098

DYNAMIC PILE MONITORING AND PILE LOAD TEST REPORT

NORTH PLATTE RIVER BRIDGE

LINGEL - BARNES ROAD

GOSHEN COUNTY, WYOMING

FHWA-DP-66-11

Federal Highway Administration

Office of Highway Operations

Demonstration Projects Program

Washington, D.C.

July 1985

DYNAMIC PILE MONITORING AND PILE LOAD TEST REPORT

NORTH PLATTE RIVER BRIDGE

LINGEL - BARNES ROAD

GOSHEN COUNTY, WYOMING

Federal Highway Administration

Office of Highway Operations

Demonstration Projects Program

Washington, D.C.

July 1985

Dynamic Pile Monitoring and Pile Load Test Report
Lingle-Barnes Road, Goshen County, Wyoming
July 1985

Introduction and Background

A formal request for the subject field demonstration was received from the Wyoming Highway Department (WHD) in October 1984. The purpose for the load test program was, (1) to demonstrate the use of newer and more accurate techniques to determine pile load capacity during driving, (2) to verify the predictions made by the newer techniques by performing static load tests, and (3) to determine design pile lengths for the Lingle-Barnes Road Bridges.

The field work was performed over a period of 3 weeks during June 1985. The dynamic and static load tests were performed by Mr. H. Clark, Civil Engineering Technician, in the Demonstration Projects Division and Mr. S. Vanikar, Geotechnical Engineer, in the Geotechnical and Materials Branch.

Location and Structure Information

The pile load test site was located on Lingle-Barnes Road in Goshen County, Wyoming. The project involves the replacement of two bridges over the North Platte River. Site 1 was located at station 849 + 80 (Abutment No. 2), and site 2 at station 1113 + 79 (Abutment No. 1). The structures will be supported on pile foundations because of scour considerations and the existence of loose to very dense medium to coarse sands and gravels to depths of up to approximately 101 feet below the existing ground surface. Bridge plans show that the pile groups will consist of vertical and batter piles.

Pile Data

Steel H and steel pipe piles were evaluated during the pile testing program. (No static load test was performed on the pipe pile.) The WHD's design for the proposed bridge foundations consisted of 14x73 H piles at the pier locations and 12x53 H piles at the abutments. These piles were evaluated both with and without tip reinforcement. The contractor Scott and Son was given permission to drive a closed end pipe pile (this was not included as part of the contract). A steel plate was welded at the pile tip to close the pipe. The pipe pile was 16-inch OD with a 3/8" wall thickness. The reaction pile system installed for the static load tests consisted of 4 12x53 H piles at site 1 and 4 14x73 H piles at site 2.

Each compression and reaction pile was driven in 40-foot long sections. Additional sections were added until the desired tip elevation or bearing resistance was obtained. One compression test pile at each location was dynamically monitored 24 hours after intial driving to determine the effects of soil set up.

Subsurface Conditions

Test site No. 1 (station 849 + 80) is located near Boring No. TH5. The boring shows a loose to very dense gravelly sand to a depth of 65-feet below the existing ground surface (Elevation 4158). Although no standard penetration tests were taken in this boring, the results of SPT tests in adjacent borings show "N" values which vary from 24 to 47.

This deposit is underlain by a 30-foot layer of dense sand and gravel. A very hard claystone bedrock exists at approximately elevation 4062. The 4 other borings (TH1-TH4) at this bridge site indicate a fairly similar subsurface profile. Both the gravelly sand, and the sand and gravel layers were evaluated using a 2" diameter drive point penetrometer. The cone information was only qualitatively used in our evaluation and will not be discussed further in this report. The results of these tests show a gradual and consistent increase in density with depth for both layers. The ground water surface in all borings was either at or above the ground surface.

Test site No. 2 (station 1113 + 79) is located near Boring No. TH1. The boring shows a loose to very dense sand and gravel layer to a depth of 101-feet below the existing ground surface (Elevation 4184). Only two standard penetration tests were taken in this boring (both tests are within 20-feet of the ground surface). Results of these tests indicate SPT "N" values varying from 27 to 41. The sand and gravel layer is underlain by hard claystone bedrock. The four other borings (TH2-TH5) at this bridge site indicate a fairly uniform subsurface profile. Dynamic cone penetrometer tests were conducted in each boring and all results show a consistent and steady increase in resistance with depth. The ground water surface in all borings was either at or above the ground surface.

