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DEVELOPMENT OF DYNAMIC TEST FOR ANALYZING FREE VIBRATIONS OF BRIDGE PIERS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

David Herschel McAlister

December, 1994

DEDICATION

This thesis is dedicated to my parents, Mr. Ed McAlister and Mrs. Rena McAlister. They have given me the love, support, and opportunities to accomplish all my academic endeavors.

ACKNOWLEDGEMENTS

I would like to express my gratitude to all the people that helped me complete this undertaking. I especially wish to thank my major professor, Dr. James Harold Deatherage. He influenced me to enter graduate school and has always been quick to lead me in the right direction. I respect him greatly and will miss his guidance. I would also like to thank my other committee members, Dr. Edwin G. Burdette and Dr. David Goodpasture. All three of these professors have guided me and I thank them for it. Last, but not least, I would like to thank Kevin Bowling and Richard Graves for their assistance on this project.

ABSTRACT

A quick release test on piles was designed to aid in the understanding of the soil-pier interface of bridge piers under seismic loads. All facets of the test were designed and proof tested in the laboratory before field tests were performed. The test was specifically for use in loess soil, but was designed to be applied to tests in any type soil.

The results of the preliminary field tests demonstrate the acceptability of the design as a quick release test to determine the dynamic characteristics of the soil-pile interface. The tests resulted in a static stiffness of 66.67 kips per inch in loess soil. The tests also provided a frequency of 15 hertz for a unit mass pile. This frequency should be reduced considerably by attaching a lumped mass to the system. A damping ratio of 13.9 percent of critical damping also was recorded. This quick release test demonstrated the acceptability of the procedure for future tests.

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CHAPTER 1

INTRODUCTION

The 1989 Loma Prieta earthquake caused an enormous amount of damage to the California transit system. States across the country became concerned about their own highway systems if an earthquake of significant magnitude was to occur in their region. As can be seen in Figure 1, the state of Tennessee is especially interested because the western portion of the state is in either seismic zone two or three, and the eastern portion of the state is in seismic zone two. Bridges in Tennessee have generally been designed for the expected traffic loads and not analyzed for seismic activity. Currently, the Tennessee Department of Transportation (TNDOT) bridge engineers are conducting a damage assessment study on the state's bridges should they experience loads caused by an earthquake. The state of Tennessee utilizes the computer program, SEISAB, which was developed by Ibsen and Associates, to analyze bridges under these conditions. The SEISAB program allows for input of six stiffness values, three for rotation and three for translation, at the soil-pile interfaces. These values are applicable for both the bridge abutments and bents. SEISAB also allows for consideration of a particular pile group arrangement and stiffness values for individual piles to be input to model the behavior of the foundation (Cook, 94).

To conduct an accurate study the support conditions must be known. The bridges in West Tennessee are founded on a wind-

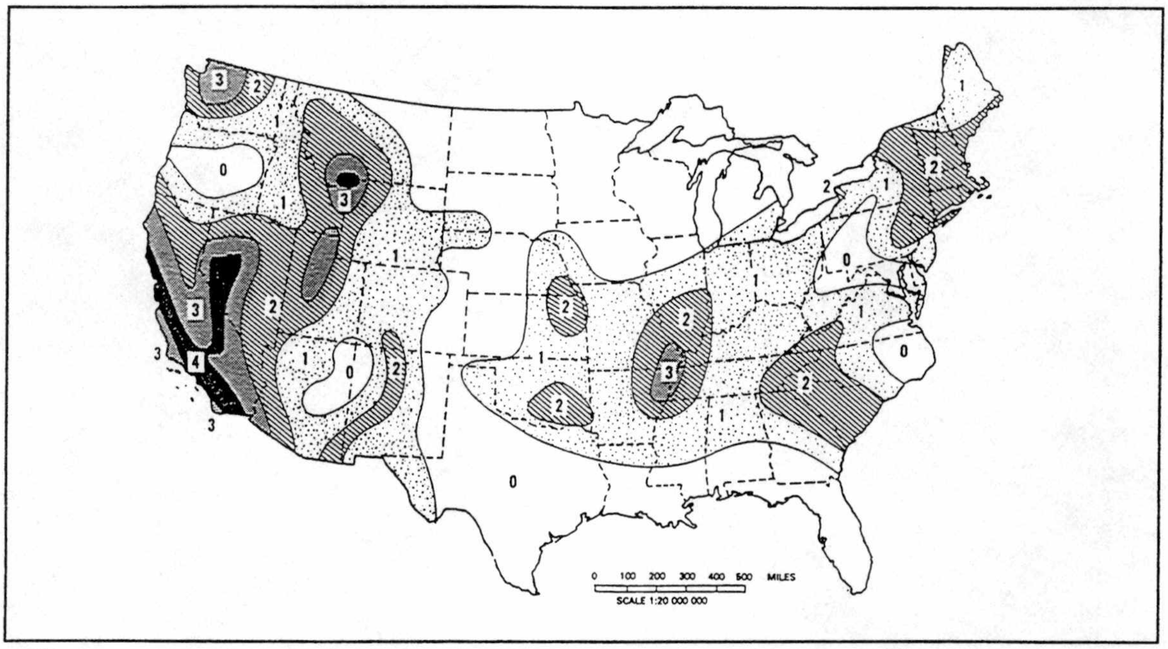

Figure 1. Map of Seismic Zones - Contiguous 48 States.

Source: American Society of Civil Engineers (1990). Minimum Design Loads for Buildings and Other Structures (ANSI/ ASCE 7-88). New York, New York. p. 33.

deposited soil known as loess which creates a challenge when attempting to model the soil-pier interface. The soil-pier interface is currently assumed to be fixed due to lack of better data. This assumption can lead to either unconservative or conservative predictions under certain loading conditions.

The Tennessee Department of Transportation (TNDOT) has contracted with the Department of Civil Engineering at the University of Tennessee to investigate the stiffness coefficients at the soil-pier interface. The initial purpose of the "bridge pier" project was to provide TNDOT with information on the actual soil-pier interface and thus lead them to either accept the fixed condition with confidence or refine the support conditions which are input in SEISAB. A sensitivity study was conducted to learn if the soil-pier interface was fixed, and if not, how the actual interface would affect the behavior of the superstructure in the event of an earthquake. Three important conclusions were derived from the sensitivity study. The most important conclusion was that varying the torsional and rotational springs had little or no effect on the column forces and moments. This allows future analysis to focus on the lateral spring stiffnesses which are critical to the column forces and moments. The second conclusion of the SEISAB analysis was that the moments and forces in the columns are not sensitive to slight variations of the soil stiffnesses. The third conclusion was that the current modeling procedure can be conservative or unconservative based on the abutment stiffnesses. For relatively low bent stiffnesses and the currently modeled abutment stiffnesses, the forces and moments are lower than the current values. If lower abutment stiffnesses are used with the lower bent stiffnesses, the forces and moments are higher than current values. These conclusions indicate a need to seek better information regarding spring coefficients.

The three conclusions from the sensitivity study and the fact that little is known about loess soil, lead us to conduct field tests to investigate the values of the spring constants at the soil-pier interface. The field tests will be "pluck" tests on single piles. These tests will be performed at a bridge site southeast of Memphis, in conjunction with the University of Memphis. The tests will consist of various "pluck tests" on two fourteen inch square concrete friction piles driven to depths of fifteen and seventeen feet. A mass will be attached to the pile cap to cause the pile to vibrate at approximately the natural frequency of the superstructure. The pile will be pulled horizontally under a static load to a predetermined deflection and suddenly released. The displacement, acceleration, and velocity time history of the pile cap will be recorded in order to obtain the dynamic properties of the soil.

The most important factor in performing the test will be the quick release of the pile. This quick release will be intended to permit free vibration of the pile at the natural frequency of the superstructure. It was determined that a frequency either significantly higher or lower than the natural frequency of the superstructure would give results of questionable usefulness. Analysis of the "pluck test" will allow the investigators to calculate the actual lateral spring coefficient of the soil.

Dr. Stu Chen from the State University of New York at Buffalo is currently conducting full scale bridge tests. The tests recorded an acceleration of 0.05g instead of the expected 0.10g. The reduction in acceleration was attributed to the fact that the release was too slow, a problem which can be remedied by effecting a instantaneous release of the pile.

The purpose of this thesis was to develop a test plan and a design for a "pluck" test of single pile foundations. Each component used in the test was proof tested, and only after an optimum confidence level was achieved could the actual field tests be conducted.

CHAPTER 2

REVIEW OF LITERATURE

The sensitivity analysis phase of the research project was conducted by Tommy Cook (1994), a University of Tennessee graduate student. His goal was to determine how dynamic analysis results were affected by varying the foundation spring coefficients. The sensitivity analysis was conducted on two different bridges, one a two-span bridge in Madison County, Tennessee; and the other a three-span, free standing pile bent bridge in Haywood County, Tennessee. Two sets of analyses were conducted on each bridge, one set with the abutments and bents at a ninety degree skew angle (no skew) and the other set with the abutments and bents at a forty five degree skew angle. Each set consisted of four analyses: one with the foundation springs at the abutments and the bents fixed, one with the foundation springs at both the abutments and the bents, one with the bent springs divided by ten, and one with both the bent springs and the abutment springs divided by ten.

The Tennessee Department of Transportation (TNDOT) currently models bridges using calculated spring coefficients for the abutments, while assuming that the bents' spring coefficients are fixed. For these analyses, the foundation spring coefficients for the bents were calculated by Tommy Cook, while the spring coefficients for the abutments were calculated by TNDOT. The bent springs were calculated using two different methods. The Madison County bridge springs were calculated using an approximate method developed by

Novak (1974), while The Haywood County bridge was calculated using beam on elastic foundation theory (Cook, 1994). The lack of knowledge of the dynamic properties of loess soil leads to the reduction of the bent spring coefficients. The abutment springs were not considered to be as critical due to the fact that the backfill surrounding the abutments can be controlled.

Results of Cook's analyses are shown in Figures 2 through 5. The maximum moment and maximum axial force in the bent columns are plotted for each analysis. The columns labeled "Bent Fixed" are the values corresponding to the current method that TNDOT utilizes to model bridges. The columns labeled "Bent Springs" are the values with the current abutment springs and the bent springs calculated by Cook (1994). The columns labeled "Bent Springs/10" are the values for the current abutment springs and the calculated bent springs divided by ten. The last column labeled "All Springs/10" are the values for all the foundation springs divided by ten.

As was stated in the introduction, there are three important conclusions in Cook's thesis. The most important conclusion was that varying the rotational and the torsional springs had a nominal effect on the column moments and axial forces. This conclusion allows for a pluck test to be conducted which concentrates only on the horizontal forces. A second conclusion showed that the forces and moments of the bents are not extremely sensitive to slight variations in spring stiffnesses. The third conclusion shows that the current modeling procedure can give either conservative and unconservative designs based on the magnitude of the bent springs. These three conclusions

