - POISSON RATIO = 0.285)	ASTM E-251 (Constant Stress Cantilever Method)
TEMPERATURE COEFFICIENT OF GAGE FACTOR	ATSM E-251 (Step Deflection Method)
THERMAL OUTPUT	ASTM E-251 (Slow Heating Rate, Continuously Recorded)
TRANSVERSE SENSITIVITY AT 24°C AND 50%H	ASTM E-251
FATIGUE LIFE	NAS-942 (Modified)
STRAIN LIMITS	NAS-942 (Modified)
GAGE THICKNESS	M-M Procedure
CREEP AND DRIFT	M-M Procedure (Similar to NAS 942 Method)

NOTE: Gage resistance, gage factor, temperature coefficient of gage factor, thermal output, and transverse sensitivity testing and information presentation are in compliance with OIML International Recommendation NO. 62. "Performance characteristics of metallic resistance strain gages." Other tests are not included in I.R. NO. 62

T001

Appendix D

ADVANCEMENTS IN STATNAMIC DATA REGRESSION TECHNIQUES

Gray Mullins[1], Christopher L. Lewis[2], Michael D. Justason[3]

ABSTRACT

Until recently, the analysis of Statnamic test data has typically incorporated the "Unloading Point Method" (Middendorp et. al, 1992) to determine an equivalent static capacity. The UP method requires that the foundation move as a rigid body, thus excluding stress wave phenomenon from the analysis. If this requirement is met the foundation capacity can be determined using this simplified method. However, many foundations do not meet the UP criteria (e.g. fixed end or relatively long piles) and have proven difficult to analyze without more complex techniques. This paper presents a new analysis method that uses measured strain data as well as the standard Statnamic test data to determine foundation capacity. This new method discretizes the foundation into smaller segments that each meet the rigid body criteria of the UP method. Thereby, a more refined inertia and viscous damping evaluation can be implemented that individually determines the contributions from the various segments. This approach, termed the "Segmental Unloading Point" (SUP) method, is developed herein and then demonstrated with results from full-scale Statnamic test data.

INTRODUCTION

Since its inception in1988, Statnamic testing of deep foundations has gained popularity with many designers largely due to its time efficiency, cost effectiveness, data quality, and flexibility in testing existing foundations. Where large capacity static tests may take up to a week to set up and conduct, the largest of Statnamic tests typically takes no more than a few days. Further, multiple smaller-capacity tests can easily be completed within a day. The direct benefit of this time efficiency is the cost savings to the client and the ability to conduct more tests within a given budget. Additionally, this test method has boosted quality assurance by giving the contractor the ability to test foundations thought to have been compromised by construction difficulties without significantly affecting production.

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[1] *Assistant Professor, University of South Florida, Department of Civil and Environmental Engineering, Tampa, FL.*

[2] *Geotechnical Engineer, Applied Foundation Testing, Green Cove Springs / Tampa, FL.*

[3] *Senior Engineer, Berminghammer Foundation Equipment, Hamilton, ON.*

Statnamic testing is designated as a rapid load test that uses the inertia of a relatively small reaction mass instead of a reaction structure to produce large forces. Rapid load tests are differentiated from static and dynamic load tests by comparing the duration of the loading event with respect to the axial natural period of the foundation (2L/C). Test durations longer than 1000 L/C are considered static loadings and those shorter than 10 L/C are consider dynamic, where L represents the foundation length and C represents the strain wave velocity (Janes et al., 2000; Kusakabe et.al, 2000). Tests with a duration between 10L/C and 1000 L/C are denoted as rapid load tests. The duration of the Statnamic test is typically 100 to 120 milliseconds, but is dependant on the ratio of the applied force to the weight of the reaction mass. Longer duration tests of up to 500 milliseconds are possible but require a larger reaction mass.

The Statnamic force is produced by quickly-formed high pressure gases that in turn launch a reaction mass upward at up to twenty times the acceleration of gravity. The equal and opposite force exerted on the foundation is simply the product of the mass and acceleration of the reaction mass. It should be noted that the acceleration of the reaction mass is not significant in the analysis of the foundation; it is simply a by-product of the test. Secondly, the load produced is not an impact in that the mass is in contact prior to the test. Further, the test is over long before the masses reach the top of their flight. The parameters of interest are only those associated with the movement of the foundation (i.e. force, displacement, and acceleration).

Typical analysis of Statnamic data relies on measured values of force, displacement and acceleration. A soil model is not required, hence, the results are not highly user dependent. A new method of analysis is introduced that extends present methods by incorporating additional measured values of strain at discrete points along the length of the foundation. In the ensuing sections a discussion of analysis methods and their applicability will be presented. Full details on the development of this method can be found elsewhere (Lewis, 1999).

PRESENT ANALYSIS PROCEDURES

The Statnamic forcing event induces foundation motion in a relatively short period of time and hence acceleration and velocities will be present. The accelerations are typically small(1-2 g's), however the enormous mass of the foundation when accelerated resists movement due to inertia and as such the fundamental equation of motion applies, Equation 1.

$$F = ma + cv + kx \qquad \text{Equation 1}$$

where,
F = forcing event
m = mass of the foundation
a = acceleration of the displacing body
v = velocity of the displacing body
c = viscous damping coefficient
k = spring constant of the displacing system
x = displacement of the body

The equation of motion is generally described using four terms: forcing, inertial, viscous damping, and stiffness. The forcing term (*F*) denotes the load application which varies with time and is equated to the sum of remaining three terms. The inertial term (*ma*) is the force which is generated from the tendency of a body to resist motion, or to keep moving once it is set in motion (Young,1992). The viscous damping term (*cv*) is best described as the velocity dependant resistance to movement. The final term (*kx)*, represents the classic system stiffness, which is the static soil resistance.

When this equation is applied to a pile/soil system the terms can be redefined to more accurately describe the system. This is done by including both measured and calculated terms. The revised equation is displayed below:

$$F_{Statnamic} = (ma)_{Foundation} + (cv)_{Foundation} + F_{Static} \qquad \text{Equation 2}$$

where, $F_{Statnamic}$ is the measured Statnamic force, *m* is the calculated mass of the foundation, *a* is the measured acceleration of the foundation, *c* is the viscous damping coefficient , *v* is the calculated velocity, and F_{static} is the derived pile/soil static response.

There are two unknowns in the revised equation F_{static} and *c*, thus the equation is under specified. F_{static} is the desired value, so the variable *c* must be obtained to solve the equation. Middendorp (1992) presented a method to calculate the damping coefficient referred to as the Unloading Point Method (UP). With the value of *c* known, the static force can be calculated. This force, termed “Derived Static,” represents an equivalent soil response to that produced by a traditional static load test.

UP DESCRIPTION

The UP is a simple method which allows the equivalent static resistance to be derived from the measured Statnamic quantities. It uses a simple single degree of freedom model to represent the foundation/soil system as a rigid body supported by a non-linear spring and a linear dashpot in parallel (see Figure 1). The spring represents the static soil response (F_{Static}) which includes the elastic response of the foundation as well as the foundation/soil interface and surrounding soil response. The dashpot is used to represent the dynamic resistance which depends on the rate of pile penetration (Nishimura, 1995).

The UP makes two primary assumptions in its determination of “c.” The first is the static capacity of the pile is constant when it plunges as a rigid body. The second is that the damping coefficient is constant throughout the test. By doing so a window is defined in which to calculate the damping coefficient. The first point of interest (1) is that of maximum Statnamic Force. At this point the static resistance is assumed to have become steady state, for the purpose of calculating “c”. Thus, any extra resistance is attributed to that of the dynamic forces (ma and cv). The next point of interest (2) is that of zero velocity which has been termed the “Unloading Point.” Figure 2 shows a typical Statnamic load-displacement curve which denotes points (1) and (2). At this point the foundation is no longer moving and the resistance due to damping is zero. The static resistance, used to calculate “c” from point (1) to (2), can then be calculated by the following equation:

$$F_{Static_{UP}} = F_{Statnamic} - (ma)_{Foundation} \qquad \text{Equation 3}$$

where, $F_{Statnamic}$, m, and a are all known parameters; $F_{Static\ UP}$ is the static force calculated at (2) and assumed constant from (1) to (2).

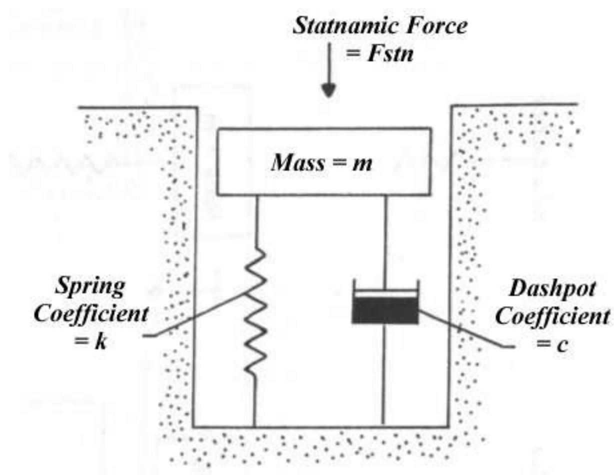

Figure 1 Single D.O.F. Model (After Das 1994)

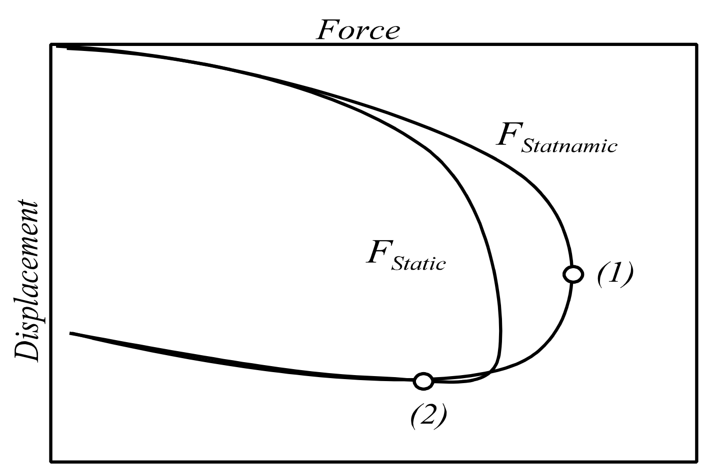

Figure 2UP time window for *C* determination.

Next, the damping coefficient can be calculated throughout this range, from maximum force (1) to zero velocity (2). The following equation is used to calculate *c*:

$$c = \frac{F_{Statnamic} - F_{Static_{UP}} - (ma)_{Foundation}}{v_{Foundation}} \qquad \text{Equation 4}$$

Damping values over this range should be fairly constant. Often the average value is taken as the damping constant, but if a constant value occurs over a long period of time it should be used (see Figure 3). Note that as v approaches zero at point (2), values of *c* can be different from that of the most representative value and therefore the entire trend should be reviewed. Finally the derived static response can be calculated as follows:

$$F_{Static} = F_{Statnamic} - (ma)_{Foundation} - (cv)_{Foundation} \qquad \text{Equation 5}$$

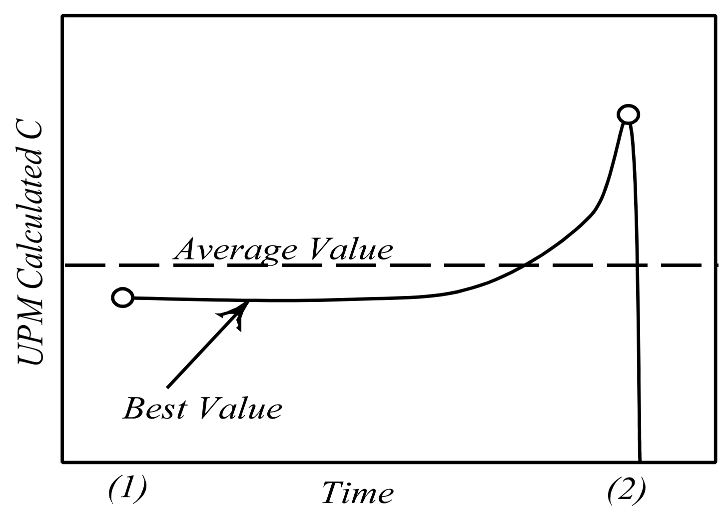

Figure 3 Variation in *C* between times (1) and (2).