Hammer Data

The following is the data for the hammer system selected by the contractor:

Mitsubishi MH15 single acting diesel hammer
Rated energy at 8-foot, 6-inch stroke = 28,100 ft. lbs.
Operating speed 42-60 blows per minute
Ram Weight = 3,310 lbs.
Hammer cushion - alternate layers of micarta and aluminum
Total thickness 3 inches
Pile Cushion - none

Dynamic Monitoring (Pile Analyzer) Results

A total of eleven piles were driven at the bridge sites 5-12x53 H piles, 5-14x73 H piles and 1-16 inch, 3/8-inch wall thickness closed end pipe pile. Seven piles were dynamically monitored 3-12x53 (site No. 1), 3-14x73 (site No. 2), 1 pipe pile (site No. 2).

Site No. 1

The compression test pile, 12x53 H, was initially driven to an embedment depth of 70-feet with tip reinforcement. The ultimate resistance at this depth was 150 tons during initial driving and no increase in capacity was noted after a 24-hour retap test. Reaction pile No. 1 was driven to 75-feet and had an ultimate capacity of 147 tons with tip reinforcement. Reaction pile No. 4 was driven to 70-feet and had an ultimate resistance of 154 tons without tip reinforcement. The design load for the 12x53 H piles is 70 tons at an estimated length of 75-feet. Safety factor is equal to 2.0.

The hammer was in good operating condition during the driving series and was operating at a speed range of 43-48 blows per foot and transfer efficiency of approximately 35 percent was recorded. The compressive stresses in the test piles reached a maximum of approximately 25 ksi which is well below the maximum recommended driving stress of 32.4 ksi.

The lack of soil setup is not surprising based on the coarse grained structure of the subsurface soils. The recorded capacity was obtained at a fairly consistent resistance of 35-50 blows per feet.

A static analysis of the compression pile's ultimate bearing capacity (using the standard penetration test results and the Nordlund design method) yielded a value of 160 tons for a driven length of 72-feet (11 percent point resistance and 89 percent skin friction). Based on the limited amount of standard penetration test information, our confidence is very limited in this analysis.

Site No. 2

The compression test pile 14x73 H was initially driven to an embedment depth of 85-feet, without tip reinforcement. The ultimate resistance at this depth was 163 tons during initial driving and approximately 200 tons during pile retap after 24 hours. This increase in capacity was also verified by the results of the static load test. Reactions pile No. 2 was driven to a depth of 88-feet and had an ultimate capacity of 178 tons with no tip resistance. Reaction pile No. 4 was driven to a depth of 86-feet and had an ultimate capacity of 165 tons with tip reinforcement. The design load for the 14x73 H piles is 96 tons at an estimated length of 85-feet. Safety factor is equal to 2.0.

The 16" closed end pipe pile was driven to an embedment length of 65-feet. At that depth, it had an ultimate bearing capacity of 193 tons. No retap test was performed on this pile. Assuming the pile would be unfilled, and have a yield strength of 35 KSI, then the allowable design load (based on pile material stresses) would be 80 tons. (Higher design loads would be permissible if it were filled with concrete.) This 80 ton design load could be obtained with an embedment depth of approximately 53-feet.

The hammer was in good operating condition during this driving series and was operating at a speed range of 44-50 blows per foot and a transfer efficiency of approximately 30-35 percent. The recorded bearing capacities were obtained at 47 blows per foot for the pipe pile and a range of 40-55 blows per foot for the H pile. Maximum compressive stresses during driving in the H piles were 24 ksi and in the pipe pile 26.5 ksi. These are both within acceptable limits.

A static analysis of the compression pile's ultimate bearing capacity (using the standard penetration test results and Nordlund's design method) yielded a value of 150 tons at an estimated length of 85-feet (8 percent point resistance and 92 percent skin resistance). A similar analysis for the 16" pipe pile yielded an ultimate capacity of 227 tons at an estimated length of 65-feet. (39 percent point resistance and 61 percent skin friction). Based on the limited about of data our confidence is in this analysis is low.

