COPY NO. 2
NCHRP PROJECT 21-5 (2)

UNKNOWN SUBSURFACE BRIDGE FOUNDATION TESTING

FINAL REPORT

Prepared for

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Larry D. Olson (PI)
Marwan F. Aouad (CO-PI)
Olson Engineering, Inc.
5191 Ward Road, Suite 1
Wheat Ridge, Colorado 80033

JUNE, 2001

ACKNOWLEDGMENT OF SPONSORSHIP

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program, which is administered by the Transportation Research Board of the National Research Council.

DISCLAIMER

This is an uncorrected draft as submitted by the research agency. The opinions and conclusions expressed or implied in this report are those of the research agency. They are not necessarily those of the Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or the individual states participating in the National Cooperative Highway Research Program.

NOTICE

During the review of the Preliminary Draft Final Report for this project several comments were made by the NCHRP Technical Oversight Panel pertaining to the research findings. The reader should be aware of the reservations expressed by some of this group of experts assembled to oversee the research project from its inception to completion.

- The report is not the definitive answer to the problem presented because of the low percentage of satisfactory investigative results. While there were several inconclusive test results, the probability of incorrect test results with no viable conformation methodology available is of even greater concern.

- I seriously question the choice of 13% +/- or 15% +/-as being acceptable accuracies. A review of the spreadsheet titled 'Data from Table VI-Summary of Predicted Depths' indicates, for blind test results only, the predicted depth accuracy within +/- 15% of the actual depth was realized in 57% of the Parallel Seismic (PS) investigations and in 39% of the Ultraseismic (US) investigations, an observation not included in the report. Further review indicates successful percentages of accuracy, for blind tests only, to be in the order of 47% (PS) and 33% (US) for a +/- 10% limit or 37% (PS) and 17% (US) for a 5% limit, perhaps more realistic expectations for investors in these technologies.

The results of Project 21-05(02) will not be published in the NCHRP report series. Instead the research results are being released only in this unedited report and accompanying guideline document completed by the principal investigator, Larry Olson, of Olson Engineering. The NCHRP feels that even though the research results from Project 21-05(02) are interesting, they are inconclusive and are not of sufficient accuracy and reliability to be implemented in normal practice at this time by state DOTs.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGMENTS

The research reported herein on "Determination of Unknown Subsurface Bridge Foundations" and "Unknown Subsurface Bridge Foundation Testing" was performed under NCHRP Projects 21-5 and 21-5 (2) with Olson Engineering, Inc., of Wheat Ridge, Colorado serving as the prime contractor. Mr. Larry D. Olson, President and Principal Engineer of Olson Engineering was the Principal Investigator. Mr. Farrokh Jalinoos (formerly a Senior Geophysicist with Olson Engineering) was the Co-Principal Investigator on Project 21-5. Dr. Marwan Aouad (formerly a Project Manager of Olson Engineering) was the Co-Principal Investigator on Project 21-5 (2) who is now with the Arizona DOT. Messrs. Dennis Sack, David Fidelman, Robert Phelps and Darrol Woods of Olson Engineering were extensively involved in the research work, particularly in the hardware and software research effort. During Phases I and II research, a large number of individuals contributed their time and their resources to this project, particularly State DOT's. We would like to acknowledge the following individuals for their effort:

Mr. Ming Liu (formerly a Project Engineer of Olson Engineering) performed the theoretical modeling using programs developed by Drs. Jose Roesset (now with Texas A&M University), Shu-Tao Liao and Chih-Peng Yu while at the University of Texas at Austin,

Drs. Kenneth H. Stokoe, II and Jose M. Roesset of the Geotechnical Engineering Center of the University of Texas at Austin contributed their expertise in nondestructive testing and evaluation from their perspectives as geotechnical and structural engineers, respectively,

Dr. Alfred H. Balch of the Colorado School of Mines in Golden provided geophysical

consulting services,

Mr. Peter J. Loris of Loris and Associates, Boulder, Colorado provided the perspective of a structural engineering consultant experienced in design and inspection of bridges to the selection of bridges and analysis of their dynamic response in the research,

Mr. Ron Akin added the perspective of a retired, experienced Colorado DOT bridge engineer to the selection of bridges for research while he was working as a bridge engineer for Range Engineering of Denver, Colorado,

Dr. Glenn J. Rix, geotechnical engineering professor of the Georgia Institute of Technology, Atlanta, served as a consultant for a feasibility evaluation of applying neural network analyses to the research problem,

Dr. Dan Brown, geotechnical engineering professor at Auburn University in Alabama and Sanjoy Chakraborty, his Ph.D. research assistant, were of much assistance in the data acquisition and coordination of the drilling and casing of boreholes at the Alabama bridge site. The Alabama DOT generously drilled these boreholes for research by Dr. Brown and their use in this NCHRP project, and provided plans for the bridge,

The help of Messrs. Takashi Kanemori and John Mims of Oyo Geospace Corporation of Houston, Texas in the acquisition of the Borehole Sonic data in Texas is acknowledged,

The Colorado DOT provided access and plans to three bridges in Colorado. Weld County, Colorado provided access to a new bridge that was constructed during the project with plans provided by Loris & Associates, the bridge designer,

The Texas DOT provided plans to a newer bridge in Bastrop, Texas while the City of Bastrop provided plans to the adjacent historic bridge,

Case history information from independent consulting work by Olson Engineering, Inc. is also included herein to provide additional helpful information to the research project. The Connecticut DOT, Greiner Engineering, and AI Engineers graciously allowed the inclusion of the nondestructive testing investigation results of an Olson Engineering investigation that was independently performed during the course of this research to determine the conditions of unknown foundations for 5 Connecticut bridges. Another independent case history from Olson Engineering work preceding this research on a timber pile bridge is also included herein with permission to publish example results of this investigation graciously granted by the Santa Fe Railroad, Long Beach Port Authority, Moffat and Nichols Engineers of Long Beach, and Woodward-Clyde Consultants of San Diego,

Drs. Soheil Nazarian, Carlos Ferregut, and Mark Baker of the University of Texas at El Paso reviewed geophysical methods and statistical evaluation methods,

Mr. Malcom Lim and Dr. Bruce Morgan of Construction Technology Laboratories, Skokie, Illinois provided expertise on ground penetrating radar. In the course of the research planning and execution of this project, a large number of individuals and organizations provided voluntary, valuable technical input that is gratefully acknowledged below:

Messrs. Mac Best, Walt Mystkowski, and Joe Siccardi of the Colorado Department of Transportation (CDOT) on candidate bridge selection,

Mr. Drew Sheltinga of Weld County Engineering, Colorado on candidate bridges,

Mr. Michael Alexander of the U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, on review of potential technologies,

Mr. Graeme Beattie of Works Consultancy Services, Central Laboratories, Gracefield, Lower

Hutt, New Zealand for providing reports on the induction field method for reinforced concrete and steel piles length determination,

Mr. Robert Brooks, Engineering Data Management, Fort Collins, Colorado on research on Longitudinal Stress waves for timber pile length determination,

Dr. Robert Douglas, North Carolina State University, Raleigh, North Carolina on research on Bending Stress waves for timber pile length determination,

Drs. Everitt and Jerry Richardson of Resource Consultants and Engineers, Inc. of Fort Collins, Colorado on candidate bridge selection and related work from NCHRP 21-3 on bridge scour evaluation methods,

Dr. Michael Forde, University of Edinburgh, Scotland for input on seismic approaches and European approaches to the determination of unknown bridge foundations,

Mr. F. P. Haeni of the Hartford, Connecticut USGS office on Seismic, Electrical, and Ground Penetrating Radar geophysical methods,

Dr. Richard O. Hansen of the Colorado School of Mines, Golden, Colorado on Gravity and Magnetic geophysical methods,

Dr. Gary R. Olhoeft of the Colorado School of Mines on Ground Penetrating Radar,

Drs. Phil Nelson and Abdel Zhody of the United States Geological Survey (USGS) Denver on Electrical and Magnetic methods,

Mr. Phil Thompson, FHWA, Washington D.C. on National Bridge Inventory information on the population of unknown bridge foundations,

Dr. Roy Trent of the FHWA's Turner-Fairbank Highway Research Center, McLean, Virginia for providing related information on unknown foundation determination,

Mr. Mark Webber of LCT, Inc., Houston, Texas for Gravity and Magnetic modeling of bridge superstructure/substructure,

Dr. Charles Stoyer, Messrs. Christo Slee and Mark Butler of Interpex House for developing the "Bridgix" software,

Mr. Edward Chang, private consultant, for adding to Olson Engineering's "TFS" software,

Mr. Sam Bandimere of Denver Grouting Services for jetting a casing at one of the bridges tested in Colorado,

Messrs. William L. Moore, III and Collin Mellor of the North Carolina DOT, Raleigh, North Carolina for providing six bridges for testing. The North Carolina DOT also contributed the drilling of the boreholes at no cost to the project,

Mr. Steve Barrett of the Minnesota DOT, Roseville, Minnesota for providing three bridges for testing. The Minnesota DOT also contributed the drilling of the boreholes at no cost to the project,

Ms. Kimberly Sharp of the New Jersey DOT, Trenton, New Jersey for providing three bridges for testing. The New Jersey DOT also contributed the drilling of the boreholes at no cost to the project,

Mr. Patrick O'Rourke of the Michigan DOT, Lansing, Michigan for providing three bridges for testing. The Michigan DOT also contributed the drilling of the boreholes at no cost to the project,

Mr. Jim Braendle of the Oregon DOT, Salem, Oregon for providing two bridges for testing. The Oregon DOT also contributed the drilling of the boreholes at no cost to the project,

Messrs. Paul Nardone and Alexander Bardow of the Massachusetts DOT, Boston,

Massachusetts for providing two bridges for testing. The Massachusetts DOT also contributed the drilling of the boreholes at no cost to the project. Ms. Martha King of Frederic. R. Harris and Mr. F. P. Haeni of the Hartford, Connecticut United States Geological Survey office assisted in the testing at one of the Massachusetts bridges. Mr. Robert Connors of STV Incorporated assisted in testing at one of the Massachusetts bridges and provided photographs of the bridge, and

Mr. Walt Mystkowski of the Colorado DOT, Denver, Colorado for providing two bridges for testing. The Colorado DOT also contributed the drilling of the boreholes at no cost to the project,

Mr. George Odom of the Texas DOT, Austin, Texas and Ms. Darlene Gayle of the Texas DOT, Bryan, Texas for providing one bridge for testing. The Texas DOT also contributed the drilling of the boreholes at no cost to the project.

Finally, the interest and input provided to the research team from the NCHRP 21-5 panel during this research and the assistance of Messrs. Lloyd Crowther and Timothy Hess, Senior Program Officers of NCHRP, are gratefully acknowledged.

ABSTRACT

This report presents the test results of a research study on the feasibility of using nondestructive test (NDT) methods for the determination of unknown depths of bridge foundations. Of the over 480,000 highway bridges over water in the National Bridge Inventory, 91,094 of these bridges are over water and have unknown foundations and consequently, unknown foundation scour risks. Foremost is the need to determine the foundation depth and then foundation type (footings or piles), geometry, and subsurface conditions. In Phase I research, a comprehensive evaluation was made of potential NDT technologies at 7 bridges in Colorado, Texas and Alabama. Prior to this research, only the surface-based Sonic Echo/Impulse Response and Bending Wave methods for timber piles, and the borehole-based Parallel Seismic and Induction Field methods had been used to determine unknown foundation depths. In Phase IIA research, a total of 21 bridges were selected to evaluate the validity and accuracy of NDT methods identified during Phase I research as potential methods for determining unknown bridge foundation depths. The foundation conditions were known to only DOT engineers, but not to the research team until all depth predictions were made. Phase IIB of Project 21-5 (2) dealt with theoretical modeling of the Ultraseismic and Parallel Seismic methods with additional experimental work performed on two Colorado bridges. In addition, several types of sources were evaluated as part of the energy generation for the Parallel Seismic tests. Case history results from independent consulting projects performed by Olson Engineering are also presented to illustrate the applications of the NDT technologies to unknown bridge foundations.

This study documents the results for six acoustic and two electromagnetic NDT methods (Sonic Echo/Impulse Response, Bending (Flexural) Wave, Spectral Analysis of Surface Waves (SASW) and Ultraseismic surface methods, the Parallel Seismic, Ground Penetrating Radar and Induction Field methods in a borehole) performed on the selected bridges, whenever the testing method is applicable. The surface Ultraseismic test method and the borehole method of Parallel Seismic were found to have the broadest application to the investigated concrete, timber, masonry and steel bridge substructures. In addition to NDT testing , software and hardware were developed including prototype systems for Parallel Seismic, Ultraseismic and Induction Field systems. The software development included an upgrade of Olson's TFS software for analyzing Sonic Echo/Impulse Response, Bending Waves data and the development of a new software program called "Bridgix", devoted to the analysis of Parallel Seismic and Ultraseismic experimental data.

SUMMARY

There are approximately 481,000 highway bridges in the National Bridge Inventory. The best estimate of the population of bridges over water with unknown foundations, as of April 15, 2000, is about 91,094. These unknown bridge foundations pose a significant problem to the state DOTs because of scour vulnerability concerns. The foundation depth information in particular is needed to perform an accurate scour evaluation at each bridge site, along with as much other information on foundation type, geometry, materials, and subsurface conditions as can be obtained.

The National Cooperative Highway Research Program (NCHRP) 21-5 and 21-5 (2) projects "Determination of Unknown Subsurface Bridge Foundations and Unknown Subsurface Bridge Foundation Testing" were introduced to evaluate and develop existing and new technologies that can determine subsurface bridge foundation characteristics, where such information is unavailable. The 21-5 Phase I research focused on the identification of potential NDT methods for determining depths of unknown bridge foundations at 7 bridge sites in Colorado, Texas and Alabama. The 21-5 (2) Phase II research focused on evaluating the validity and accuracy of the identified NDT methods for determining depths of unknown bridge foundations at 21 bridge sites in North Carolina, Minnesota, New Jersey, Michigan, Oregon, Massachusetts and Colorado. Phase II research also involved the development of hardware and software needed to perform the NDT testing.

Phase I research involved field nondestructive testing (NDT) investigations of bridges with detailed foundation plans, and frequently, as-built foundation depth information. The work also involved theoretical modeling of selected bridge substructure responses for the Sonic Echo/Impulse Response, Dynamic Foundation Response, and Parallel Seismic tests for comparison with field data. Nondestructive testing was performed at seven bridge sites with four bridges located in Colorado, two in Texas, and one in Alabama under NCHRP 21-5. Also, results of two investigation case histories to determine <u>unknown</u> bridge foundation depths and conditions were reported. The field work included the performance of the seven selected NDT methods (where possible) at each bridge site. They included the four *surface* techniques of Sonic Echo/Impulse Response, Bending (Flexural) Wave, Ultraseismic and Dynamic Foundation Response; and the three *borehole* techniques of Parallel Seismic, Borehole Sonic and Borehole Radar. Additionally, the surface Spectral Analysis of Surface Waves (SASW) method was found to have some specific applications for wall-shaped substructures. The surface techniques require access from the exposed parts of the bridge substructure as opposed to the borehole methods that require access from a nearby boring.

Phase II research involved the performance of 5 acoustic and 1 electromagnetic NDT methods (Sonic Echo/Impulse Response, Bending (Flexural) Wave, Spectral Analysis of Surface Waves (SASW) and Ultraseismic surface methods, and the borehole based Parallel Seismic, Ground Penetrating Radar and Induction Field methods on the selected bridges, whenever the testing method was applicable.

Brief Description of Surface Methods. In the Sonic Echo/Impulse Response test, the source and receiver are placed on the top and/or sides of the exposed pile or columnar shaped substructure. The depth of the reflector is calculated using the identified echo time(s) for SE tests, or resonant peaks for IR tests. The Bending Wave test is based on the dispersion characteristics and echoes of bending waves traveling along very slender members like piles. The method was first developed for timber piles. The method involves mounting two horizontal receivers a few feet apart on one side of an exposed pile, and then impacting the pile horizontally on the opposite side of the pile a few feet above the topmost receiver. The Ultraseismic test involves impacting exposed substructure to generate and record the travel of compressional or flexural waves down and up substructure at multiple receiver locations on the substructure.

The Spectral Analysis of Surface Wave (SASW) test involves determining the variation of surface wave velocity vs. depth in layered systems. The bottom depths of exposed substructures or footings are indicated by slower velocities of surface wave travel in underlying soils. The Dynamic Foundation Response test was proposed mainly in an attempt to differentiate between shallow foundations and foundations with piles (or other deep foundations) underlying the visible bridge substructure. The method is based on the differences in the dynamic vibration responses of a shallow footing on piles (pilecap) and without piles (footing alone) subjected to vertical and horizontal modes of vibrations.

Brief Description of Borehole Methods. A Parallel Seismic test consists of impacting exposed foundation substructure either vertically or horizontally with an impulse hammer to generate

compressional or flexural waves which travel down the foundation and are refracted to the surrounding soil. The refracted compressional (or shear) wave arrival is tracked at regular intervals by a hydrophone receiver suspended in a water-filled cased borehole (past procedure) or by a clamped three-component geophone receiver (new procedure-better for shear) in a cased or uncased borehole (if it stands open without caving). The Induction Field method is similar in its application to the Parallel Seismic method, but employs the use of electromagnetic waves instead of stress (sound) waves. The Borehole Sonic test is a proposed new method which involves lowering a source and a receiver unit in the same or separate boreholes and measuring the reflections of compressional or shear waves from the side of the bridge substructure foundation using essentially horizontal raypaths. The Borehole Radar test uses a transmitter/receiver radar antenna to measure the reflection of radar echoes from the side of the bridge substructure foundation.

Summary of Phase I Results (Project 21-5). The results of this research indicate that of all the surface and borehole methods, the Parallel Seismic test was found to have the broadest applications for determining the bottom depth of substructures. Of the surface tests (no boring required), the Ultraseismic test has the broadest application to the determination of the depths of unknown bridge foundations, but provides no information on piles below larger substructure (pilecaps). The Sonic Echo/Impulse Response tests, Bending Wave method, Spectral Analysis of Surface Wave (SASW) method, and Borehole Radar method all had more specific applications. Summary evaluations of all tested NDT methods are presented in Tables I and II below for the surface and borehole tests, respectively.

Table I- Summary Evaluation of the Applicable <u>Surface</u> NDT Methods.

Ability to Identify Foundation Parameters	Sonic Echo (SE)/Impulse Response (IR) Test (Compressional Echo)	Bending Wave (BW) Test (Flexural Echo)
Foundation Parameters:		
Depth of Exposed Piles	Fair-Good	poor-good
Depth of Footing/Cap	Poor-Good	Poor-Fair?
Piles Exist Under Cap?	N/A	N/A
Depth of Pile below Cap?	N/A	N/A
Geometry of Substructure	N/A	N/A
Material Identification	N/A	N/A
Access Requirements:		
Bridge Substructure	Yes	Yes
Borehole	No	No
Subsurface Complications:		
Effect of soils on response	Low-High	Medium-High
Relative Cost Range:		
Operational Cost/SSU*	\$500-\$2,000	\$5,00-\$2,000
Equipment Cost	\$6,000-\$16,000	\$10,000-\$20,000
Required expertise:		
Field Acquisition	Technician	Technician
Data Analysis	Engineer	Engineer
Limitations:	Most useful for columnar or tabular structures. Response complicated by bridge superstructure elements. Stiff soils and rock limit penetration.	Only useful for purely columnar substructure, softer soils, and shorter piles. Response complicated by various bridge superstructure elements, and stiff soils may show only depth to stiff soil layer.
Advantages:	Lower cost equipment and inexpensive testing. Data interpretation for pile foundations may be able to be automated using neural network. Theoretical modeling should be used to plan field tests.	Lower cost equipment and inexpensive testing. Theoretical modeling should be used to plan field tests. The horizontal impacts are easy to apply.

*SSU = Substructure Unit cost is for consultant cost only - DOT to supply 1-2 people. Operational cost range estimates are for testing few to many SSU's per contract. N/A=Not Applicable

Table I- Summary Evaluation of the Applicable Surface NDT Methods (cont).

Ultraseismic (US) Test (Compressional and Flexural Echo)	Spectral Analysis of Surface Wave (SASW) Test	Surface Ground Penetrating Radar (GPR) Test
Fair-Excellent Fair-Excellent N/A N/A Fair N/A	N/A Fair-Good N/A N/A Poor-Good Good	N/A Poor Fair-Poor Poor Poor-Good Poor-Fair
Yes No	Yes No	Yes No
Low-High	Low	High
$1,000-$2,500 $17,000	$1,000-$2,500 $20,000	$1,000-$2,500 $20,000-$30,000+
Technician Engineer	Technician-Engineer Engineer	Technician-Engineer Engineer
Cannot image piles below cap. Difficult to obtain foundation bottom reflections in stiff soils.	Cannot image piles below cap. Use restricted to bridges with flat, longer access for testing.	Signal quality is highly controlled by environmental factors. Adjacent substructure reflections complicate data analysis. Higher cost equipment.
Lower equipment and testing costs. Can identify the bottom depth of foundation inexpensively for a large class of bridges. Combines compressional and flexural wave reflection tests for complex substructures.	Lower equipment and testing costs. Also shows variation of bridge material and subsurface velocities (stiffnesses) vs. depth and thicknesses of accessible elements.	Fast testing times. Can indicate geometry of accessible elements and bedrock depths. Lower testing costs.