Currently software is available to the public that can be used in conjunction with Statnamic test data to calculate the derived static pile capacity using the UP Method (Garbin, 1999). This software was developed by the University of South Florida and the Federal Highway Administration and can be downloaded from [*www.eng.usf.edu/˜gmullins*] under the Statnamic Analysis Workbook (SAW™) heading.

UP SHORTCOMINGS

The UP has proven to be a valuable tool in predicting damping values when the foundation acts as a rigid body. However, as the pile length increases an appreciable delay can be introduced between the movement of the pile top and toe, hence negating the rigid body assumption. This occurrence also becomes prevalent when an end bearing condition exists; in this case the lower portion of the foundation is prevented from moving jointly with the top of the foundation.

Middendorp (1995) defines the "Wave Number" (N_w) to quantify the applicability of the UP. The wave number is calculated by dividing the wave length (D) by the foundation depth (L). D is obtained by multiplying the wave speed c in length per second by the load duration (T) in seconds. Thus, the wave number is calculated by the following equation:

$$N_w = \frac{D}{L} = \frac{cT}{L} \qquad \text{Equation 6}$$

Through empirical studies Middendorp determined that the UP would accurately predict static capacity, from Statnamic data, if the wave number is greater than 12. Nishimura (1995) established a similar threashold at a wave number of 10. Using wave speeds of 5000 m/s and 4000m/s for steel and concrete respectively and a typical Statnamic load duration, the UP is limited to piles shorter than 50 m (steel) and 40 m (concrete). Wave number analysis can be used to determine if stress waves will develop in the pile. However, this does not necessarily satisfy the rigid body requirement of the UP.

Statnamic tests cannot always produce wave numbers greater than 10, and as such there have been several methods suggested to accommodate stress wave phenomena in Statnamically tested long piles (Middendorp, 1995). Due to limitations on paper length these methods are not presented.

MODIFIED UNLOADING POINT METHOD

Given the shortcomings of the UP, users of Statnamic testing have developed a remedy for the problematic condition that arises most commonly. The scenario involves relatively short piles (N_w>10) that do not exhibit rigid body motion, but rather elastically shorten within the same magnitude as the permanent set. This is typical of rock-socketed drilled shafts or piles driven to dense bearing strata that are not fully mobilized during testing. The consequence is that the top of pile response (i.e. acceleration, velocity, and displacement) is significantly different from that of the toe. The most drastic subset of these test results show zero movement at the toe while the top of pile elastically displaces in excess of the surficial yield limit (e.g. upwards of 25 mm). Whereas with plunging piles (rigid body motion) the difference in movement (top to toe) is minimal and the average acceleration is essentially the same as the top of pile acceleration; tip restrained piles will exhibit an inertial term that is twice as large when using top of pile movement measurements to represent the entire pile.

The Modified Unloading Point Method (MUP), developed by Justason (1997), makes use of an additional toe accelerometer that measures the toe response. The entire pile is still assumed to be a single mass, m, but the acceleration of the mass is now defined by the average of the top and toe movements. A standard UP is then conducted using the applied top of pile Statnamic force and the average accelerations and velocities. The derived static force is then plotted against top of pile displacement as before. This simple extension of the UP has successfully overcome most problematic data sets. Plunging piles instrumented with both top and toe accelerometers have shown little analytical difference between the UP and the MUP. However, MUP analyses are now recommended whenever both top and toe information is available.

NEED FOR ADVANCEMENT

Although the MUP provided a more refined approach to some of the problems associated with UP conditions, there still exists a scenario where it is difficult to interpret Statnamic data with present methods. This is when the wave number is less than ten (relatively long piles). In these cases the pile may still only experience compression (no tension waves) but the delay between top and toe movements causes a phase lag. Hence an average of top and toe movements does not adequately represent the pile.

SUP METHOD DESCRIPTION AND GENERAL PROCEDURE

The fundamental concept of the SUP is that the acceleration, velocity, displacement, and force on each segment can be determined using strain gage measurements along the length of the pile. Individual pile segment displacements are determined using the relative displacement as calculated from strain gage measurements and an upper or lower measured displacement. The velocity and acceleration of each segment are then determined by numerically differentiating displacement then velocity with respect to time. The segmental forces are determined by calculating the difference in force from two strain gage levels.

Typically the maximum number of segments is dependent on the available number of strain gage layers. However, strain gage placement does not necessitate assignment of segmental boundaries; as long as the wave number of a given segment is greater than 10, the segment can include several strain gage levels within its boundaries. The number and the elevation of strain gage levels are usually determined based on soil stratification; as such, it can be useful to conduct an individual segmental analysis to produce the shear strength parameters for each soil strata. A reasonable upper limit on the number of segments should be adopted because of the large number of mathematical computations required to complete each analysis. Figure 4 is a sketch of the SUP pile discretization.

The notation used for the general SUP case defines the pile as having m levels of strain gages and $m+1$ segments. Strain gage locations are labeled using positive integers starting from 1 and continuing through m. The first gage level below the top of the foundation is denoted as GL^1 where the superscript defines the gage level. Although there are no strain gages at the top of foundation,this elevation is denoted as GL^0. Segments are numbered using positive integers from 1 to $m+1$, where segment 1 is bounded by the top of foundation (GL^0) and GL^1. Any general segment is denoted as segment n and lies between GL^{n-1} and GL^n. Finally, the bottom segment is denoted as segment $m+1$ and lies between GL^m and the foundation toe.

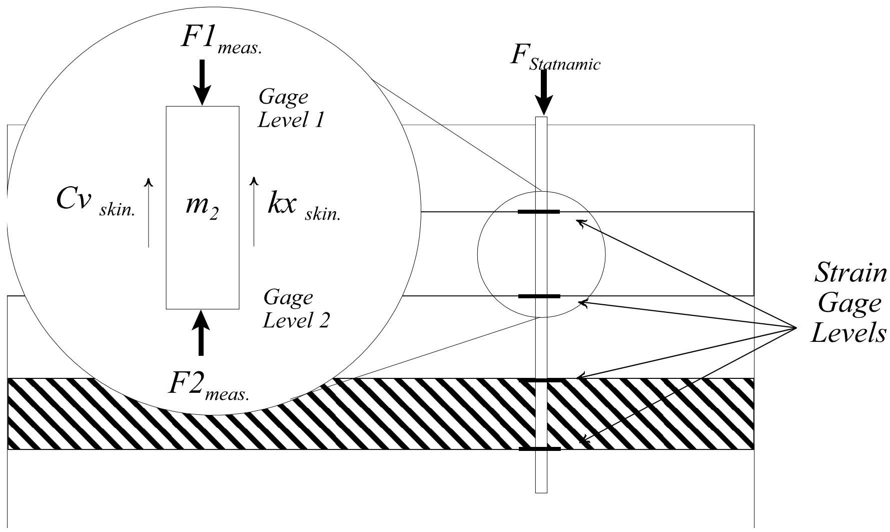

Figure 4 Segmental Free Body Diagram

CALCULATION OF SEGMENTAL MOTION PARAMETERS

The SUP analysis defines average acceleration, velocity, and displacement traces that are specific to each segment. In doing so, strain measurements from the top and bottom of each segment and a boundary displacement are required. Boundary displacement may come from the Statnamic laser reference system (top), top of pile acceleration data, or from embedded toe accelerometer data.

The displacement is calculated at each gage level using the change in recorded strain with respect to an initial time zero using Equation 7. Because a linearly-varying strain distribution is assumed between gage levels, the average strain is used to calculate the elastic shortening in each segment.

Level displacements

$$x_n = x_{n-1} - \Delta\epsilon_{average\ seg\ n}\, L_{seg\ n} \qquad \text{Equation 7}$$

where

x_n = the displacement at the nth gage level
$\Delta\epsilon_{average\ seg\ n}$ = the average change in strain in segment n
$L_{seg\ n}$ = the length of the nth segment

To perform an unloading point analysis, only the top-of-segment motion needs to be defined. However, the MUP analysis, which is now recommended, requires both top and bottom parameters. The SUP lends itself naturally to providing this information. Therefore, the average segment movement is used rather than the top-of-segment; hence, the SUP actually performs multiple MUP analyses rather than standard UP. The segmental displacement is then determined using the average of the gage level displacements from each end of the segment as shown in the following equation:

$$x_{seg\ n} = \frac{x_{n-1} + x_n}{2} \qquad \text{Equation 8}$$

where $x_{seg\ n}$ is the average displacement consistent with that of the segment centroid.

The velocity and acceleration, as required for MUP, are then determined from the average displacement trace through numerical differentiation using Equations 9 and 10, respectively:

$$v_n = \frac{x_{n_t} - x_{n_{t+1}}}{\Delta t} \qquad \text{Equation 9}$$

$$a_n = \frac{v_{n_t} - v_{n_{t+1}}}{\Delta t} \qquad \text{Equation 10}$$

where v_n = the velocity of segment *n*
a_n = the acceleration of segment *n*
Δt = the time step from time *t* to *t+1*

It should be noted that all measured values of laser displacement, strain, and force are time dependent parameters that are field recorded using high speed data acquisition computers. Hence the time step, Δt, used to calculate velocity and acceleration is a uniform value that can be as small as 0.0002 seconds. Therefore, some consideration should be given when selecting the time step to be used for numerical differentiation.

The average motion parameters (x, v, and a) for segment *m+1* can not be ascertained from measured data, but the displacement at GL^m can be differentiated directly providing the velocity and acceleration. Therefore, the toe segment is evaluated using the standard UP. These segments typically are extremely short (1 - 2 m) producing little to no differential movement along its length.