Static Pile Load Tests

The FHWA provided the load test frame and accessory equipment including the precision load measuring equipment. The FHWA personnel also provided the technical assistance for conducting the load tests. One static compression load test was conducted at each site. One 12x53 steel H pile and one 14x73 steel H pile were load tested to failure.

The load applied was measured by electronic load cells and also by a pressure gage. The pile deflection at the top was measured by two dial gages 180° apart from each other. The load versus settlement curves were plotted by using load cell measurements for the axial load and the pile settlement was computed from dial gage readings. The average dial reading was used at site 1 but only dial gage No. 2 was used at site No. 2 due to variations between the two gages. The deflection data from gage No. 1 at site No. 2 was considered suspect and not used. The compression piles were not instrumented along their lengths for determining load-transfer distribution. The quick load test procedure as per ASTM D 1143-81 was used for both load tests.

Figure 1 shows the load-settlement curve for the compression pile at site no. 1 and Figure 2 presents the results at site no. 2. The scale used was as per Professor Davissons' recommendations for estimating failure loads. In the absence of instrumentation for obtaining load transfer distribution along the pile length, the elastic compression line was plotted by assuming that all the load was transferred to the pile tip.

At site no. 1, a sudden plunging failure occurred at a load of 175 tons when a constant load could not be maintained on the pile. Figure 1 shows that the "Davisson Criteria" provides a failure load of 140 tons. The rebound curve shows a residual settlement of 0.4 inch, after all the load on the pile was removed.

At Site No. 2, a disagreement between the dial gages on the order .6 inches (at 240 tons) caused us to examine each gage separately. Based on the shape of the load settlement curve we placed more confidence in the deflection measurements of gage No. 2. The cause of the gage disagreement is unknown and unfortunately no backup measurement system was used during the program.

Figure 2 shows that the "Davisson Criteria" provides a failure load of 210 tons (using dial gage No. 2) the rebound curve shows a residual settlement of 0.7 inches after all the load on the pile was removed.

Comparison of Static Analysis, Dynamic Monitoring (Pile Analyzer) Prediction with Static Load Tests.

Table 1 shows a comparison of static analysis, pile analyzer predictions, and static load test failure loads for the 12x53 steel H pile. It should be noted that the pile analyzer was also used for estimating ultimate capacities of two 12x53 H reaction piles. The ultimate predictions for these piles were 145 tons (75-feet) and 157 ton (70-feet). No soil set up appears to be occuring at this site. Fairly good agreement exists between the three methods indicating that the design load of 70 tons is obtainable at the length of 70-75 feet. A back analysis using static design methods indicates that the 14x73 H piles can obtain a 96 ton design load at an estimated length of 80-feet. Assuming this analysis is valid driven lengths of 85-90 feet should yield the necessary resistance. Our confidence in this analysis is limited based on the fact that no 14x73 H piles were driven at site No. 1 and the limited number of soil samples and SPT data which is available.

Site No. 1 TABLE 1

Comparison of Static Analysis and Dynamic Monitoring (Pile Analyzer) Predictions with static load tests - 12x53 steel H pile.

Method used for Determining Ultimate Capacity	Ultimate (Failure) Load
Static Analysis	160 tons
Prediction by dynamic monitoring	150 tons (147 and 154 tons reaction piles)
Static Load test	140 tons

Conclusions and Recommendations

1. The dynamic monitoring equipment (pile analyzer) performed well in monitoring driving stresses and hammer performance. The predicted ultimate pile load capacities by the analyzer compare very well with the results of the static load tests. The analyzer also provides a tool to detect and assess pile damage. This demonstrates the tremendous advantages provided by the equipment.

2. The static analysis and dynamic monitoring (during initial driving) compared well with the results of static load test at site No. 1. The static analysis under estimated the pile capacity for the H pile and pipe pile at site No. 2.

3. Little confidence could be placed in the static analysis based on limited subsurface samples and SPT tests. Additional data should be obtained on subsequent projects to insure a higher degree of confidence. Also the extrapolation of the conclusions in the report to other substructure locations is done with little confidence.