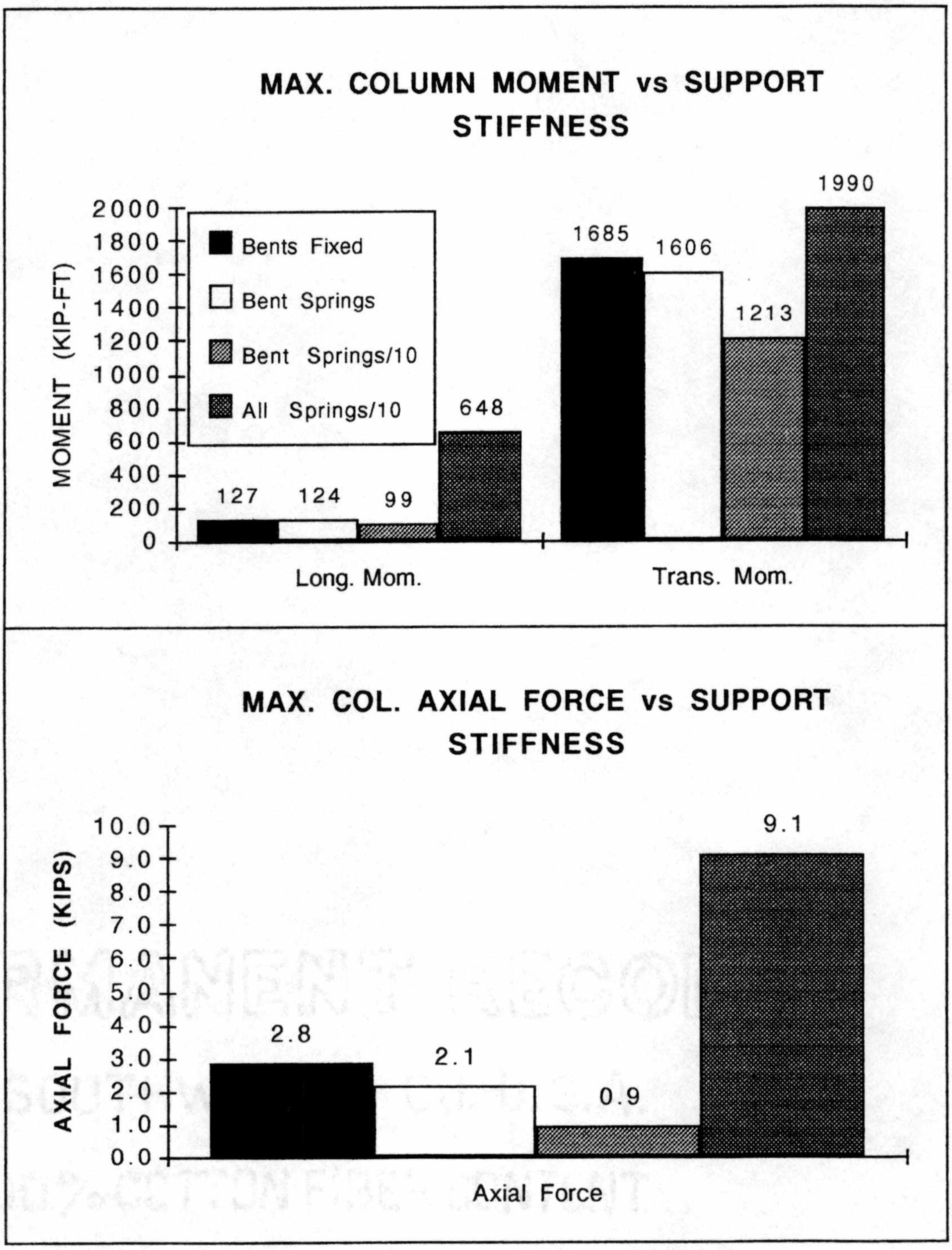

Figure 2. Madison County Bridge - 90 Degree Skew

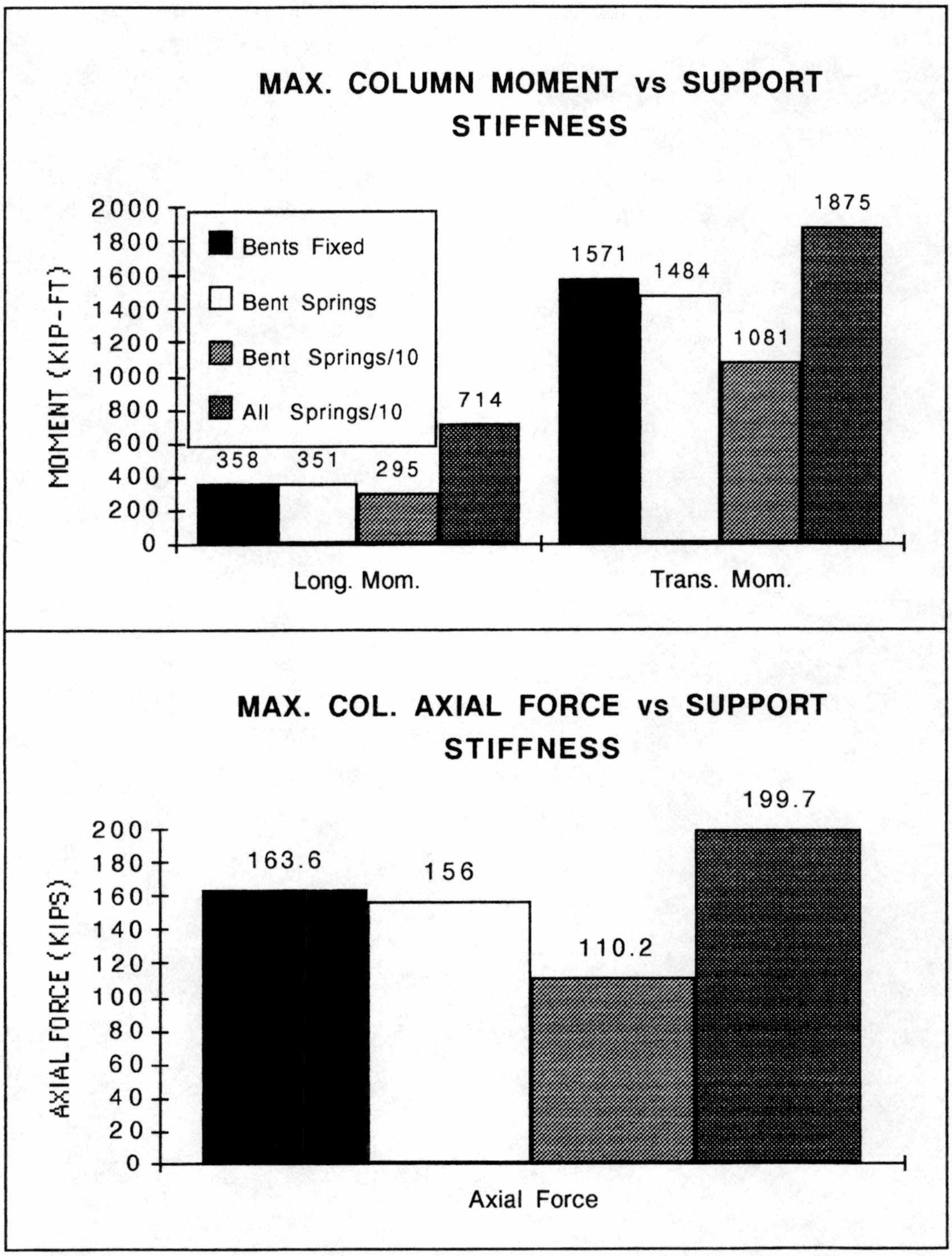

Figure 3. Madison County Bridge - 45 Degree Skew

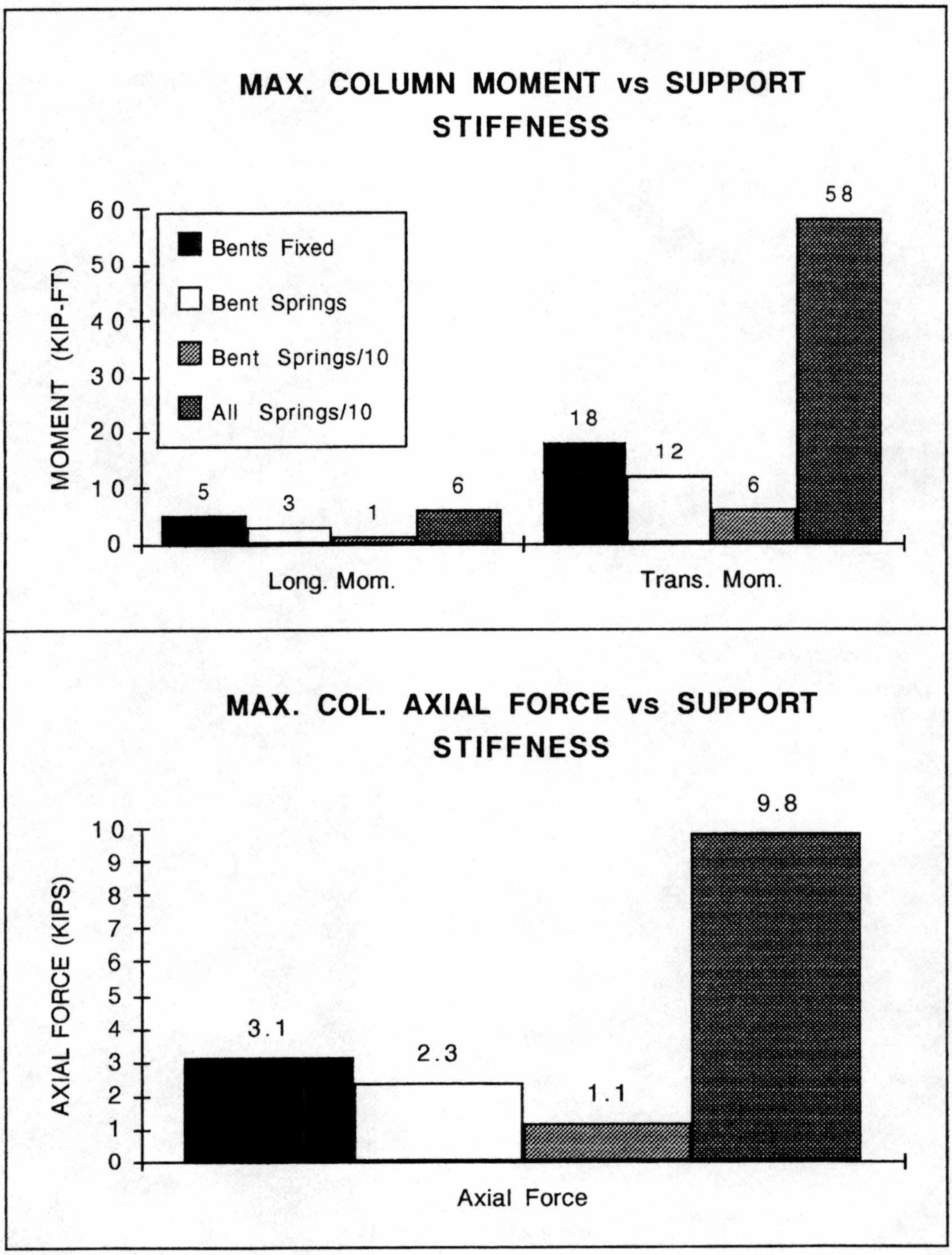

Figure 4. Haywood County - 90 Degree Skew

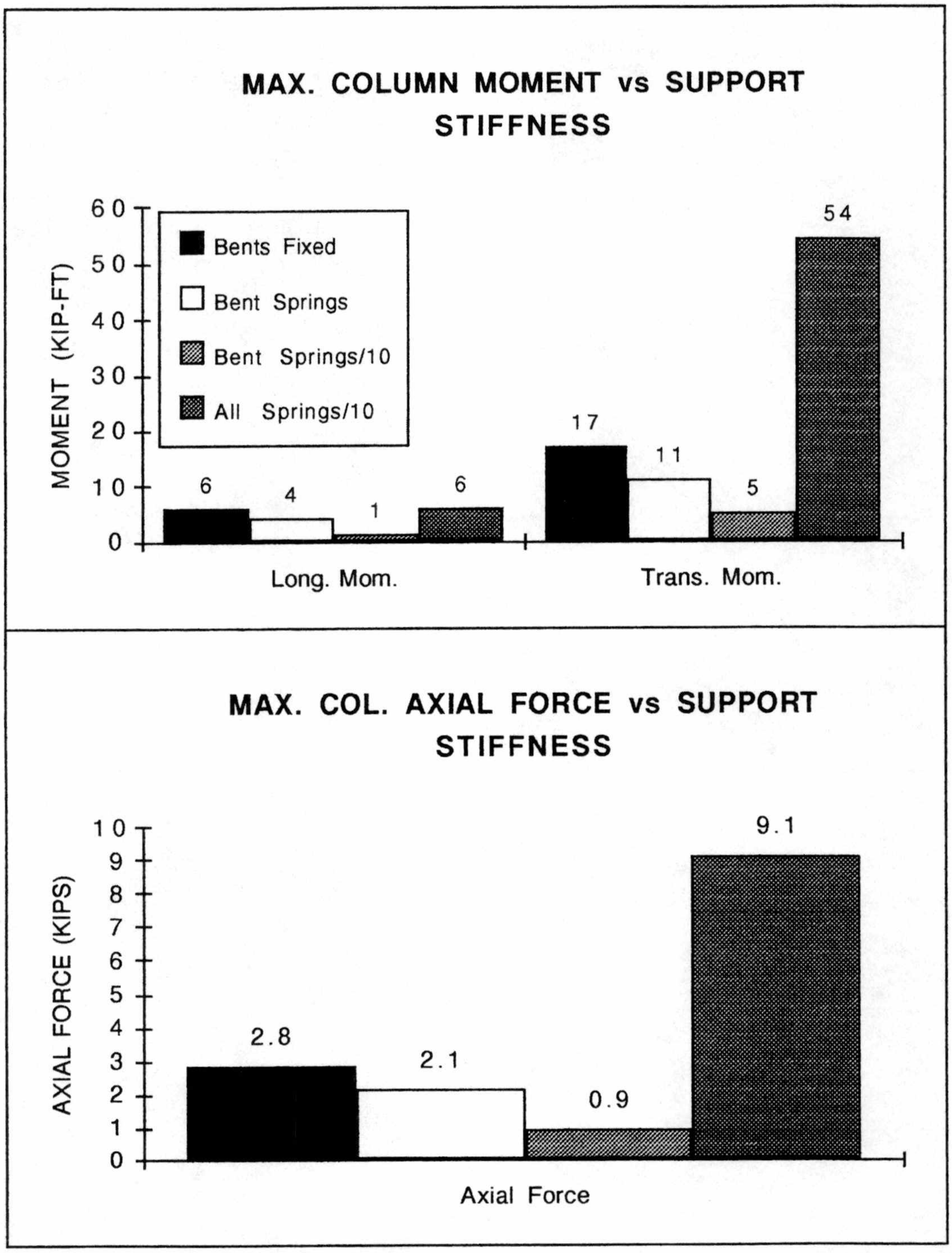

Figure 5. Haywood County Bridge - 45 Degree Skew

show the need for additional study for the purpose of calculating approximate spring coefficients of bridges in loess (Cook, 1994).

Many researchers have conducted dynamic tests to gain insight into the behavior of the soil-pile interface, yet there is very little data for piles in loess. Gle and Woods (1984) developed a system of dynamic tests for the evaluation of deep foundations supporting vibratory equipment. In order to analyze the effect of vibrations on the soil-pile interface, both a steady-state vibration and a pluck test were developed. The tests were conducted on eleven steel-pipe piles placed in both cohesive and cohesionless soils. A mass was attached to the pile cap such that a well-defined dynamic response curve would result.

A primary objective of the tests was to get the mass located as close to the surface of the ground as possible. This allowed the dynamic lateral response of the system to be analyzed as a single degree of freedom. As the mass is raised the dynamic response becomes that of a cantilever system where the structural response of the beam controls instead of the soil-pile interface, which results in a multi-degree of freedom system. Test results showed that the soil-pile interaction increased greatly as the mass was lowered from 1.5 meters above the ground surface to just above the ground surface (Gle and Woods, 1984).

The steady-state vibration tests were their main emphasis, with the pluck tests used to gather supplementary vibratory information. The pluck tests were conducted by applying an impulse force to the mass and monitoring the free vibration of the system.

The impulse force was applied by striking the mass with a "wooden plank." From measurements of the free vibration, the critical damping and the damped natural frequency of the soil-pile system were obtained. These tests were conducted at the conclusion of the steady-state tests as the amplitude of vibration could not be controlled. The soil had been previously disturbed by the steady-state tests so the free vibration results were not of great value (Gle and Woods, 1984).

Blaney and O'Neil (1991) conducted similar tests at the University of Houston to obtain a better understanding of the dynamic properties of pile foundations under earthquake loads. The tests consisted of a single pile installed at a bridge site prior to construction. A mass was attached rigidly to the pile above the ground surface. They suggest that the mass size and the distance that it is attached above the ground surface be chosen so that the fundamental frequency of the system be similar to that of the superstructure that the pile is supporting. Typically, systems such as these described are more flexible in the horizontal direction than in rocking, so the horizontal dynamic response is analyzed to a greater degree. The tests were conducted by applying a cyclic horizontal load to the pile, and the soil-pile deflection was recorded. As was the case with Gle and Woods, a free vibration pluck test was conducted after the cyclic tests had been completed. Once again this is not of great importance in understanding the soil-pile interface in loess (Blaney and O'Neil, 1991).

Stuart Chen at the State University of New York at Buffalo conducted a full-scale dynamic bridge test designed to determine the stiffness of the bridge piers from quick release tests. The design criteria called for a system that could be mobile, yet powerful enough to deliver 300 kips of horizontal load. It also had to release quick enough to induce free vibration in the range of fourteen hertz. The quick release consisted of a fuse-bar in line with the jacking system. The bar had been machined to a specific diameter, and designed to fail at a pre-determined load. This particular test focused on the above-ground structural response and not the soil-pile interface (Chen, 1993).

Presently a test is under way to conduct a full-scale bridge test with the soil-pile interface as its primary concern, but at this time results have not been published. The test set-up includes jacking the south bound bridge against the north bound bridge. The quick release method used in this case consisted of dumping the hydraulic fluid out of the jack once the design load was reached. Pre-tests that were conducted in June 1994 resulted in the acceleration reaching only 0.05g instead of the expected 0.10g. The current status of the project is not known (Chen, 1994).

As can be seen, there has been very little work conducted on the dynamic behavior of piles in loess. Thus, the proposed tests described herein take on added importance.

CHAPTER 3
QUALIFICATION TESTS OF EQUIPMENT

3.1 Introduction

The following chapter covers the initial design and testing of the equipment which will be used on the actual pluck test. All components of the pluck test were proof tested to prevent problems in the field. According to researchers at the University of Nevada Reno, the pluck tests will be repeatable only at a particular displacement. Two tests at the same displacement should give similar answers. Once the displacement has been increased, one could not expect to obtain similar results if a test at the initial displacement was repeated. The field tests will consist of pluck tests on two separate concrete piles and tests already conducted on an H-pile which was used for an instrumentation check-out.

3.2 Initial Quick Release Test

The ability to quick release the pile is a key factor in obtaining useful data. The quick release mechanism described later herein, was initially tested on an existing frame in the University of Tennessee (UT) structures lab. A schematic of the test is shown in Figure 6. The main purpose of the test was to determine the performance of the quick release mechanism and to proof test all connections which will be used in the actual field test. The expected maximum load of the field tests is approximately ten kips. Each

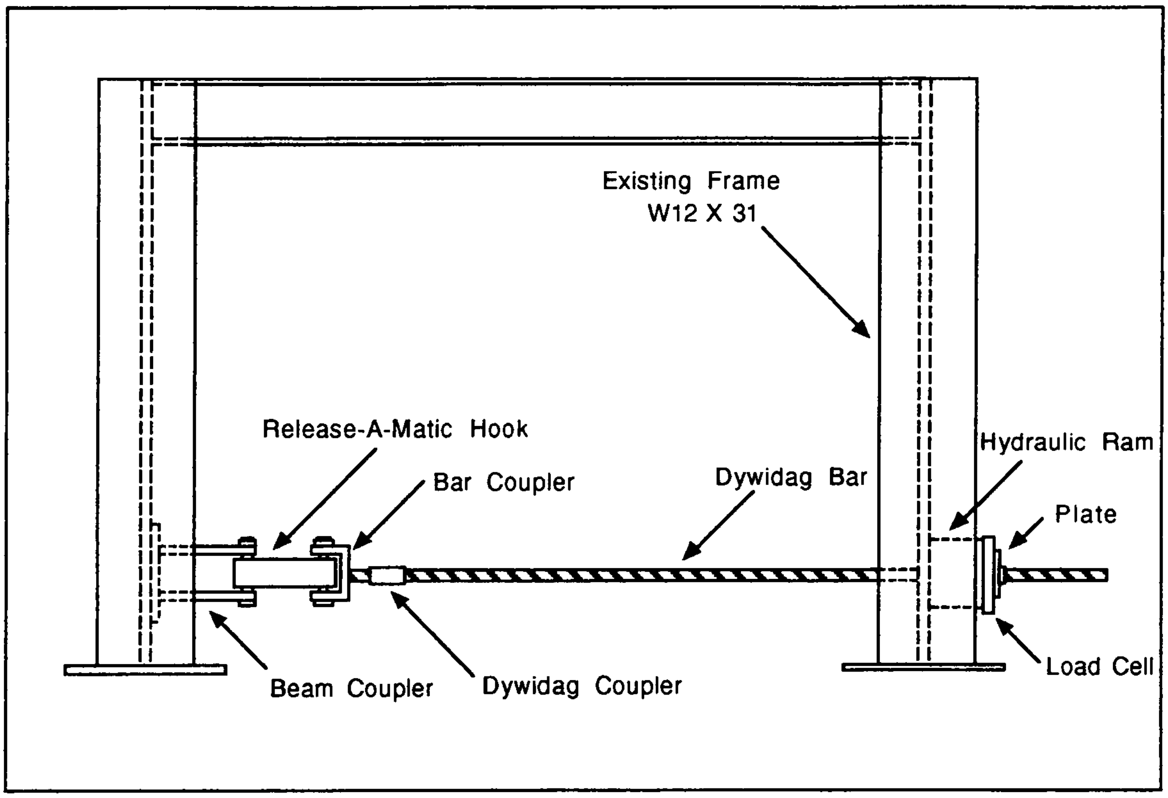

Figure 6. Initial Quick Release Test Schematic.

component of the test was designed for twenty kips to insure an appropriate factor of safety.