Table II- Summary Evaluation of the Applicable Borehole NDT Methods.

Ability to Identify Foundation Parameters	Parallel Seismic (PS) Test	Borehole Radar (BHR) Test	Induction Field (IF) Test
Foundation Parameters:			
Depth of Exposed piles	Good-Excellent	Poor-Excellent	None-Excellent
Depth of Footing/Cap	Good	Poor-Good	N/A
Piles Exist Under Cap?	Good	Fair-Good	None-Excellent
Depth of Pile below cap	Good-Excellent	Fair-Good	None-Excellent
Geometry of Substructure	Fair	Fair-Excellent	N/A
Material Identification	Poor-Fair	Poor-Fair	Poor-Fair
Access Requirements:			
Bridge Substructure	Yes	No	Yes
Borehole	Yes	Yes	Yes
Subsurface Complications:			
Effect of soils on response	Medium	High	Medium-High
Relative Cost Range:			
Operational Cost/SSU*	\$2,000-\$4,000	\$2,000-\$4,000	\$2,000-\$4,000
Equipment Cost	\$15,000-\$25,000	\$35,000+	\$20,000
Required expertise:			
Field Acquisition/SSU*	Technician-Engineer	Engineer	Technician
Data Analysis	Engineer	Engineer	Engineer
Limitations:	Difficult to transmit large amount of seismic energy from pile caps to smaller (area) piles.	Radar response is highly site dependent (very limited response in conductive, clayey, salt-water saturated soils).	It requires the reinforcement in the columns to be electrically connected to the piles underneath the footing. Only applicable to steel or reinforced substructure.
Advantages:	Lower equipment and testing costs. Can detect foundation depths for largest class of bridges and subsurface conditions.	Commercial testing equipment is now becoming available for this purpose. Relatively easy to identify reflections from the foundation; however, imaging requires careful processing.	Low equipment costs and easy to test. Could work well to complement PS tests and help determine pile type.

*SSU = Substructure Unit cost is for consultant cost only - DOT to supply 1-2 people + does not include drilling costs. Operational cost range estimates are for testing few to many SSU's in a contract.

N/A = Not Applicable.

Summary of Phase II Results (Project 21-5 (2)). During Phase II research, 21 bridge sites were selected in the States of of North Carolina, Minnesota, New Jersey, Michigan, Oregon, Massachusetts and Colorado (another bridge was selected in the State of Texas, but no bridge plans are available) to evaluate the accuracy of the NDT methods identified in Phase I research as potential methods for the unknown foundation problem.

The field work included the performance of the selected methods (where possible) from the research planning stage at each bridge site. They included the four *surface* techniques of Sonic Echo/Impulse Response, Bending (Flexural) Wave, Ultraseismic and Spectral Analysis of Surface Waves tests; and the two *borehole* techniques of Parallel Seismic and Induction Field tests. The surface techniques require access from the exposed parts of the bridge substructure as opposed to the borehole methods that require access from a nearby boring. Table III is a summary of the results of tests performed at the 21 bridges including blind predictions and post-processed predictions.

Results from Spectral Analysis of Surface Waves (SASW) tests performed at two bridges, one in New Jersey and one in Michigan, were inconclusive. Due to limited lateral access, wavelengths in SASW tests greater than the depth of the wall-shaped foundations were not generated, thus, the results are considered to be inconclusive.

Table III- Summary of Results of the NDT Methods Performed.

Test Method	Blind Predictions	Post-Processed Predictions
Parallel Seismic	Depths of 12 out of the 19 bridges tested were predicted to within ± 13% of the actual bottom of foundation depths. Bottom foundation depths of the other 6 bridges were incorrectly predicted	Depths of 16 out of the 19 bridges tested were predicted to within ± 13% of the actual depths. There was no indication in the PS test results to support the actual depths for the remaining three bridges
Ultraseismic	Results were inconclusive on 6 out of the 18 bridges tested. Depths of 8 out of the 18 bridges were predicted within ± 14% of the actual depths. Depths of the other 4 bridges were incorrectly predicted	Test results were inconclusive on 6 out of the 18 bridges tested. Depths of 11 out of the 18 bridges were predicted within ± 14% of the actual depths. There was no indication in the US data to support the actual depth reported for the Johnston County Bridge # 33.
Sonic Echo/Impulse Response	Results were inconclusive on 14 out of the 15 bridges tested. Only one bridge with timber piles foundation in North Carolina showed an echo which corresponded to the tip of the pile	Results were inconclusive on 14 out of the 15 bridges tested. Only one bridge with timber piles foundation in North Carolina showed an echo which corresponded to the tip of the pile
Bending Waves	Results were inconclusive on all 7 timber pile bridges tested with the BW method	Results were inconclusive on all 7 timber pile bridges tested with the BW method
Induction Field	Tests were performed on two bridges, one in Colorado and one in Texas. The IF results showed a drop in amplitude below the tip of the steel pile at the Colorado bridge with good agreement with the depth predicted from Parallel Seismic tests. The IF results at the steel-pile bridge in Texas were inconclusive	Tests were performed on two bridges, one in Colorado and one in Texas. The IF results showed a drop in amplitude below the tip of the steel pile at the Colorado bridge with good agreement with the depth predicted from Parallel Seismic tests. The IF results at the steel-pile bridge in Texas were inconclusive

In addition to the blind testing of 21 bridges, Phase II research involved development of related hardware and software. Olson's TFS software was improved for Sonic Echo/Impulse Response, Bending Waves, Spectral Analysis of Surface Waves and Parallel Seismic test data analysis. The "Bridgix" software was developed by Interpex House. The "Bridgix" software is fully dedicated to the analysis of Parallel Seismic and Ultraseismic test data. In relation to hardware development, a prototype battery operated field computer was designed and manufactured. Also, developed was a prototype 8-channel hydrophone string for rapid field Parallel Seismic data collection. Two prototype systems for Ultraseismic and Induction Field tests were developed.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 INTRODUCTION.

The NCHRP 21-5 and 21-5 (2) Phase I and Phase II research projects for "Determination of Unknown Subsurface Bridge Foundations and Unknown Subsurface Bridge Foundations Testing" were conceived to evaluate, develop, and test concepts, methods and equipment that will allow the determination of subsurface bridge foundation characteristics where information is unavailable, unknown, or uncertain. The 21-5 project objective is to provide practical, accurate and cost-effective means to determine unknown foundation conditions for as many different bridge types as feasible. The foundation data will be used as input into scour evaluation studies of existing bridges over water. This report presents the combined results of Phase I and II research.

There are approximately 481,000 highway bridges in the National Bridge Inventory that are over water. It is estimated that 91,094 of these bridges have unknown foundations in terms of the type and/or depth (1). For a large number of older non-federal-aid bridges, and to a lesser extent federal-aid bridges, there are no design or as-built bridge plans, and there is little or no information available to document the type, depth, geometry, or materials incorporated in the foundations. These unknown bridge foundations pose a significant problem to state DOT's from a scour safety evaluation perspective. Because of the risk of scour undermining bridge foundations and the threat to public safety, the Federal Highway Administration (FHWA) and state transportation departments face the need to screen and evaluate all bridges, over rivers, streams and in coastal areas, both on-

and off-state systems, to determine their susceptibility to scour. The problem is that foundation type and depth information is needed to perform an accurate scour evaluation of each bridge. State bridge engineers are faced with the dilemma that their budgets cannot support the required investigations with conventional excavation, coring or boring methods to determine unknown bridge foundation conditions and simultaneously support normal maintenance operations at required funding levels.

Substantial progress has been made in the use of nondestructive testing (NDT) methods for the determination of unknown depths of bridge foundations in Phase I Research. Accomplishments in the application of NDT methods for unknown foundation depth determination included the following: i) development of the new Ultraseismic method, ii) a new application of the Spectral Analysis of Surface Waves method, iii) finite element modeling of compression and flexural wave travel for bridge substructures, iv) the use of geophysical processing, 3-component geophone recording, and finite element modeling in the borehole Parallel Seismic method, v) the application of Borehole Radar to the unknown bridge foundation problem for the first time, vi) research on a new Borehole Sonic method and vii) initial studies of the use of NDT to determine the type of foundations.

Based on the results of the Phase I research, additional Phase II funding was provided to evaluate the validity and accuracy of some of the NDT methods researched during Phase I and to develop instrumentation for use by highway departments and agencies. This was accomplished by initial blind testing of 21 bridges, and through additional research on the Ultraseismic and Parallel Seismic methods.

1.2 UNKNOWN FOUNDATION PROBLEM DISCUSSION.

Before nondestructive testing methods to detect unknown bridge foundations can be selected and studied, a general framework for the problem needs to be established. This framework is set forth in this section. The types of bridge superstructures and substructures which make up the population of bridges with unknown foundation conditions are discussed in Section 1.2.1. Representative unknown bridge foundation conditions are discussed in Section 1.2.2. The physical properties of interest for unknown bridge foundations and their relation to NDT methods are presented in Section 1.2.3.

1.2.1 Characteristics of The Unknown Bridge Foundation Population.

A national status report for the scour evaluation program was prepared by Pagan-Ortiz (1) of the Federal Highway Administration on April 15, 2000. The following information from the Pagan-Ortiz study was gathered from the National Bridge Inventory (NBI) of approximately 481,000 bridges over water.

1. Unknown foundations are considered by states as a foundation whose type is known, but its bottom elevation is unknown, or a foundation whose type and elevation are unknown. The study further characterized foundation units as either piers on land or in water, or as abutments.

2. Approximately 481,000 bridges are built over waterways. For the bridges over water, about 332,000 bridges (69.0%) are at low risk for scour.

3. 91,094 bridges (18.9%) have unknown foundation depths. 32,553 bridges (6.8%) are scour susceptible and 24,831 bridges (5.2%) are scour critical.

4. Most states have completed the evaluation of at least 90% of the bridges that are scour susceptible.

1.2.2 Bridge Superstructure, Substructure, Geology, And Channel Variables.

Before discussing possible approaches to the problem of unknown bridge foundations over water, it is useful to consider the components of the bridge system. The bridge superstructure is defined as all structure above the bridge bearing elevation and bridge substructure consists of everything below the superstructure. Therefore, bridge substructure incorporates all foundation elements such as columns, wall piers, footings, pile caps, piles, drilled shafts, etc. In this report, the terms "bridge foundation" and "bridge substructure" are used interchangeably. The challenge of nondestructive determination of unknown bridge foundation characteristics can be appreciated by considering the multitude of variables that could impact a given NDT method as listed below:

1. The bridge may be a single span with abutments only, or multiple spans with abutments and piers.

2. Foundation materials may be concrete, steel, timber, or masonry.

3. The tops of footings and pile caps may be buried below riprap, backfill and/or channel

soils

4. The bridge channel conditions may range from dry riverbed to marsh to flowing water with water conditions ranging from fresh to brackish to saltwater.

5. Subsurface conditions may range from soft silts, clays and loose sands to very stiff and dense soils to bedrock.

6. The foundation types may be shallow footings or deep foundations. Footings are most likely square or rectangular with some massive cofferdam footings, but pedestal stone footings may also exist. Such masonry foundations are older and will typically not have piles underneath. Piles might be timber, concrete (round, octagonal and square), or steel (H and round pipe sections); with or without concrete pile caps; and may be battered or vertical. Other deep foundation types used for bridges recently are concrete drilled shafts and even more recently auger-cast concrete piles. However, because of their comparatively recent use, few bridges of the unknown bridge foundation population are believed to have drilled shafts and almost none of them will have auger-cast piles.

7. The bridge substructure and superstructure are highly variable in geometry and materials. The superstructure may consist of steel, concrete, timber, or a combination of materials. The substructure is generalized herein as an abutment or pier, which can be made of steel, concrete, masonry and/or timber.

As a typical example, Mr. William L. Moore, III of the North Carolina DOT provided a detailed list and drawings of bridge substructures that was prepared by the NCDOT Bridge Maintenance Unit. They categorized North Carolina bridge substructure into three categories: 1. Abutments, 2. End Bents, and 3. Bents and Piers. Abutments were subcategorized as Full Height or Stub. End Bents were subcategorized as Reinforced Concrete, Pile or Pile Footing (capped piles). Bents and Piles were subcategorized as Concrete, Mass Concrete, Post and Beam, Steel Bent, Sill, Pile Bent, Hammerhead, Single Column, Bascule Pier, Lift Span Pier, Swing Span, Other, Steel Rigid Frame, Post and Beam with Concrete Piles, Pile Bents, and Repair Pile Bents. All told, 6 pages of differing substructure were required to list the various types of North Carolina bridge substructures. This listing shows the wide variation of bridge substructure conditions that must be considered in the application of NDT methods.

1.2.3 Physical Property Considerations of NDT of Unknown Bridge Foundations.

The following items are considered important information to be ideally determined by the nondestructive testing (NDT) methods:

1. Foundation Depth - bottom of footing, pile or combined system;
2. Foundation Type - shallow (footings), deep (piles or shafts) or a combination;
3. Foundation Geometry - buried substructure dimensions, pile locations;
4. Foundation Materials - steel, timber, concrete, and masonry;
5. Foundation Integrity - corroded steel, rotted timber, cracked concrete, etc.

The foundation depth and the foundation type (if unknown) were consistently indicated by the NCHRP panel members and others to be the two most critical items on which bridge engineers want accurate data for input into scour studies. The other items may be judged to be of secondary importance, since the evaluation of scour susceptibility is less dependent on these variables, although knowledge of the foundation substructure geometry is certainly useful in scour evaluations.

In order to decide which nondestructive testing (NDT) methods might be useful in determining unknown bridge foundation conditions, one must first consider which physical properties can be nondestructively detected to delineate the unknown bridge foundation components from the water and earth environments around the foundation substructure. Secondly, one must consider what positive or negative impacts the differing geometry and materials of the bridge superstructure and substructure will have with varying water and subsurface geological conditions on the potential NDT methods.

An unknown bridge foundation almost always has different material properties from the surrounding geological and hydrological environment in which it is constructed. The foundation material may be steel, wood, concrete, or masonry. The bridge foundation shape may be that of a footing, a pile or a combination of the two. The environment around the bridge substructure is composed of air, water, riprap materials, soils, and/or rock and is generally approximated by a horizontally layered medium of these various materials. Thus, methods to detect and delineate a bridge foundation need to primarily consider the wide ranges of substructure, geological and hydrological conditions at a particular bridge site. Depending on the NDT method, consideration

may also need to be given to the superstructure conditions of a bridge as well.

The differing material types and geometries of foundations are the two most important factors to be considered in nondestructively determining bridge foundation data for widely varying geological and hydrological conditions. Superstructure can have an adverse effect on the results of some NDT methods also. However, it is not as important a factor because since the superstructure is visible and known, <u>corrections can many times be made to account for and/or remove undesired effects from superstructure on NDT results</u>. The above characteristics of the unknown bridge foundation environment provide the background used to identify NDT methods with potential applications for foundation type and depth determination. Any NDT methods must delineate between foundation substructure and surrounding subsurface conditions. Accordingly, a wide range of possible NDT technologies based on stress waves, electrical/electromagnetic, magnetic, and gravity measurements to sense the difference between the foundation and its environment were reviewed and investigated. Existing NDT methods that have been applied to unknown bridge foundations prior to this research are introduced below.

1.3 PROVEN AND POTENTIAL NDT METHODS FOR UNKNOWN FOUNDATIONS

Four existing NDT methods were identified that had been used to investigate depths and conditions of unknown bridge foundations. In addition, a wide-ranging review was conducted of civil, aerospace, industrial and medical NDT, and geophysical technologies to identify methods that potentially could be useful for investigating unknown foundations. A summary evaluation and discussion is presented in Section 1.3.2 following the discussion of existing NDT methods below.

1.3.1 Summary Classification of Considered NDT Methodologies.

A summary of all the methodologies considered in this research to evaluate their potential application to unknown bridge foundations is presented in Table 1V. The methods are categorized based on four general Geophysical/NDT techniques of Stress Wave, Electric/Electromagnetic,

Table IV. Summary of Proven and Potential NDT Methods for Determination of Unknown Bridge Foundations.

Proven/Potential Application to Bridge Foundations:	
	A. **Stress Wave Techniques**
	1. NDT Stress Wave Methods from Substructure or Superstructure
Proven*	a. Sonic Echo/Impulse Response with Compressional Waves for Piles
Proven*	b. Dispersion of Bending Wave Energy (recent research by others and in this study)
Proven*	c. Ultraseismic (new research after Sonic Echo and Bending Wave tests)
Potential*	d. Dynamic Foundation Response (new research for shallow/deep).
	2. Surface Seismic Methods for bridge substructures and ground
None	a. Refraction (more for soils)
Minimal	b. Reflection (footing/cap top at best)
Proven**	c. Spectral Analysis of Surface Waves (independent study case history)
	3. Borehole Methods
Proven*	a. Parallel Seismic (for foundation depth with hydrophones and new research with geophones)
Potential*	b. Borehole Sonic (new research for substructure image)
Potential	c. Crosshole Seismic Tomography/Imaging (multiple boreholes for image of substructure and soils/bedrock)
	B. **Electrical and Electromagnetic Techniques**
None	1. DC-Resistivity Method on Ground Surface (supporting soils data)
	2. Ground Penetrating Radar (GPR)
Minimal	a. Surface GPR (footing/cap top at best)
Proven*	b. Borehole GPR (substructure image)
Proven	3. Induction Electromagnetic Field Method with Borehole (steel rebar/pile required)
Minimal	4. Time Domain Reflectometry from Substructure (steel rebar required - may not work)
	C. **Magnetic Techniques**
None	1. Surface Magnetic Surveys (not applicable by modeling)
Minimal	2. Borehole Magnetic Surveys (steel only - other methods better)
	D. **Gravity Technique**
None	1. Micro-Gravity Surface Survey (not applicable by modeling)

(*NCHRP 21-5 research - **Olson Case History only)

Magnetic, and Gravity methods. Each method is identified as having proven, potential, minimal, or no potential (None) by the research team for the determination of the unknown depth of the bridge foundation problem.

The methodologies are further sub-categorized in Table IV on the basis of the physical access required to apply the method. Possible test locations include: (1) the top of the bridge super/substructure or exposed sides, (2) the surface of the water or the riverbed surface, and (3) a nearby boring.

1.3.2 Discussion of Proven and Potential NDT Methods.

Obviously, methods that allow analyses from the surface of the exposed superstructure/substructure are the simplest and in most cases the least expensive tests to perform. Some methods, like the Sonic Echo/Impulse Response and Bending Wave Methods had already been successfully performed prior to this research for the determination of unknown lengths of deep foundations under existing bridges when the substructure was columnar.

Due to the predominantly vertical orientation of most foundations, borehole methods are most suited for a high-resolution characterization of the bridge substructure. Specifically, for near-vertical piles underneath pile caps, it is doubtful that any usable reflection data can be acquired with surface reflection or radar methods in most cases. In general, borehole methods are most useful for discerning pile/shaft depths, pile geometry and distribution, and a general image of the subsurface condition including riprap and scour. However, the additional cost of borings would be incurred.

Most tests require drilling only one boring, but a few, like Crosshole Seismic Tomography surveys, need two or more boreholes for the investigation to be performed. The drilling for borehole methods generally involve shallow geotechnical borings.

Generally, stress wave methods seem to offer the best combination of high resolution and low attenuation under the range of conditions found in the bridge substructure environment. They can, therefore, be more universally applied than the other NDT/geophysical methods in many different bridge, water and subsurface conditions. The advantages of stress wave (seismic) methods lie in their potential ability to image the bridge substructure directly (for Borehole Sonic, and Crosshole Seismic Tomography methods). An image of the substructure may also be able to be directly obtained from application of Borehole Radar in reflection and Crosshole Tomography tests. However, the depth of penetration of this method is very restricted by the presence of conductive materials, clay minerals, and brackish/salt water in the subsurface soils, which limits its applicability.

Electrical/electromagnetic methods are best applied when steel materials are used in the bridge. Pile depths below concrete caps can be assessed by the electric/electromagnetic methods only if reinforcing steel can be attached to in the superstructure/substructure and are electrically connected to steel piles or reinforced concrete piles. Borehole magnetic methods may also be useful in computing the depth of steel piles as the Induction Field method is known to work in this application. Some techniques (such as gravitational prospecting and surface magnetic methods) do not appear to be particularly well suited to the application at hand.

1.4 RESEARCH AND DEVELOPMENT APPROACHES.

Phase I research dealt primarily with the identification of the most promising NDT techniques for determining unknown bridge foundations. Phase II research dealt primarily with the verification and the accuracy of some of the NDT methods identified in Phase I research and instrumentation development.