   Without the availability of soil grain size information and other index tests for the entire soil profile there is no way to confidently predict soil setup. We suggest that provisions be made for the actual driven pile lengths to be a minimum of 10-feet longer than indicated in our recommendations.

4. No soil setup can be expected at site No. 1 and the resistance obtained during driving should be indicative of the pile's ultimate capacity. At site No. 2, 35 tons of soil setup was measured by the pile analyzer and this was verified by the static load test. During the production driving several piles at site No. 2 should be retapped to verify that setup is in fact occurring.

5. The 12x53 H piles should provide at design load of 70 tons, at each site, length of approximately 70-75 feet.

6. The 14x73 H piles should provide a design load of 96 tons at site 1 from 80-85 feet and site 2 from 85-90 feet.

7. The 16" diameter pipe pile should provide a design load of 80 tons at 53-feet of embedment. This estimate does not account for any setup as would be expected at this site based on the results obtained for the 14x73 H pile. On future projects, with similar subsurface profiles, consideration should be given to using a displacement pile type (closed end pipe or precast concrete) rather than H piles. These pile types would yield a more economical foundation.

8. No ~~different~~ difference was recorded during driving of the H piles with or without tip reinforcement. Hence, all H-piles can be driven without tips and no pile damage should occur.

9. The lack of a backup settlement measurement system for the static load tests made interpretation of the results at site No. 2 very difficult. A mirror and wire system or survey measurements should be used on all subsequent pile test projects.

10. The data contained within this report should be used to reevaluate the wave equation analyzes for the project. The revised analyzes should be based on analyzer and driving information recorded during the test program. By conducting these analyzes at several pile lengths (± 10-feet of estimated length for each pile type) construction personnel can control production driving. Figure 3 and 4 show the results of these revised analyses for a 70-foot long 12x53 H and an 85-foot 14x73 H pile. A copy of the wave equation output and input are being transmitted to the State with their copy.