3.2.1 Design of Beam Coupler

The beam coupler, which was designed using the Load and Resistance Factor Design (LRFD) method for steel design, is the component which attaches the quick release to the existing frame and is shown in Figures 7 and 8. The coupler consists of two 8" by 3" by 1/2" lugs connected to an 8" by 8" by 1" base plate. The base plate of the coupler has 3/4" holes placed five inches on center symmetrically in the event that the quick release needs to be pulled

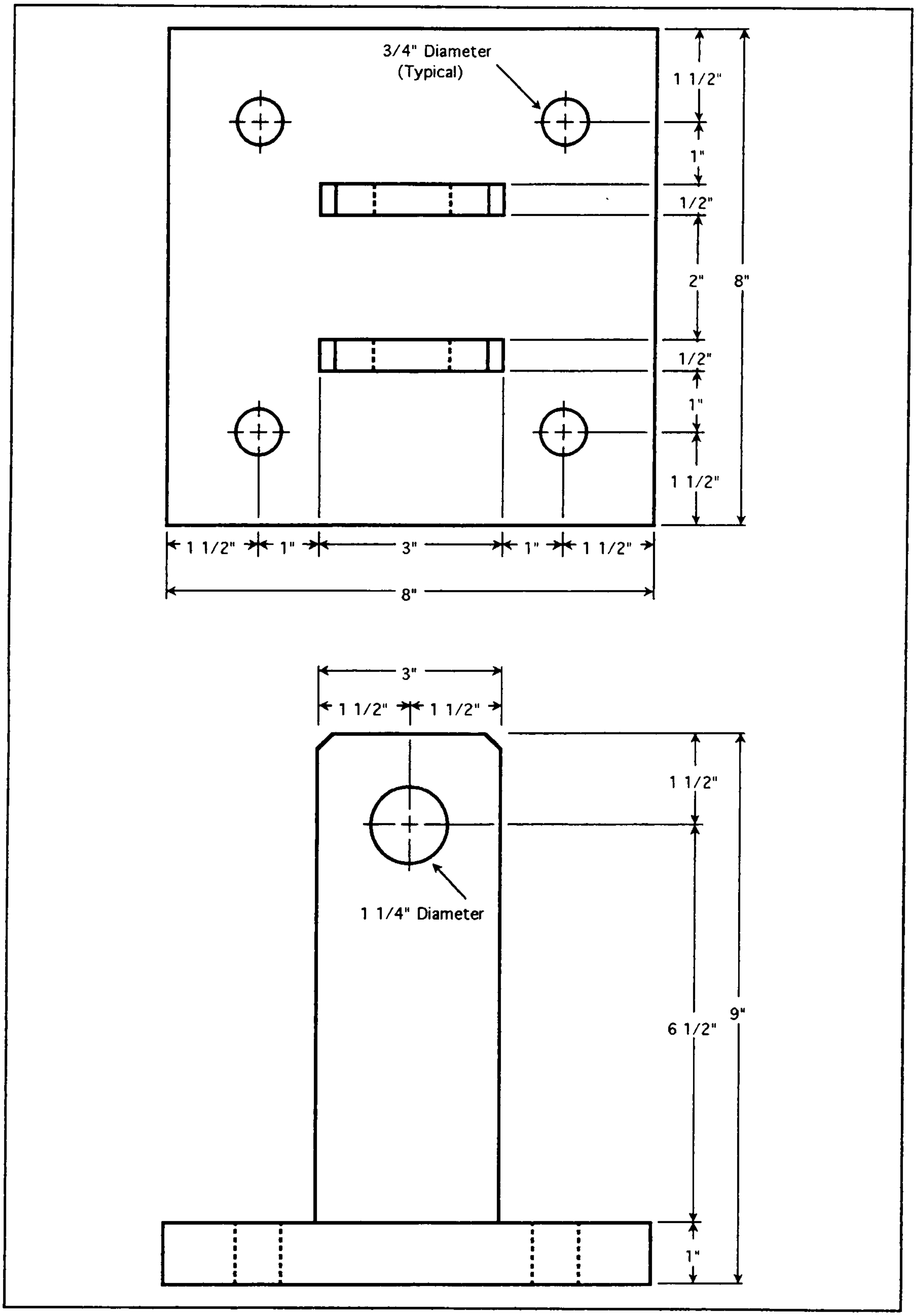

Figure 7. Beam Coupler Details

Figure 8. Beam Coupler

vertically instead of horizontally. The throat was made eight inches long to give sufficient clear swing when the quick release is disengaged. The critical part of the beam coupler was the bolt. The bolt could not exceed 1.25" diameter as the hole in the quick release was just slightly larger than this. For a twenty kip load the maximum bending stress on the bolt was calculated to be 53.78 ksi. Grade 8 bolts were used since their maximum allowable stress is 72 ksi.

3.2.2 Quick Release

The purpose of the quick release is to allow for free vibration of the pile. Free vibration of the pile is necessary to calculate actual spring coefficients for the embedded pile. Dr. Stuart Chen at the State University of New York (SUNY) at Buffalo suggested the Release-A-Matic Hook, manufactured by Peck and Hale. Dr. Chen had considered using this particular quick release on full-scale bridge tests which were being conducted at SUNY. The Release-A-Matic Hook (H44-9) appeared to be ideal for the loads which were expected in the proposed pluck tests and is shown open in Figure 9 and closed in Figure 10. According to Peck and Hale, the H44-9 has a minimum breaking strength of 71.7 kips and a safe working load of 18 kips.

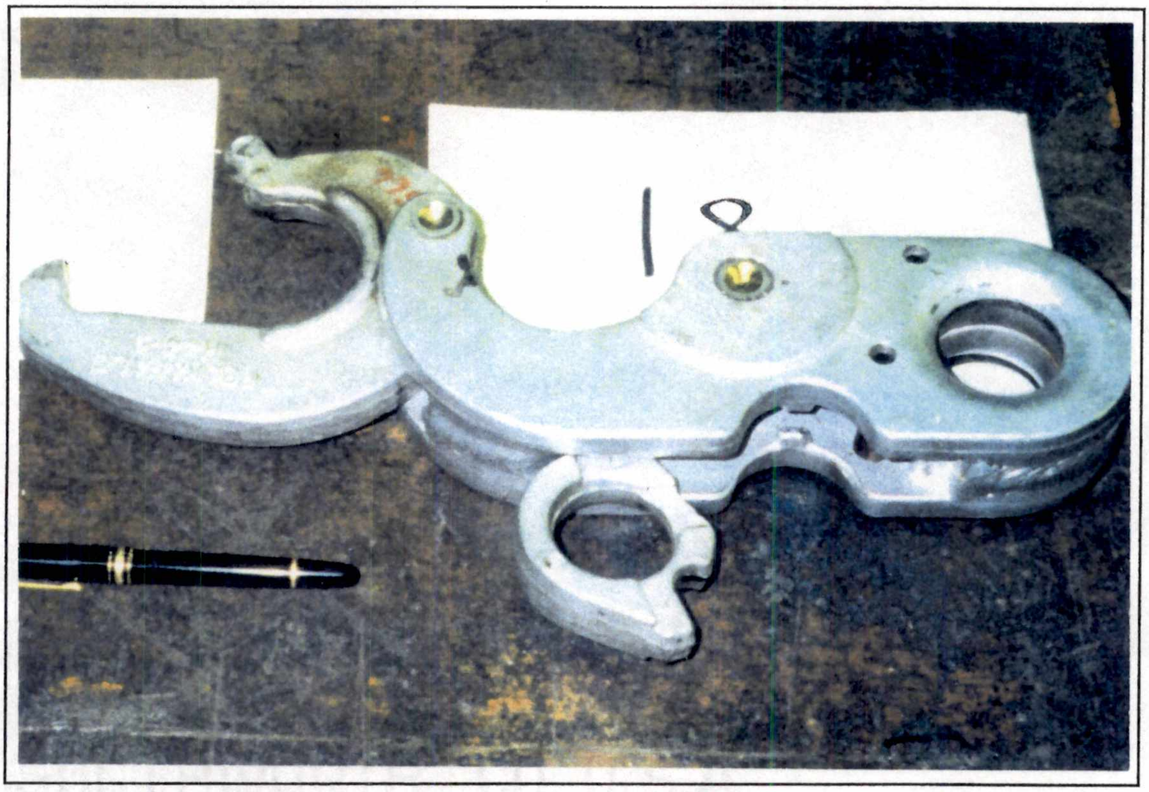

Figure 9. Release-A-Matic Hook, Open

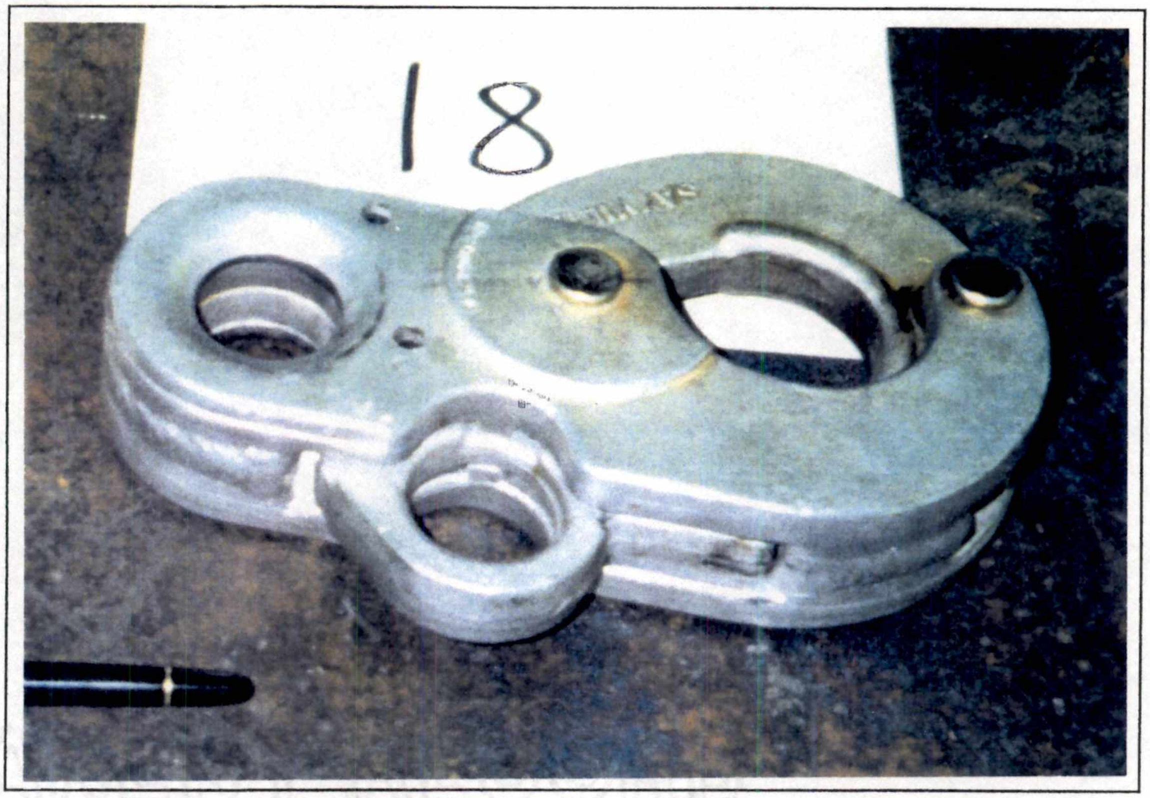

Figure 10. Release-A-Matic Hook, Closed

For an applied load of ten kips, the pull strength required to disengage the quick release is approximately one hundred pounds. The latch ring allows for a rope to be attached to the quick release and enables the system to be disengaged from a safe distance.

3.2.3 Design of Bar Coupler

The bar coupler is the apparatus which connects the Dywidag bar to the quick release and was also designed by the LRFD method. The bar coupler, as can be seen in Figures 11 and 12, consists of two 4" by 4" by 1/2" plates welded to a 4" by 3.5" by 1/2" base plate. A three inch section of Dywidag bar was welded to the bottom of the