Phase I- A literature search was performed to document the NDT methods that have been used for determining unknown bridge foundation conditions. The literature search led to the development of a research approach on which methods need to be investigated and improved in relation to the unknown foundation bridge problem.

Several NDT methods were investigated as part of Phase I research. Of all NDT methods researched, the borehole Parallel Seismic method was found to be the most applicable for a wide range of bridge foundations. Of the surface methods, the Ultraseismic method was found to be the most applicable for certain types of foundations. The results of Phase I research led to Phase II research continuation.

Phase IIA- For Phase II, it was first recommended that the surface and borehole NDT technologies with the greatest application range be fully validated. This was achieved by testing 21 bridges (20 bridges were originally proposed) with known bridge foundations to DOT engineers, but not to the NDT research team. The blind predictions and the subsequent feedback would clearly indicate the accuracy of the various NDT methods for a wide range of bridge substructures. This

also allowed for continued advancement in the understanding of the NDT methods, and served to indicate where improvements are needed during the course of the research.

From the bridge substructure, Ultraseismic echo tests (including Sonic Echo/Impulse Response (SE/IR), Bending Waves (BW) and Ultraseismic (US)) were recommended and used, when applicable. From boreholes or driven/jetted access tubes, the Parallel Seismic (PS) method using hydrophones and 3-component geophones with limited Induction Field testing were recommended and used at bridges with boreholes drilled adjacent to the foundations. The testing was supported with limited analytical modeling in conjunction with Ultraseismic and Parallel Seismic testing.

Secondly, prototype testing systems were developed for a surface system of Ultraseismic (US) and a borehole system of Parallel Seismic (PS) and Induction Field (IF) tests. The aim of developing the prototype hardware and software systems is to make instruments that are user friendly so that technicians can acquire field data and engineers can analyze the results. The system development included software packages for the US, SE/IR, BW and PS methods.

Coordination with State DOT's who were interested in the unknown foundation problem was critical to the success of the performed research. The research was performed to evaluate the actual capabilities of the NDT methods for determining unknown foundations by applying them to bridges with known substructure conditions (known only to DOT engineers until predictions were made). Also the interested State DOT's provided the boreholes needed for PS and IF testing at no cost to

the project.

Phase IIB- This phase initially involved testing of 20 bridge sites around the country to demonstrate and apply the technology developed during this research. The Phase IIB research was modified and approved by the panel members to include evaluation of energy sources for the PS method and theoretical modeling for the PS and US methods.

1.5 NCHRP 21-5 AND 21-5 (2) RESEARCH OBJECTIVES.

1.5.1 NCHRP 21-5 Research Objectives - Tasks 1 Through 7.

The initial phase of the research, Tasks 1-4, was to determine the feasibility of adapting/developing practical methods and equipment for the determination of subsurface bridge foundation characteristics such as type, depth, geometry, and material type where little or no information is available. The second phase of the research, Tasks 5-7, involved the performance of the research recommended in Task 4 in the Interim Report. The tasks set forth in the NCHRP 21-5 request for proposals are presented below.

Task 1 (begun April 26, 1992). Review and summarize existing, proposed, and conceptualized domestic and foreign technologies having promise for use in determining unknown subsurface bridge foundation characteristics such as type, depth, geometry, and materials. The review should be interdisciplinary, comprehensive, and consider technology transferable from other sources in addition to the highway industry.

Task 2. Develop an analytical process for screening and evaluating concepts, methods, and equipment which can be applied to the determination of unknown subsurface bridge foundation characteristics. As a minimum, the process must consider the ability of the concept, method, or equipment to identify the foundation type, foundation material, and geometry (including depth, size, and number of elements) under differing geology, hydraulic, and hydrologic conditions.

Task 3. Evaluate the concepts, methods, and equipment that were identified in Task 1 using the screening and evaluation process developed in Task 2 noting advantages, limitations, developmental costs, initial and operational costs, and other important features and considerations. Provide recommendations on the concepts, methods, and equipment that show promise for further development and testing.

Task 4. Document the findings of Tasks 1 through 3 in an interim report to be submitted not later than 10 months after initiation of the study. The interim report shall include a detailed research plan for evaluating and testing as many of the recommended concepts, methods, and equipment as are feasible under the remaining project budget. The detailed research plan may require the performance of mathematical or physical model studies, and/or field evaluations. NCHRP approval of the interim report and the detailed research plan shall be required before proceeding with the remaining tasks.

Task 5 (begun August, 1993). Perform the studies and evaluations in accordance with the NCHRP approved research plan.

Task 6. Summarize the results of the studies and evaluations conducted under Task 5. Recommend concepts, methods, and equipment that provide a means for determining subsurface foundation characteristics, specifically noting the advantages and limitations of each. Provide an estimate of the initial and operational costs for those methods and equipment that can be readily implemented in the field. Provide a plan which includes estimates of the cost and time necessary to fully develop and validate the recommended concepts, methods, or equipment which cannot be readily implemented in the field at this time.

Task 7. Submit a final report documenting all research findings. The final report for Phase I research was submitted in August, 1995.

1.5.2 NCHRP 21-5 (2) Research Objectives - Tasks 1 Through 5.

The initial phase of the research (Phase IIA), Tasks 1-3, was to determine the feasibility of adapting/developing practical methods and equipment for the determination of subsurface bridge foundation characteristics, particularly the depths of the foundations. The second phase of the research (Phase IIB), Tasks 4-5, involves the performance of the semi-blind study of additional 20 bridges. The tasks set forth in the NCHRP 21-5 (2) request for proposals are presented below.

Task 1. Submit a research workplan within 2 weeks of funding start. Evaluate in greater detail those methods that were identified in the Phase I study as having most promise to the unknown depth of bridge foundation problem. Perform the research planning and evaluations in accordance with the

NCHRP approved research plan. Conduct initial "blind" NDT studies of 20 bridge sites with known foundations to determine accuracy of methods. Task 1 was further subdivided into the following subtasks:

Subtask	Workplan
1.0	Administration/Travel/Research Planning
1.1	Ultraseismic (including SE/IR and BW methods)
1.2	Parallel Seismic (PS)
1.3	Spectral Analysis of Surface Waves (SASW)
1.4	Induction Field (IF)/Electromagnetic logging

Task 2. Develop prototype Ultraseismic, Parallel Seismic and Induction Field instrument systems.

Task 3. Submit an interim report documenting all research findings (report submitted in April, 1998). Summarize the results of the studies and evaluations conducted under Task 1. Report developments of specific concepts, methods and equipment that provide a means for determining subsurface foundation characteristics, for the selected methods in Phase I. Propose a plan for the demonstration project of 20 bridge sites in 5 regions.

Task 4. Execute the semi-blind demonstration study of 20 bridge sites in 5 different regions. This task was modified and approved by the panel members to include evaluation of energy sources for the PS method and theoretical modeling of the PS and US methods.

Task 5. Submit a final report documenting all research findings and the results of the demonstration study and other DOT case histories. This task was also slightly modified to include a guidelines document that can be used by state DOT's and other agencies on their applications for determining unknown bridge foundations.

1.6 REPORT ORGANIZATION.

Chapter 2 of the report presents an overview of the NDT methods used in the research. The intent was to give the reader an insight on the NDT methods used during this research.

Chapter 3 contains the results and conclusions of Phase I research. Chapter 4 represents the results and conclusions of Phase II research. Recommended future research and conclusions are presented in Chapter 5. Appendix A contains descriptions and photographs of the bridges tested during Phase I and Phase II research. Also show in Appendix A are photos of many of the NDT methods that were used in the research.

CHAPTER 2

REVIEW OF NONDESTRUCTIVE TESTING METHODS

The state-of-practice for nondestructive determination of unknown foundation conditions has primarily involved two methods: the surface Sonic Echo/Impulse Response tests, and the borehole Parallel Seismic method. The Sonic Echo/Impulse Response tests involve measurement of the echoes (reflections) of compression (longitudinal) stress waves from foundation bottoms that are generated by as vertical an impact as possible to rod-like shaped concrete piles, timber piles, drilled shafts, and other columnar shaped foundation elements. The SE/IR method has been used on bridge piles and shafts by the authors and others (2). Reflection times and frequencies measured by receivers mounted on the tops and/or sides of the deep foundations are used with the compression wave velocity to calculate foundation bottom reflector depths. The Sonic Echo test for piles and shafts was first developed for quality assurance of the integrity and length of newly constructed driven piles (concrete and timber) and drilled shafts (concrete). The Parallel Seismic method has broad application to a wide range of substructure foundations since it involves impacting the exposed substructure to generate seismic wave energy that travels down the foundation that is sensed by a receiver in a nearby borehole. The seismic wave energy travels faster with more energy when the receiver is next to or parallel to the unknown foundation than when it is below it. Conversely, as the wave energy reaches the bottom of a foundation and is transmitted into the underlying soils, the wave velocity and energy of the waves sensed by the nearby borehole receiver are much lower at depths below the foundation bottom than when the receiver is in the same depth range as the foundation. Plots of the wave arrival and/or energy thus indicate the foundation depth.

Two additional methods have also been used to indicate unknown foundation depths for specific types of piles: the borehole Induction Field method for piles with steel in them, and the surface Bending Wave (Flexural Wave) method for timber piles. The Induction Field method is the electromagnetic analog to the Parallel Seismic method and involves generating an electromagnetic field with a source which is electrically attached to steel foundation substructure and then measuring the magnetic field in a borehole adjacent to the unknown foundation. This method is only applicable to foundations of steel (H-piles, pipe piles, and reinforced concrete piles and shafts), and will not work on timber piles, plain concrete or unreinforced masonry foundations because these foundations are non-conductive. Mr. Graeme Beattie of Works Consultancy Services, Lower Hutt, New Zealand indicated that the Induction Field method (3) was researched about 15 years ago by the New Zealand Department of Scientific and Industrial Research (4). He stated that the method is typically applied about once a year in New Zealand, and he was unaware of any other NDT methods for determination of unknown bridge foundations.

A research study on determining lengths of timber piles with dispersive bending wave propagation methods has recently been conducted at North Carolina State University (5, 6). The Bending Wave method is limited to more slender piles such as timber piles, and is similar to the Sonic Echo/Impulse Response tests in that one uses the bending wave velocity and the reflection time for the bending wave energy to return from the pile bottom to calculate its depth. Horizontal impacts are applied to the exposed portion of the timber pile and bending wave travel is monitored by horizontal receivers (preferably mounted below the impact point). During the course of this research, the Spectral Analysis of Surface Waves method was used on a limited basis on wall-shaped

bridge foundations. Brief descriptions of the NDT methods are presented below (for a detailed description of the NDT methods, see Olson et al. (7).

2.1 SONIC ECHO (SE)/IMPULSE RESPONSE (IR) METHOD.

The Sonic Echo/Impulse Response (SE/IR) method was developed for testing the integrity and length of single, rod-like, columnar shaped deep foundations such as drilled shafts and driven piles. A schematic of the SE/IR method is shown in Fig. 1a. The method is based on the principle that stress waves will reflect from significant changes in stiffness (i.e., changes in acoustic impedances which is velocity x mass density x cross-sectional area for foundation substructures). Much like sonar or a fish-finder, the Sonic Echo/Impulse Response test of bridge substructure involves measuring the velocity of wave travel in the known substructure, tracking the reflection events as to whether they are coming from above or below the source/receivers locations, and then calculating the reflector depth corresponding to the foundation bottom (or other significant change in stiffness).

The Sonic Echo/Impulse Response method classically involves impacting the top of a deep foundation with a hammer to generate a downward traveling compressional wave (5,6); See Fig. A.3a for an actual SE/IR test on a timber pile. The wave energy reflects back to the surface from changes in stiffness, cross-sectional area, and density (i.e. the reflections are from changes in acoustic impedance). A neck or break has lower impedance relative to a sound pile section while a bulb or a much stiffer soil or bedrock layer has a higher impedance. The arrival of the reflected compressional wave energy is sensed by a receiver (accelerometer or vertical geophone). Analyses

are done in the time domain for the Sonic Echo test and in the frequency domain (mobility transfer function, i.e. velocity/force) for the Impulse Response test.

A reflection from a change in impedance (velocity * density * area) is seen as an increase in amplitude and change in phase of the receiver response versus time in the Sonic Echo test. The same reflection event is seen as evenly spaced frequency peaks that correspond to the resonant echo in the Impulse Response test. Test equipment typically includes an impulse hammer (measures impact force), accelerometer (acceleration) and vertical geophone (velocity) receivers, and a micro-processor based recording and processing system, e.g., a dynamic signal analyzer, or a commercial SE/IR system.

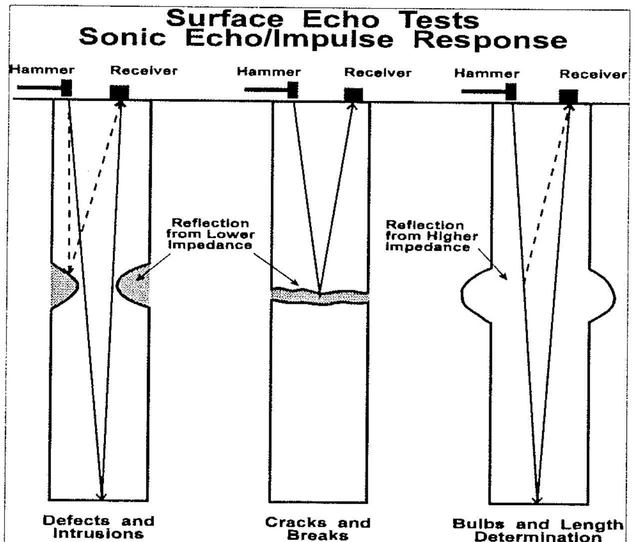


Figure 1a- Schematic of the Sonic Echo/
Impulse Response Method.

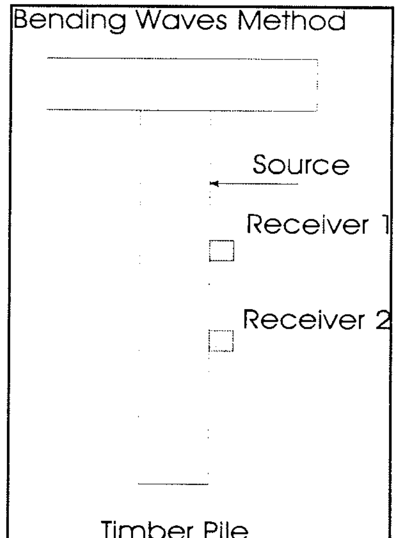


Figure 1b- Schematic of the
Bending Waves Method.

The Sonic Echo/Impulse Response method is most applicable to columnar substructures on drilled shafts, or deep foundations that are exposed above the ground or water. Side-mounting receivers and setting nails, screws, blocks or other impact contact points can be accomplished without much effort for such conditions. Ideally, the compressional wave velocity is measured between the two side-mounted receivers to improve the accuracy of the length prediction. Using two receivers when testing on the sides of substructure is highly recommended in order to determine whether wave reflection events are coming from the bottom or top of the substructure.

For concrete and steel piles, lengths can be predicted to within 5% to 10%. Brooks et. al. (2) estimate that the lengths of timber piles can be conservatively predicted within 15% due to the greater variation of compressional wave velocity in wood than in concrete.

In most cases, exponential amplification with time of the receiver trace is required to enhance weak echoes and compensate for the damping of the wave energy as it travels up and down a foundation (7). For wavelengths that are long relative to the foundation diameter, then for rod-like foundations the bar compression wave velocity, V, is equal to the square root of Young's modulus, E, divided by the square root of mass density (ρ, unit weight divided by gravity) or $V = (E/\rho)^{0.5}$. The depth of the reflector is calculated as follows:

$$D = \Delta t * V / 2$$

where D is the reflector depth, Δt is the time interval between two echoes and V is the bar velocity of compressional waves.

The Impulse Response (also called Sonic Mobility and Transient Dynamic Response) test is similar to the Sonic Echo test, but analysis of data is performed in the frequency domain (9). The transfer and the coherence functions, and the auto power spectrum of the receiver are typically recorded in an Impulse Response test. The transfer function or the auto power spectrum is used to calculate the depth of reflectors. The coherence function is used to judge the quality of data. The depth of reflector is calculated as follows:

$$D = V / (2 * \Delta f)$$

where Δf is the frequency interval between two or more evenly spaced resonant peaks (1^{st}, 2^{nd} and 3^{rd} modes, etc.)in the transfer function or the auto power spectrum plots.

<u>SE/IR Example Results</u> The source/receiver layout for Sonic Echo/Impulse Response tests on Pile 1 of Bent 4 of the Wake County bridge # 207, North Carolina is shown in Fig. 2. The Sonic Echo test results form Pile 1 of Bent 4 are presented in Fig. 3. A possible echo was identified at a depth of 7.22 m (23.7 ft) for an assumed compression wave velocity of 3,660 m/sec (12,000 ft/sec). The upper trace in Fig. 3 represents the accelerometer output and the lower trace represents the upper trace after integration and exponential amplification. This echo was calculated based on a $\Delta t = 3.95$ ms as shown in Fig. 3. The actual length of the pile was equal to 7.6 m (25 ft) based on drilling information. The difference in length between the predicted and actual length of the pile was equal to 0.4 m (1.3 ft), a good agreement of -5% difference. This was one of the rare cases where Sonic Echo tests were able to determine the depth of a timber pile.

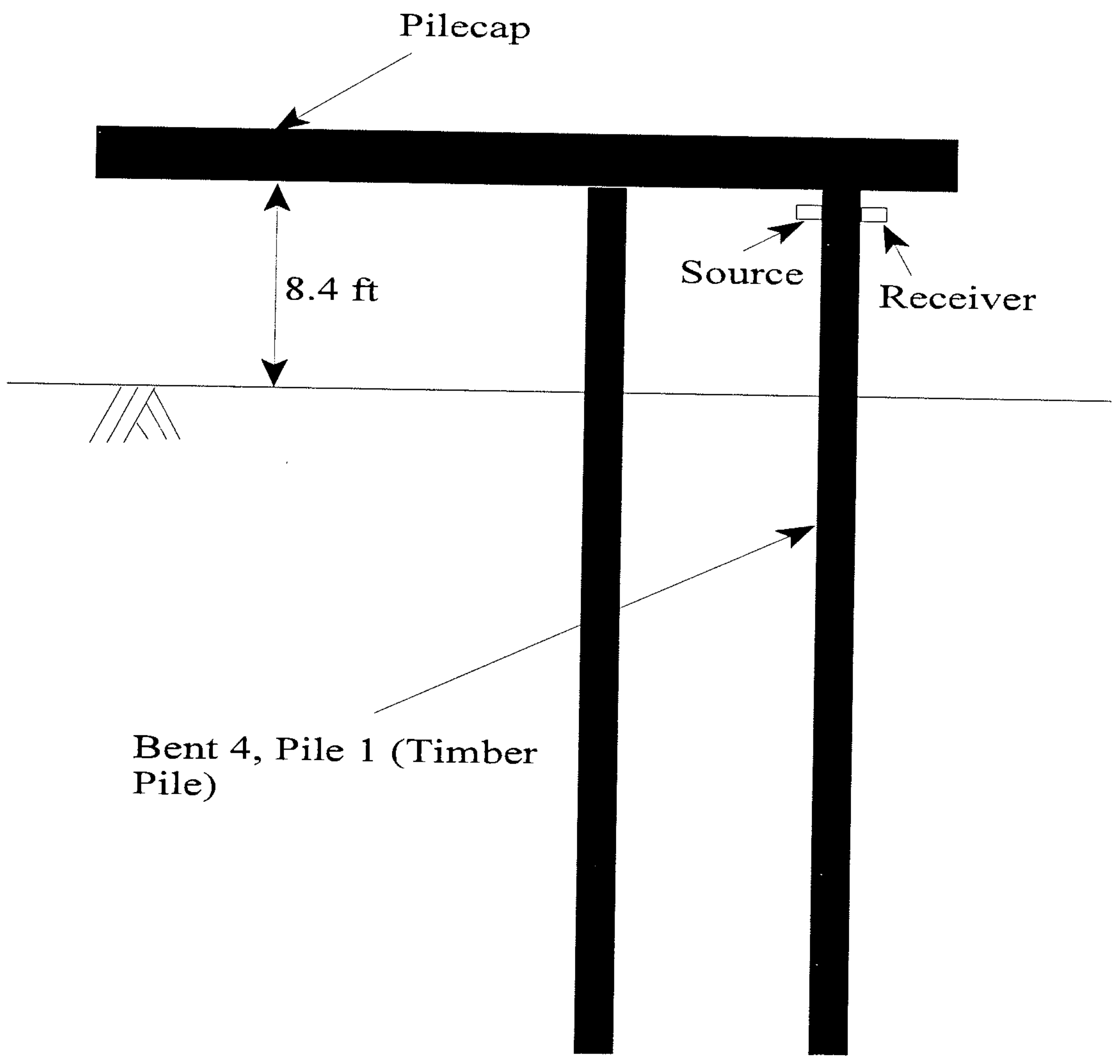


Figure 2- Source/Receiver Layout for Sonic Echo/Impulse Response Tests Performed at Bent 4, Pile 1, Wake County Bridge # 207, North Carolina.