CALCULATION OF SEGMENTAL STATNAMIC AND DERIVED STATIC FORCES

Each segment in the shaft is subjected to a forcing event which causes movement and reaction forces. This segmental force is calculated by subtracting the force at the top of the segment from the force at the bottom. The difference is due to side friction, inertia, and damping for all segments except the bottom segment. This segment has only one forcing function from GL^m and the side friction is coupled with the tip bearing component. The force on segment *n* is defined as:

$$S_n = A_{(n-1)}\, E_{(n-1)}\, \epsilon_{(n-1)} - A_n\, E_n\, \epsilon_n \qquad \text{Equation 11}$$

where S_n = the applied segment force from strain measurements
E_n = the composite elastic modulus at level *n*
A_n = the cross sectional area at level *n*
ϵ_n = the measured strain at level *n*

Once the motion and forces are defined along the length of the pile, an unloading point analysis on each segment is conducted. The segment force defined above is now used in place of the Statnamic force in Equation 2. Equation 12 redefines the fundamental equation of motion for a segment analysis:

$$S_n = m_n\, a_n + c_n\, v_n + S_{n\ Static} \qquad \text{Equation 12}$$

where, $S_{n\ Static}$ = the derive static response of segment *n*
m_n = the calculated mass of segment *n*
c_n = the damping constant of segment *n*

The damping constant (in Equation 13) and the derived static response (Equation 14) of the segment are computed consistent with standard UP analyses:

$$c_n = \frac{S_n - S_{n\ Static}}{v_n} \qquad \text{Equation 13}$$

$$S_{n\ Static} = S_n - m_n\, a_n - c_n\, v_n \qquad \text{Equation 14}$$

Finally the top-of-foundation derived static response can be calculated by summing the derived static response of the individual segments as displayed in the following equation:

$$F_{Static} = \sum_{n=1}^{m+1} S_{n\ Static} \qquad \text{Equation 15}$$

Software capable of performing SUP analyses (SUPERSAW™) is currently being developed at the University of South Florida in cooperation with the Federal Highway Administration.

SITE CHARACTERISTICS AND SUP APPLICATION

Prepared in this section are examples of the motion parameters, segment forces, and load displacement trends as analyzed by SUP. The foundation was instrumented with four strain gage levels (m = 4) which produced five segments. Data was obtained at the 17th Street Causeway Replacement Bridge project as part of an extensive load test program implemented by the Florida Department of Transportation (FDOT), which included Statnamic load tests. Statnamic load testing was performed using a 30MN Statnamic device equipped with a gravel catch structure. Shaft instrumentation consisted of standard Statnamic equipment as well as, resistive type strain gages, and a toe accelerometer. Instrumentation elevations are presented in Table 1.

The test shaft had a planned diameter of 1.22 m and was 22 m in length. It was constructed using a temporary casing method and sea water as the drilling fluid. The 1.22 m O.D. steel casing (1 cm wall thickness) was installed to elevation-18.96 using a vibratory hammer. The concrete was placed using a tremie method, then the casing tip was pulled to elevation -0.9 m, using a vibratory hammer.

A soil boring performed at the test shaft location indicated that the natural ground elevation was approximately 1.5 m. The water table was reported to exist at elevation 0.3 m. SPT testing was initiated at the ground surface (elevation +3 m) and extended to elevation -28.15 m. The upper two meters of soil consisted of compacted limestone fill with SPT “N” values ranging from 27 to 16.

Table 1 Instrumentation Schedule

Instrumentation Elevation (m)	Number of Transducers	Type of Transducer
3.0	4	Calibrated Load Cell, 2 Accelerometers, and Laser Reference System
-1.8	3	Strain Gage
-4.2	3	Strain Gage
-17.0	3	Strain Gage
-18.3	3	Strain Gage
-19.0	1	Accelerometer

The following strata was reported as fine sand with fragments of limestone and shell. This strata extended to elevation -14.7m, "N" values ranged from 9 to 57. From elevation -14.7m throughout, the rock socket length averaged 34% RQD at 80% recovery. RQD values ranged from 18% to 73% in the limestone below the shaft tip. Recovery values in this strata are generally greater than 70%.

Figure 5 shows the measured change in strain with respect to time $t = 0$ for each gage level. Figures 6 through 8 illustrate the motion parameters determined for each of the five segments. Figure 9 shows the forces calculated at each gage level with the true measured strain. Figure 10

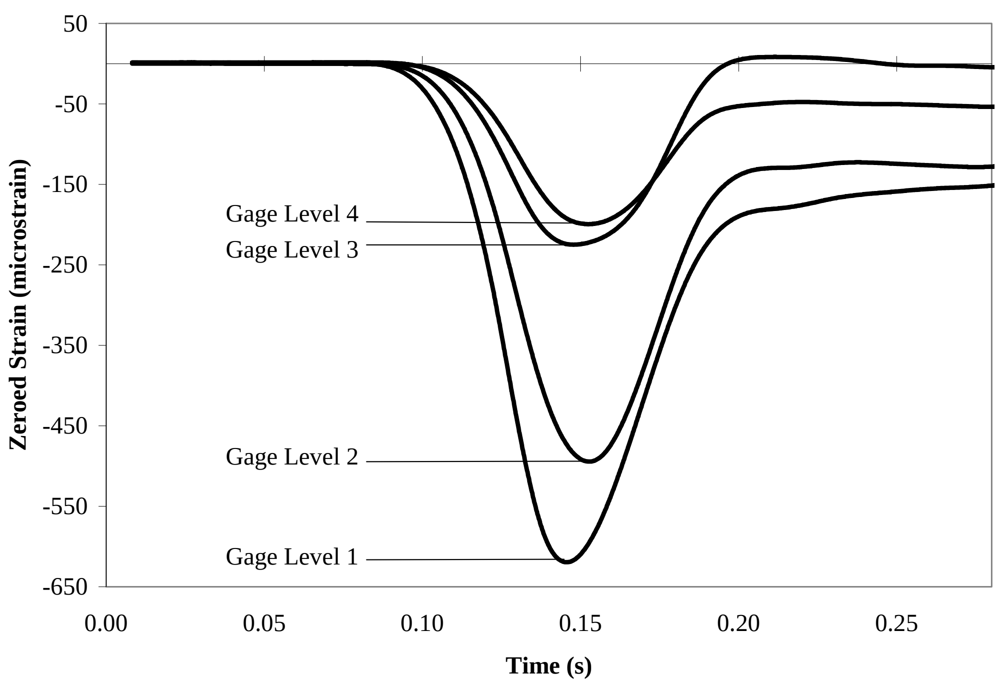

Figure 5 Strain versus Time

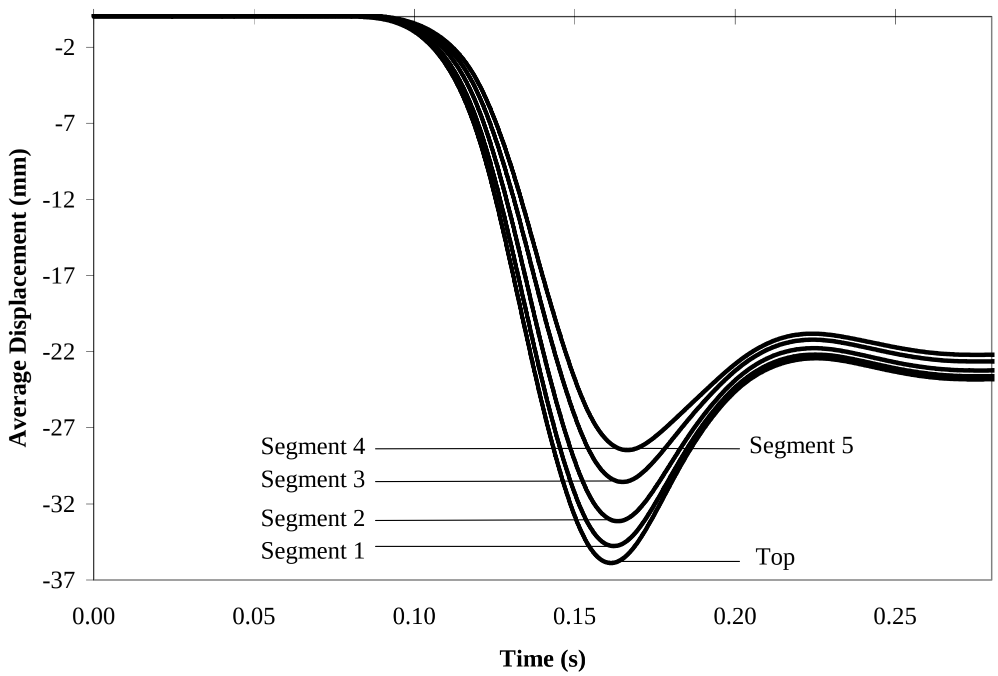

Figure 6 Segmental Average Displacements

shows the dynamic forces on each of the segments as calculated by Equation 11.