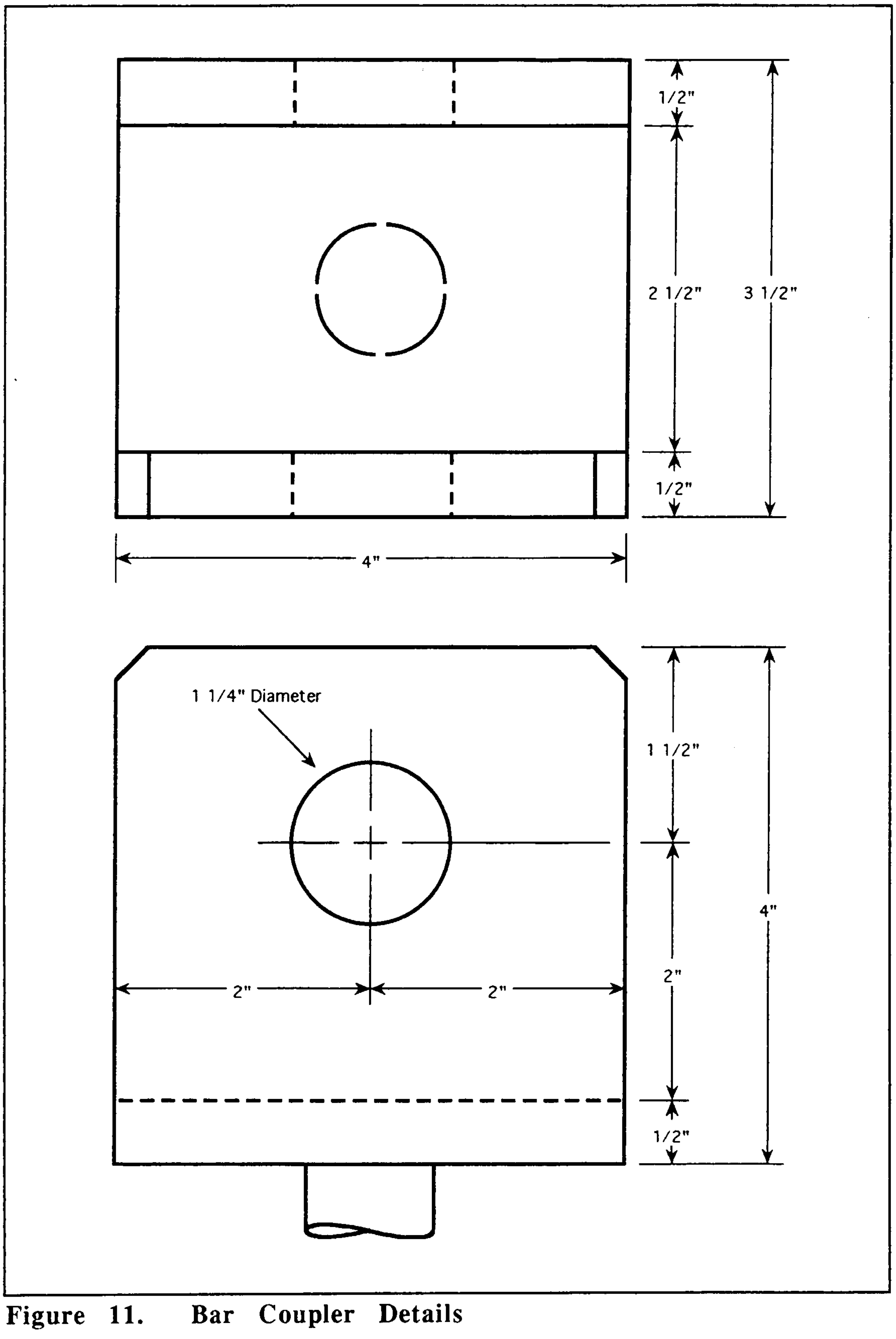

Figure 11. Bar Coupler Details

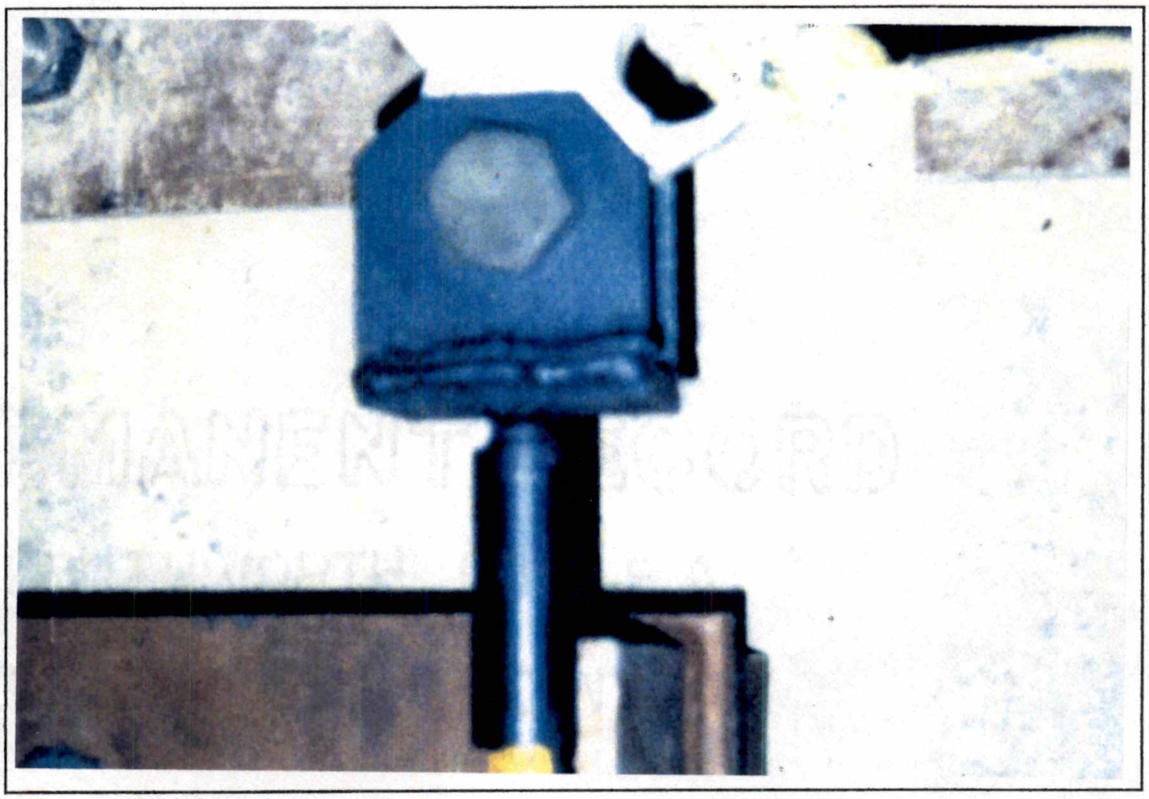

Figure 12. Bar Coupler

base plate to allow for connection in the system. Once again the most critical element was the bolt. The portion of the Release-A-Matic Hook which attaches to the bar coupler is wider than the portion that attaches to the beam coupler. This spreads out the load and reduces the stress on the bolt. Grade 8 bolts were once again used.

3.2.4 Dywidag Bars

The initial quick release test as well as the actual field tests require some way to transfer the load from a load frame to the quick release. A cable was rejected because of the safety concerns. Dywidag Bars, manufactured by Dywidag Systems International

(1994), are high strength threaded bars used mainly for post tensioning concrete. The bars have a continuous rolled pattern of deformations along the entire length. These deformations act like threads and are more durable than machined threads. Since the deformations are rolled along the entire length of the bars, anchors and couplers can be placed anywhere they are needed as illustrated in Figure 13.

For one inch bars, as were used in the proof tests, the ultimate stress is 150 kips per square inch and the ultimate strength is 127.5

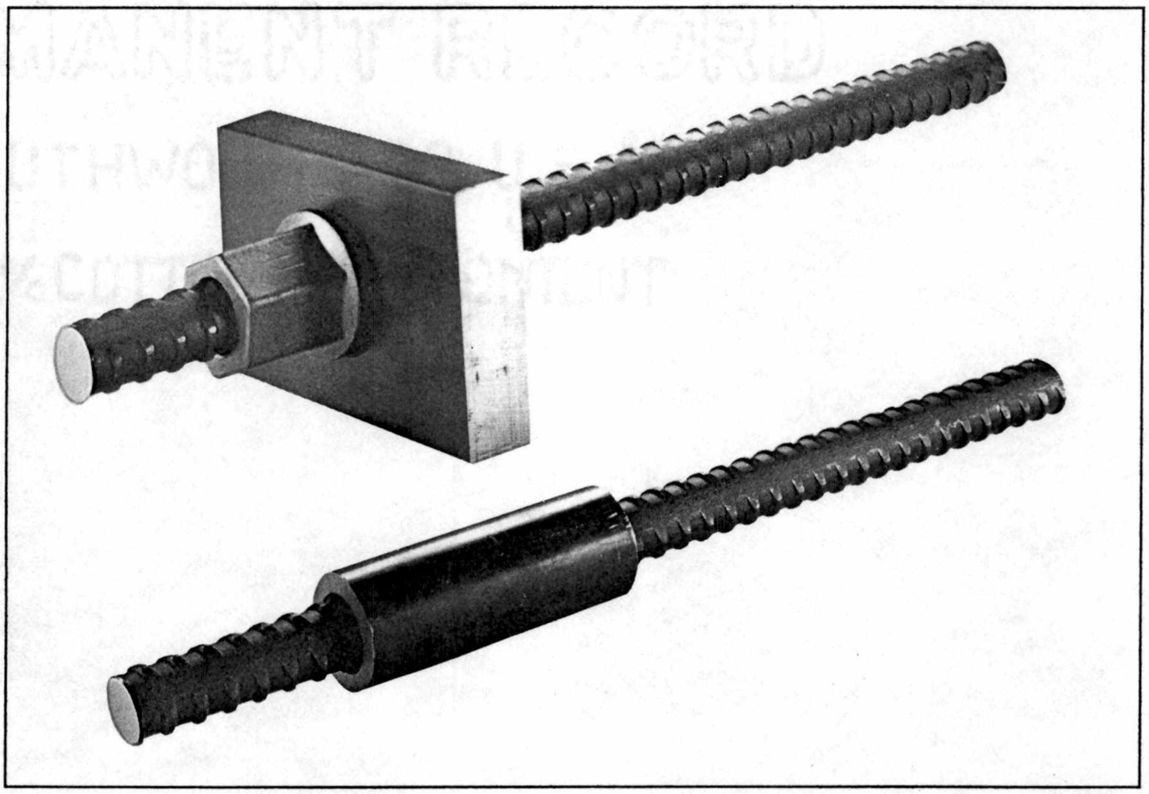

Figure 13. Dywidag Anchors and Couplers

Source: Dywidag Systems International (1994). "Dywidag Threadbar Post-Tensioning System." Bolingbrook, Illinois.

kips, based on the cross sectional area of 0.85 square inches. Dywidag bars may be stressed to 102 kips. The strength and the stress of the Dywidag bars were substantially greater than the anticipated maximum applied loads.

3.2.5 Load Application

The proof test load was applied to the system using a hydraulic ram and jack. The hydraulic ram was an Enerpac RCH 603 center hole ram, and the jack was an Enerpac P84. The jacking load was monitored by an Interface 100 kip doughnut load cell. The Dywidag bar was inserted through the ram and load cell and a half inch plate was bolted to the bar for the ram to jack against. An additional half inch plate was bolted to the bar inside the frame to arrest the energy within the system after the load was released.

3.2.6 Results of Initial Quick Release Test

An initial 2.5 kip load was applied and subsequent loads of 5, 7.5, 10, and 15 kips were then applied. A maximum load of fifteen kips was used to proof test all components. The load increments were conducted to determine if the quick release would disengage in a similar manner at various load levels. The actual pile test will be loaded in much the same way as this, controlled by the deflection of the pile itself. All connections, bars, and the Release-A-Matic Hook held firm at the maximum load that was applied. The bolts which were critical in the design of the couplers experienced no apparent bending.

The Release-A-Matic Hook was disengaged easily by one person at loads of 7.5 kips and below, but required two people to release it at the ten and fifteen kip loads. It was also determined that the rope used to disengage the quick release needed to be static rope. Static rope, often used for rappelling, is designed specifically not to elongate significantly as a load is applied. The Release-A-Matic Hook performed adequately and it was determined that it would indeed be ideal for the purpose of the proposed field tests.

CHAPTER 4
PRELIMINARY FIELD TESTS

4.1 Introduction

After all qualification tests were completed, the preliminary field tests were performed. The preliminary field tests took place at the Nonconnah Parkway Bridge in Shelby County, Tn, southeast of Memphis. The preliminary field tests consisted of pluck tests on an existing H-pile, the main purpose of which was to check out instrumentation and to insure that the test set-up was applicable to future tests.

4.2 H-Pile Test Set-Up

The preliminary field tests conducted on the existing H-pile have the same set-up as that planned for future pluck tests. The schematic of the H-pile tests is shown in Figure 14. The tests consist of an existing H-pile, load frame, slab system, and all aspects of instrumentation.

4.2.1 Slab System

A foundation had to be designed to support the load frame and supply mass to counteract the pulling force. The slab system, which is shown in Figure 15, is six feet square by two and a half feet deep and was placed twenty-five feet from the pile. The slab was cast in place with a minimal amount of formwork. Six anchor bolts were embedded in the slab for attaching the load frame. The anchor

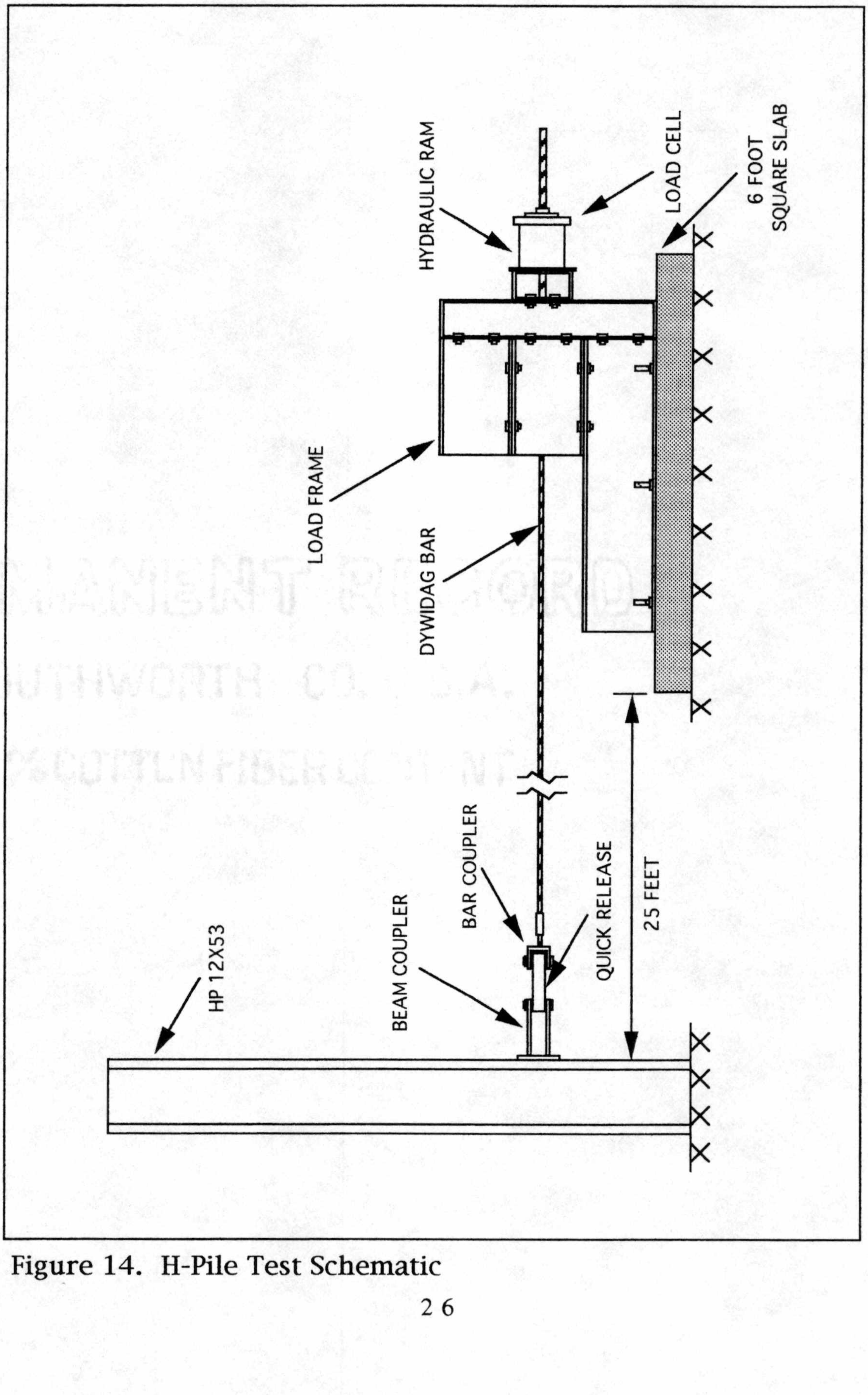

Figure 14. H-Pile Test Schematic

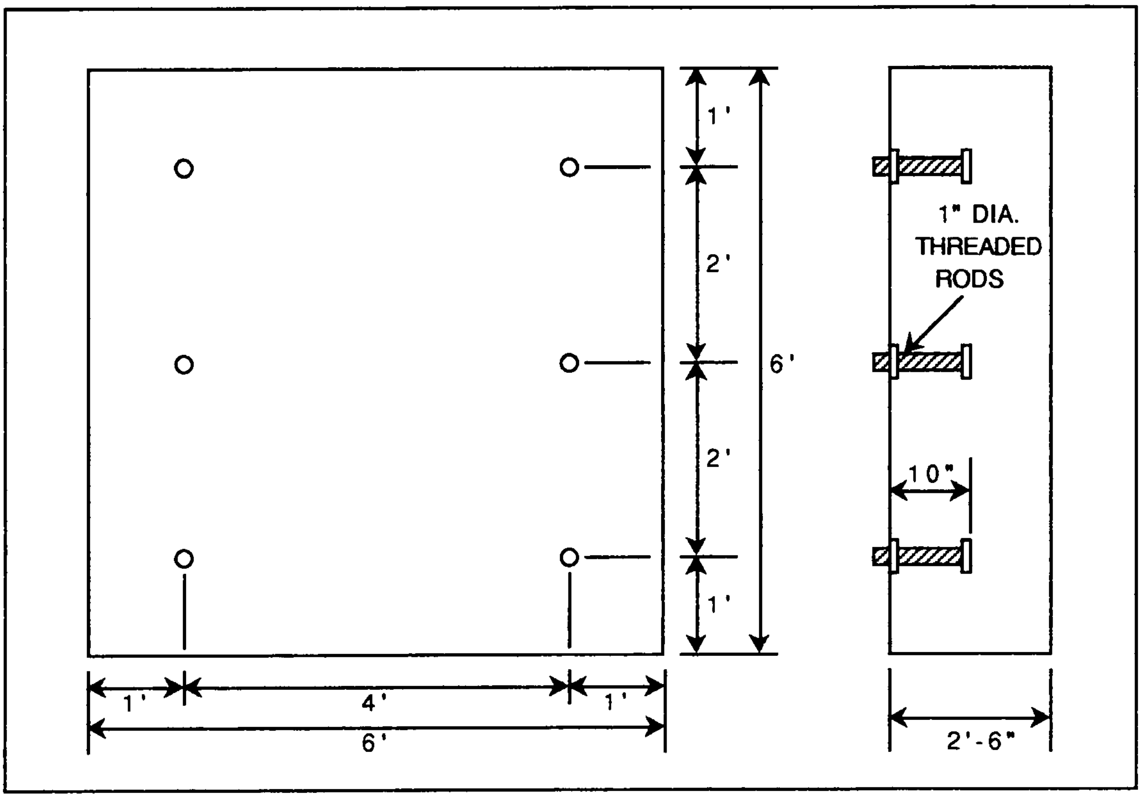

Figure 15. Slab System

bolts were twelve inches long, one inch diameter threaded rods with a nut placed at the base of the rod. These bolts were placed with the use of a template which was fastened to the form to insure that the bolts were located in the proper location. The bolts were embedded ten inches to ensure sufficient pullout strength.