Comments:
Receiver and source are placed at 1 ft below the top of the pile
Possible echo identified at $\Delta t = 3.95$ ms
Assumed wave velocity of 12,000 ft/sec
Bottom depth = $(V^*\Delta t/2) = 23.7$ ft (reference is top of pile)

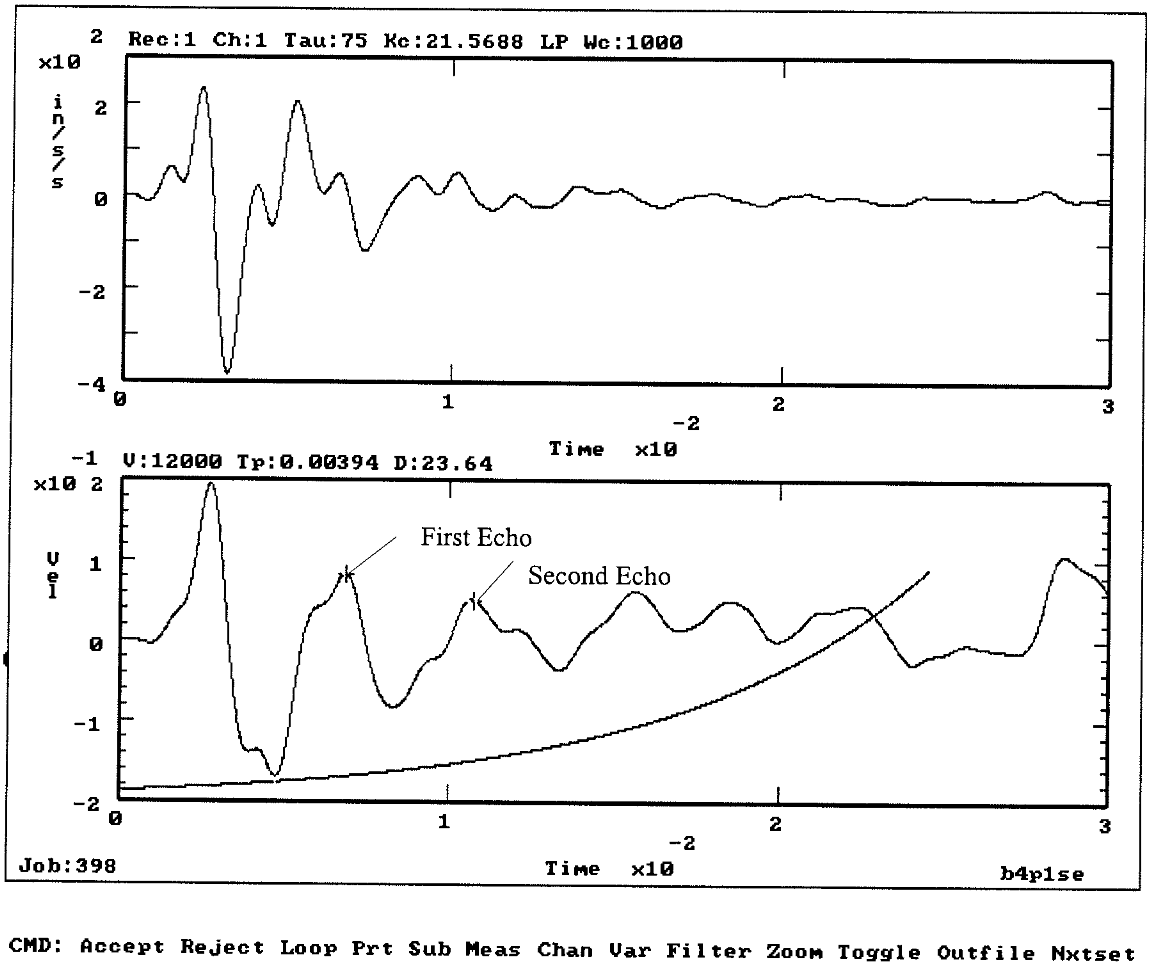


Figure 3- Sonic Echo Test Results Using a 3-lb Vertical Hammer Hit and a PCB 308B02 Accelerometer, Bent 4, Pile 1, Wake County Bridge # 207, North Carolina.

The Impulse Response test results from Pile 1, Bent 4 are presented in Fig. 4. A possible echo was identified at a depth of 6.9 m (22.6 ft) for an assumed wave velocity of 3,660 m/sec (12,000 ft/sec). The upper trace in Fig. 4 represents the coherence function and the lower trace represents the mobility function (velocity/force). This echo was calculated based on a peak $\Delta f =$ 265.5 Hz as shown in Fig. 4. The actual length of the pile was equal to 7.6 m (25 ft) based on drilling information. The difference in length between the predicted and actual length of the pile was equal to 0.7 m (2.4 ft), a good agreement of -10% difference.

2.2 BENDING WAVES METHOD.

This method was first researched for timber piles for depth prediction by Dr. R.A. Douglas of North Carolina State University and Dr. J.D. Holt who performed a North Carolina DOT research project to predict timber pile lengths on bridges. The method is based on the principles of bending wave propagation in slender, rod-like media. The method involves exciting dispersive bending (flexural) waves in a pile, measuring the varying (dispersive) bending wave velocity, using horizontal receivers to track bending wave reflections in time from boundaries (head, toe, stiff soil interfaces, other significant stiffness, or "impedance" changes) and calculating the depths of the reflectors (5, 6), see Fig. 1b for the Bending Waves Schematic and Fig. A.9 for an actual test on a timber pile. Recent research on the Bending Waves method included work by Chen (10), Qian and Kim (11) and Hughes et al. (12). Chen performed theoretical modeling to better characterize the dispersive characteristics of bending waves. Qian and Kim used the Bending Waves method to predict the remaining cross-sectional areas of timber piles while Hughes et al performed modal analysis using bending waves. The Bending Waves method involves horizontally impacting the pile

Comments:
Receiver and source were placed at 1 ft below the top of the pile.
A possible echo was identified at Δf = 265.5 Hz. Then for
an assumed wave velocity of 12,000 ft/sec, a predicted pile bottom is at
depth = (V/Δf*2) = 22.6 ft (reference is top of pile)

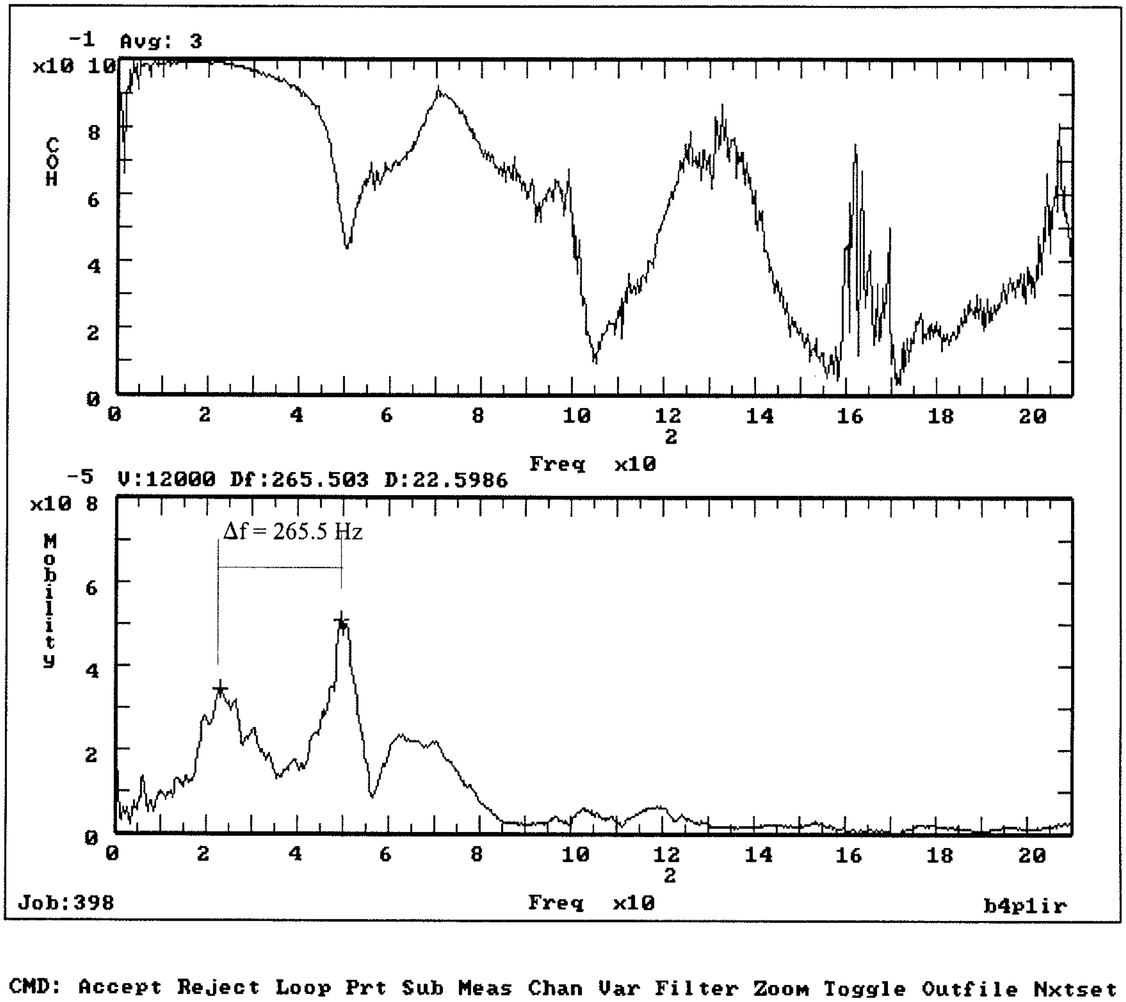


Figure 4- Impulse Response Test Results Using a 3-lb Vertical Hammer Hit and a PCB 308B02 Accelerometer, Bent 4, Pile 1, Wake County Bridge # 207, North Carolina.

to generate flexural or bending waves that travel up and down the pile. The bending wave propagation is monitored by two horizontal accelerometer receivers mounted on the same side of the pile from the. The Bending Wave method with the Short Kernel Method analysis advanced by Douglas (5) can be thought of as being the bending wave equivalent of the Sonic Echo method, which uses the faster compressional (longitudinal) waves. Both methods involve determining the velocity of wave travel, then identifying initial wave arrivals and subsequent reflections (echoes) from impedance changes to calculate the depths and locations of pile foundation bottoms.

Douglas and Holt (5, 6) detail the research and development of the use of dispersion of bending (flexural) wave energy to predict pile depths. Dispersion of stress waves means that the velocity of wave travel is not a constant, but is a function of frequency or wavelength. Stress wave velocity (V), frequency (f) and wavelength (λ) are related by the following equation.

$$V = f\,\lambda$$

Bending waves in piles are highly dispersive. The bending wave velocity decreases with increasing wavelength (lower frequency) with most of the velocity decrease occurring at wavelengths that are longer than the pile diameter, and these longer waves propagate as flexural or bending wave energy. Correspondingly, as wavelengths become shorter than the diameter of a pile, the bending wave velocity limit is approximately that of the surface (Rayleigh) wave velocity, and this wave energy propagates as surface waves. Compressive waves are also dispersive in piles, but in a different way that in practice results in a bar velocity decrease only when a deep foundation has a low length to diameter ratio (slenderness) of about 2:1 or less which is uncommon for deep

foundations (13,14).

Douglas and Holt used the Short Kernel Method (SKM) to analyze the data. The method is similar to narrow band cross-correlation procedures between the input (the hammer blow) and the output (receiver response(s)). However, instead of measuring the hammer blow, a periodic function of 1 or more cycles is used as a "Kernel Seed", and a number of seeds of frequencies ranging from 500 to 4000 Hz may be cross-correlated with the receiver responses. The SKM correlation procedure amplifies bending wave energy responses with the selected seed frequency and in a way bandpass filters the response data since frequencies higher and lower than the seed frequency are filtered out. Two receivers are used in order to measure the bending wave velocity (distance divided by elapsed time for between the bending wave arrival peaks) between them as determined from the peak responses in the cross-correlated data of the two receivers. The use of two receivers also allows one to determine whether the reflections of the bending wave energy are traveling back up the pile (the bottommost receiver senses the wave energy first) after reflection from the pile bottom, or if the bending wave energy is traveling back down the pile (the topmost receiver senses the wave energy first) after reflection from the pile top or beam. This is identical to the procedures used in Sonic Echo tests when 2 receivers are used. The dispersion of the bending wave velocity is thus accounted for by calculating the bending wave velocity for each Kernel seed frequency.

Bending Waves Example Results. The source/receivers layout for Bending Waves (BW) tests on the east Pile of Bent 1, Wilson County Bridge No. 5, North Carolina is shown in Fig. 5. The Kernel seed used in the analysis was 1-cycle of a 1000 Hz sine wave. The bending wave velocity

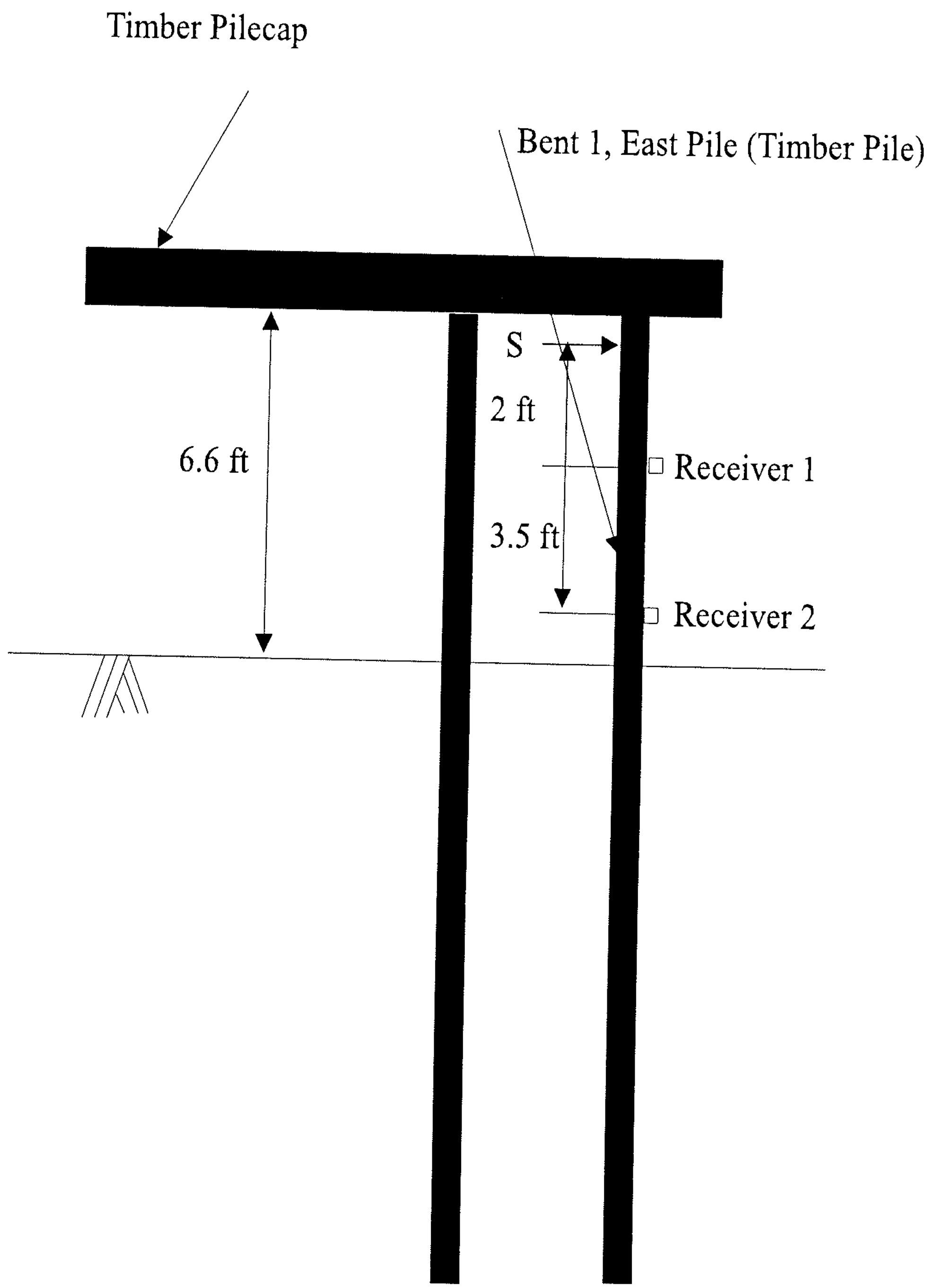


Figure 5- Source/Receiver Layout for Bending Waves Tests Performed at Bent 1, East Pile, Wilson County Bridge # 5, North Carolina.

Comments:
Distance between receivers = 3.5 ft
Δt = 1.28 ms (Using a Kernel of 1000 Hz)
Velocity = $3.5/(1.52*10^{-3})$ = 2,300 ft/sec

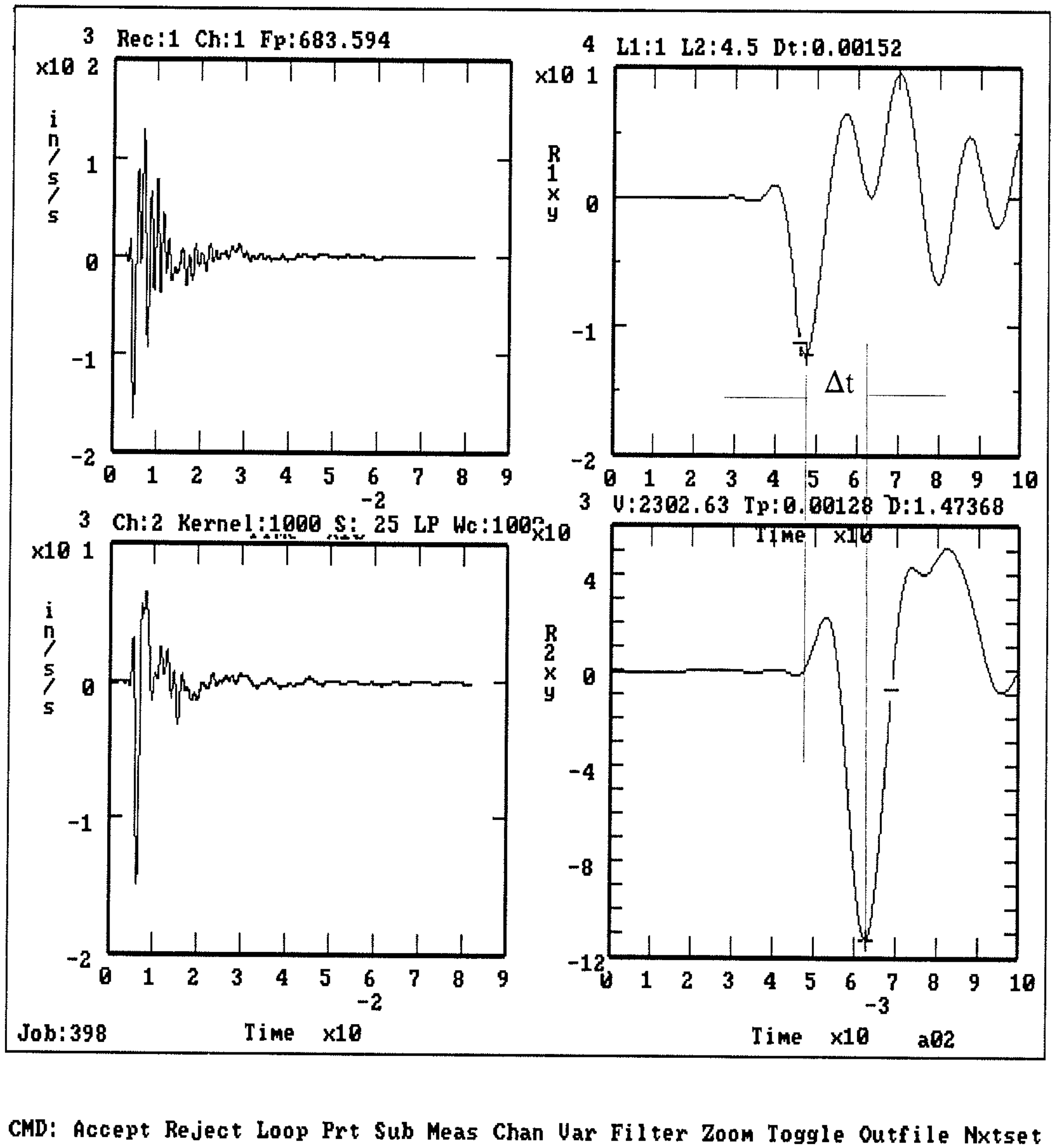


Figure 6- Velocity Calculation from Bending Waves Tests
Wilson County Bridge # 5, Bent 1, East Pile, North Carolina.

was found to be 700 m/sec (2,300 ft/sec) for the initial bending wave arrival time peaks in the Short Kernel Method cross-correlation records presented in Fig. 6. No apparent echoes from the tip of the pile are present in Fig. 7. For all of the BW tests performed on 7 timber pile bridges during Phase II research, the results were inconclusive. This is due to the difficulty in identifying a reflection based on two traces only and the severe attenuation of bending wave energy in stiffer soils. Reflections from other boundaries are present in the data which obscured the interpretation of the BW results. These reflections may be caused by surface wave energy reflected from many boundaries in a bridge substructure and reflection of bending waves at the air/ground surface interface. Based on this limited study of BW tests, the BW method cannot be considered as a reliable method for the determination of unknown bridge foundations, except for cases of short piles in soft soils.