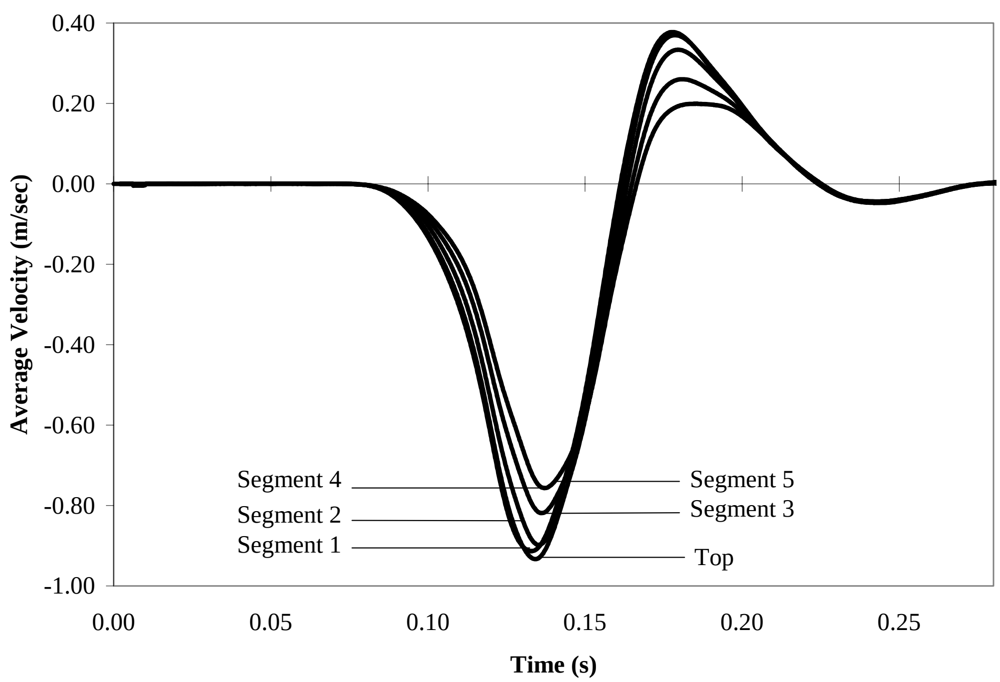

Figure 7 Segmental Average Velocity

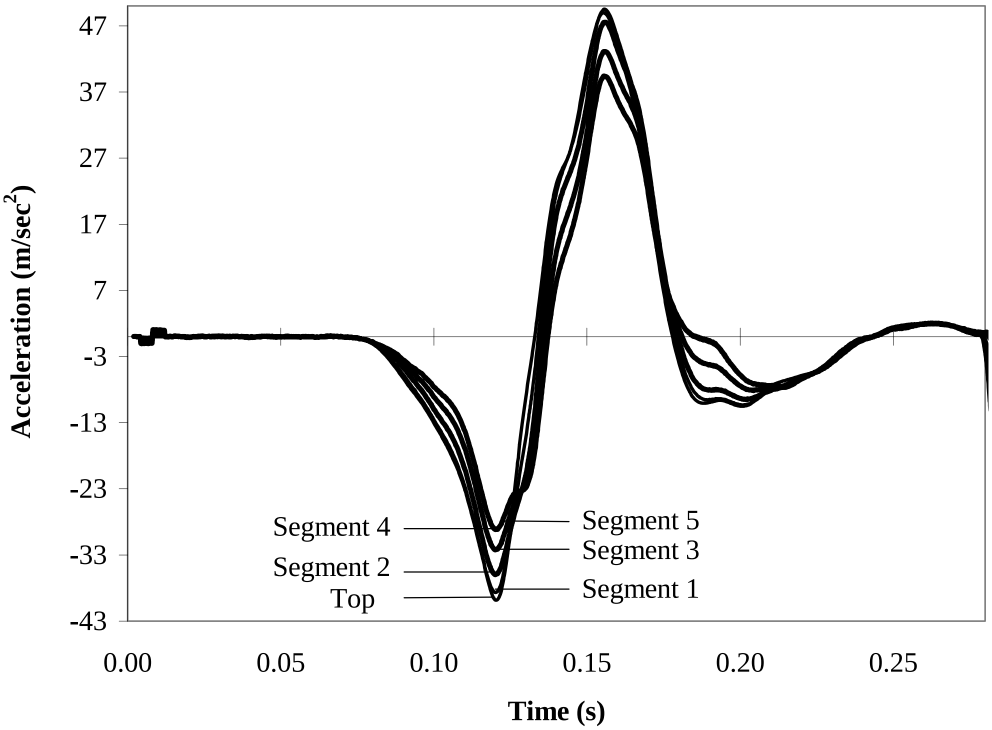

Figure 9 Segmental Average Acceleration

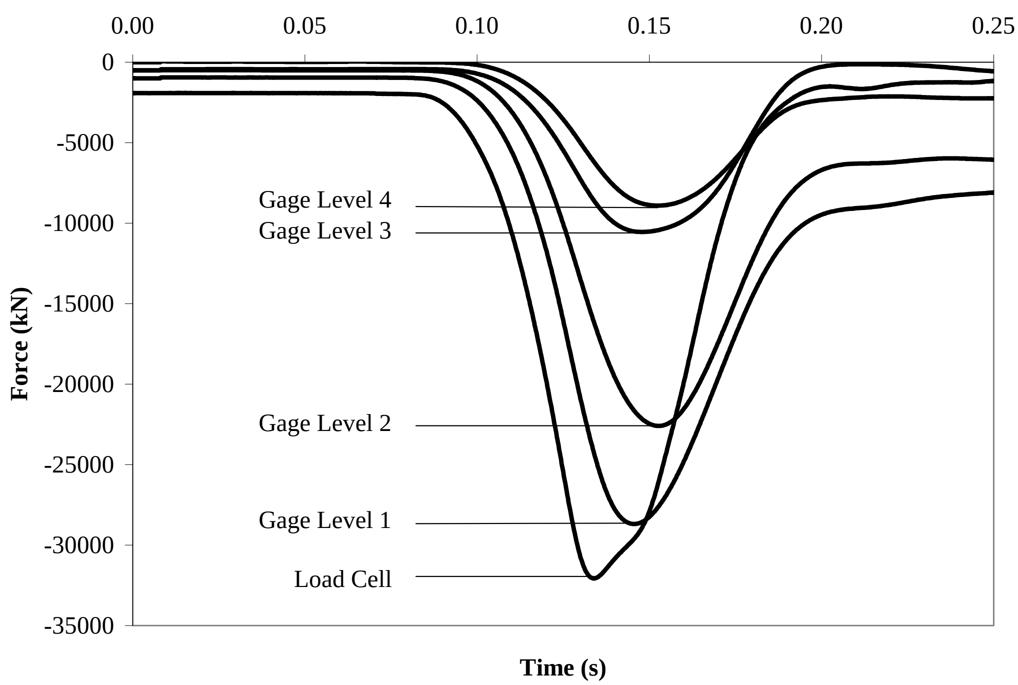

Figure 8 Force at Gage Levels

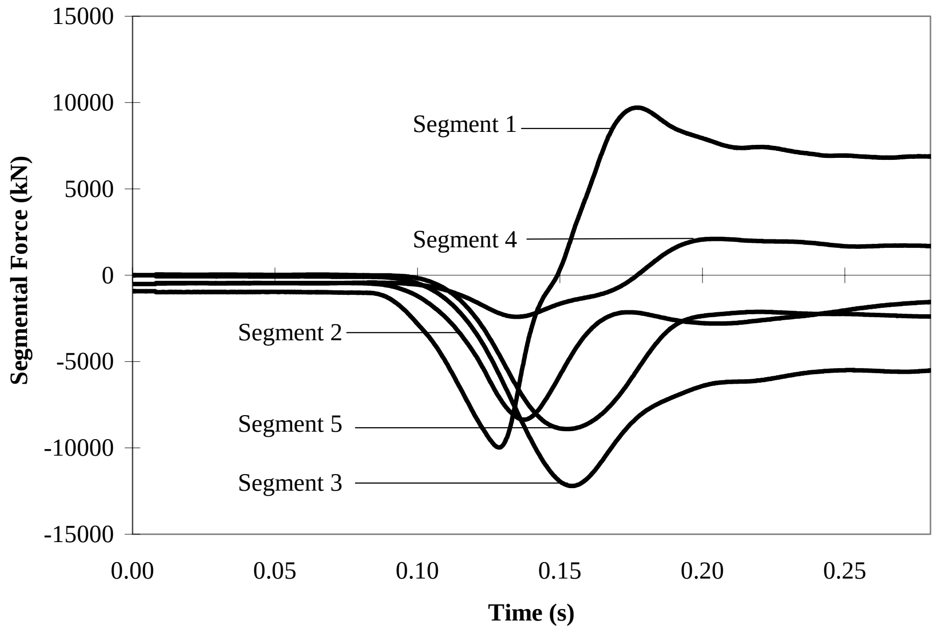

Figure 10 Segmental Statnamic Forces

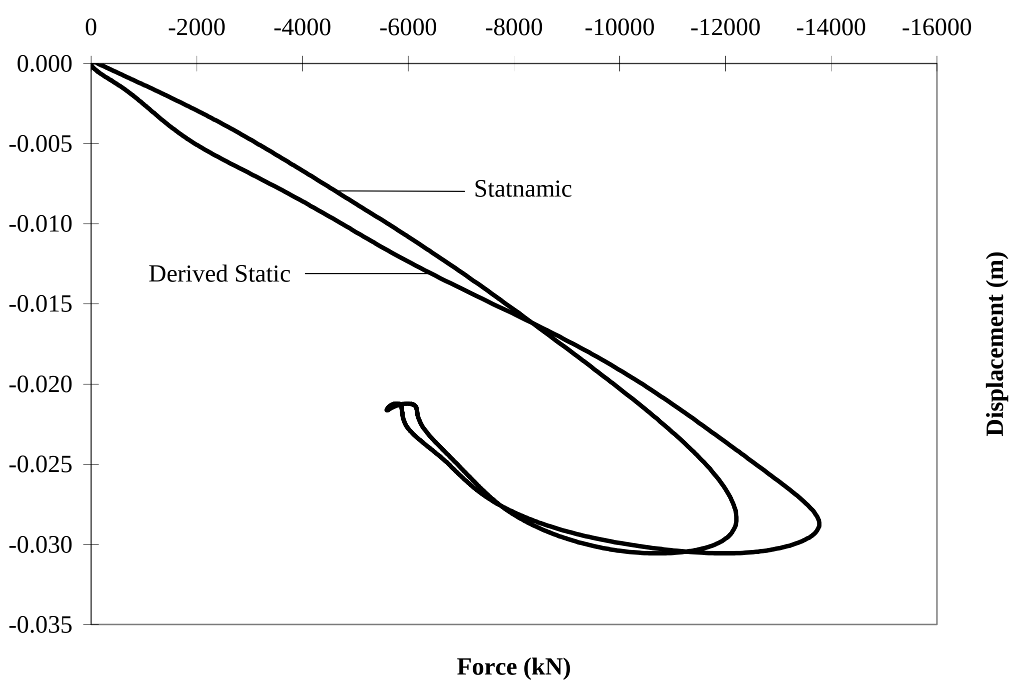

Figure 11 Segment Load Displacement Curves

Using the segmental forces, the derived static soil resistance is determined for each of the segments. Figure 11 shows a typical segment load versus segment displacement curve. By simply dividing the segment force by circumferential surface area this curve can be converted in to a shear stress versus displacement (T-Z) curve for that specific soil strata.

It can be seen in Figure 10 that the peak of each segment force may occur at different times and therefore at different top-of-pile displacements. This can effect the pile capacity in that the ultimate shear strengths of the strata should not be simply summed. This is most probably not a significant concern with this pile due to the plunging nature of the failure. This is evidenced by the similar top and toe movements shown in Figure 6. However, to be technically correct, SUP uses the summation of segment forces as they were developed. This accounts for upper soil layers that may fully mobilize and become residual in nature while lower soil layers begin to develop ultimate strengths. Figure 12 shows the raw Statnamic load displacement curve as well as SUP, MUP, and UP derived static capacity versus the top-of-pile displacement.

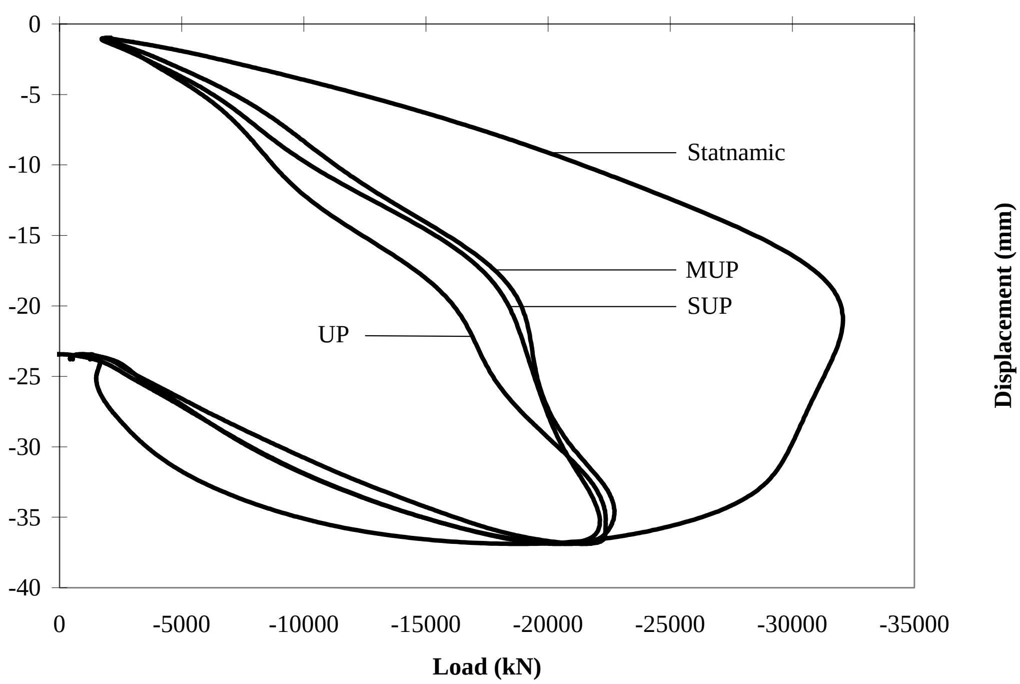

Figure 12 Top of Pile Load versus Displacement

SUMMARY

A new method of analysis called the Segmental Unloading Point Method (SUP) was presented that evaluates Statnamically loaded foundations as segments whose lengths are defined by embedded strain gage elevations. The recorded strain measurements are used to determine both the segmental motion parameters as well as the segmental force traces. Each segment is then treated as an individual foundation whose static response is derived using either the UP or MUP methods. The summation of each segment contribution with respect to time provides a top of foundation response that more closely incorporates the actual distribution of inertial and damping forces throughout the foundation. This is most important in the analysis of relatively long or fixed-ended piles. Although the UP and MUP methods of analysis are sufficient for most loading conditions, SUP provides information for soil strata T-Z curves as well as cut-off elevation load-displacement curves.