PREPARED BY

Jerry DiMaggio 7/17/85

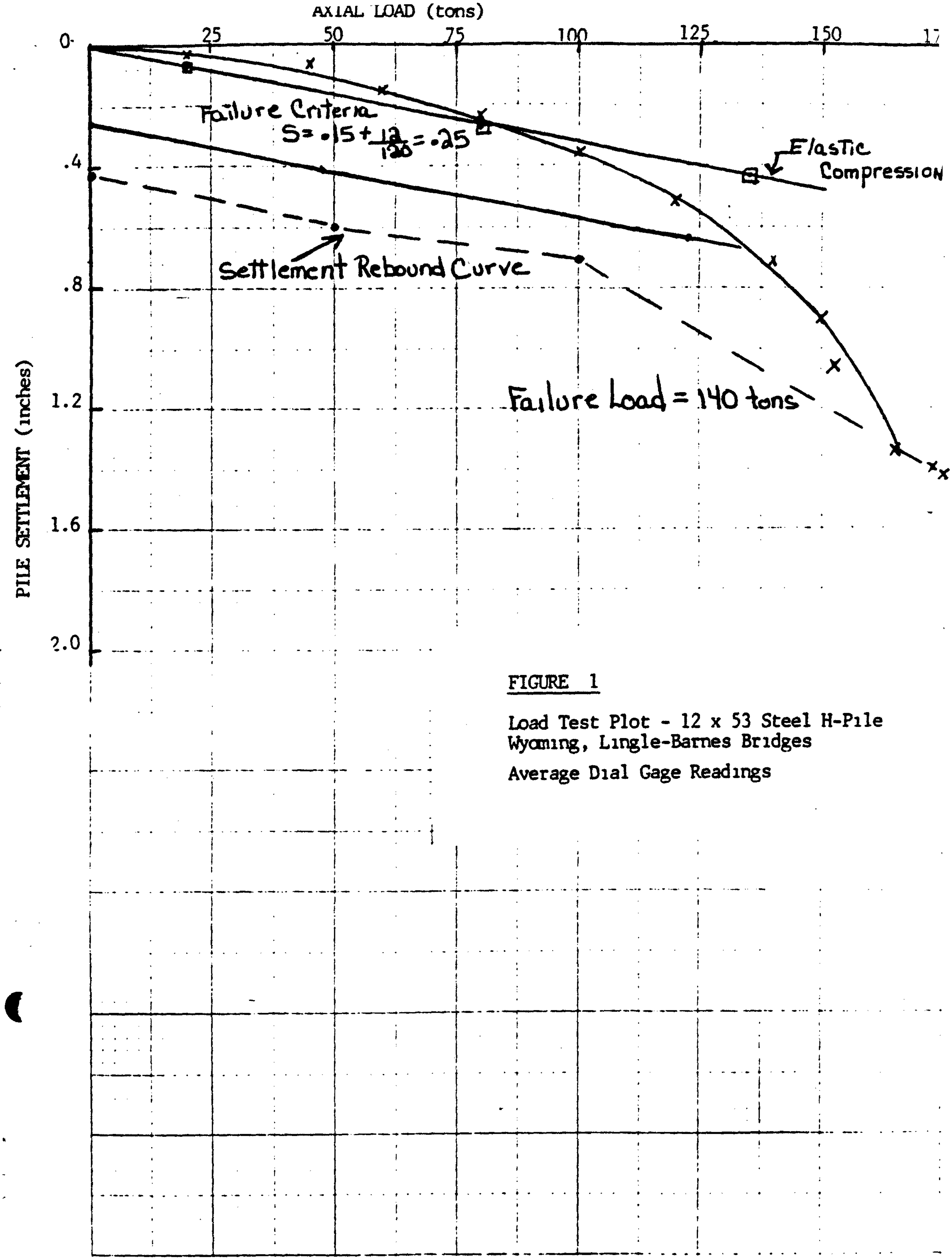


FIGURE 1

Load Test Plot - 12 x 53 Steel H-Pile
Wyoming, Lingle-Barnes Bridges
Average Dial Gage Readings