The slab had to withstand both sliding force and overturning moment during the pluck tests. The slab was designed for a horizontal load of twenty kips to ensure that a failure did not occur in the field. The soil density of loess soil is approximately 100 pounds per cubic feet (pcf); the friction angle is thirty degrees; and the cohesion is expected to be no less than 127 pcf. For a depth of two and a half feet the passive pressure is 5,625 pounds, based on the six foot width. Combined with the cohesive resistance of 12,366 pounds, the total sliding resistance is 17991 pounds or 17.99 kips. The maximum pulling force was fifteen kips, so the resistance to sliding was considered acceptable.

The overturning moment was analyzed for the load applied nine inches above the slab. The resisting forces were 54.00 kip-feet, while the overturning forces were only 24.38 kip-feet; therefore, the overturning moments were acceptable. Even though the load was to be applied nine inches above the slab, the resisting forces allowed the load to be applied as high as twenty-seven inches above the slab, a location almost at the load frame's full height.

Future tests are expected to have a twenty kip pulling force and to apply the load at both the base of the piles and at the center of gravity of the pile cap; thus, the slab needs to be slightly larger than in the preliminary field tests. The enlarged slab is eight feet square by two and a half feet deep. The passive resistance is approximately 7500 pounds for the eight foot width with a cohesive resistance of approximately 22000 pounds. The total combined passive and cohesive resistance is 29500 pounds or 29.50 kips, greater than the maximum expected load of twenty kips.

The overturning resistance had to be greater than the overturning forces for the load applied both nine inches and thirty-two inches above the slab. The thirty-two inch application controlled with an overturning moment of 65.8 kip-feet, compared to the 27.5 kip-feet overturning moment at nine inches. The resisting moment of 96.0 kip-feet is more than adequate to resist overturning. The enlarged slab system is designed to resist an applied load of twenty kips at the full vertical range of the load frame.

4.2.2 Load Frame

The purpose of the load frame, which is shown in Figures 16 and 17, is to transfer the load from the hydraulic ram to the H-pile. The frame is designed for a working load of twenty kips and with a safety factor of 4.0; thus, it should experience no yielding at a load up to eighty kips. The base of the frame consists of two W12X40's, five feet long, placed four feet apart. Stiffeners were added to compensate for the fact that the beams are bolted to the slab on only one side. Two additional levels of two foot long W12X40's were placed on top of the base beam. A W8X31 was placed upright and attached to the W12X40's. A 10X6X5/16 structural tube was bolted to the upright beams to provide a location for the load. A plate was placed at the location of the load to spread the stress over the depth of the tube. Holes were placed four inches on center at the outer flange of the W8X31 to allow for the load to be applied anywhere from four to thirty-two inches off the slab. This was necessary to enable the load to be moved between the base and the center of gravity of pile on future tests. The tube also had a slotted hole to allow for the load to be placed at the precise location desired. The load frame was proof tested to twenty-five kips in the lab.

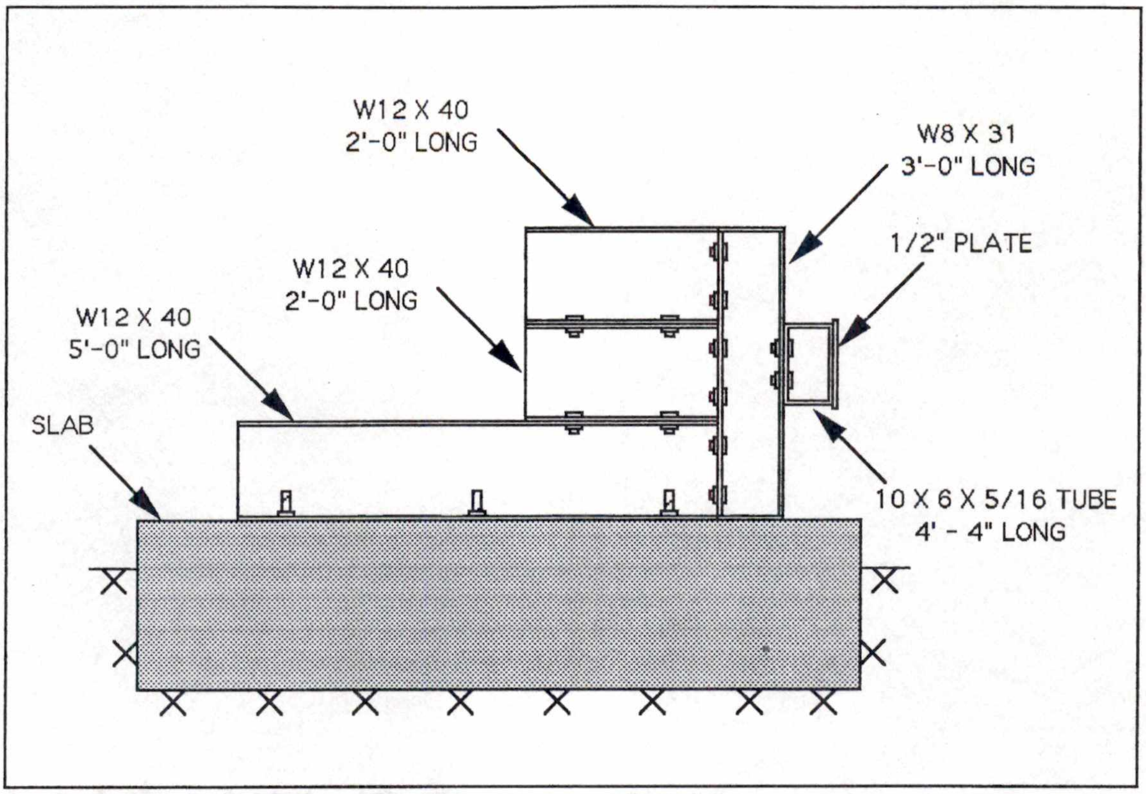

Figure 16. Load Frame

Figure 17. Load Frame

4.2.3 Instrumentation

The instrumentation locations can be seen in Figure 18. Since acceleration and displacement are mathematically related, if one variable is known, the other can be derived from it. To insure the accuracy of the measured results both variables were recorded.

One Linear Variable Displacement Transducer (LVDT) was used and placed approximately two feet above the load on the opposite side of the H-pile from where the load was applied. This insured that only the deflection of the pile itself was recorded and not any shifting in the loading system. The LVDT support structure would not move when the pile deflected, but also would not disturb the soil

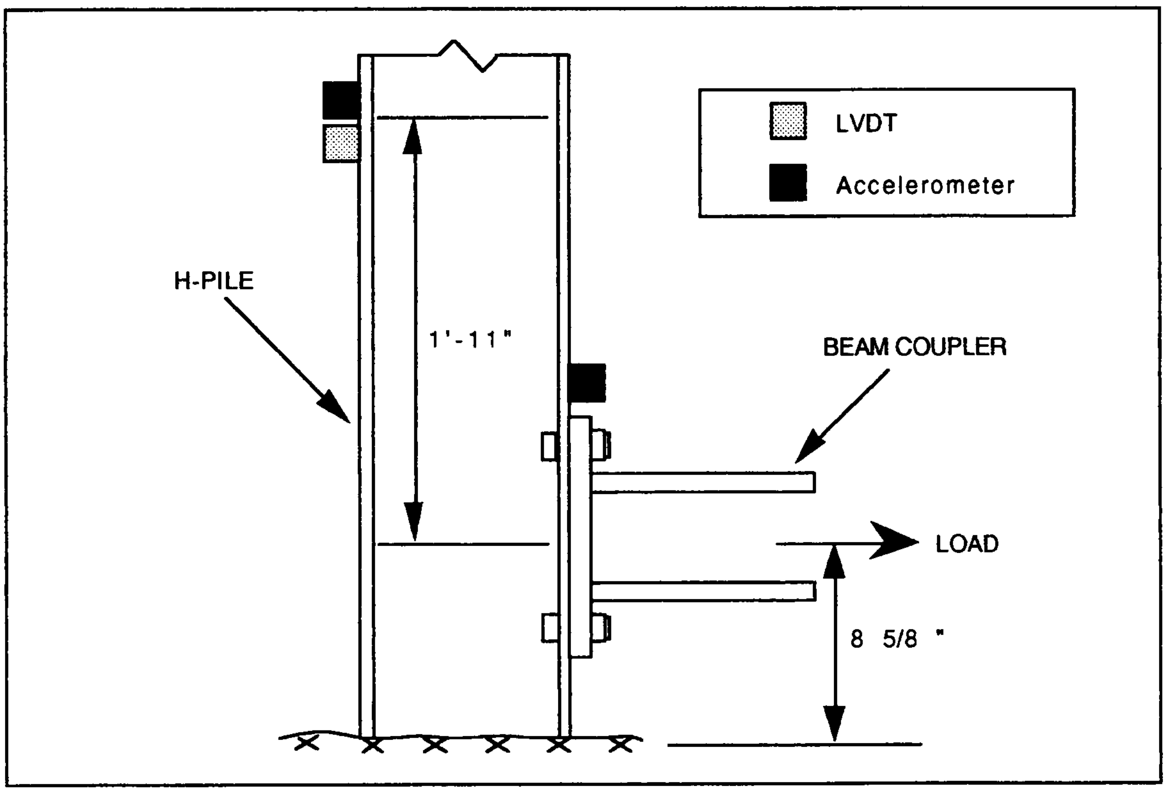

Figure 18. Instrumentation Location

directly around the H-pile. The LVDT support structure consisted of a two inch square tube spanning between supports which were partially embedded in the ground. The tip of the LVDT was glued to the H-pile to insure that it was always in constant contact with the H-pile. The LVDT Model LDC1000A, manufactured by RDP Electronics, was used and had a range ±1.25 inch.

Two accelerometers were used for the preliminary field tests. One was placed at the same height as the LVDT, which was one foot eleven inches above the load. The other was placed at the same height as the load but on the front face of the H-pile. Two accelerometers were used to check each other and ensure that accurate results were obtained when the load was disengaged. The accelerometers had a range ±1g and a sensitivity of 5V/g. Ultra Low Frequency Seismic Accelerometer, Model 731-201A, manufactured by Wilcoxon Research Incorporated, was used for the tests. The accelerometers required a voltage source of 18-30 direct current volts. A Four Channel Power Unit Model P703B, manufactured by Wilcoxon Research, was used to power both accelerometers simultaneously. The power unit itself was powered by a standard 110 alternate current voltage source, a generator.

A data acquisition system received the signals which were sent by the sensors: the accelerometer and the LVDT. The system consisted of a screw terminal box to attach the cables, a data acquisition board, a 486DX computer, and software (DTVEE for Windows) to view the information. The data acquisition board Model DT2836, manufactured by Data Translation, was chosen for these tests. This particular board had a vertical resolution of sixteen bits and eight differential channels. The board had a voltage range of ±10 volts and could be programmed to measure inputs of ±10, ±5, ±2.5, or ±1.25 volts with full vertical resolution. Data acquisition boards measure analog signals from sensors and convert them into digital signals for computer storage or analysis. They also allow computers to convert digital signals into analog signals so that a computer may run a chart recorder or other types of analog recorders.

A data acquisition board must be chosen so that it is compatible with the voltage range which is being measured. A board which has a range of ±10 volts that is used with sensors which have ranges of ±5 volts would result in a reduction in the usable resolution of one half. On the other hand, a board with a range of ±5 volts used to record sensors that have ranges of ±10 volts is essentially useless with only half of the input signal able to be recorded by the board.

The DT2836 had eight differential input channels. Differential inputs take up two multiplexer channels, so the input channels are reduced by half compared to single ended input channels. Differential inputs are used primarily because they are immune to noise common in single-ended input channels. This noise, in the form of voltage, is common in long cables. In these tests the data acquisition board was between fifty and sixty feet away from the sensors so differential inputs were a must.

The rate that data are sampled is an important factor to consider. According to the Nyquist Theorem, a data acquisition system should sample input at a rate at least twice the speed of the highest input frequency. This prevents aliasing, which is where high frequencies appear to be lower frequencies, a situation which occurs when the samples taken leap from point to point while missing entire peaks. This is illustrated in Figure 19. The DT2836 had an input rate of thirty-three kilohertz, which meant 33,000 samples could be taken per second. Since all eight channels could be used, this translated to 4.125 kilohertz per channel. If ten points per wave are desired, two points being the minimum to prevent aliasing, the

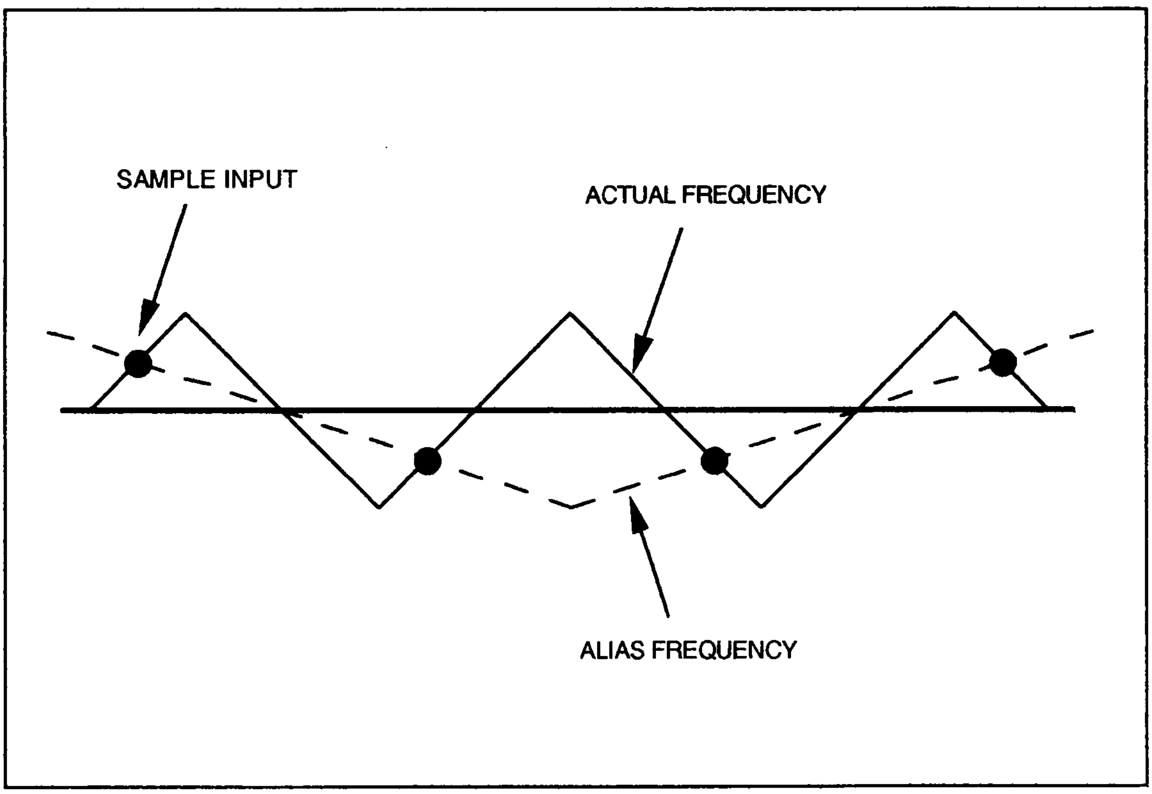

Figure 19. Signal Aliasing

maximum frequency allowed was 412.5 hertz per channel. With the natural frequency experienced during testing, this test frequency was satisfactory.

Accuracy of the input is the final factor which must be addressed when choosing a data acquisition board. Since the DT2836 had a sixteen bit vertical resolution, the range could be divided into 2^{16} or 65,536 pieces. This allowed a 0.00003g change to be recorded in the system. This data acquisition board was optimally suited for these tests and allowed precise data to be recorded.