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Appendix D

ADVANCEMENTS IN STATNAMIC DATA REGRESSION TECHNIQUES

Gray Mullins[1], Christopher L. Lewis[2], Michael D. Justason[3]

ABSTRACT

Until recently, the analysis of Statnamic test data has typically incorporated the "Unloading Point Method" (Middendorp et. al, 1992) to determine an equivalent static capacity. The UP method requires that the foundation move as a rigid body, thus excluding stress wave phenomenon from the analysis. If this requirement is met the foundation capacity can be determined using this simplified method. However, many foundations do not meet the UP criteria (e.g. fixed end or relatively long piles) and have proven difficult to analyze without more complex techniques. This paper presents a new analysis method that uses measured strain data as well as the standard Statnamic test data to determine foundation capacity. This new method discretizes the foundation into smaller segments that each meet the rigid body criteria of the UP method. Thereby, a more refined inertia and viscous damping evaluation can be implemented that individually determines the contributions from the various segments. This approach, termed the "Segmental Unloading Point" (SUP) method, is developed herein and then demonstrated with results from full-scale Statnamic test data.

INTRODUCTION

Since its inception in1988, Statnamic testing of deep foundations has gained popularity with many designers largely due to its time efficiency, cost effectiveness, data quality, and flexibility in testing existing foundations. Where large capacity static tests may take up to a week to set up and conduct, the largest of Statnamic tests typically takes no more than a few days. Further, multiple smaller-capacity tests can easily be completed within a day. The direct benefit of this time efficiency is the cost savings to the client and the ability to conduct more tests within a given budget. Additionally, this test method has boosted quality assurance by giving the contractor the ability to test foundations thought to have been compromised by construction difficulties without significantly affecting production.

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Statnamic testing is designated as a rapid load test that uses the inertia of a relatively small reaction mass instead of a reaction structure to produce large forces. Rapid load tests are differentiated from static and dynamic load tests by comparing the duration of the loading event with respect to the axial natural period of the foundation (2L/C). Test durations longer than 1000 L/C are considered static loadings and those shorter than 10 L/C are consider dynamic, where L represents the foundation length and C represents the strain wave velocity (Janes et al., 2000; Kusakabe et.al, 2000). Tests with a duration between 10L/C and 1000 L/C are denoted as rapid load tests. The duration of the Statnamic test is typically 100 to 120 milliseconds, but is dependant on the ratio of the applied force to the weight of the reaction mass. Longer duration tests of up to 500 milliseconds are possible but require a larger reaction mass.

The Statnamic force is produced by quickly-formed high pressure gases that in turn launch a reaction mass upward at up to twenty times the acceleration of gravity. The equal and opposite force exerted on the foundation is simply the product of the mass and acceleration of the reaction mass. It should be noted that the acceleration of the reaction mass is not significant in the analysis of the foundation; it is simply a by-product of the test. Secondly, the load produced is not an impact in that the mass is in contact prior to the test. Further, the test is over long before the masses reach the top of their flight. The parameters of interest are only those associated with the movement of the foundation (i.e. force, displacement, and acceleration).

Typical analysis of Statnamic data relies on measured values of force, displacement and acceleration. A soil model is not required, hence, the results are not highly user dependent. A new method of analysis is introduced that extends present methods by incorporating additional measured values of strain at discrete points along the length of the foundation. In the ensuing sections a discussion of analysis methods and their applicability will be presented. Full details on the development of this method can be found elsewhere (Lewis, 1999).

PRESENT ANALYSIS PROCEDURES

The Statnamic forcing event induces foundation motion in a relatively short period of time and hence acceleration and velocities will be present. The accelerations are typically small(1-2 g's), however the enormous mass of the foundation when accelerated resists movement due to inertia and as such the fundamental equation of motion applies, Equation 1.

$$F = ma + cv + kx \qquad \text{Equation 1}$$

where, F = forcing event
m = mass of the foundation
a = acceleration of the displacing body
v = velocity of the displacing body
c = viscous damping coefficient
k = spring constant of the displacing system
x = displacement of the body

The equation of motion is generally described using four terms: forcing, inertial, viscous damping, and stiffness. The forcing term (F) denotes the load application which varies with time and is equated to the sum of remaining three terms. The inertial term (ma) is the force which is generated from the tendency of a body to resist motion, or to keep moving once it is set in motion (Young,1992). The viscous damping term (cv) is best described as the velocity dependant resistance to movement. The final term (kx), represents the classic system stiffness, which is the static soil resistance.

When this equation is applied to a pile/soil system the terms can be redefined to more accurately describe the system. This is done by including both measured and calculated terms. The revised equation is displayed below:

$$F_{Statnamic} = (ma)_{Foundation} + (cv)_{Foundation} + F_{Static} \qquad \text{Equation 2}$$

where, $F_{Statnamic}$ is the measured Statnamic force, m is the calculated mass of the foundation, a is the measured acceleration of the foundation, c is the viscous damping coefficient , v is the calculated velocity, and F_{static} is the derived pile/soil static response.

There are two unknowns in the revised equation F_{static} and c, thus the equation is under specified. F_{static} is the desired value, so the variable c must be obtained to solve the equation. Middendorp (1992) presented a method to calculate the damping coefficient referred to as the Unloading Point Method (UP). With the value of c known, the static force can be calculated. This force, termed "Derived Static," represents an equivalent soil response to that produced by a traditional static load test.

UP DESCRIPTION

The UP is a simple method which allows the equivalent static resistance to be derived from the measured Statnamic quantities. It uses a simple single degree of freedom model to represent the foundation/soil system as a rigid body supported by a non-linear spring and a linear dashpot in parallel (see Figure 1). The spring represents the static soil response (F_{Static}) which includes the elastic response of the foundation as well as the foundation/soil interface and surrounding soil response. The dashpot is used to represent the dynamic resistance which depends on the rate of pile penetration (Nishimura, 1995).

The UP makes two primary assumptions in its determination of "c." The first is the static capacity of the pile is constant when it plunges as a rigid body. The second is that the damping coefficient is constant throughout the test. By doing so a window is defined in which to calculate the damping coefficient. The first point of interest (1) is that of maximum Statnamic Force. At this point the static resistance is assumed to have become steady state, for the purpose of calculating "c". Thus, any extra resistance is attributed to that of the dynamic forces (ma and cv). The next point of interest (2) is that of zero velocity which has been termed the "Unloading Point." Figure 2 shows a typical Statnamic load-displacement curve which denotes points (1) and (2). At this point the foundation is no longer moving and the resistance due to damping is zero. The static resistance, used to calculate "c" from point (1) to (2), can then be calculated by the following equation:

$$F_{Static_{UP}} = F_{Statnamic} - (ma)_{Foundation} \qquad \text{Equation 3}$$

where, $F_{Statnamic}$, m, and a are all known parameters; $F_{Static\ UP}$ is the static force calculated at (2) and assumed constant from (1) to (2).

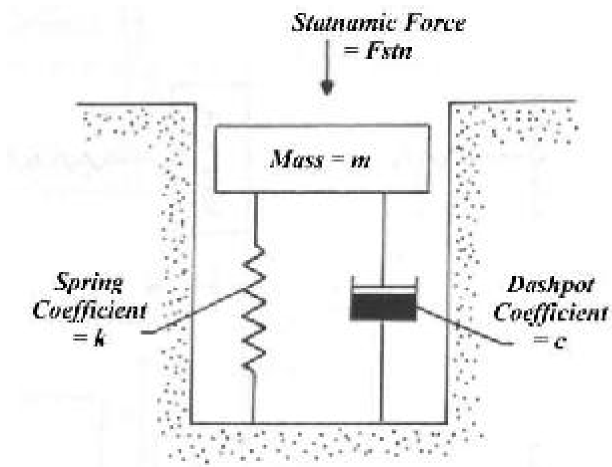

Figure 1 Single D.O.F. Model (After Das 1994)

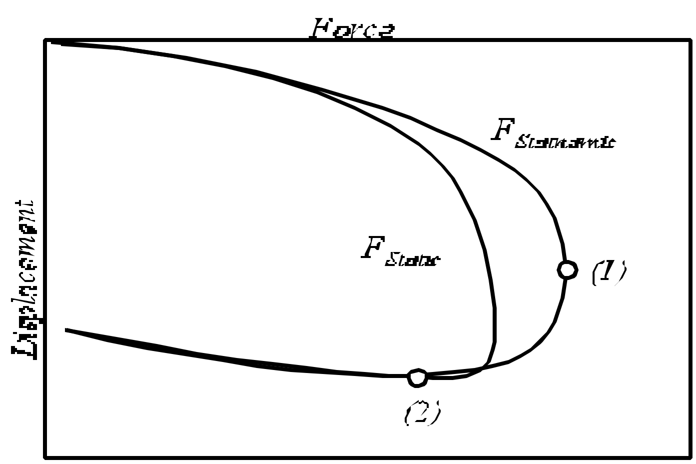

Figure 2UP time window for *C* determination.

Next, the damping coefficient can be calculated throughout this range, from maximum force (1) to zero velocity (2). The following equation is used to calculate *c*:

$$c = \frac{F_{Statnamic} - F_{Static_{UP}} - (ma)_{Foundation}}{v_{Foundation}} \qquad \text{Equation 4}$$

Damping values over this range should be fairly constant. Often the average value is taken as the damping constant, but if a constant value occurs over a long period of time it should be used (see Figure 3). Note that as v approaches zero at point (2), values of *c* can be different from that of the most representative value and therefore the entire trend should be reviewed. Finally the derived static response can be calculated

$$F_{Static} = F_{Statnamic} - (ma)_{Foundation} - (cv)_{Foundation} \qquad \text{Equation 5}$$

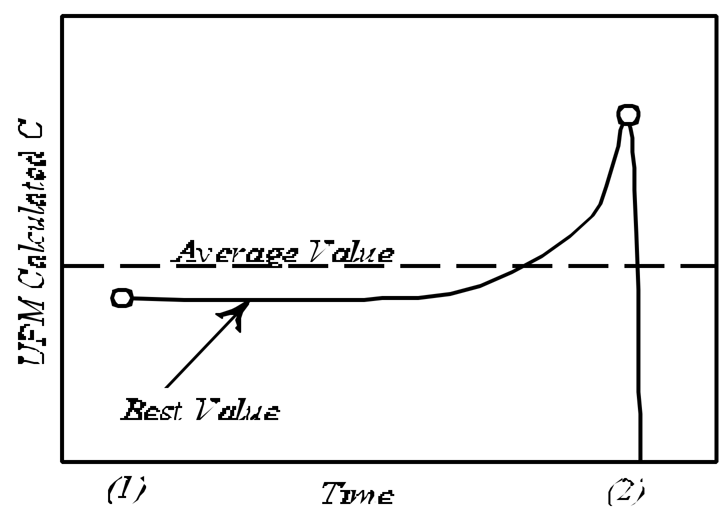

Figure 3 Variation in *C* between times (1) and (2).

as follows:

Currently software is available to the public that can be used in conjunction with Statnamic test data to calculate the derived static pile capacity using the UP Method (Garbin, 1999). This software was developed by the University of South Florida and the Federal Highway Administration and can be downloaded from www.eng.usf.edu/~gmullins under the Statnamic Analysis Workbook (SAW™) heading.