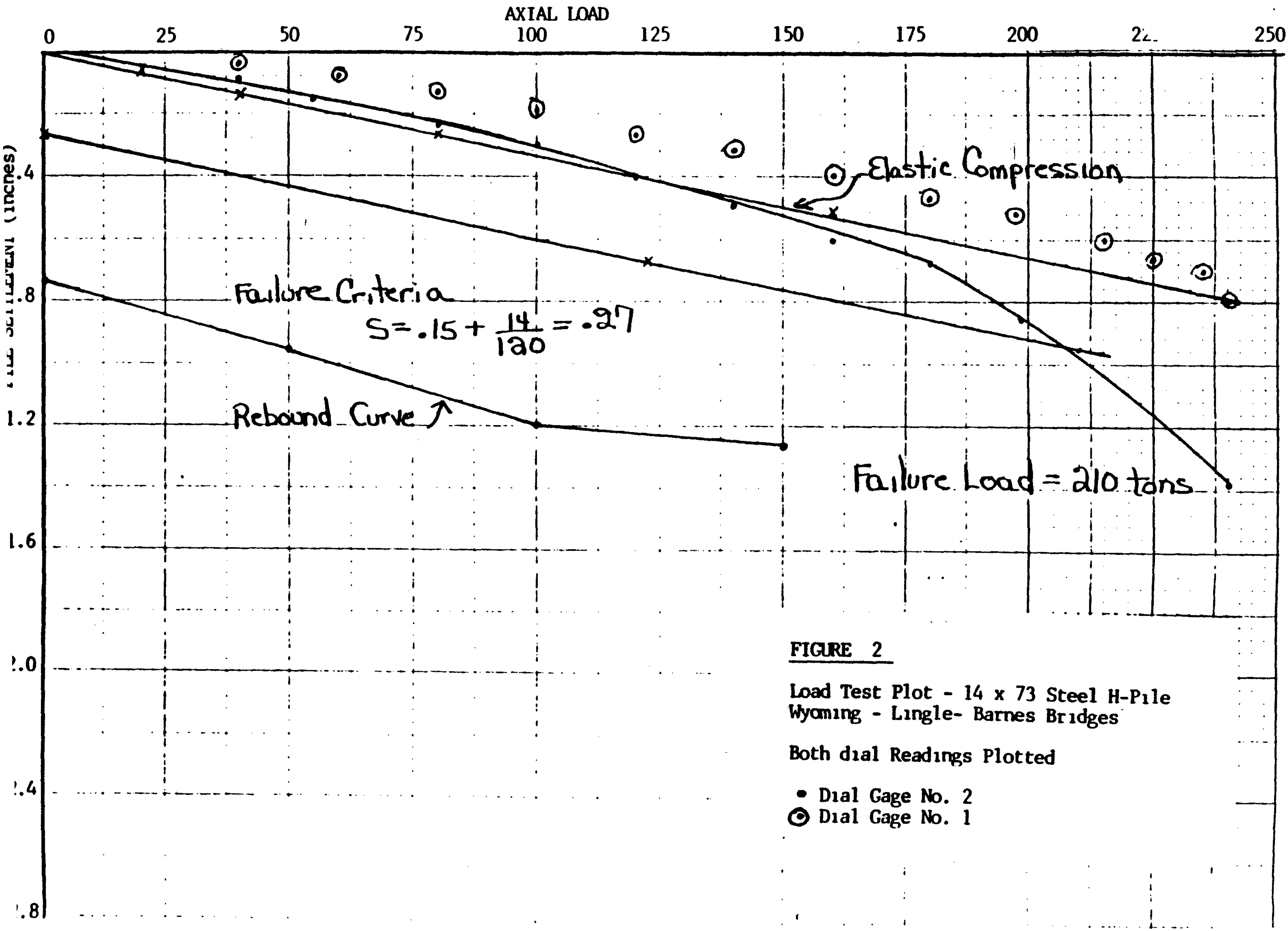