2.3 ULTRASEISMIC METHOD.

The Ultraseismic (US) method was researched and developed during the NCHRP 21-5 research for determination of the unknown depth of bridge foundation . The Ultraseismic method is a <u>sonic reflection technique</u> that uses geophysical digital data processing techniques to analyze the propagation of induced compressional and flexural waves as they reflect from foundation substructure boundaries (impedance changes, i.e. changes in the multiple of wave velocity x density x cross-sectional area). This is the same principle that the Sonic Echo/Impulse Response and Bending Wave methods rely on as well, but the data acquisition and processing for the US method involves recording and display of multiple channels of data as discussed below. A schematic of the Ultraseismic method is shown in Fig. 8 (See Fig. A.5b for an actual US test on a steel pile). The

Comments:

No echoes from the bottom of the shaft, Results are inconclusive

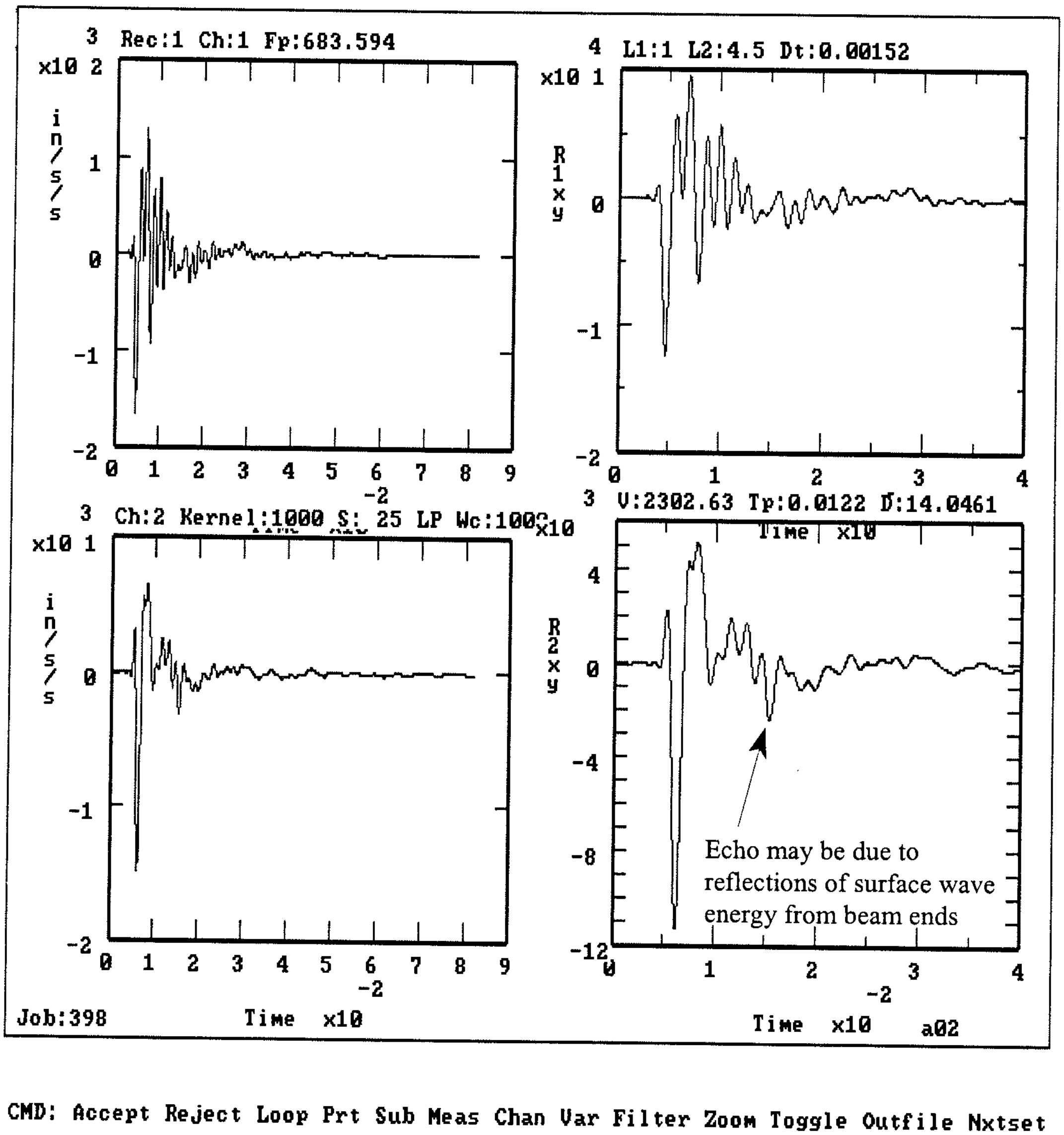


Figure 7- Bending Waves Test Results
Wilson County Bridge # 5, Bent 1, East Pile, North Carolina.

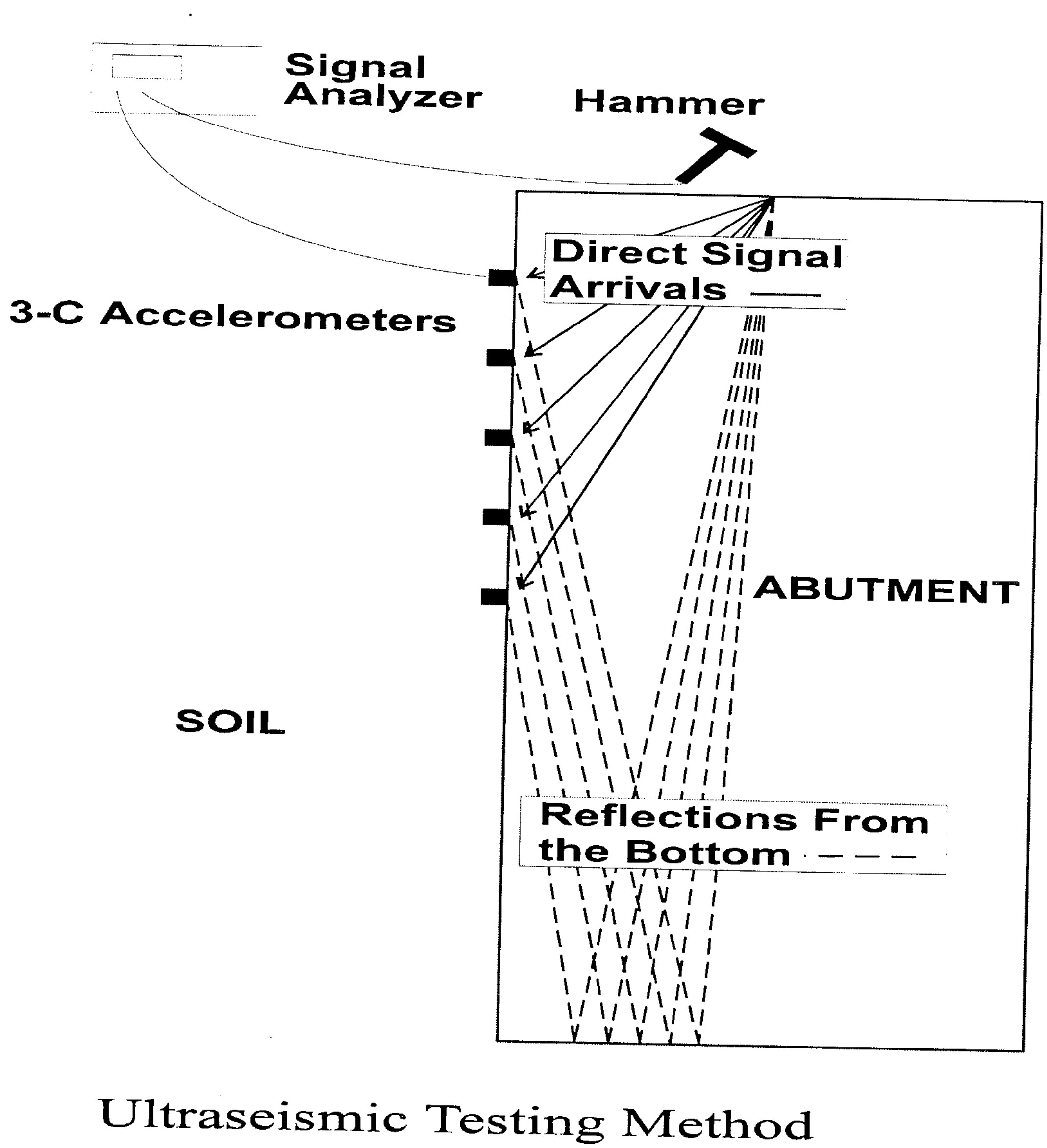


Figure 8- Schematic of the Ultraseismic Method.

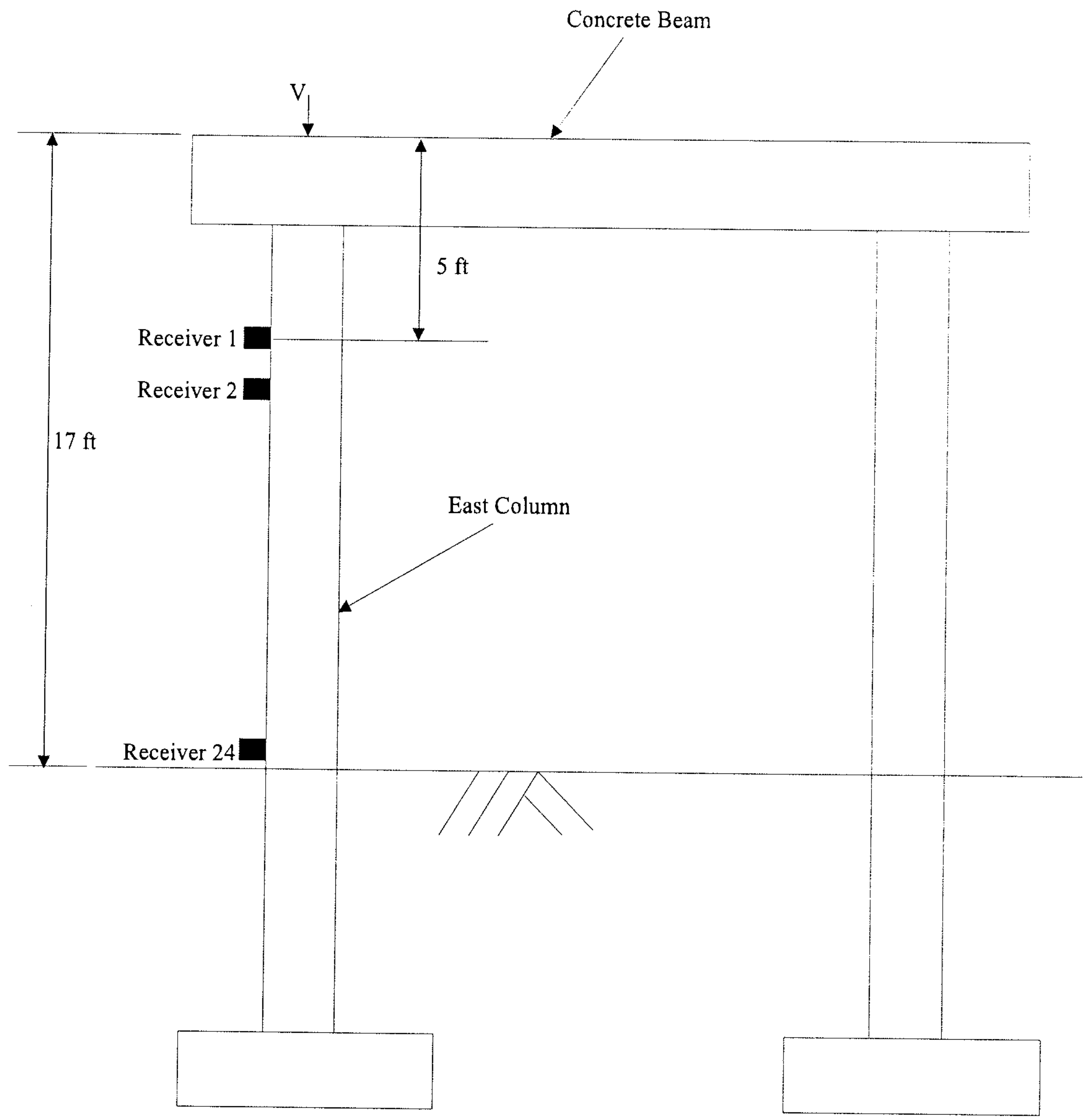
Ultraseismic method was researched and developed by Olson Engineering in response to the difficulties encountered by the Sonic Echo/Impulse Response method and the Bending Wave method tests on non-columnar and complex columnar bridge substructures. The Ultraseismic method is a broad application of geophysical processing to both the Sonic Echo/Impulse Response and Bending Wave tests in that the initial arrivals of both compressional and bending waves and their subsequent reflections are analyzed to predict unknown foundation depths. Two types of Ultraseismic test geometries have been specifically introduced for this problem:

1. For a one dimensional imaging of the foundation depth and tracking the upgoing and downgoing events, the term Vertical Profiling (VP) test method is used. In this method, the bridge column or abutment is hit from the top or bottom (both vertically and horizontally) and the resulting wave motion is recorded at regular intervals down the bridge substructure element. Typically, three-component recording of the wavefield is taken in order to analyze all types of ensuing wave motion. A VP line can be run in *both* a columnar (like a bridge pier or pile foundation) and a tabular (like a wall shaped bridge abutment) substructure.

2. For two-dimensional imaging of the foundation depth, the term Horizontal Profiling (HP) test geometry is used. In this method, the reflection echoes from the bottom are analyzed to compute the depth of the foundation. The source and receiver(s) are located horizontally along the top of accessible tabular shaped substructure, or any accessible face along the side of the substructure element, and a full survey is taken.

The Ultraseismic method uses multi-channel, 3-component (vertical and two perpendicular horizontal receivers, i.e. triaxial receiver) recording of acoustic data followed by computer processing techniques adapted from seismic exploration methods. Ultraseismic records were collected by using 0.1, 0.5, 1.5 and 5 kg (0.2, 1, 3, and 12 lb) impulse hammers as the source and accelerometers as receivers that are mounted on the surface or side of the accessible bridge substructure at intervals of 0.3 m (1 ft) or less. The bridge substructure element is used as the medium for the transmission of the seismic energy. All the usual wave modes traveling down or reflected back (echoes from the bottom) can be recorded by this method. The seismic processing can greatly enhance data quality by identifying and clarifying reflection events that are from the foundation bottom and minimizing the effects of undesired wave reflections from the foundation top and attached beams. For concrete bridge elements, useful wave frequencies up to 4-5 kHz are commonly recorded. All data are de-biased to remove any DC shift, bandpass zero-phase filtered (0-0.5-3-4 kHz trapezoidal filter), and Automatic Gain Controlled (AGC) as desired.

Ultraseismic Example Results. The source/receiver layout for Ultraseismic Vertical Profiling tests on one of the pier columns of Bridge No. 5188, Minnesota Highway 58 in Zumbrota is shown in Fig. 9. The source was located vertically on top of the concrete beam using a 1.36 Kg (3 lb) hammer. A 3-component accelerometer was mounted on the side of the exposed portion of the column at intervals of 15 cm (6 in) from 1.5 m (5 ft) below the top of the beam to near the ground surface. Field data for a Vertical Profiling test done to measure flexural waves is shown in Fig. 10. The depth shown in Fig. 10 represents the depth below the top of the concrete beam. All data were de-biased to remove any DC shift and f-k filtered to enhance upgoing waves. A clear



V = Vertical Hammer Hit Location
24 Accelerometer Receiver Locations at Intervals of 0.5 ft

Figure 9- Source/Receiver Layout for Ultraseismic Tests Performed at Bridge No. 5188, Minnesota Hwy 58, Zumbrota, Minnesota.

Comments:
Depth shown in Figure is depth below top of pier
Depth of first reflector = 16.5 + 6.5 = 23 ft (reference is top of pier)
Depth of second reflector = 16.5 + 14 = 30.5 ft (reference is top of pier)

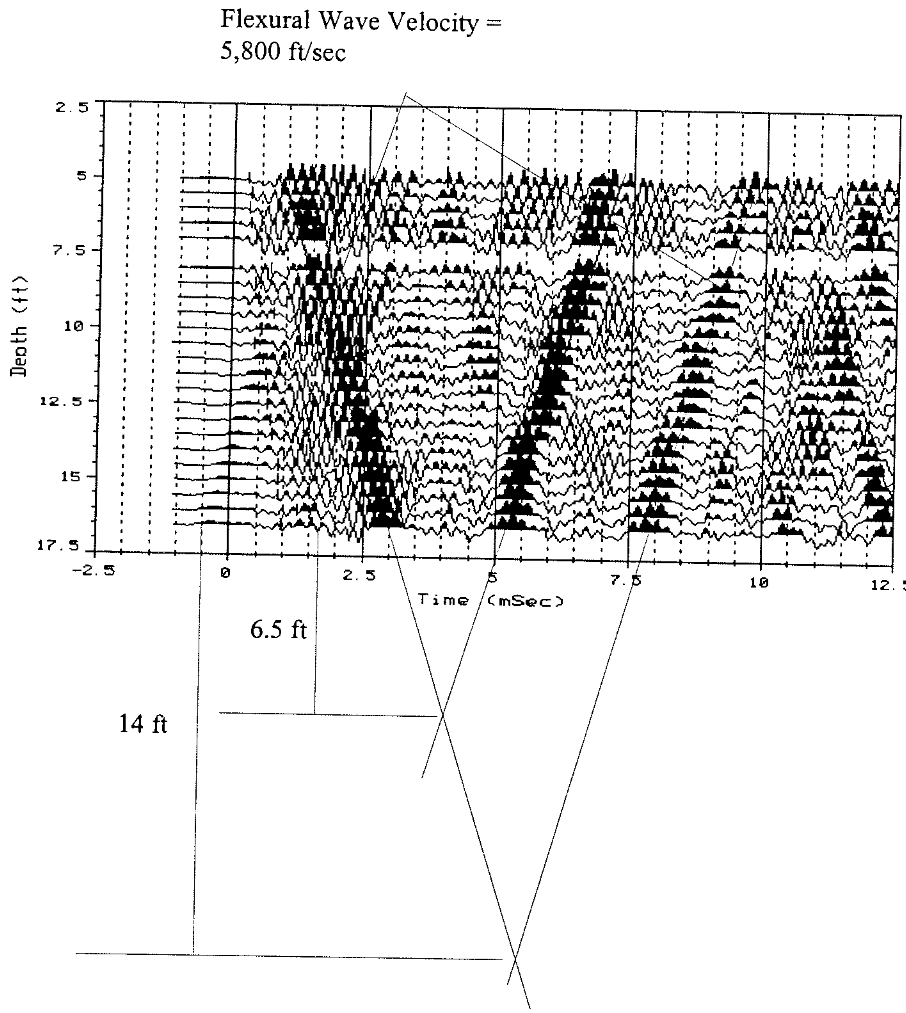


Figure 10- Ultraseismic Data from a 3-lb Vertical Hammer Hit and Radial Component Recording f-k Filtered to Enhance Upgoing Waves Bridge No. 5188, Minnesota Highway 58, Zumbrota, Minnesota.

reflector at a depth of 2 m (6.5 ft) below Receiver 24 at a depth of 5 m (16.5 ft) below top of concrete beam was identified. The flexural wave velocity of 1,770 m/sec (5,800 ft/sec) was determined from the slope of the initial downgoing and reflected upgoing waves. This reflection corresponds to a depth of 7 m (23 ft) from the top of the concrete beam. The actual depth of the foundation was equal to 9.4 m (31 ft). The difference in length between the predicted and actual depth of the foundation was equal to 2.4 m (8 ft), a difference of -25%.

By inspecting Fig. 10, there is another reflector at a depth of 4.3 m (14 ft) below Receiver 24 at a depth of 5 m (16.5 ft) below top of concrete beam. This reflection corresponds to a depth of 9.3 m (30.5 ft) from the top of the concrete beam. The actual depth of the foundation was equal to 9.4 m (31 ft). The difference in length between the predicted and actual depth of the foundation was equal to 0.1 m (0.5 ft), a difference of -2%. The first reflector at a depth of 7 m (23 ft) that was reported may have corresponded to a change in the material properties at this elevation. It is difficult in this case to make a final determination on the depth of the foundation based on the US results. If a borehole had been drilled at this site, PS tests would have given additional information.