UP SHORTCOMINGS

The UP has proven to be a valuable tool in predicting damping values when the foundation acts as a rigid body. However, as the pile length increases an appreciable delay can be introduced between the movement of the pile top and toe, hence negating the rigid body assumption. This occurrence also becomes prevalent when an end bearing condition exists; in this case the lower portion of the foundation is prevented from moving jointly with the top of the foundation.

Middendorp (1995) defines the "Wave Number" (N_w) to quantify the applicability of the UP. The wave number is calculated by dividing the wave length (D) by the foundation depth (L). D is obtained by multiplying the wave speed c in length per second by the load duration (T) in seconds. Thus, the wave number is calculated by the following equation:

$$N_w = \frac{D}{L} = \frac{cT}{L} \qquad \text{Equation 6}$$

Through empirical studies Middendorp determined that the UP would accurately predict static capacity, from Statnamic data, if the wave number is greater than 12. Nishimura (1995) established a similar threashold at a wave number of 10. Using wave speeds of 5000 m/s and 4000m/s for steel and concrete respectively and a typical Statnamic load duration, the UP is limited to piles shorter than 50 m (steel) and 40 m (concrete). Wave number analysis can be used to determine if stress waves will develop in the pile. However, this does not necessarily satisfy the rigid body requirement of the UP.

Statnamic tests cannot always produce wave numbers greater than 10, and as such there have been several methods suggested to accommodate stress wave phenomena in Statnamically tested long piles (Middendorp, 1995). Due to limitations on paper length these methods are not presented.

MODIFIED UNLOADING POINT METHOD

Given the shortcomings of the UP, users of Statnamic testing have developed a remedy for the problematic condition that arises most commonly. The scenario involves relatively short piles (N_w>10) that do not exhibit rigid body motion, but rather elastically shorten within the same magnitude as the permanent set. This is typical of rock-socketed drilled shafts or piles driven to dense bearing strata that are not fully mobilized during testing. The consequence is that the top of pile response (i.e. acceleration, velocity, and displacement) is significantly different from that of the toe. The most drastic subset of these test results show zero movement at the toe while the top of pile elastically displaces in excess of the surficial yield limit (e.g. upwards of 25 mm). Whereas with plunging piles (rigid body motion) the difference in movement (top to toe) is minimal and the average acceleration is essentially the same as the top of pile acceleration; tip restrained piles will exhibit an inertial term that is twice as large when using top of pile movement measurements to represent the entire pile.

The Modified Unloading Point Method (MUP), developed by Justason (1997), makes use of an additional toe accelerometer that measures the toe response. The entire pile is still assumed to be a single mass, m, but the acceleration of the mass is now defined by the average of the top and toe movements. A standard UP is then conducted using the applied top of pile Statnamic force and the average accelerations and velocities. The derived static force is then plotted against top of pile displacement as before. This simple extension of the UP has successfully overcome most problematic data sets. Plunging piles instrumented with both top and toe accelerometers have shown little analytical difference between the UP and the MUP. However, MUP analyses are now recommended whenever both top and toe information is available.

NEED FOR ADVANCEMENT

Although the MUP provided a more refined approach to some of the problems associated with UP conditions, there still exists a scenario where it is difficult to interpret Statnamic data with present methods. This is when the wave number is less than ten (relatively long piles). In these cases the pile may still only experience compression (no tension waves) but the delay between top and toe movements causes a phase lag. Hence an average of top and toe movements does not adequately represent the pile.

SUP METHOD DESCRIPTION AND GENERAL PROCEDURE

The fundamental concept of the SUP is that the acceleration, velocity, displacement, and force on each segment can be determined using strain gage measurements along the length of the pile. Individual pile segment displacements are determined using the relative displacement as calculated from strain gage measurements and an upper or lower measured displacement. The velocity and acceleration of each segment are then determined by numerically differentiating displacement then velocity with respect to time. The segmental forces are determined by calculating the difference in force from two strain gage levels.

Typically the maximum number of segments is dependent on the available number of strain gage layers. However, strain gage placement does not necessitate assignment of segmental boundaries; as long as the wave number of a given segment is greater than 10, the segment can include several strain gage levels within its boundaries. The number and the elevation of strain gage levels are usually determined based on soil stratification; as such, it can be useful to conduct an individual segmental analysis to produce the shear strength parameters for each soil strata. A reasonable upper limit on the number of segments should be adopted because of the large number of mathematical computations required to complete each analysis. Figure 4 is a sketch of the SUP pile discretization.

The notation used for the general SUP case defines the pile as having m levels of strain gages and $m+1$ segments. Strain gage locations are labeled using positive integers starting from 1 and continuing through m. The first gage level below the top of the foundation is denoted as GL^1 where the superscript defines the gage level. Although there are no strain gages at the top of foundation,this elevation is denoted as GL^0. Segments are numbered using positive integers from 1 to $m+1$, where segment 1 is bounded by the top of foundation (GL^0) and GL^1. Any general segment is denoted as segment n and lies between GL^{n-1} and GL^n. Finally, the bottom segment is denoted as segment $m+1$ and lies between GL^m and the foundation toe.

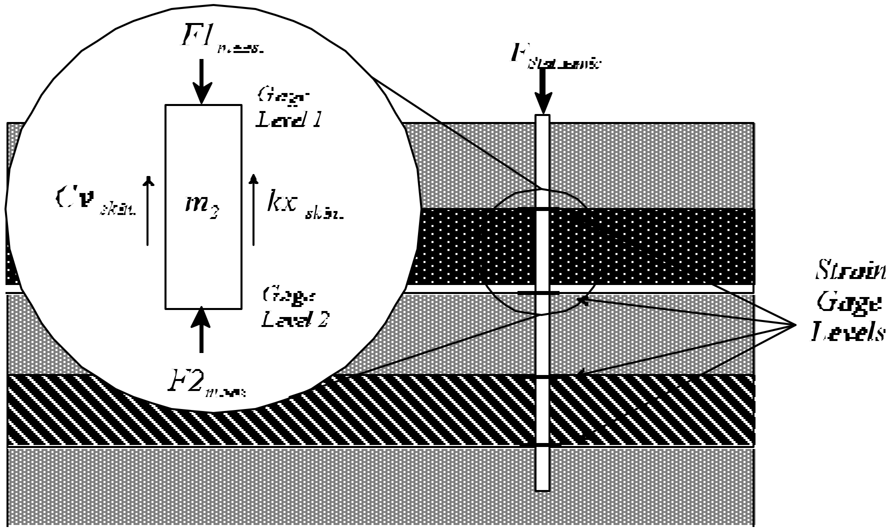

Figure 4 Segmental Free Body Diagram

CALCULATION OF SEGMENTAL MOTION PARAMETERS

The SUP analysis defines average acceleration, velocity, and displacement traces that are specific to each segment. In doing so, strain measurements from the top and bottom of each segment and a boundary displacement are required. Boundary displacement may come from the Statnamic laser reference system (top), top of pile acceleration data, or from embedded toe accelerometer data.

The displacement is calculated at each gage level using the change in recorded strain with respect to an initial time zero using Equation 7. Because a linearly-varying strain distribution is assumed between gage levels, the average strain is used to calculate the elastic shortening in each segment.

Level displacements

$$x_n = x_{n-1} - \Delta\varepsilon_{average\ seg\ n} L_{seg\ n} \qquad \text{Equation 7}$$

where

x_n = the displacement at the nth gage level

$\Delta\varepsilon_{average\ seg\ n}$ = the average change in strain in segment n

$L_{seg\ n}$ = the length of the nth segment

To perform an unloading point analysis, only the top-of-segment motion needs to be defined. However, the MUP analysis, which is now recommended, requires both top and bottom parameters. The SUP lends itself naturally to providing this information. Therefore, the average segment movement is used rather than the top-of-segment; hence, the SUP actually performs multiple MUP analyses rather than standard UP. The segmental displacement is then determined using the average of the gage level displacements from each end of the segment as shown in the following equation:

$$x_{seg\ n} = \frac{x_{n-1} + x_n}{2} \qquad \text{Equation 8}$$

where $x_{seg\ n}$ is the average displacement consistent with that of the segment centroid.

The velocity and acceleration, as required for MUP, are then determined from the average displacement trace through numerical differentiation using Equations 9 and 10, respectively:

$$v_n = \frac{x_{n_t} - x_{n_{t+1}}}{\Delta t} \qquad \text{Equation 9}$$

$$a_n = \frac{v_{n_t} - v_{n_{t+1}}}{\Delta t} \qquad \text{Equation 10}$$

where	v_n	=	the velocity of segment n
	a_n	=	the acceleration of segment n
	Δt	=	the time step from time t to $t+1$

It should be noted that all measured values of laser displacement, strain, and force are time dependent parameters that are field recorded using high speed data acquisition computers. Hence the time step, Δt, used to calculate velocity and acceleration is a uniform value that can be as small as 0.0002 seconds. Therefore, some consideration should be given when selecting the time step to be used for numerical differentiation.

The average motion parameters (x, v, and a) for segment $m+1$ can not be ascertained from measured data, but the displacement at GL^m can be differentiated directly providing the velocity and acceleration. Therefore, the toe segment is evaluated using the standard UP. These segments typically are extremely short (1 - 2 m) producing little to no differential movement along its length.

CALCULATION OF SEGMENTAL STATNAMIC AND DERIVED STATIC FORCES

Each segment in the shaft is subjected to a forcing event which causes movement and reaction forces. This segmental force is calculated by subtracting the force at the top of the segment from the force at the bottom. The difference is due to side friction, inertia, and damping for all segments except the bottom segment. This segment has only one forcing function from GL^m and the side friction is coupled with the tip bearing component. The force on segment n is defined as:

$$S_n = A_{(n-1)} E_{(n-1)} \varepsilon_{(n-1)} - A_n E_n \varepsilon_n \qquad \text{Equation 11}$$

where	S_n	=	the applied segment force from strain measurements
	E_n	=	the composite elastic modulus at level n
	A_n	=	the cross sectional area at level n
	ε_n	=	the measured strain at level n

Once the motion and forces are defined along the length of the pile, an unloading point analysis on each segment is conducted. The segment force defined above is now used in place of the Statnamic force in Equation 2. Equation 12 redefines the fundamental equation of motion for a segment analysis:

$$S_n = m_n a_n + c_n v_n + S_{n\,Static} \qquad \text{Equation 12}$$

where, $S_{n\,Static}$ = the derive static response of segment n
m_n = the calculated mass of segment n
c_n = the damping constant of segment n

The damping constant (in Equation 13) and the derived static response (Equation 14) of the segment are computed consistent with standard UP analyses:

$$c_n = \frac{S_n - S_{n\,Static}}{v_n} \qquad \text{Equation 13}$$

$$S_{n\,Static} = S_n - m_n a_n - c_n v_n \qquad \text{Equation 14}$$

Finally the top-of-foundation derived static response can be calculated by summing the derived static response of the individual segments as displayed in the following equation:

$$F_{Static} = \sum_{n=1}^{m+1} S_{n\,Static} \qquad \text{Equation 15}$$

Software capable of performing SUP analyses (SUPERSAW™) is currently being developed at the University of South Florida in cooperation with the Federal Highway Administration.