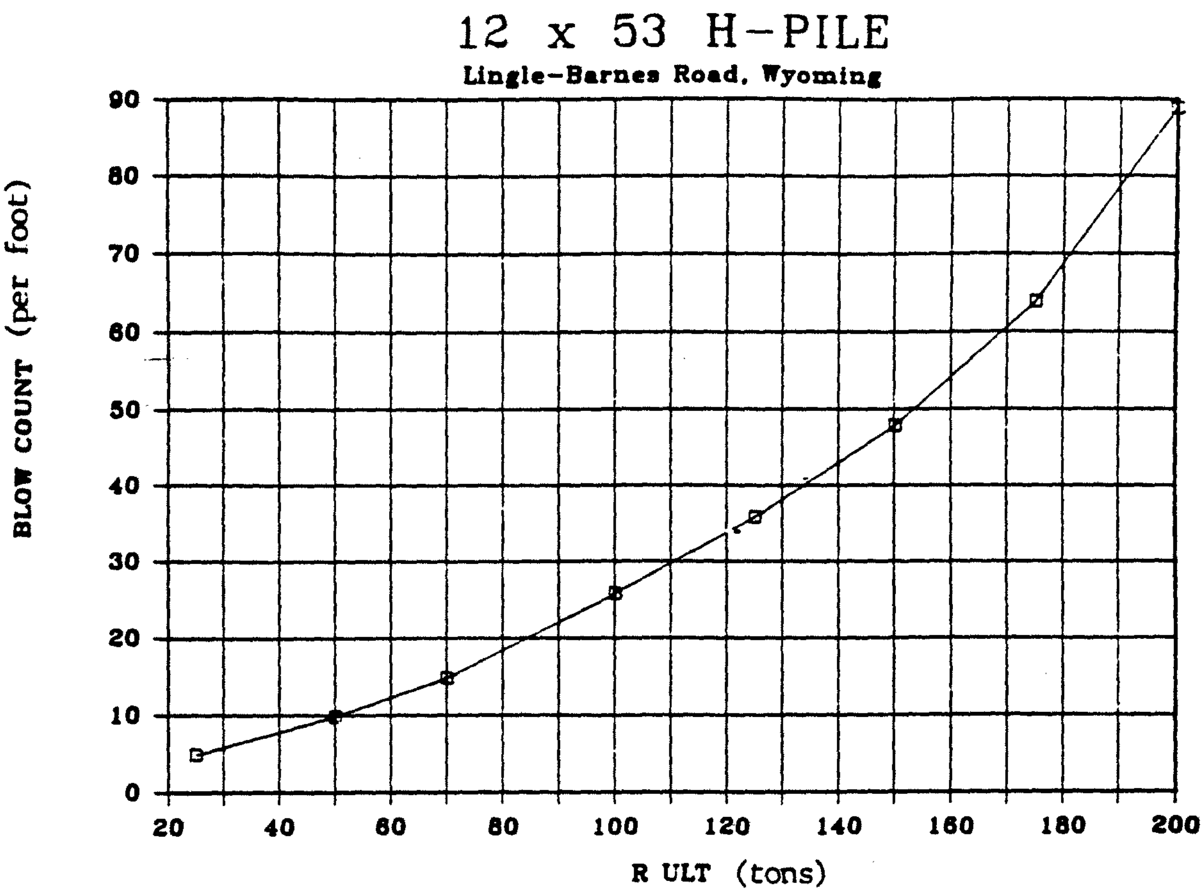