4.2.4 Additional Equipment

Equipment for the field tests included the Dywidag bars which were used in the initial quick release test. Plates were used both to jack against and to arrest the energy when the load was disengaged. A twenty-five kip load cell, manufactured by Interface, was used to record the actual load applied. An Enerpac RCH 603 center hole ram and Enerpac P84 hydraulic ram were used to apply the load to the Dywidag bars. The beam coupler, bar coupler, and Release-A-Matic Hook, from the initial quick release test, were also used to attach the load to the H-pile. A static rope was used to disengage the Release-A-Matic Hook and initiate free vibration.

4.2.5 H-Pile

The H-pile used for the preliminary field tests was placed by the bridge contractor and used as an anchor while placing girders. The exact length that the H-pile was embedded in the ground was not known. Both the size of the H-pile and the length that it was embedded in the ground influence the analysis of the pluck test results.

Based on field measurements, the H-pile was an HP 12X53. This particular H-pile has a moment of inertia of 393 in^4 and a weight of fifty-three pounds per foot. The moment of elasticity of steel was taken as 29,000 ksi. All three of these factors are involved in the analysis of the H-pile. All the properties except the embedded length could be determined from field inspections. The length of the pile may have a large effect on the dynamic analysis; thus, the analysis may be skewed because the exact length that the H-pile is embedded in the ground is not known.

4.3 Preliminary Field Test Data

After the test set-up was inspected and the instrumentation had been placed at the proper locations, the data acquisition system was checked out by hitting the pile with a sledge hammer. Readings were taken and the results inspected to ensure that the system was acting properly. After the data monitoring system was deemed acceptable, the tests commenced.

The H-pile was loaded as shown in Table 1. The static deflection at each load can be seen in Figure 20. The measured deflections are the values which were measured by the LVDT. The calculated deflections used in the analysis are the values for the actual deflections adjusted to the point of load application. It was important to get these values for the analysis.

Table 1. Load Applied Per Test

TEST	A	B	C	D	E
LOAD (KIPS)	5	5	6	10	15

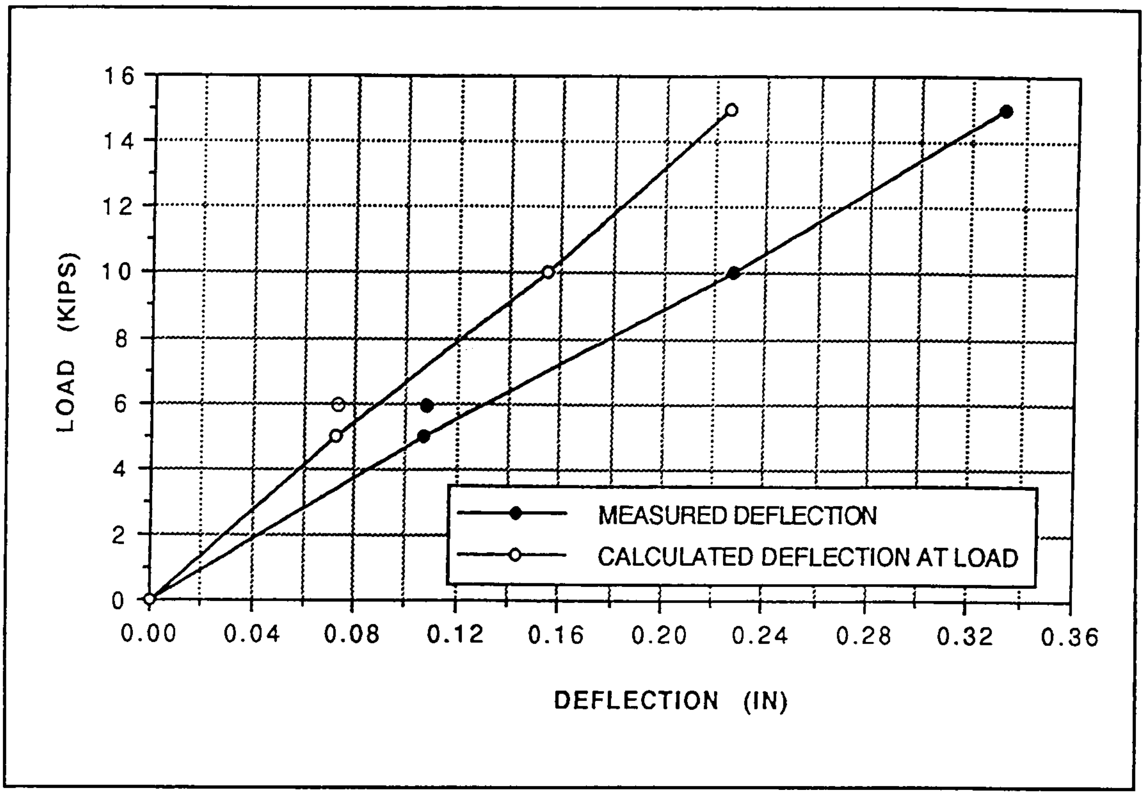

Figure 20. Load-Deflection Curve for H-Pile Tests

The deflection versus time was recorded by the LVDT and plotted by the data acquisition system. The deflection-time curves for each loading are shown in Figures 21 through 25. The data acquisition system did not record the start of the loading for Test A. The specific data points for the deflection versus time curves are tabulated in Appendix A.

Due to a malfunction of the data acquisition system, the accelerations of the H-pile were not recorded.