SITE CHARACTERISTICS AND SUP APPLICATION

Prepared in this section are examples of the motion parameters, segment forces, and load displacement trends as analyzed by SUP. The foundation was instrumented with four strain gage levels (m = 4) which produced five segments. Data was obtained at the 17th Street Causeway Replacement Bridge project as part of an extensive load test program implemented by the Florida Department of Transportation (FDOT), which included Statnamic load tests. Statnamic load testing was performed using a 30MN Statnamic device equipped with a gravel catch structure. Shaft instrumentation consisted of standard Statnamic equipment as well as, resistive type strain gages, and a toe accelerometer. Instrumentation elevations are presented in Table 1.

The test shaft had a planned diameter of 1.22 m and was 22 m in length. It was constructed using a temporary casing method and sea water as the drilling fluid. The 1.22 m O.D. steel casing (1 cm wall thickness) was installed to elevation-18.96 using a vibratory hammer. The concrete was placed using a tremie method, then the casing tip was pulled to elevation -0.9 m, using a vibratory hammer.

A soil boring performed at the test shaft location indicated that the natural ground elevation was approximately 1.5 m. The water table was reported to exist at elevation 0.3 m. SPT testing was initiated at the ground surface (elevation +3 m) and extended to elevation -28.15 m. The upper two meters of soil consisted of compacted limestone fill with SPT "N" values ranging from 27 to 16.

Table 1 Instrumentation Schedule

Instrumentation Elevation (m)	Number of Transducers	Type of Transducer
3.0	4	Calibrated Load Cell, 2 Accelerometers, and Laser Reference System
-1.8	3	Strain Gage
-4.2	3	Strain Gage
-17.0	3	Strain Gage
-18.3	3	Strain Gage
-19.0	1	Accelerometer

The following strata was reported as fine sand with fragments of limestone and shell. This strata extended to elevation -14.7m, "N" values ranged from 9 to 57. From elevation -14.7m throughout, the rock socket length averaged 34% RQD at 80% recovery. RQD values ranged from 18% to 73% in the limestone below the shaft tip. Recovery values in this strata are generally greater than 70%.

Figure 5 shows the measured change in strain with respect to time *t = 0* for each gage level. Figures 6 through 8 illustrate the motion parameters determined for each of the five segments. Figure 9 shows the forces calculated at each gage level with the true measured strain. Figure 10 shows the dynamic forces on each of the segments as calculated by Equation 11.