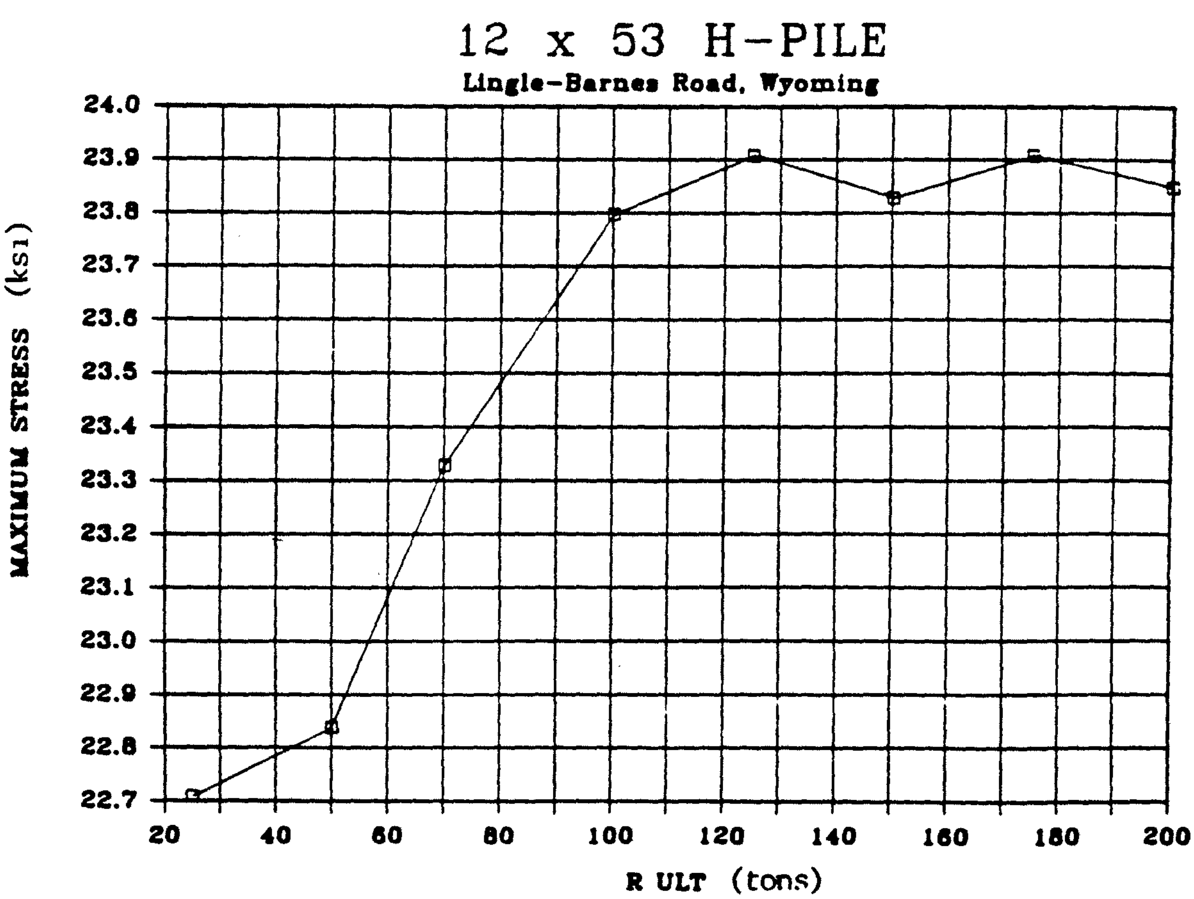
FIGURE 2

Load Test Plot - 14 x 73 Steel H-Pile
Wyoming - Lingle- Barnes Bridges

Both dial Readings Plotted

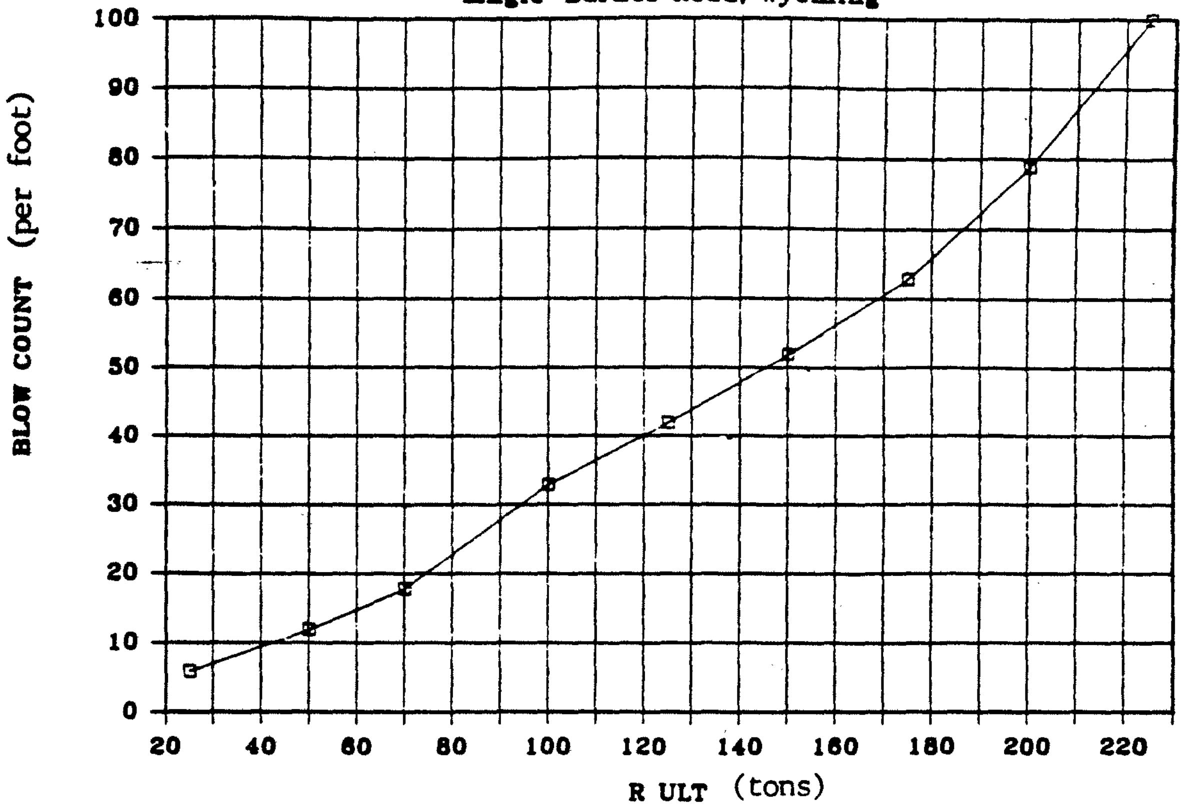
• Dial Gage No. 2
⊙ Dial Gage No. 1


12 x 53 H-PILE
Lingle-Barnes Road, Wyoming
BLOW COUNT (per foot)
R ULT (tons)


12 x 53 H-PILE
Lingle-Barnes Road, Wyoming
MAXIMUM STRESS (ksi)
R ULT (tons)

14 x 73 H-PILE

Lingle-Barnes Road, Wyoming



14 x 73 H-PILE

Lingle-Barnes Road, Wyoming