2.4 SPECTRAL ANALYSIS OF SURFACE WAVES METHOD.

Research on the SASW method was initiated at the University of Texas at Austin By Dr. Kenneth H. Stokoe, II and his colleagues in the late 1970's with primarily funding from the Texas DOT (15,16). The SASW method has unique capabilities to nondestructively determine layer thicknesses and velocity (stiffness) versus depth for soft over stiff over soft layers that other methods such as Seismic Refraction are not capable of doing unless velocity increases with depth. One

advantage of the SASW method for investigation unknown foundation depths of bridges is that measurements are performed using a source and two receivers which can be placed on top of a horizontal surface such as the exposed surface of an abutment. In the last 16 years, active research has been conducted at the UT Austin and other universities to improve the theoretical and practical aspects of the method (17,18,19). The method has been successfully applied for the determination of shear wave velocity profiles for soils (20) and for pavement systems (21). Lately, the SASW testing has been adopted for offshore/underwater use (22). A schematic of the SASW method is shown in Fig. 11.

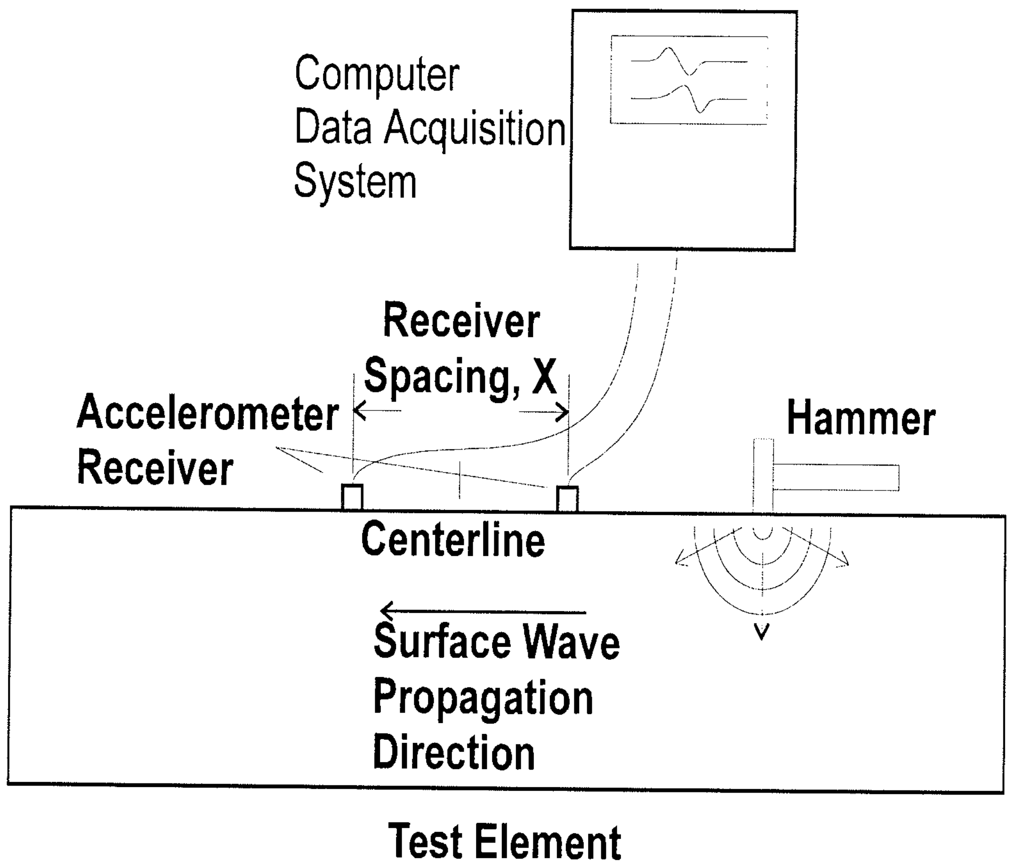


Figure 11- Schematic of the Spectral Analysis of Surface Waves Method.

When SASW measurements are performed, a source and two receivers are placed in-line on the surface such that the distance from the source to the first receiver (D) is equal to the distance between the two receivers. Testing is performed by impacting the surface and recording the passage of predominant Rayleigh (surface) wave energy past the two receivers. A series of receiver spacing is used, and testing is performed in forward and reverse directions at each receiver spacing.

A dynamic signal analyzer is used to capture and process the receiver outputs denoted as x(t) and y(t) for receivers 1 and 2, respectively. The time domain outputs x(t) and y(t) are then transformed to the frequency domain (X (f) and Y(f)) through the employment of a Fast Fourier Transform. X(f) and Y(f) are used to calculate the cross power spectrum between the two receivers denoted by G_{XY} ($G_{XY} = X(f)^* \cdot Y(f)$; * denotes the complex conjugate). The surface wave velocity and wavelength associated with each frequency, $V_R(f)$ and $\lambda_R(f)$, respectively, are then calculated from the following equations:

$$t(f) = \phi_{XY}(f) / (360 \cdot f)$$

$$V_R(f) = D / t(f)$$

$$\lambda_R(f) = V_R(f) / f$$

where t(f) = time delay between receivers as a function of frequency f,

$\phi_{XY}(f)$ = phase shift of the cross power spectrum in degrees, and

D = distance between receivers.

A plot of the surface wave velocity versus wavelength is called a dispersion curve. Once the dispersion curve is determined, one can obtain the shear wave velocity profile of the structure or soil being tested. This is accomplished through a process called forward modeling. Dr. Jose Roesset and his students at UT Austin have developed computer programs for the forward modeling procedure (23, 24). When a good match is obtained between the experimental and theoretical dispersion curves, the assumed profile is considered to be a good representation of the actual profile. Accuracies of velocity profiles and layer thicknesses vary with the variability of the pavement/soil/bedrock or other layers being tested, but theoretically modeled values are typically accurate to within 10 to 15 % of actual values.

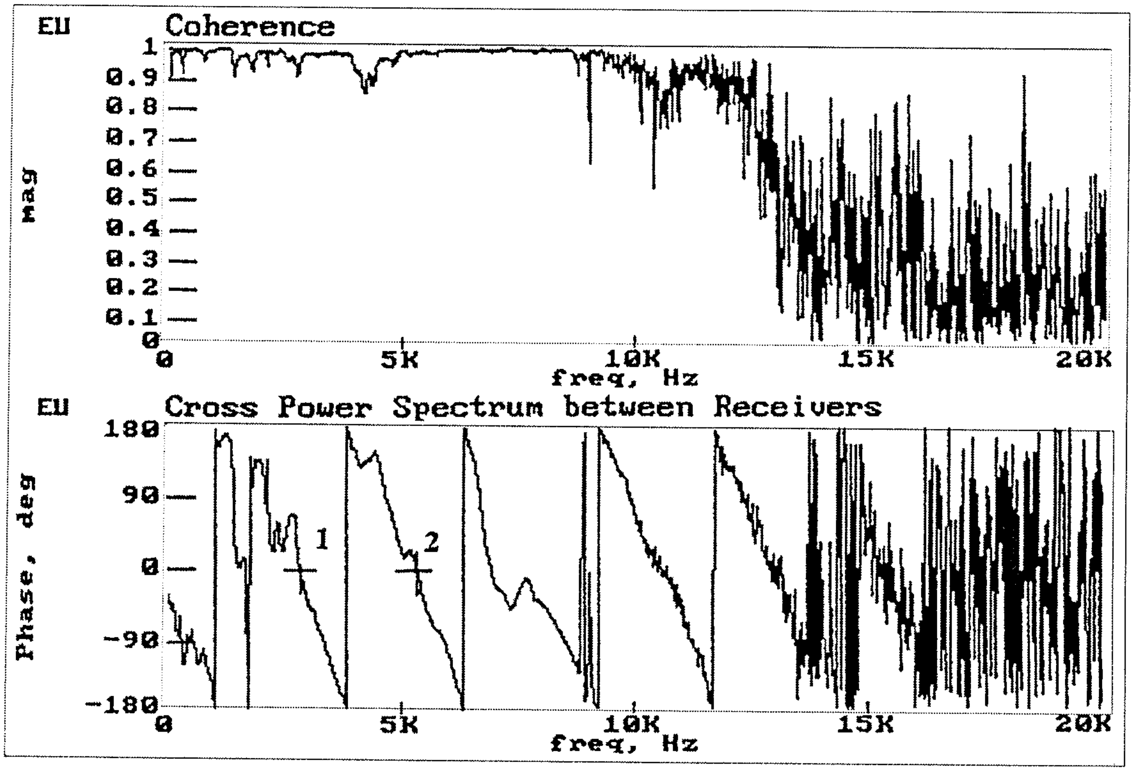
Spectral Analysis of Surface Waves Example Results. Initially, the Spectral Analysis of Surface Waves (SASW) method was primarily viewed as a supporting method to determine the shear wave velocity profile versus depth from the ground surface without drilling a boring. Subsequent independent consulting experience by the authors for determining unknown foundation depths has shown that SASW tests can be quite useful in determining the depths of more massive abutments, piers and footings provided the substructure geometry allows for proper access. Access for the SASW test in terms of unknown bridge foundations means that the foundation is more massive (a wall, abutment, pier, or exposed footing/pilecap) and has an exposed fairly flat ledge or top surface on which impacts are applied and a pair of receivers placed.

The SASW test was successfully used to predict the depths of foundations for 2 of the 5 Connecticut DOT bridges (Olson Engineering's case study presented in Phase I report submitted in

August, 1995 (7)). Since 2 out of 5 bridges could be tested with the method, and the results were very good, the SASW method is now considered to have real applications to some of the bridges with massive abutment/pier unknown foundations.

A typical phase plot of the cross power spectrum and the coherence function is shown in Fig. 12 for a receiver spacing of 3 ft used in SASW measurements at the Hamden Bridge in Connecticut. Spacings of 3, 6 and 12 ft between receivers were used to obtain the SASW phase data. A measured dispersion curve (velocity versus wavelength from the phase data) for the Hamden bridge is presented in Fig. 13. Examination of Fig. 13 indicates an average velocity of 7,700 ft/sec for wavelengths of 1 to 9 ft which is indicative of concrete velocity. For wavelengths greater than 9 ft, there is a sharp velocity drop until for wavelengths of 10 ft and longer there is a constant velocity layer of approximately 4,000 ft/sec which is indicative of a medium hard bedrock. There is no need to model this dispersion curve since the concrete layer has a relatively uniform velocity, then wavelengths are equal to depth, and the depth of the abutment is inferred to be approximately 9 ft.

One limitation of the SASW method could occur if a bridge substructure is much deeper than its width available for testing. In this case, the width of the substructure may be too small to generate the required longer wavelengths necessary to reach the bottom of the foundation and penetrate into the supporting strata.



At Point 1:
Phase Shift = 360 degrees (1 cycle)
Frequency = f = 2,835 Hz
Wavelength = λ = distance between receivers = 3 ft
Surface Wave Velocity = f x λ = 8,500 ft/sec

At Point 2:
Phase Shift = 360 degrees (2 cycles)
Frequency = f = 5,298 Hz
Wavelength = λ = ½ x distance between receivers = 1.5 ft
Surface Wave Velocity = f x λ = 7,950 ft/sec

Figure 12- Coherence Function and Phase Shift of the Cross Power Spectrum between the Two Receivers Determined from SASW Measurements, R1-R2 = 3 ft, West Abutment, Hamden Bridge, Connecticut.

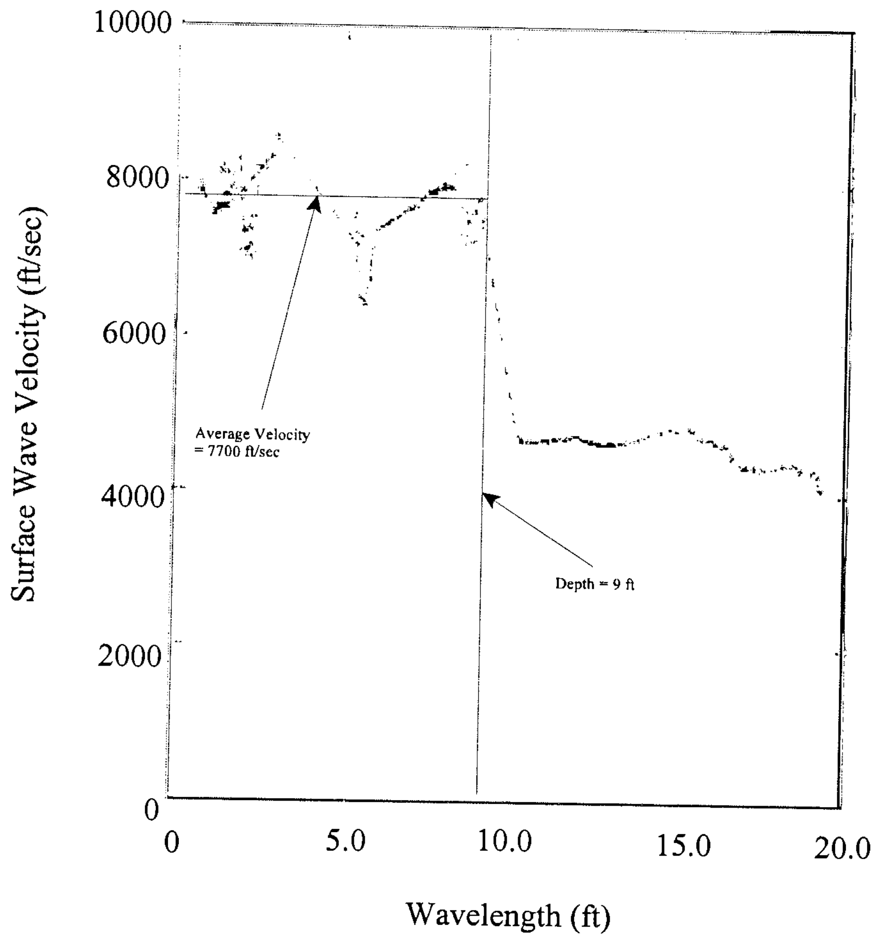


Figure 13- SASW Dispersion Curve for the West Abutment, Connecticut.

2.5 PARALLEL SEISMIC METHOD.

The borehole Parallel Seismic (PS) method was researched and developed specifically to determine the depths of unknown foundations by the CEBTP research organization headquartered in Paris, France (25). The Parallel Seismic method is based on the principle that an impact to the exposed structure generates wave energy that travels down the foundation and can be tracked by depth with receivers in a nearby parallel boring to determine when the signal weakens, and slows down which indicates the receiver has gone beyond the bottom of the foundation, and the depth is determined. The method has been previously used in a number of consulting projects to determine the depths of unknown foundations below buildings and bridges (25,26). The method has been used with good success for determining the unknown depths of rod-like deep foundations, like driven piles and drilled shafts, but not as much was known about its capabilities for the full range of substructure types that make up the unknown bridge foundation population at the start of the research.

A schematic of the Parallel Seismic method is shown in Fig. 14a (see Fig. A.5a for an actual PS test on a steel pile). Typical Parallel Seismic (PS) test equipment includes an impulse hammer, hydrophone or geophone receiver, and dynamic signal analyzer or oscilloscope. A portable PC - based digital oscilloscope was used to record the Parallel Seismic data in this study.

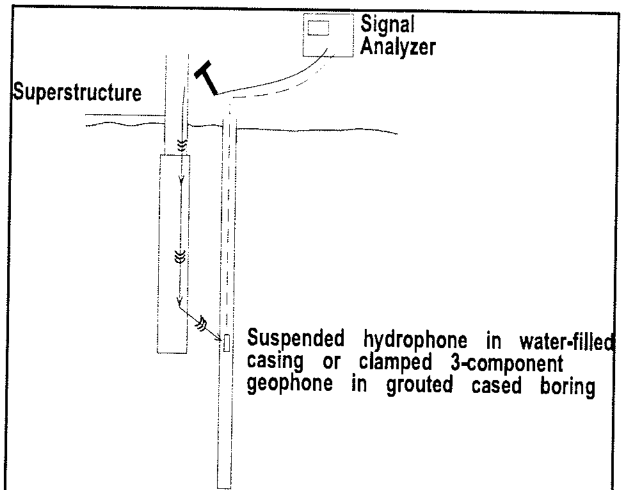


Figure 14a- Schematic of the Parallel Seismic Method.

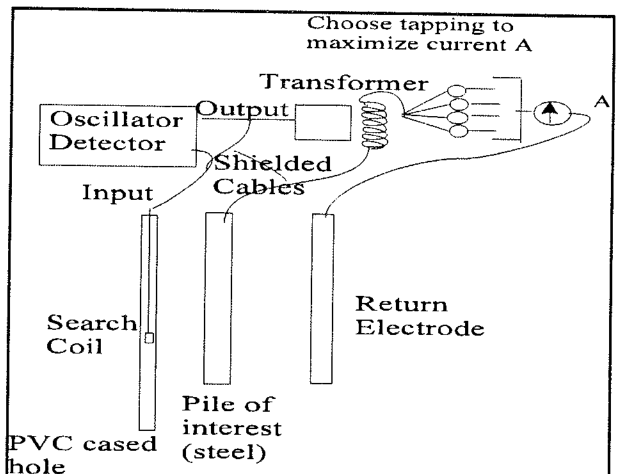


Figure 14b- Schematic of the Induction Field Method.

The Parallel Seismic (PS) method involves impacting the side or top of exposed bridge substructure with a 3 lb or 12 lb (preferred for larger foundations) hammer to generate wave energy which travels down the foundation and is refracted to the adjacent soil. The refracted wave arrival is tracked by a hydrophone receiver suspended in a water-filled cased borehole in the conventional approach to the test. A hydrophone receiver is sensitive to pressure changes in the water-filled tube, but it is also subject to contaminating tube wave energy. PS research was performed for the first time during this project on the use of clamped three-component geophones in empty PVC cased and grouted borings to better examine the wave propagation behavior with reduced tube wave energy noise. The boring is drilled typically within 0.9 to 1.5 m (3 to 5 feet) of the foundation edge and should extend at least 3 m (10 feet) (4.5 m (15 ft) is preferred) deeper than the anticipated and/or minimum required foundation depth for the depth to be determined.

It is preferred that all borings for the Parallel Seismic (PS) test to be cased (either with plastic or steel) with an inside casing diameter of 50 mm (2 inches) or larger. Open hole PS testing is also acceptable but this requires mechanical clamping geophones and the user runs the risk of losing the tool due to soil caving. The casing and boring must be filled with water before testing if hydrophones are to be used for compression wave arrivals. The casing should be dry if geophones are to be used for shear and compression wave arrivals. It is also important for the PS test that borings be drilled with as little deviation from vertical as possible. The void between the soil and casing should ideally be cement-grouted for obtaining the best PS results with geophones. Grouting should be done in compliance with the ASTM D4428/D4428M standard for Crosshole Seismic Testing. Basically, the grout mixture needs to be non-shrinking with a density and strength similar

to the material that is removed during drilling.