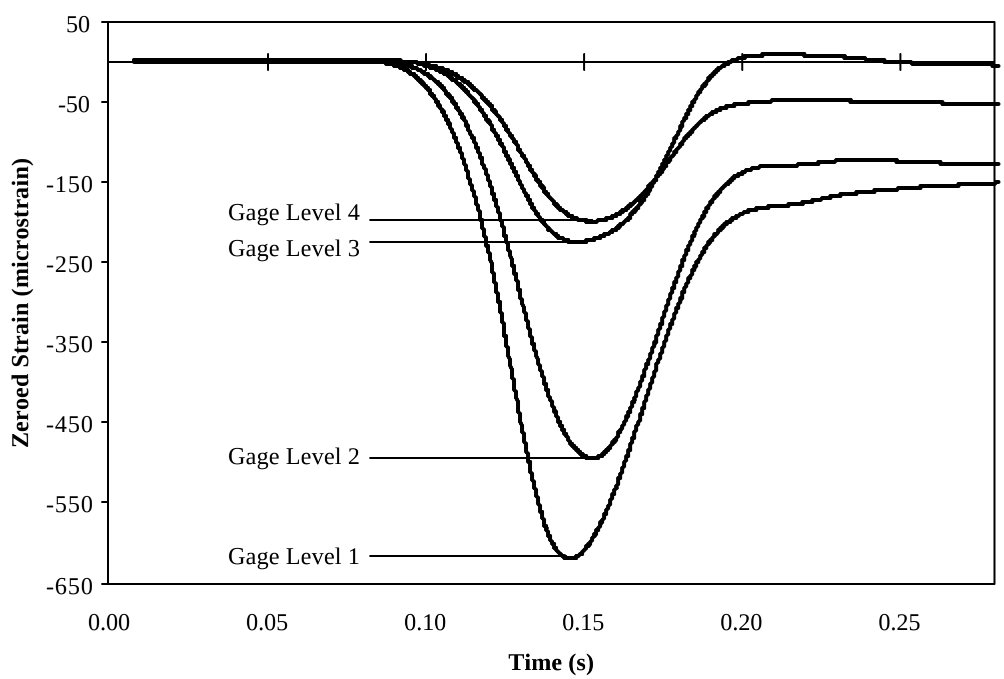

Figure 5 Strain versus Time

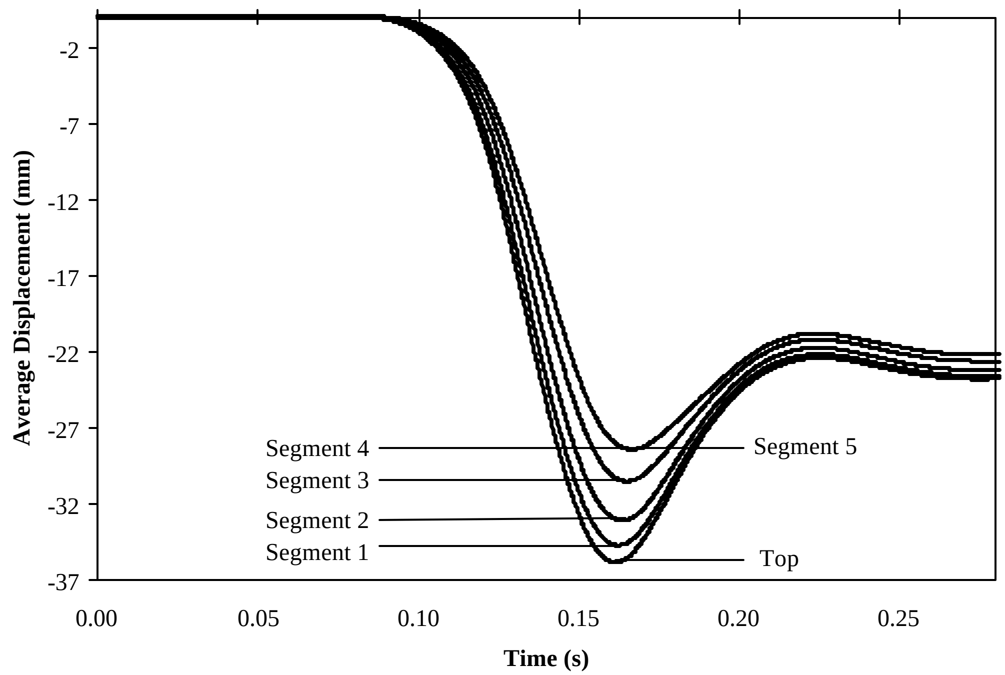

Figure 6 Segmental Average Displacements

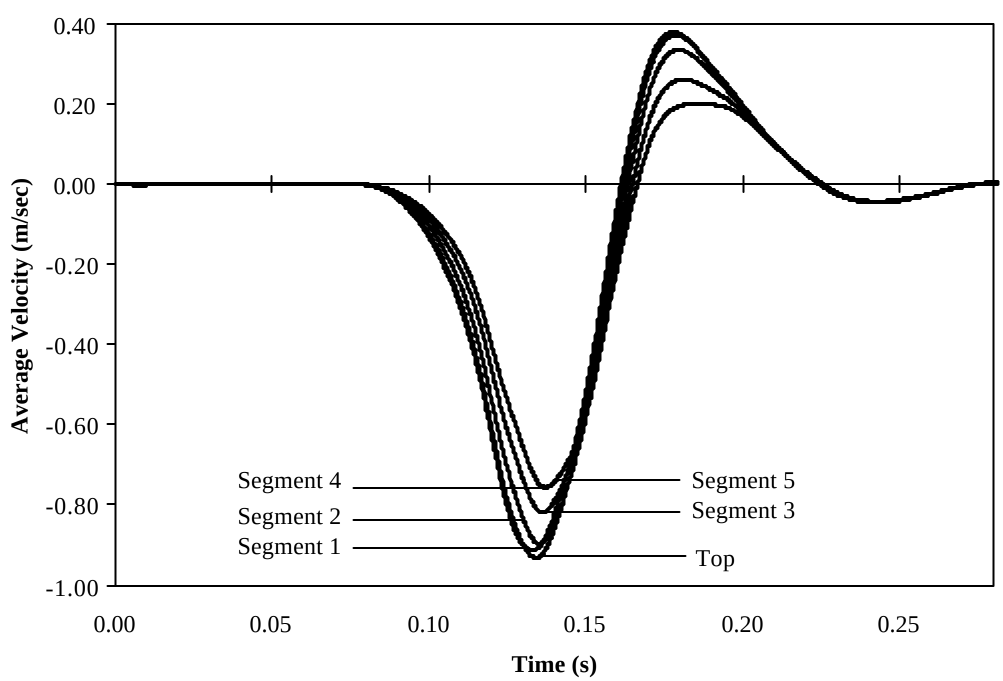

Figure 7 Segmental Average Velocity

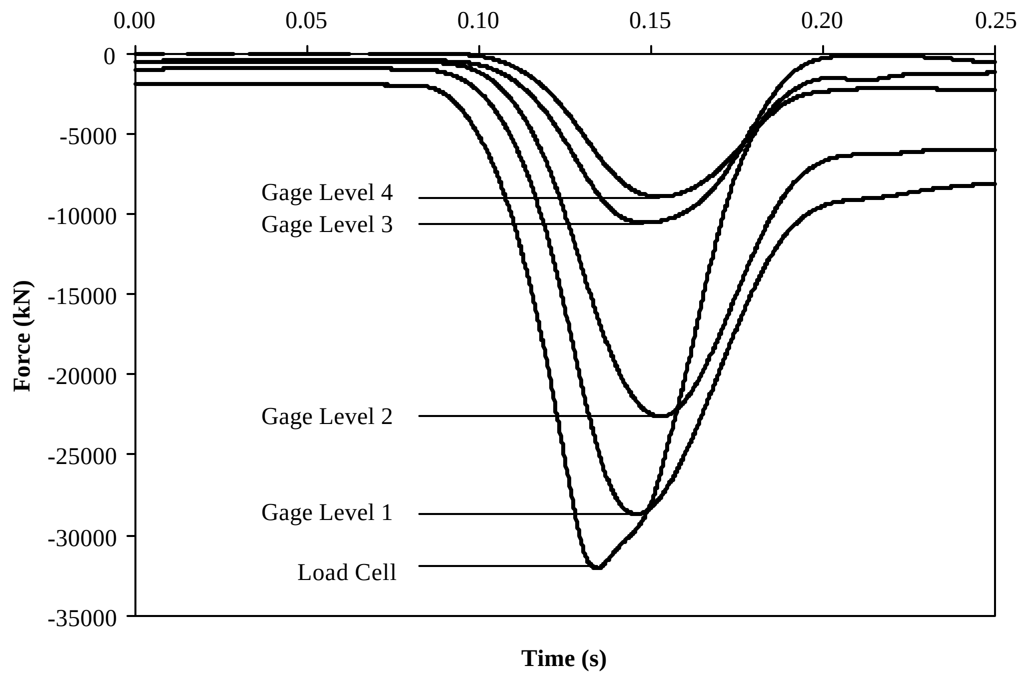

Figure 8 Force at Gage Levels

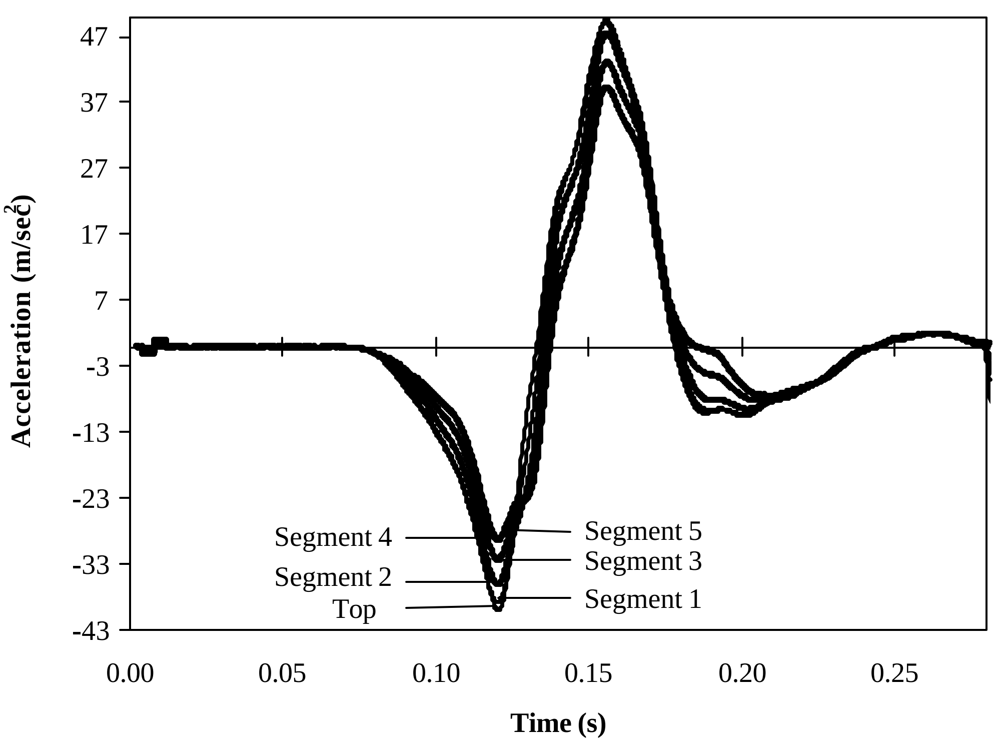

Figure 9 Segmental Average Acceleration

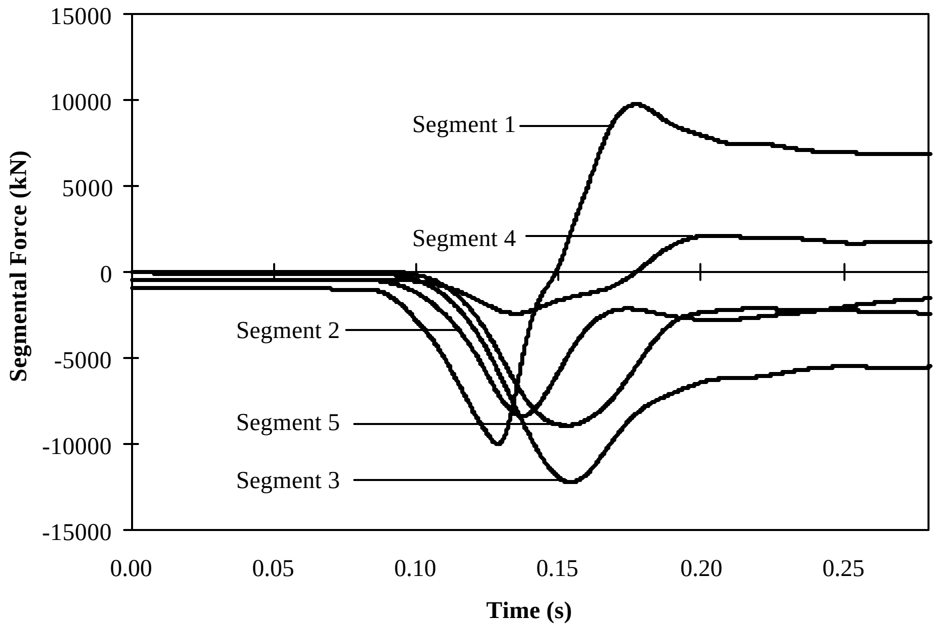

Figure 10 Segmental Statnamic Forces

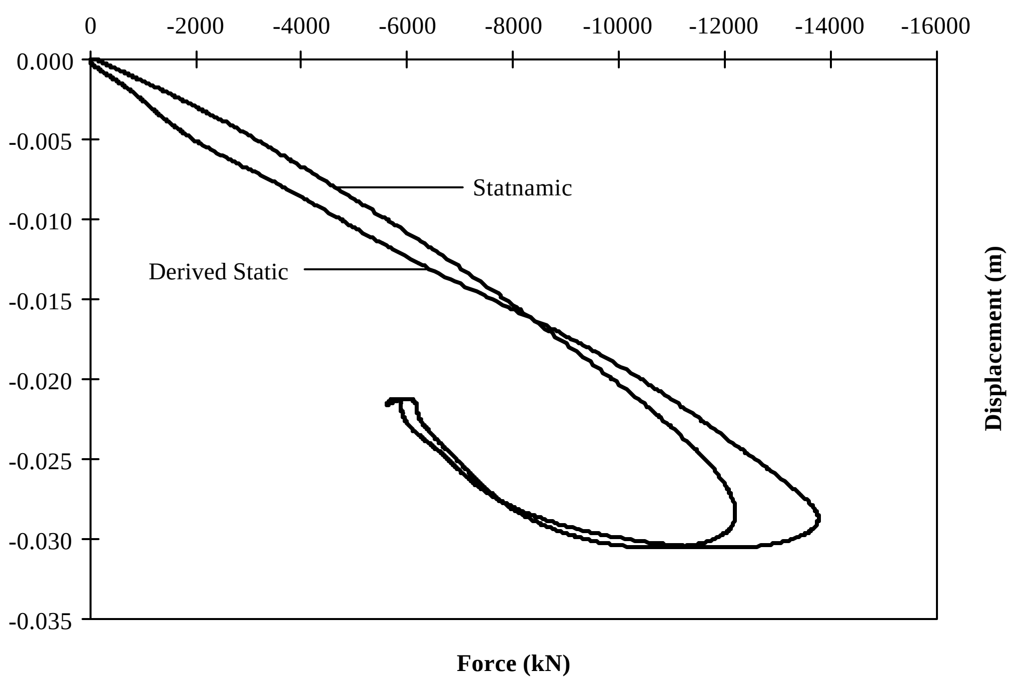

Figure 11 Segment Load Displacement Curves

Using the segmental forces, the derived static soil resistance is determined for each of the segments. Figure 11 shows a typical segment load versus segment displacement curve. By simply dividing the segment force by circumferential surface area this curve can be converted in to a shear stress versus displacement (T-Z) curve for that specific soil strata.

It can be seen in Figure 10 that the peak of each segment force may occur at different times and therefore at different top-of-pile displacements. This can effect the pile capacity in that the ultimate shear strengths of the strata should not be simply summed. This is most probably not a significant concern with this pile due to the plunging nature of the failure. This is evidenced by the similar top and toe movements shown in Figure 6. However, to be technically correct, SUP uses the summation of segment forces as they were developed. This accounts for upper soil layers that may fully mobilize and become residual in nature while lower soil layers begin to develop ultimate strengths. Figure 12 shows the raw Statnamic load displacement curve as well as SUP, MUP, and UP derived static capacity versus the top-of-pile displacement.

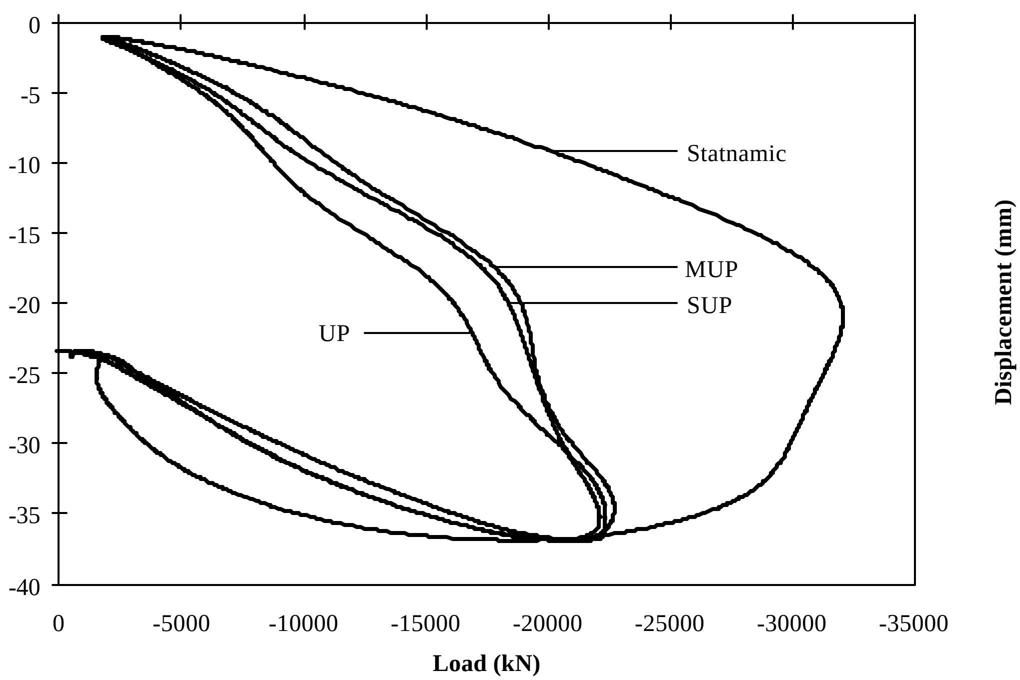

Figure 12 Top of Pile Load versus Displacement

SUMMARY

A new method of analysis called the Segmental Unloading Point Method (SUP) was presented that evaluates Statnamically loaded foundations as segments whose lengths are defined by embedded strain gage elevations. The recorded strain measurements are used to determine both the segmental motion parameters as well as the segmental force traces. Each segment is then treated as an individual foundation whose static response is derived using either the UP or MUP methods. The summation of each segment contribution with respect to time provides a top of foundation response that more closely incorporates the actual distribution of inertial and damping forces throughout the foundation. This is most important in the analysis of relatively long or fixed-ended piles. Although the UP and MUP methods of analysis are sufficient for most loading conditions, SUP provides information for soil strata T-Z curves as well as cut-off elevation load-displacement curves.

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