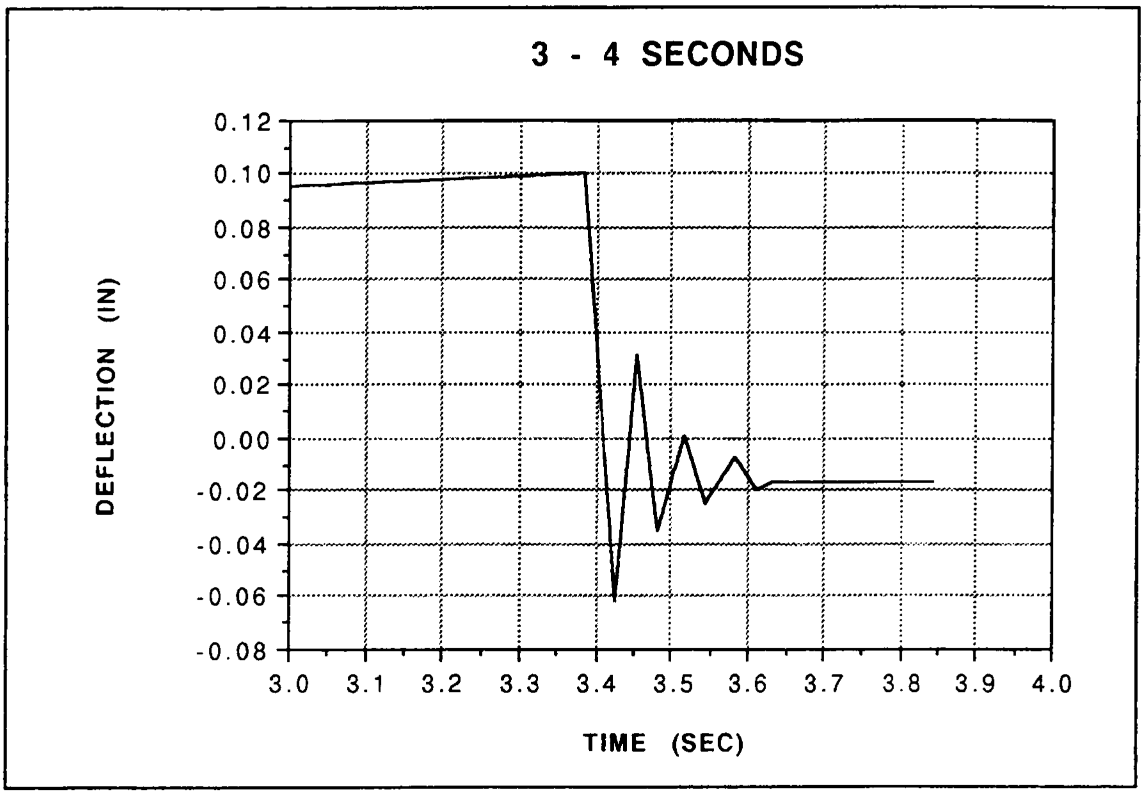

Figure 21. Deflection-Time Curve for 5 Kip Load, Test A

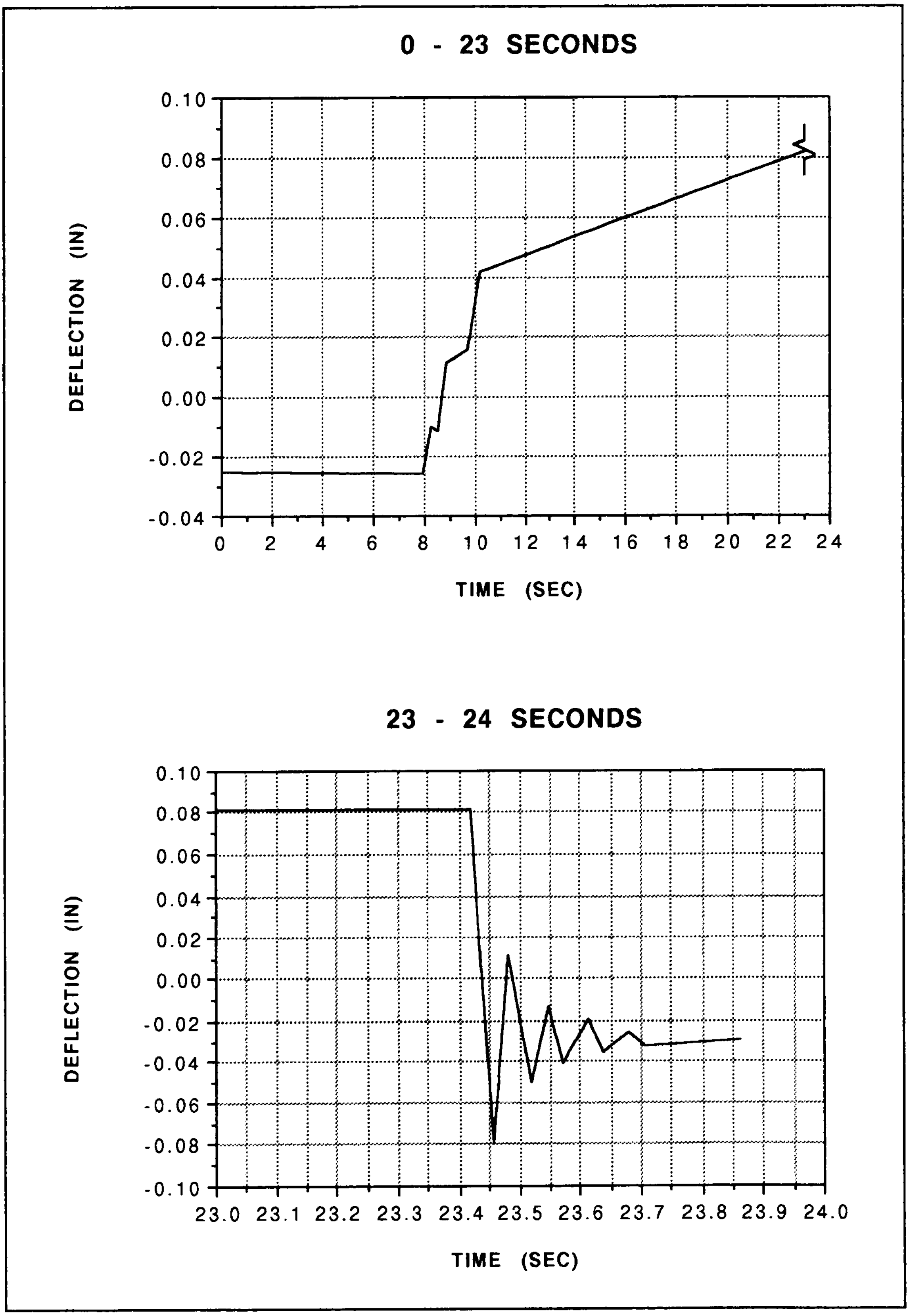

Figure 22. Deflection-Time Curve for 5 Kip Load, Test B

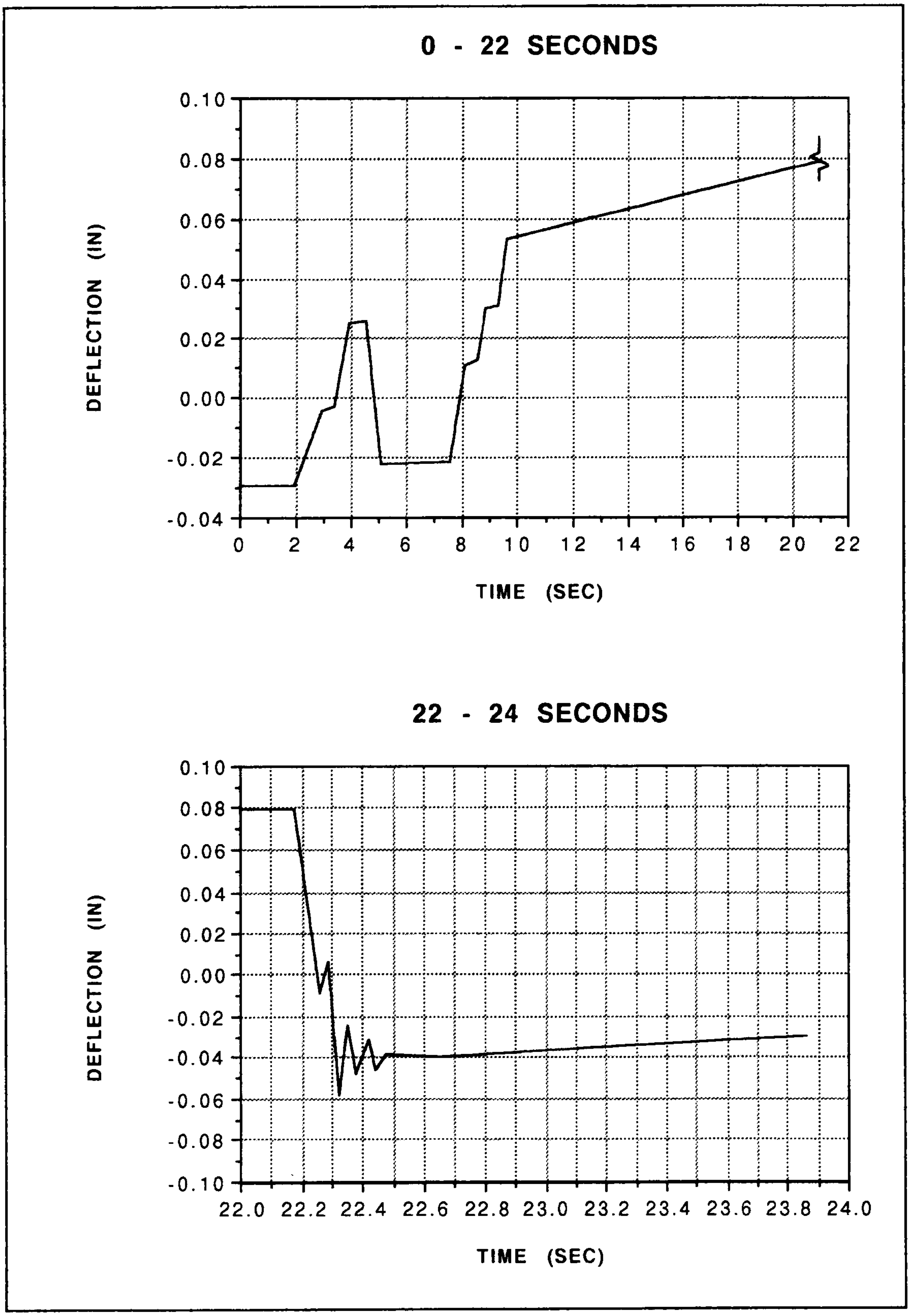

Figure 23. Deflection-Time Curve for 6 Kip Load, Test C

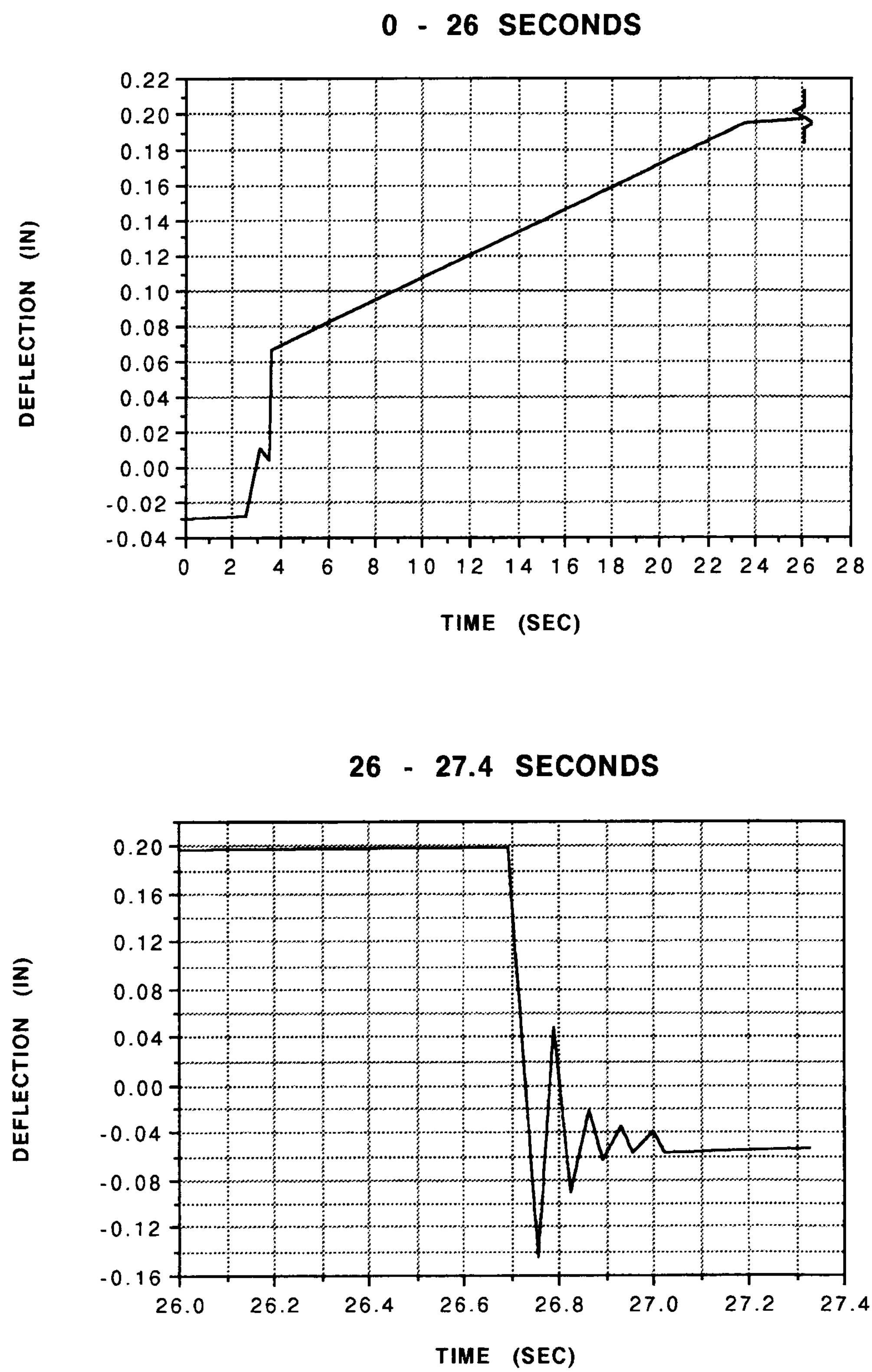

Figure 24. Deflection-Time Curve for 10 Kip Load, Test D

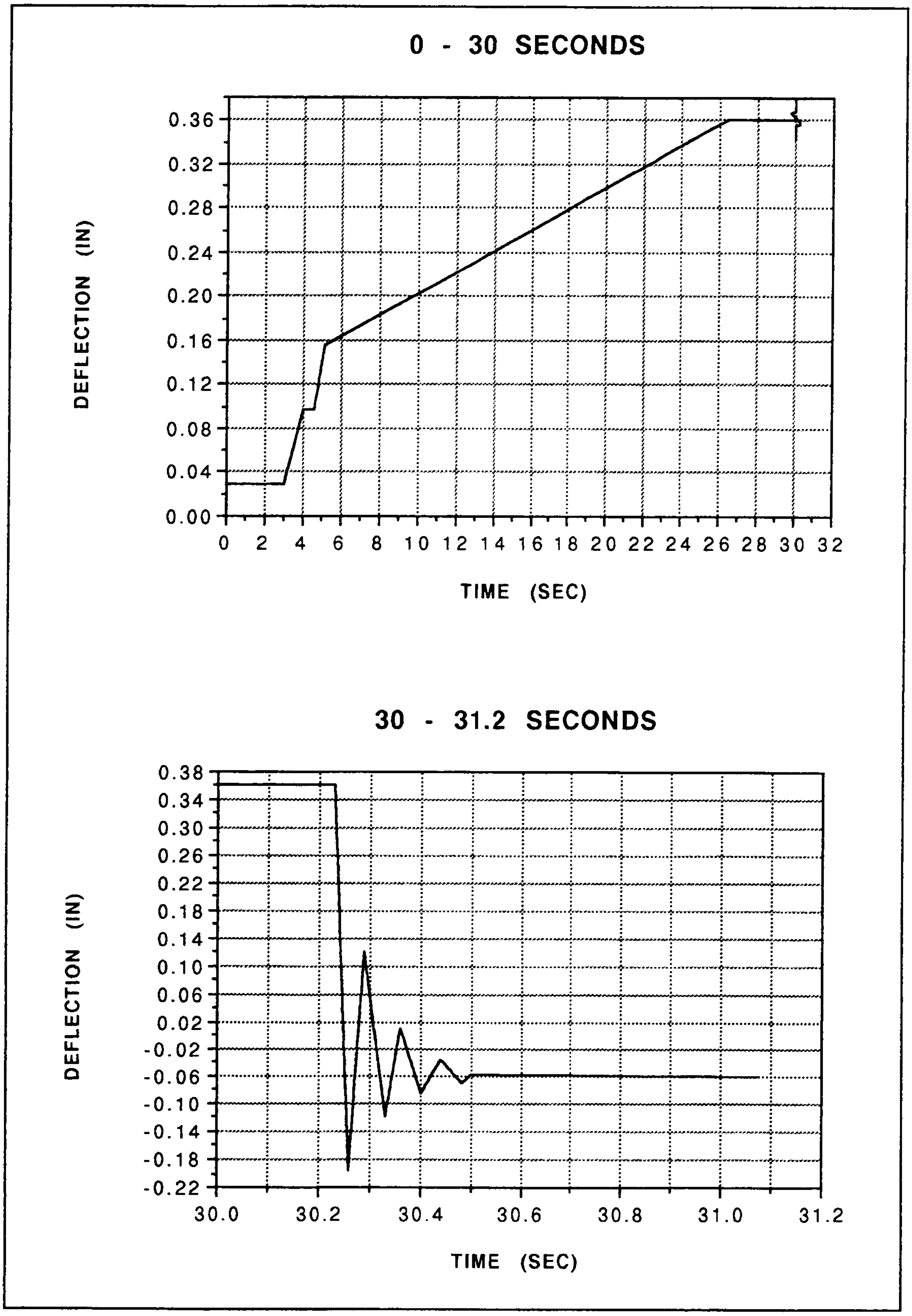

Figure 25. Deflection-Time Curve for 15 Kip Load, Test E

CHAPTER 5

ANALYSIS OF PRELIMINARY FIELD TESTS

5.1 Introduction

The analysis of the preliminary field tests included the determination of the frequency and damping of the H-pile and the static stiffness of the soil. The analysis includes sample calculations for Test D, the ten kip load, so that this thesis can serve as a reference for future tests and their analysis. The recorded data points for Test D are listed in Table 2, and the deflection versus time curve is presented again in Figure 26. The data points in Table 2 represent the positive and negative peaks of the time-deflection curve and were extrapolated from the data acquisition system.

Table 2. Time-Deflection Data for 10 Kip Load

LVDT #927	CONVERSION 2.287 V/in	
VOLTS	TIME (SEC)	INCHES
-0.06531	0.000	-0.029
-0.06470	2.582	-0.028
0.02533	3.142	0.011
0.00916	3.530	0.004
0.15410	3.592	0.067
0.44310	23.558	0.194
0.45380	26.694	0.198
-0.32930	26.758	-0.144
0.11200	26.788	0.049
-0.20390	26.822	-0.089
-0.04944	26.862	-0.022
-0.14440	26.892	-0.063
-0.07904	26.930	-0.035
-0.12700	26.956	-0.056
-0.09064	26.998	-0.040
-0.13030	27.022	-0.057
-0.11780	27.326	-0.052

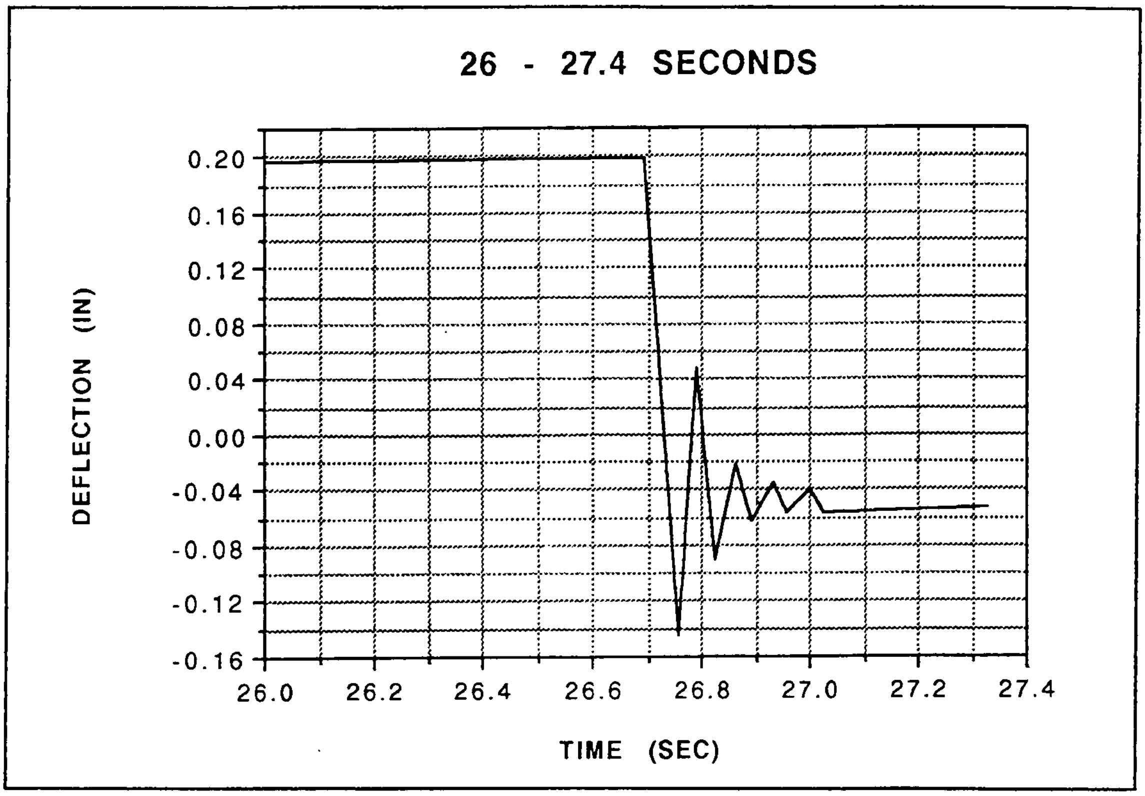

Figure 26. Deflection-Time Curve for 10 Kip Load, Test D

5.2 Frequency of the H-pile

The frequency is easily found by taking the reciprocal of the period, which is the number of seconds per cycle. Theoretically, the period of a wave would be constant, but in actuality the period varies slightly so the average of periods was taken. In Test D the first positive peak occurred at 26.788 seconds and the second peak occurred at 26.862 seconds. By taking the difference, a period of 0.074 seconds per cycle was calculated. The reciprocal of this number is 13.51 hertz. This same procedure can be done from the second positive peak to the third positive peak, and so on. The analysis can also be conducted on the negative peaks. For example, the first negative peak occurred at 26.758 seconds and the second negative peak at 26.822 seconds. This translates into a period of 0.055 seconds per cycle and a frequency of 18.18 hertz. The average of all the positive peaks gave a frequency of 14.93 hertz, while the average of all the negative peaks gave a frequency of 16.30 hertz. An average for both sets of peaks gave a final frequency of 15.62 hertz. Table 3 shows the positive and negative frequencies and the average of these values for each test. The first cycle in Tests C and D had abnormally large periods and were excluded in the study. The average frequency of all tests run was 15.10 hertz.

Table 3. Frequency Results of H-Pile Test

TEST	LOAD (KIPS)	POS. FREQUENCY (HERTZ)	NEG. FREQUENCY (HERTZ)	AVG. FREQUENCY (HERTZ)
A	5	15.00	16.30	15.65
B	5	15.27	16.00	15.64
C	6	14.29	15.15	14.72
D	10	14.93	16.30	15.62
E	15	14.15	13.64	13.90

5.3 Damping of the H-Pile

The second area of analysis is damping, which reflects the loss of energy in the system. Critical damping is the amount of damping that removes all the vibration from a structure (Biggs,1964). The damping ratio, ξ, is the percent of critical damping that occurs in a particular system. The peak displacements of a wave are utilized in calculating the damping ratio. Every peak or every other peak may be used to calculate the ratio with an average of the calculated values being used for the final ratio. The end deflection was not recorded as zero on the LVDT so the deflection values had to be adjusted so the wave converged to zero. As can be seen in Table 4, the deflection values from Table 2 were adjusted by adding 0.052 inches to each measured deflection. If the values are not adjusted to a zero line, the damping ratio calculated by the positive peaks is significantly higher than the actual ratio and the ratio calculated by the negative peaks is significantly lower than the actual ratio since the wave converges to a value below zero.

Table 4. Adjusted Time-Deflection Values for 10 Kip Load, Test D

LVDT #927	
TIME (SEC)	**INCHES**
26.694	0.250
26.758	-0.092
26.788	0.101
26.822	-0.037
26.862	0.030
26.892	-0.011
26.930	0.017
26.956	-0.004
26.998	0.012
27.022	-0.005
27.326	0.000

Test D with a ten kip load is once again used for example purposes. The equation for the damping ratio is taken from Bennett (1994) and stated on the following page.

$$\xi = \sqrt{\frac{\left(\ln \frac{V_n}{V_{n+j}}\right)^2}{(2\pi j)^2 + \left(\ln \frac{V_n}{V_{n+j}}\right)^2}}$$

Where:

ξ	=	Damping Ratio
j	=	Number of cycles over which damping is calculated
V_n	=	First displacement peak
V_{n+j}	=	Second displacement peak

The number of cycles over which damping is calculated dictates which peaks are used. If j = 1, every positive or negative peak of the wave is included. If j = 2, every other positive or negative peak is used. The load was released at time 26.694 seconds with a positive displacement of 0.250 inches. With j = 1, the second positive displacement peak occurred at 26.788 seconds with a value of 0.101 inches. The equation yielded a damping ratio of 14.277 percent of critical damping. If j = 2, the second positive displacement had a value of 0.050 inches and occurred at 26.862 seconds. For this case the equation produced a damping ratio of 12.704 percent of critical damping.

The same method was used on the negative displacement. The first negative displacement peak had a value of -0.092 inches at a time of 26.758 seconds. With j = 1, the second negative displacement peak occurred at 26.822 seconds and had a value of -0.037 inches. The equation gave a damping ratio of 14.347 percent of critical damping. If j = 2, the second negative displacement peak had a value of -0.011 inches at a time of 26.862 inches. Using the equation again gave a damping ratio of 16.665 percent of critical damping.

The damping ratios for positive and negative displacements using both j = 1 and j = 2 are listed in Table 5. The average of the positive damping ratios was 11.964 percent of critical damping using j = 1 and 12.766 percent of critical damping with j = 2. The average for both positive methods was 12.32 percent of critical damping. The average of the negative damping ratios was 15.251 percent of critical damping with j = 1 and 16.197 percent of critical damping with j = 2. The average for both negative methods was 15.72 percent of critical

Table 5. Damping Results for 10 Kip Load, Test D

	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.25	0.101	1	0.14277	14.277
EVERY	0.101	0.05	1	0.11121	11.121
POINT	0.05	0.017	1	0.16922	16.922
	0.017	0.012	1	0.05535	5.535
			AVG	**0.11964**	**11.964**
EVERY OTHER	0.25	0.05	2	0.12704	12.704
POINT	0.101	0.017	2	0.14040	14.040
	0.05	0.012	2	0.11284	11.284
			AVG	**0.12676**	**12.676**
NEGATIVE	-0.092	-0.037	1	0.14347	14.347
EVERY	-0.037	-0.011	1	0.18956	18.956
POINT	-0.011	-0.005	1	0.12451	12.451
			AVG	**0.15251**	**15.251**
EVERY OTHER	-0.092	-0.011	2	0.16665	16.665
POINT	-0.037	-0.005	2	0.15729	15.729
			AVG	**0.16197**	**16.197**

damping. The total average for the wave was 13.90 percent of critical damping. The damping ratio for the positive and negative displacements, as well as the total average for the system, is shown in Table 6. The calculated values for both positive and negative damping ratios using j = 1 and j = 2 for each test are in Appendix B.