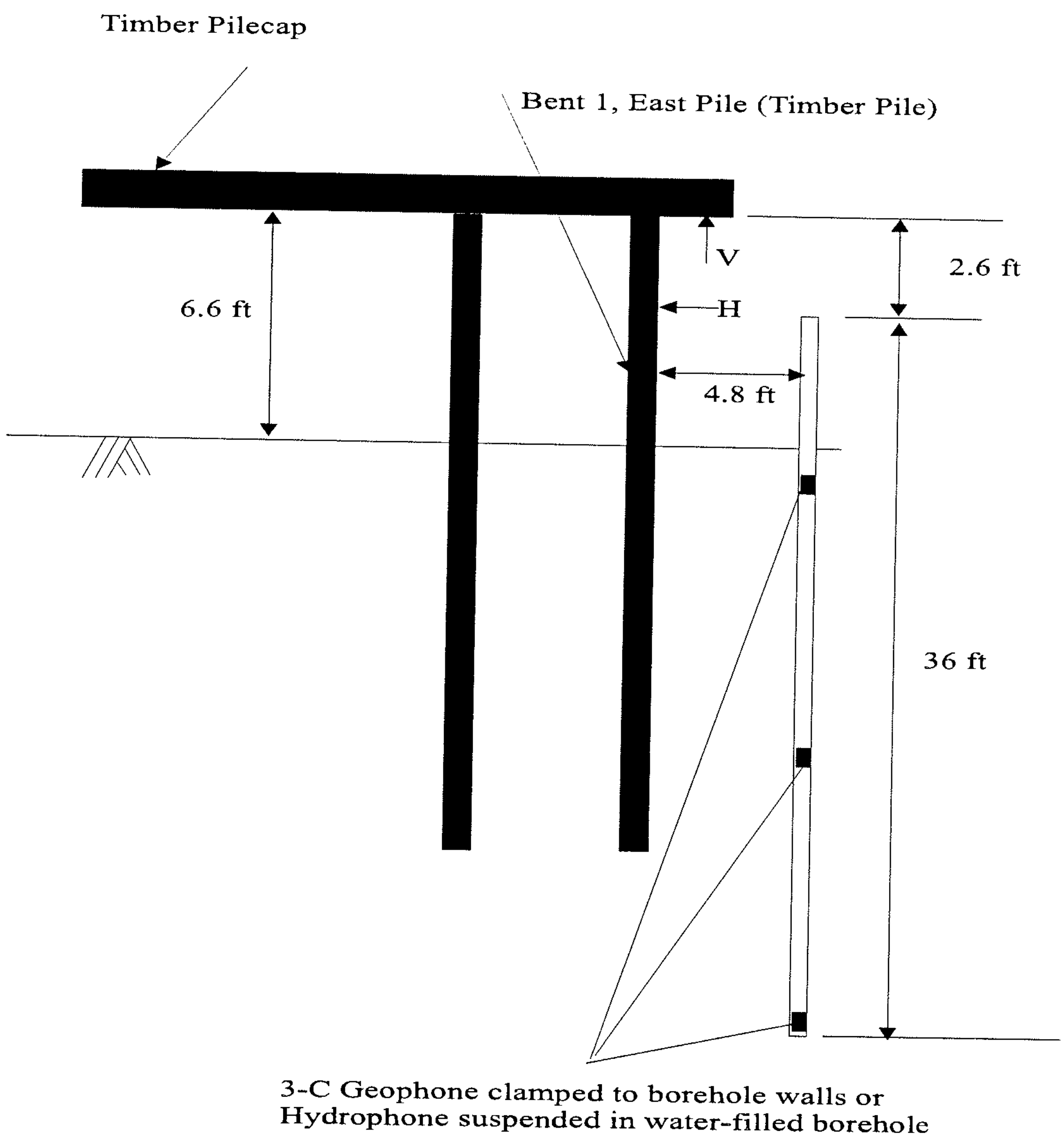
For fully saturated sites below the water-table, the use of geophones and grouting is not as critical; and we have performed number of PS tests with hydrophones in slotted plastic casing and no grouting at these sites. This is because the water couples the compressional wave energy through the soils to the hydrophones in the water-filled boring and casing. In partially saturated soils, we have also poured loose sand to fill the void between the soil and the casing but this practice is obviously not as satisfactory for the use of geophones as grouting with a cement-bentonite mixture.

The PS test typically involves lowering the receiver (hydrophone or geophone) to the bottom of the borehole, impacting the substructure as close to the ground surface as practical, recording the receiver response and then raising the receiver to the next test depth, typically in 0.3 to 1 m (1 to 3 ft) increments depending on the desired accuracy. This test sequence is repeated until the top of the cased boring is reached. Downward, vertical impacts are ideal to generate compressional waves, but angled to horizontal impacts to substructures work also. A horizontal impact is used to generates flexural and shear waves, and a second opposing horizontal impact may be used to cause a reversal of polarity in the soils shear wave arrival at the receiver to enhance its identification. Geophysical seismic processing was used to display and analyze the data in this study.

Parallel Seismic Example Results. Figure 15 shows the source receiver layout for PS tests performed at Wilson County Bridge No. 5, North Carolina. Both hydrophone and geophone receivers at depth intervals of 0.3 m (1 ft) were used in the data collection with a 5.4 kg (12-lb)

horizontal and vertical hammer hits.

Figure 16 shows the vertical component recording of the geophone due to an upward, vertical hammer hit to the bottom of the timber beam. Note that the depth shown in Fig. 16 is the depth below the top of the borehole. By inspecting Fig. 16, one can easily notice the drop in amplitude below a depth of 7.3 m (24 ft), which corresponds to the tip of the pile in our interpretation. It is preferred to measure the depth of a foundation with respect to a fixed reference point. In this case, the top of the borehole was 0.8 m (2.6 ft) below the top of pile. Thus, the total length of the pile is equal to 8.1 m (26.6 ft) with a reference point as the bottom of the timber beam. The actual length of the pile was equal to 7.7 m (25.4 ft). The difference in length between the predicted and actual length of the pile was equal to 0.4 m (1.2 ft), an excellent agreement of +5% difference.



V = Vertical Hammer Hit Location
H = Horizontal Hammer Hit Location
33 geophone locations were used at intervals of 1 ft (from 4 to 36 ft below top of borehole)
32 hydrophone locations were used at intervals of 1 ft (from 5 to 36 ft below top of borehole)

Figure 15- Source/Receiver Layout for Parallel Seismic Tests Performed at Bent 1, East Pile, Wilson County Bridge # 5, North Carolina.

Comments:
Depth shown in Figure is depth below top of borehole
Drop in amplitude below a depth of 24 ft
Bottom depth = 24 ft (reference is top of borehole)
Top of borehole is 2.6 ft below top of pile
Pile length = 24+2.6 = 26.6 ft (reference is top of pile)

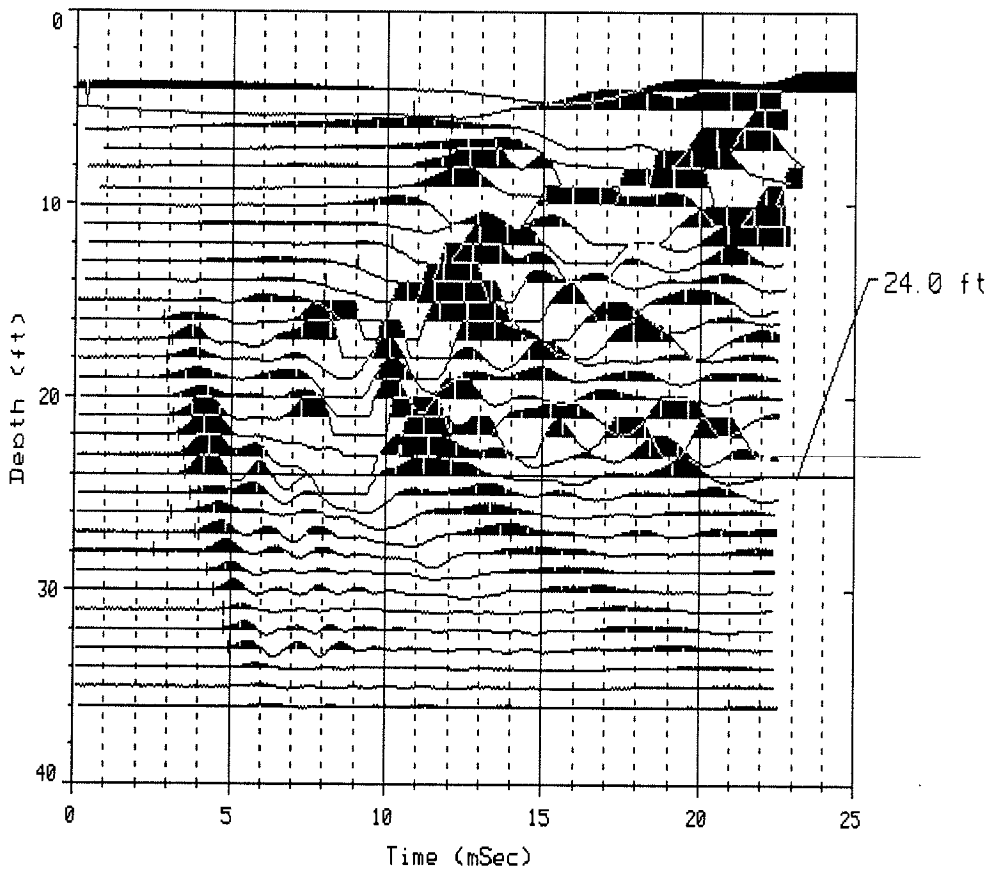


Figure 16- Parallel Seismic Data from a 12-lb Vertical Hammer Hit and Vertical Component Recording of Geophone Wilson County Bridge # 5, Bent 1, East Timber Pile, North Carolina (True Amplitude Plot).

Figure 17 shows the same results presented in Fig. 16 but with traces adjusted to represent the maximum amplitude of each trace. The tip of the pile acted as a point energy emitter, from which the apparent shear wave velocity of the soil below the tip of the pile at a depth of 7.3 m (24 ft) was calculated to be equal to 350 m/sec (1,150 ft/sec).

Figure 18 shows the PS results using a hydrophone receiver. The results are plotted using the maximum amplitude of each trace. Note the well defined velocity of 1,490 m/sec (4,900 ft/sec) below a depth of 7.3 m (24 ft) measured from the top of the borehole. This velocity corresponds to the compression wave velocity of water. Thus, the point at which the velocity starts to correspond to the velocity of water is interpreted to correspond to the tip of the pile at a depth of 7.3 m (24 ft), a result consistent with the geophone PS test results. The velocity above a depth of 24 ft was equal to 3,700 m/sec (12,000 ft/sec), which is the velocity of the timber pile. It is difficult in the hydrophone data to determine the shear wave velocity of soil below the tip of the pile due to the dominant tube wave energy.

2.6 INDUCTION FIELD METHOD.

In the Induction Field (IF) method, an AC current flow is impressed into a steel pile (or the rebar in a reinforced concrete pile) from which the current couples into the subsurface and finally to a return electrode (see Fig. 14b for a schematic of the IF method). The return electrode can be another pile, or it can be a pipe or piece of rebar driven into the ground. A receiver coil which is suspended in a nearby boring is then used as a sensor of the magnetic field induced by the alternating current flow between the pile and return electrode. The basic limitation of this method is that the

Comments:
Depth shown in Figure is depth below top of borehole
Bottom depth = 24 ft (reference is top of borehole)
Top of borehole is 2.6 ft below top of pile
Pile length = 24+2.6 = 26.6 ft (reference is top of pile)

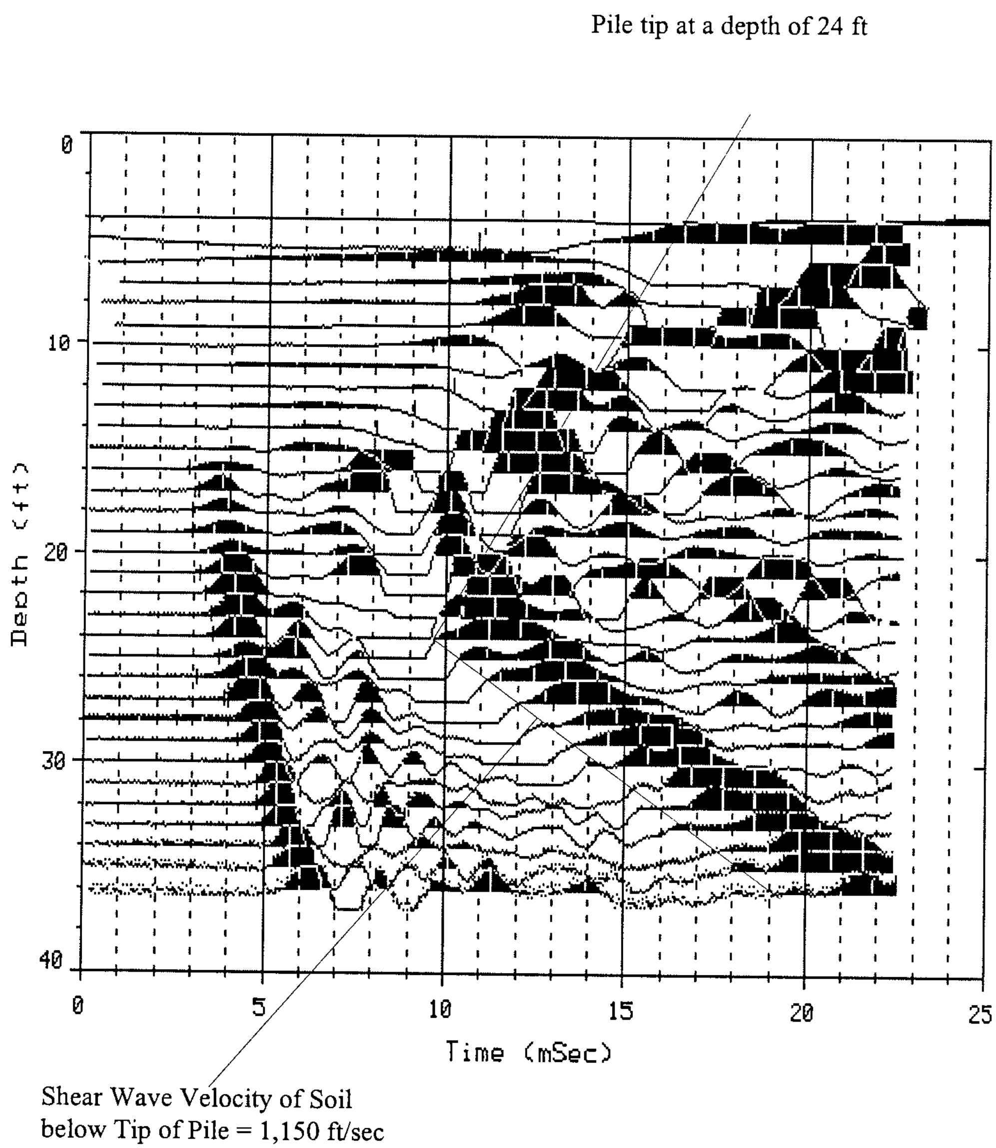


Figure 17- Parallel Seismic Data from a 12-lb Vertical Hammer Hit and Vertical Component Recording of Geophone Wilson County Bridge # 5, Bent 1, East Pile, North Carolina (Amplitude Plot adjusted to reflect the maximum of each trace).

Comments:
Depth shown in Figure is depth below top of borehole
Bottom depth = 24 ft (reference is top of borehole)
Top of borehole is 2.6 ft below top of pile
Pile length = 24+2.6 = 26.6 ft (reference is top of pile)

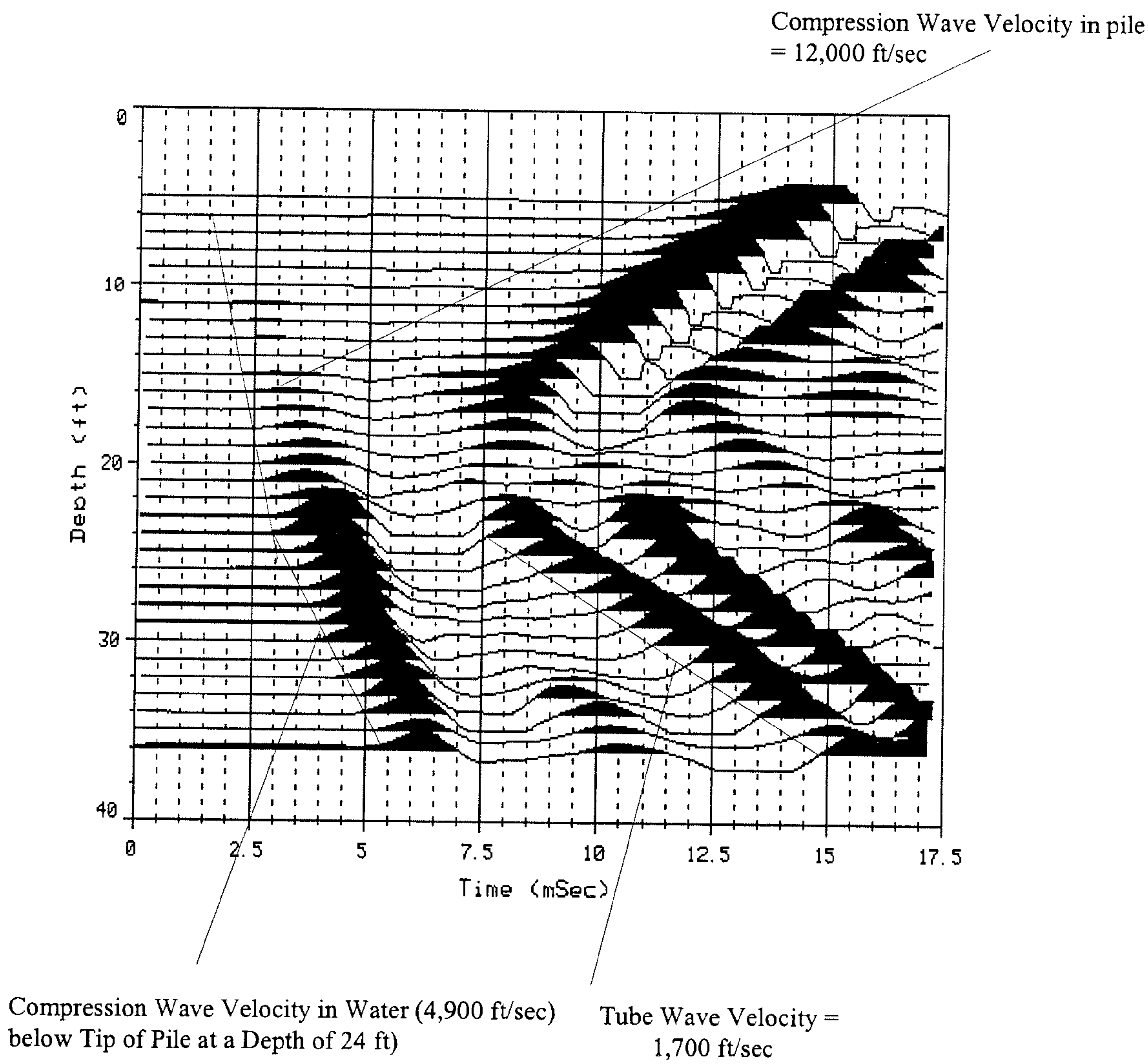


Figure 18- Parallel Seismic Data from a 12-lb Vertical Hammer Hit and Hydrophone Recording
Wilson County Bridge # 5, Bent 1, East Pile, North Carolina
(Amplitude Plot adjusted to reflect the maximum of each trace).

foundation substructure must contain electrically continuous steel for its entire length, and the steel used must be accessible at the top to allow the electrical connection. Steel pile and reinforced concrete pile depths can be obtained from this type of survey. However, it is doubtful that the distribution of piles can be discerned. In addition, piles which are not electrically connected to a pile cap will not be seen unless some other electrical connection can be made to the pile steel directly.

This method was developed in New Zealand for foundation length determination of reinforced concrete piles and steel piles (3,4). The method is the electromagnetic analog to the Parallel Seismic (PS) test. A current is passed down the reinforcement or steel of a deep bridge foundation to a return electrode which can be a metal object with an area of about 1 m^2 (11 ft^2) or an adjacent deep foundation having no direct electrical contact with the test foundation. A magnetic field is generated alongside the pile, by the current flowing vertically down the pile steel, that is measured using a search coil in a boring. This search coil is then connected to a detecting instrument which shows the relative field strength of the magnetic field. As the depth of the search coil increases, the induced voltage decreases linearly down the length of the pile, provided there is a constant current leakage down the pile.

By plotting the magnitude of the induced voltage versus the depth of the search coil, an indication of the length of the pile is provided. The presence of a ground water-table will somewhat effect the results of this measurement, but as long as the foundation bottom is not at the exact depth of the water-table, this should not be a big factor. Once the search coil is below the bottom of the foundation, the measured induced voltage tends to stabilize at a low value because of the residual

conductivity of the soil or bedrock.

Capabilities. As seen from the discussion above, the Induction Field method is a proven technology for the determination of the depth of steel piles and reinforced concrete piles. One important consideration with respect to unknown bridge foundations is that while the method could detect the presence of piles under a buried footing, it requires that the piles be electrically connected through the footing to the bridge substructure or some other accessible element to allow connection of the current source. Another important consideration is that the method requires a PVC plastic cased boring (metal casing absorbs the magnetic field). However, the method could be performed in conjunction with the Parallel Seismic method, which also requires a borehole.

Limitations. Interpretation of the data from the Induction Field method is complicated by the existence of ferrous or other conductive materials in the bridge structure, and can be further complicated by the presence of conductors (such as cables or pipes) in the ground around the pile. Also, these tests can only work for reinforced concrete or steel piles that have electrically connected rebar which is accessible at the ground surface. The boring should be drilled within 1 m (3 ft) or less of the foundation and extend about 3 m (10 ft) below the bottom of the foundation.