Table 6. Percent of Critical Damping for H-Pile Tests

	5 KIPS	5 KIPS	6 KIPS	10 KIPS	15 KIPS
POSITIVE	13.80	12.86	15.21	12.32	15.05
NEGATIVE	14.01	10.92	10.08	15.72	12.96
AVG	13.90	11.89	12.65	14.02	14.01

5.4 Static Stiffness of the Soil

The static stiffness of soil is the amount of load that is required to displace the soil a given amount. In the preliminary field tests, the static stiffness is simply the applied load divided by the recorded deflection. As stated before, the measured deflections had to be adjusted so that they reflected the deflection at the point of load application. The measured deflections, the calculated deflections at load, and the resulting static stiffness' are listed in Table 7. The static deflection for Test A was not recorded by the data acquisition system. For the ten kip load in Test D the measured deflection was 0.227 inches. This is the recorded distance that the pile moved when the ten kip load was applied. A deflection of 0.155 inches was calculated for the point of load application (Rourke, 1965). The ten kip load was divided by the 0.155 inch deflection which gave a static stiffness of 64.52 kips per inch.

Table 7. Deflection and Static Stiffness for H-Pile Tests

MEASURED LOAD-DEFLECTION DATA			
TEST	LOAD (KIPS)	DEFLECTION (IN)	STATIC STIFFNESS (K/IN)
B	5	0.107	46.73
C	6	0.108	55.56
D	10	0.227	44.06
E	15	0.332	45.18
REVISED LOAD-DEFLECTION DATA			
TEST	LOAD (KIPS)	DEFLECTION (IN)	STATIC STIFFNESS (K/IN)
B	5	0.073	68.49
C	6	0.074	81.10
D	10	0.155	64.52
E	15	0.225	66.67

The static stiffness for the six kip load was excluded from further analysis due to the significant difference in its static stiffness. The average static stiffness for the tests, excluding the six kip load, was 66.67 kips per inch.

5.5 Method of Deflection Adjustment

The deflections were adjusted using formulae for a cantilever with an intermediate load (Roarke, 1965). As stated before, the deflections had to be adjusted to the point of load application to calculate the true static stiffness. The equation shown on the following page found the effective length of the cantilever by using the known deflection at the LVDT and the applied load. The deflection at the point of load could be found using this length. An effective length was calculated for each load and the values ranged from 171 inches, at the six kip load, to 177 inches at the ten kip load. Excluding the six kip load, the average effective length was 176.2 inches.

$$y = \frac{W}{6EI}\left(-a^3 + 3a^2 l - 3a^2 x\right)$$

Where:

y	=	Deflection at desired location, in
W	=	Load applied, kips
E	=	Modulus of Elasticity of beam, ksi
I	=	Moment of Inertia, in^4
a	=	Distance from base of cantilever to point of load application, in
l	=	Total length of beam, in
x	=	Distance from end of cantilever to point of deflection, in

For the H-Pile tests, the distance from the end of the cantilever to the point of deflection was 72 inches and the distance from the end of the cantilever to the applied load was 96 inches. The ten kip load test is used for example purposes. Initially the measured deflection of 0.227 inches was used with x equal to 72 inches and the total length equal to a + 96 inches. These values were used to calculate a length from the base of the cantilever to the load to be 81 inches. The total length became 177 inches.

Once the length to the base of the cantilever was known, the deflection at the point of load could be calculated by using x = 96 inches. This resulted in a deflection at the point of load application

CHAPTER 6
SUMMARY AND CONCLUSIONS

A quick release test was designed to study the free vibrations of piles in loess. This test was designed to apply various loads to concrete piles topped with an attached mass. Preliminary field tests were conducted to assess the acceptability of the designed test.

A load frame was designed to apply a twenty kip load to the pile anywhere from four to thirty-two inches above the base of the pile. Hence, the pile may be analyzed with the load applied at both the base of the pile and at the center of gravity of the attached mass. The load frame was designed with a safety factor of 4.0 so that an ultimate load of eighty kips can be applied without yielding the frame. The load frame was proof-tested in the laboratory and in the field and thus can be used with confidence in the tests to be conducted.

The load frame was attached to a foundation to give it the necessary resistance to apply the load to the test H-pile. The foundation consisted of a six foot square slab two and one-half feet deep with one inch threaded rods embedded ten inches into the slab. The foundation to be used in future tests is an eight foot square slab of the same depth. The slab provided sufficient resistance to resist sliding and overturning forces. The larger slab was designed to utilize the entire vertical range of the load frame at an applied load of twenty kips, and it can resist a greater load if the load is applied at the base of the frame.

All facets of the pluck test were proof-tested in the laboratory as well as in the field during the preliminary field tests. The most important facet of the pluck tests was the quick release. A Peck and Hale Release-A-Matic Hook was used to initiate free vibration. The quick release was effected by pulling a pin, which in turn disengaged the hook. The hook was satisfactory as an instantaneous release, but there were problems disengaging it at high loads. The hook could always be disengaged, but the use of a static release line to effect the release needs improvement. A mechanical winch attached to the trailer hitch of a vehicle is suggested to be a more effective way to disengage the hook.

The recorded data from the preliminary field tests gave a static load-deflection curve that was nearly linear with a static stiffness of 66.7 kips per inch. The static stiffness in future tests are expected to be similar to this value. The average frequency of the tests was 15.1 hertz. The preliminary tests were conducted on an H-pile with a distributed-mass, while future tests will include a lumped mass pile cap. The frequency of future tests should be close to three hertz. The average damping ratio recorded on the H-pile was 13.9 percent of critical damping. Again, the damping ratio recorded in future tests is expected to be on this order.

From the data which were recorded in the preliminary field tests, as well as in the proof-testing which was conducted on all the equipment used in the tests, the quick release test designed can be expected to perform satisfactorily in future vibration tests.

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APPENDIXES

APPENDIX A

	Bridge Pier Project		
	H-Pile Data		
	Test - A	Load-5 kips	
LVDT #927			
CONVERSION	**VOLTS**	**TIME (SEC)**	**INCHES**
2.287	0.21820	0.000	0.095
2.287	0.22920	3.382	0.100
2.287	-0.14190	3.426	-0.062
2.287	0.07385	3.452	0.032
2.287	-0.08087	3.480	-0.035
2.287	0.00153	3.516	0.001
2.287	-0.05646	3.542	-0.025
2.287	-0.01587	3.582	-0.007
2.287	-0.04425	3.610	-0.019
2.287	-0.03723	3.632	-0.016
2.287	-0.03723	3.842	-0.016

Bridge Pier Project	
H-Pile Data	
Test - B	Load-5 kips

LVDT #927			
CONVERSION	**VOLTS**	**TIME (SEC)**	**INCHES**
2.287	-0.05798	0.000	-0.025
2.287	-0.05890	7.940	-0.026
2.287	-0.02197	8.252	-0.010
2.287	-0.02502	8.554	-0.011
2.287	0.02838	8.866	0.012
2.287	0.03662	9.696	0.016
2.287	0.09705	10.198	0.042
2.287	0.18650	23.418	0.082
2.287	-0.18160	23.456	-0.079
2.287	0.02777	23.482	0.012
2.287	-0.11530	23.520	-0.050
2.287	-0.03052	23.546	-0.013
2.287	-0.09186	23.572	-0.040
2.287	-0.04456	23.612	-0.019
2.287	-0.08087	23.636	-0.035
2.287	-0.05798	23.680	-0.025
2.287	-0.07233	23.706	-0.032
2.287	-0.06714	23.862	-0.029

Bridge Pier Project	
H-Pile Data	
Test - C	Load-6 kips

LVDT #927			
CONVERSION	**VOLTS**	**TIME (SEC)**	**INCHES**
2.287	-0.06592	0.000	-0.029
2.287	-0.06653	1.894	-0.029
2.287	-0.00977	2.894	-0.004
2.287	-0.00763	3.390	-0.003
2.287	0.05829	3.872	0.025
2.287	0.05890	4.538	0.026
2.287	-0.04974	5.066	-0.022
2.287	-0.04822	7.494	-0.021
2.287	0.02625	8.066	0.011
2.287	0.03082	8.472	0.013
2.287	0.06897	8.804	0.030
2.287	0.07050	9.284	0.031
2.287	0.12020	9.570	0.053
2.287	0.18070	22.172	0.079
2.287	-0.02081	22.258	-0.009
2.287	0.01617	22.284	0.007
2.287	-0.13210	22.320	-0.058
2.287	-0.05430	22.348	-0.024
2.287	-0.10960	22.378	-0.048
2.287	-0.07202	22.418	-0.031
2.287	-0.10410	22.442	-0.046
2.287	-0.08610	22.474	-0.038
2.287	-0.08940	22.646	-0.039

Bridge Pier Project			
H-Pile Data			
Test - D	Load-10 kips		
LVDT #927			
CONVERSION	**VOLTS**	**TIME (SEC)**	**INCHES**
2.287	-0.06531	0.000	-0.029
2.287	-0.06470	2.582	-0.028
2.287	0.02533	3.142	0.011
2.287	0.00916	3.530	0.004
2.287	0.15410	3.592	0.067
2.287	0.44310	23.558	0.194
2.287	0.45380	26.694	0.198
2.287	-0.32930	26.758	-0.144
2.287	0.11200	26.788	0.049
2.287	-0.20390	26.822	-0.089
2.287	-0.04944	26.862	-0.022
2.287	-0.14440	26.892	-0.063
2.287	-0.07904	26.930	-0.035
2.287	-0.12700	26.956	-0.056
2.287	-0.09064	26.998	-0.040
2.287	-0.13030	27.022	-0.057
2.287	-0.11780	27.326	-0.052

Bridge Pier Project			
H-Pile Data			
Test - E	Load-15 kips		
LVDT #927			
CONVERSION	VOLTS	TIME (SEC)	INCHES
2.287	0.06561	0.000	0.029
2.287	0.06805	3.018	0.030
2.287	0.22190	4.054	0.097
2.287	0.22280	4.602	0.097
2.287	0.35430	5.124	0.155
2.287	0.82430	26.400	0.360
2.287	0.82580	30.228	0.361
2.287	-0.44830	30.260	-0.196
2.287	0.28230	30.290	0.123
2.287	-0.27370	30.330	-0.120
2.287	0.02380	30.360	0.010
2.287	-0.19170	30.400	-0.084
2.287	-0.07721	30.440	-0.034
2.287	-0.15990	30.480	-0.070
2.287	-0.13210	30.500	-0.058
2.287	-0.13640	31.070	-0.060

APPENDIX B

	BRIDGE PIER PROJECT				
	DAMPING				
	TEST A LOAD-5 KIPS				
	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.116	0.048	1	0.13907	13.907
EVERY	0.048	0.017	1	0.16299	16.299
POINT	0.017	0.009	1	0.10071	10.071
			AVG	**0.13426**	**13.426**
EVERY OTHER	0.116	0.017	2	0.15107	15.107
POINT	0.048	0.009	2	0.13204	13.204
			AVG	**0.14155**	**14.155**
NEGATIVE	-0.046	-0.019	1	0.13935	13.935
EVERY	-0.019	-0.009	1	0.11809	11.809
POINT	-0.009	-0.003	1	0.17224	17.224
			AVG	**0.14323**	**14.323**
EVERY OTHER	-0.046	-0.009	2	0.12874	12.874
POINT	-0.019	-0.003	2	0.14533	14.533
			AVG	**0.13704**	**13.704**

	BRIDGE PIER PROJECT				
	DAMPING				
	TEST B LOAD-5 KIPS				
	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.114	0.041	1	0.16064	16.064
EVERY	0.041	0.016	1	0.14811	14.811
POINT	0.016	0.01	1	0.07460	7.460
	0.01	0.004	1	0.14431	14.431
			AVG	**0.13191**	**13.191**
EVERY OTHER	0.114	0.016	2	0.15439	15.439
POINT	0.041	0.01	2	0.11158	11.158
	0.016	0.004	2	0.10965	10.965
			AVG	**0.12521**	**12.521**
NEGATIVE	-0.05	-0.021	1	0.13677	13.677
EVERY	-0.021	-0.011	1	0.10237	10.237
POINT	-0.011	-0.006	1	0.09602	9.602
	-0.006	-0.003	1	0.10965	10.965
			AVG	**0.11120**	**11.120**
EVERY OTHER	-0.05	-0.011	2	0.11963	11.963
POINT	-0.021	-0.006	2	0.09920	9.920
	-0.011	-0.003	2	0.10285	10.285
			AVG	**0.10722**	**10.722**

BRIDGE PIER PROJECT					
DAMPING					
TEST C LOAD-6 KIPS					
	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.117	0.045	1	0.15035	15.035
EVERY	0.045	0.014	1	0.18270	18.270
POINT	0.014	0.007	1	0.10965	10.965
			AVG	**0.14757**	**14.757**
EVERY OTHER	0.117	0.014	2	0.16659	16.659
POINT	0.045	0.007	2	0.14648	14.648
			AVG	**0.15653**	**15.653**
NEGATIVE	-0.053	-0.02	1	0.15327	15.327
EVERY	-0.02	-0.01	1	0.10965	10.965
POINT	-0.01	-0.008	1	0.03549	3.549
			AVG	**0.09947**	**9.947**
EVERY OTHER	-0.053	-0.01	2	0.13156	13.156
POINT	-0.02	-0.008	2	0.07272	7.272
			AVG	**0.10214**	**10.214**

	BRIDGE PIER PROJECT				
	DAMPING				
	TEST D LOAD-1O KIPS				
	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.25	0.101	1	0.14277	14.277
EVERY	0.101	0.05	1	0.11121	11.121
POINT	0.05	0.017	1	0.16922	16.922
	0.017	0.012	1	0.05535	5.535
			AVG	**0.11964**	**11.964**
EVERY OTHER	0.25	0.05	2	0.12704	12.704
POINT	0.101	0.017	2	0.14040	14.040
	0.05	0.012	2	0.11284	11.284
			AVG	**0.12676**	**12.676**
NEGATIVE	-0.092	-0.037	1	0.14347	14.347
EVERY	-0.037	-0.011	1	0.18956	18.956
POINT	-0.011	-0.005	1	0.12451	12.451
			AVG	**0.15251**	**15.251**
EVERY OTHER	-0.092	-0.011	2	0.16665	16.665
POINT	-0.037	-0.005	2	0.15729	15.729
			AVG	**0.16197**	**16.197**

	BRIDGE PIER PROJECT DAMPING TEST E LOAD-15 KIPS				
	Vn	Vn+j	j	Damping	% Damping
POSITIVE	0.419	0.181	1	0.13241	13.241
EVERY	0.181	0.068	1	0.15395	15.395
POINT	0.068	0.024	1	0.16352	16.352
			AVG	**0.14996**	**14.996**
EVERY OTHER	0.419	0.068	2	0.14321	14.321
POINT	0.181	0.024	2	0.15874	15.874
			AVG	**0.15098**	**15.098**
NEGATIVE	-0.138	-0.062	1	0.12632	12.632
EVERY	-0.062	-0.026	1	0.13701	13.701
POINT	-0.026	-0.012	1	0.12214	12.214
			AVG	**0.12849**	**12.849**
EVERY OTHER	-0.138	-0.026	2	0.13167	13.167
POINT	-0.062	-0.012	2	0.12958	12.958
			AVG	**0.13063**	**13.063**

VITA

David Herschel McAlister was born December 31, 1970 in Knoxville, Tennessee. He attended Christian Academy of Knoxville and graduated in June of 1989. Later that year, he entered the University of Tennessee at Knoxville in the School of Engineering. He received a Bachelor of Science degree in Civil Engineering in August of 1993. In December of 1994, he received a Master of Science degree in Civil Engineering.