Induction Field Example Results. Figure 19 shows the source/receiver layout for IF tests. Figure 20 shows the Induction Field results from tests performed on one of the steel H-piles. The IF measurements were performed at intervals of 0.3 m (1 ft) using a 3-axis magnetometer to measure the strength of the magnetic field generated around the steel pile by an AC current flow through the

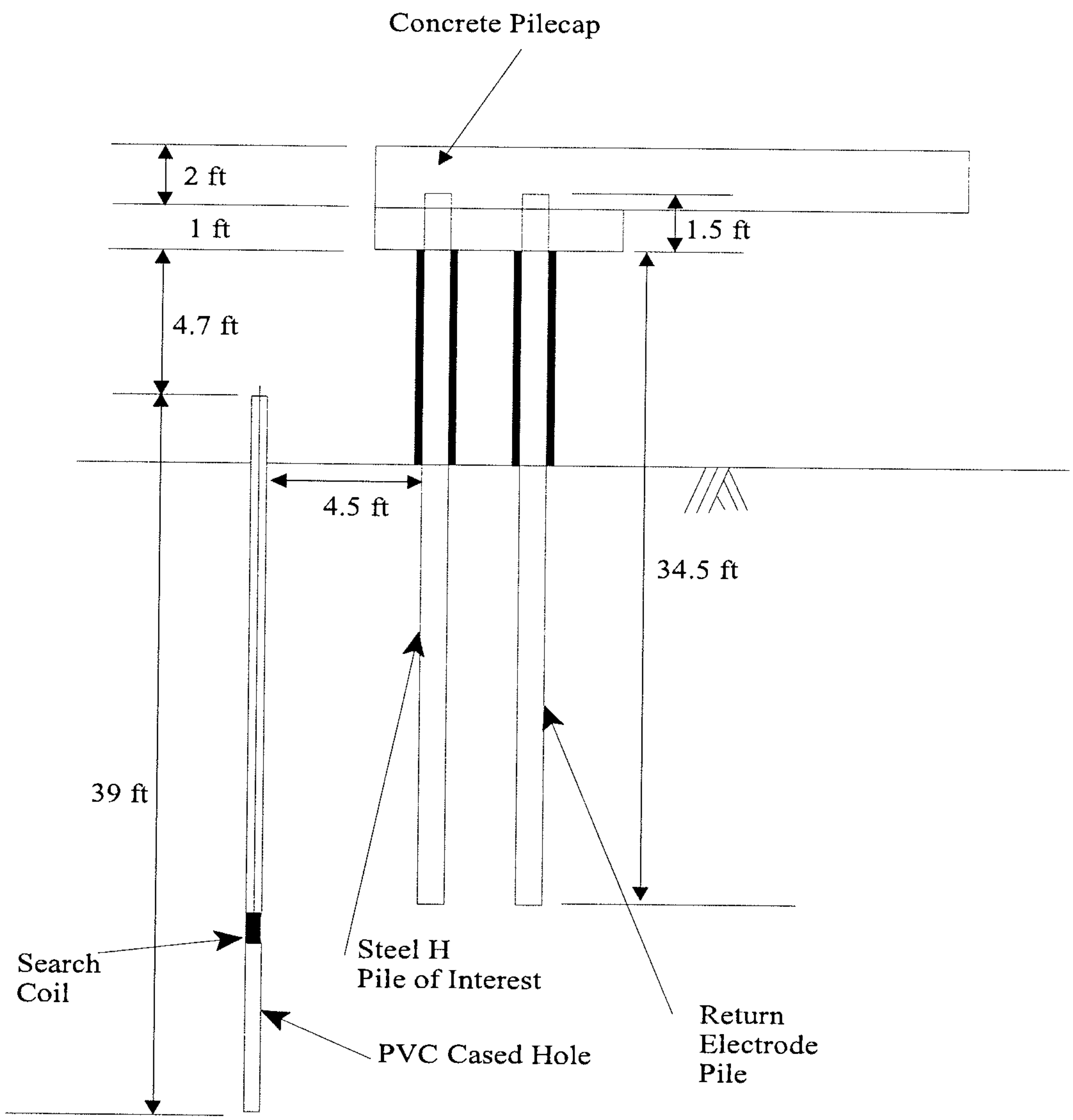


Figure 19- Source/Receiver Layout for Induction Field Tests Performed at Bridge on US 287, Structure # C-16-C, over Little Thompson River, Near Longmont, Colorado.

Comments:
Depth shown in Figure is depth below top of borehole
Drop in Amplitude below a depth of 27 ft
Bottom depth = 27 ft (reference is top of borehole)
Top of borehole is 4.7 ft below top of pile
Pile length = 27+4.7 = 31.7 ft (reference is top of pile)

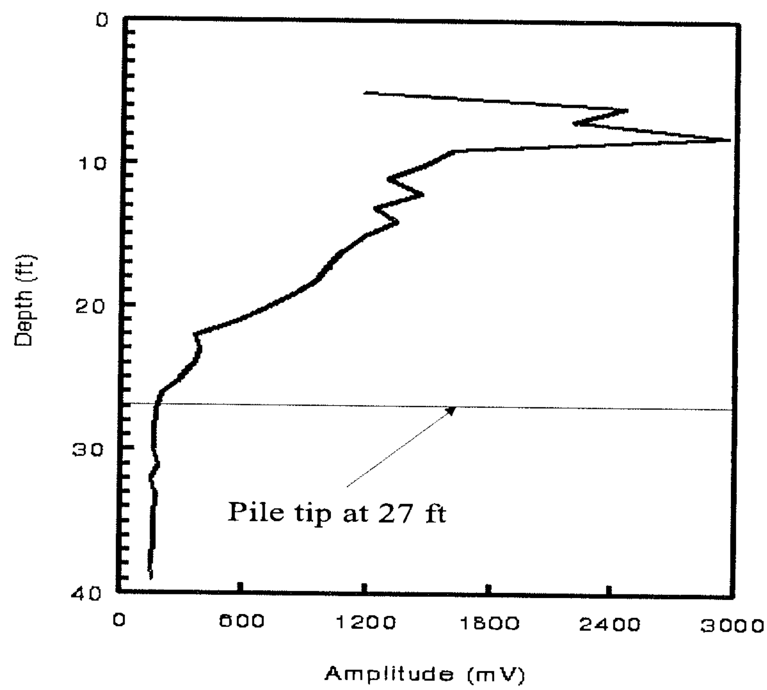


Figure 20- Induction Field Test Results
Bridge on US 287, Structure # C-16-C, over Little Thompson River, Near Longmont, Colorado.

pile. The current was generated by a function generator operating at a frequency of 5 kHz, and then and through the soil to a reference pile spaced 4-5 m (13-16 ft) away. The depth shown in Fig. 20 is the depth from the top of the borehole. The predicted depth was equal to 8.23 m (27 ft). This is the depth at which there was a drop in amplitude below the tip of the pile. In this case, the top of the borehole was 1.43 m (4.7 ft) below the top of the pile. Thus, the length of the pile is equal to 9.66 m (31.7 ft) with a reference point as the top of the pile. The actual length of the pile was equal to 10.5 m (34.5 ft). The difference in length between the predicted and actual length of the pile was equal to 0.84 m (2.8 ft), a good agreement with a difference of -8%.

Other NDT methods that were used in this research on a limited basis include the Dynamic Foundation Response, Borehole Sonic and Borehole Radar methods. Brief descriptions of these methods follow.

2.7 DYNAMIC FOUNDATION RESPONSE METHOD.

None of the other surface methods discussed to this point, Sonic Echo/Impulse Response, Bending Wave, Ultraseismic, and Spectral Analysis of Surface Waves, were able to detect the presence of piles underlying pilecaps. The Dynamic Foundation Response (DFR) method was proposed by Drs. Stokoe and Roesset of the University of Texas at Austin in an attempt to address this problem and differentiate between shallow, footing foundations, and shallow pilecaps (similar to footings) supported on piles for more massive bridge substructures. <u>The method is based on the principle that all other things being the same, then the vibration response of a given bridge</u>

substructure will exhibit lower resonant frequency responses when excited for a shallow foundation versus the comparatively higher resonant frequency response of a deep foundation system (see Fig. A.2a for an actual DFR test).

The method is unproven for this use in bridges, but is based on the dynamic analysis theory for vibration design of foundations (soil dynamics) and geotechnical analyses of foundations subjected to earthquake loading based on the theoretical work presented by Novak (27). Novak analyzed the problem of a simple shallow footing foundation with and without piles for vertical and horizontal modes of vibrations on the ground surface and embedded. Novak found that the vibration response of the pile foundations differ from the response of shallow foundations. The pile foundation is more rigid and exhibits higher amplitudes of vibrations than the shallow foundations. The pile supported foundations also have a much higher natural frequency than the footing. The initial NCHRP research lead to a related research project for the FHWA entitled, "Dynamic Bridge Substructure Evaluation and Monitoring System" by Olson and Liu (31).

2.8 BOREHOLE SONIC METHOD.

The Borehole Sonic (BHS) method was proposed for research as a promising, but unproven approach for application in the determination of unknown bridge foundation depths, and perhaps even foundation geometry. Like Sonar, the method is based on the principle of generating compression wave energy of sufficient energy and frequency so that such waves will reflect back from the much stiffer bridge foundation substructure to be sensed by receivers in the BHS tool. The complication is that soil is a highly attenuative media for stress wave travel, even when saturated,

which limits the bandwidth of the received signal. The method involves lowering a source and a receiver unit in the same borehole and measuring the reflection echoes from the sides of the bridge foundation substructure using near horizontal raypaths.

Two potential systems for performing Borehole Sonic tests were evaluated in field tests of the concrete caisson and pile foundations of the Old Bastrop bridge and the drilled shaft foundation of the New Bastrop Bridge. The first system evaluated was a mocked-up prototype made up of University of Texas at Austin Geotechnical Engineering Center source and receiver components for Crosshole Seismic Testing. The source used a solenoid impactor to strike a casing wall to generate compression and shear wave energy. The receiver was a 3-component geophone that was also used in the Parallel Seismic tests. The second system evaluated was a commercial full-waveform, singlehole sonic logging tool known as the PS Suspension logging tool by the OYO Corporation of Japan. We understand a new Borehole Sonic tool is made by a firm in Finland, but it was not used in this research.

2.9 BOREHOLE RADAR METHOD.

The Borehole Radar method was unproven in its application to the determination of unknown bridge foundations. The method is analogous to the Borehole Sonic Method in that it depends on reflections of waves from materials of differing properties as well, only the difference in those properties is not stiffness, but dielectric constants that cause reflections (28). Ground Penetrating Radar (GPR) employs radio waves in the frequency range of 1 to 2,000 MegaHertz (MHz) to map structures in the ground and man-made structures. Surface radar studies have been done to

investigate conditions of scour around bridge piers (29), but surface radar has limited application to unknown foundation depth determination because the reflecting targets tend to be largely vertical. In addition, adjacent substructure can complicate the results and make resolution of even shallow footings difficult. Borehole Radar is unproven in its application to the determination of unknown foundation conditions, but it has the promise of being able to obtain direct reflections from the vertical foundation substructure. Borehole Radar has been used in Europe for mining studies with a directional tool in both singlehole and crosshole studies, and newer equipment is now commercially available.

CHAPTER 3

SUMMARY OF NCHRP 21-5 PHASE I RESEARCH

DETERMINATION OF UNKNOWN SUBSURFACE BRIDGE FOUNDATIONS

Prior to this research, only four NDT methods had been previously used to determine unknown foundation depths. At the conclusion of the research, 3 of the 4 existing methods had been advanced (Sonic Echo/Impulse Response, Bending Wave, and Parallel Seismic - no research on Induction Field), 3 methods extended (Spectral Analysis of Surface Waves, Parallel Seismic with geophones, and Borehole Radar), and 1 new method developed (Ultraseismic Vertical and Horizontal Profiling) for unknown foundation depth determination. Significant advances were also made in the use of theoretical modeling of compressional and flexural wave travel in substructures, which is useful for several of the NDT methods. This chapter is focused on the current status of the applicable NDT methods in terms of their capabilities, limitations, applications, and implementation. The results of this research were summarized in an NCHRP results digest in 1996 (30).

Significant advancements have been made in our understanding and ability to use NDT methods to accurately determine unknown depths of many more types of bridge foundations than was previously available. The research findings are examined from an overall perspective in this chapter in order to comparatively evaluate the strengths and weaknesses of the various NDT methods for determining unknown foundation conditions of depth, type, geometry and materials for different substructure types. Detailed presentations of the NDT results of Phase I research can be found in the final report submitted in August, 1995 (7).

A summary interpretation of the findings for the nondestructive testing (NDT) methods is presented by substructure in Section 3.1. A review of the capabilities and limitations of the NDT methods is presented in Section 3.2. Guidelines to the performance of unknown foundation investigations are given in Section 3.3 for the NDT methods with immediate applications at this time. Implementation requirements in terms of equipment costs, training, and operational costs are discussed in Section 3.4.

3.1 INTERPRETATION OF NDT RESULTS BY SUBSTRUCTURE.

A comparison of the actual plan or as-built foundation element depths versus the NDT-based depth predictions is presented in Table V. The results are arranged by NDT method for the seven Phase I study bridges. Brief descriptions of the bridges are presented in Appendix A, which shows photographs of the tested bridges along with a summary of the foundation type. NDT results from the 7 bridges are further discussed below.

North Pier, Golden Bridge, Colorado. Sonic Echo/Impulse Response (SE/IR) tests from the top of the beam over the east column were unsuccessful due to the trapping of the wave energy in the large beam. However, Ultraseismic (US) Vertical Profiling tests on the column with compressional and flexural waves predicted the footing bottom to be at 12.8 m (42 ft) and 13.1 m (42.9 ft) below the top of the beam, respectively (4.3 m (14.0 ft) and 4.5 m (14.9 ft) below grade). This compares very well with the plan dimension of 13 m (42.8 ft) from the top to the bottom (footing depth of 4.5 m (14.8 ft)). The US Vertical Profiling (VP) tests are geophysically processed

Table V- Known Foundation Depths vs. NDT Predicted Depths by Substructure.

Substructure & Bridge	Substructure Description	Plan Depth (m)	NDT Foundation Element Depths (below-grade in meters)							
			SE	IR	USC	USF	BW	PS	BHR	BHS
Concrete North Pier Golden, CO	Columns on footings with breast wall	4.5	inc	inc	4.3	4.5	--	nb	nb	nb n/a
Pier 4 Coors, CO	Concrete columns to pilecap	1.5	inc	inc	1.5	1.0	--	inc	inc	n/a
	Columns to pilecap to steel piles	8.8	n/a	n/a	n/a	n/a	n/a	8.8_h 8.2_g	inc	n/a
Pier 2 Coors, CO	Concrete columns on footings	1.4	inc	inc	inc	1.3	--	--	inc	n/a
NE Wing/ Middle Pier Franktown, Colorado	Exposed timber piles in wingwall	6.4	6.9	6.4	inc	inc	6.2	nb	nb	nb n/a
	Cap beam on timber piles	7.6	6.3	6.2	6.1	inc	--	nb	nb	nb n/a
West Abutment/ West Pier Weld, CO	Stubwall on H-piles (top @ 0 ft)	1.8	2.0	2.0	--	--	--	nb	nb	nb
	Concrete wall on pilecap H-pile	3.3	--	--	inc	3.0	--	nb	nb	nb n/a
Steel Pile Substructure Bent 4 Alabama	East Battered Steel BP pile	11.9	inc	inc	--	--	--	9.1_g	9.4	n/a
	Center Vertical Steel BP Pile	11.9	inc	inc	10.4- 10.7?	10.7?	--	10.5_h 9.6_g	8.6	n/a
Concrete Caisson Old Bastrop, Texas	N. Column top to Bell top @ 0 ft	0	0.6	inc	--	--	--	n/a	n/a	n/a
	bottom of bell shaped section	5.5	--	--	inc	5.7?	n/a	inc	inc	inc
	bottom of rectangular footing	10.4	--	--	inc	11.4?	n/a	11.4_h 10.5_g	inc	10.1- 10.0?
Piles Old Bastrop, Texas	Column on exposed pilecap on concrete piles	10.1	inc	inc	n/a	n/a	n/a	10.1_h 9.8_g	inc	inc
Drilled Shaft New Bastrop, TX	Concrete Beams on columns on shafts	11.6	11.6	inc	inc	11.6?	n/a	11.7_h 10.8_g	inc	inc

Legend for Table V

SE - Sonic Echo

IR - Impulse Response

USC - Ultraseismic Vertical Profiling with Compressional Waves

USF - Ultraseismic Vertical Profiling with Flexural Waves

BW - Bending Wave with Short Kernel Analysis

PS - Parallel Seismic with hydrophone (h) or geophone (g)

BHR - Borehole Radar

BHS - Borehole Sonic

inc - inconclusive test results for foundation element depth prediction

n/a - the method was judged to not be applicable for depth prediction of the substructure

nb - indicates no borehole tests were performed because no boreholes were drilled

-- - indicates the nondestructive test was not performed for that substructure

? - tentative, weaker prediction that may or may not be accurate

+- distance above top of bell

8.8_h - denotes a foundation element depth prediction from a hydrophone PS test

8.2_g - denotes a foundation element depth prediction from a 3-component geophone PS test

multiple channel results that are based on the same raw compressional and flexural wave data from substructure side used in the SE/IR, and Bending (flexural) Wave tests. No boreholes were drilled at this site.

Pier 4, South Column, Coors Bridge, Colorado. Sonic Echo/Impulse Response tests were unsuccessful from the top of the beam over the South Column of Pier 4, again due to the large beam trapping the wave energy. Ultraseismic VP compressional tests on the column showed the shallow pilecap to be at 9.5 m (31.1 ft) below the top of the beam (1.5 m (4.8 ft) below-grade) which agrees with the plan dimension of 9.5 m (31.1 ft) (pilecap bottom at depth of 1.5 m (4.8 ft)). Ultraseismic VP flexural tests showed the shallow pilecap to be at 1 m (3.4 ft) below-grade which is 0.4 m (1.4 ft) short of the actual depth. As expected, none of the surface tests provided any information on the steel BP piles. The pilecap provides a large stiffness (impedance) contrast between the column and the 4 steel piles that reflects most of the energy back up the column, with little reaching the pile heads.

Results of the borehole Parallel Seismic tests showed pile depths of 8.8 m (29 ft) and 8.2 m (27 ft) below-grade for hydrophone and geophone receivers, respectively. These depths compare well with the plan depth of 8.8 m (28.8 ft) in Table V. No indication of the pilecap was apparent in the data. The prototype tool used in the Borehole Radar tests did not indicate the presence of either the pilecap or the steel BP piles due to man-made sources of noise (buried power lines).

Pier 2, North Column, Coors Bridge, Colorado. Sonic Echo/Impulse Response tests were unsuccessful from the North Column of Pier 2 due to the energy being trapped in the large beam. Ultraseismic VP flexural wave tests on the column indicated a reflector depth of 7.9 m (25.9 ft) below the beam top (1.3 m (4.4 ft) below-grade) which agrees very well with the plan dimensions of 7.9 m (26 ft) (footing bottom depth of 1.4 m (4.5 ft)). Borehole Radar did not indicate the presence of the shallow footing due to cultural noise from buried power/utility lines.

Northeast Wing, Franktown Bridge, Colorado Wing. Sonic Echo and Impulse Response tests from the accessible top of the Northeast Wing timber Pile 2 indicated the bottom of the pile was 9.1 m (29.8 ft) and 8.5 m (27.9 ft) below the top of the wall, respectively, (6.9 m (22.8 ft) and 6.4 m (20.9 ft) below-grade). Bending (flexural) Wave tests of this pile with the Short Kernel Method predicted a pile depth of 8.3 m (27.3 ft) below the top of the wall (6.2 m (20.3 ft) below-grade). As shown in Table V, these results compare well with the plan depth of 8.5 m (28.0 ft) below the top of the wall (6.4 m (21.0 ft) below-grade). Ultraseismic VP compressional and flexural wave results were dominated by reflections from the grade.

Center Pier, Franktown Bridge, Colorado Pier. Sonic Echo and Impulse Response tests from the accessible side and top of Pile 1 indicated the timber pile was 7.3 m (23.8 ft) and 7.1 m (23.2 ft) below the bottom of the beam (6.3 m (20.8 ft) and 6.2 m (20.2 ft) below-grade). Ultraseismic VP flexural wave results were inconclusive, but a tentative bottom reflection was identified in the compressional results that predicts a pile length of 7 m (23.0 ft) (6.1 m (20.0 ft) below-grade). As shown in Table V, these results do not compare well with the plan pile length of 8.5 m (28.0 ft) (7.6

m (25.0 ft) below-grade). It is possible that the piles of the center pier were driven to the same depth as the Wing piles, and their tops cut off by 1.2-1.5 m (4-5 ft) at the pier, since that is the elevation difference between the higher Wing versus the lower Pier piles. No borings were drilled at the Franktown bridge site.

West Abutment, Weld County Bridge, Colorado. Sonic Echo and Impulse Response tests predicted stubwall abutment depths of 2 m (6.6 ft) below the top of the wall. This compares reasonably well with an as-built abutment depth of 1.8 m (6.0 ft). The surface tests did not provide any information on the 10.7 m (35 ft) deep steel H-piles under the abutment. This bridge was designed by Loris and Associates, and NDT was able to be done while it was being built. No borings were drilled at this site.

West Pier, Weld County Bridge, Colorado. Ultraseismic VP tests with flexural waves predicted a pilecap bottom at 5.5 m (18.0 ft) below the top of the pier (depth of 3 m (9.7 ft) below-grade). This compares well with an as-built pilecap depth of 5.8 m (18.9 ft) below the top of the pier (3.3 m (10.7 ft) below-grade). The Ultraseismic tests did not provide any information on the 7.6 m (25 ft) long steel H-piles (embedded 0.3 m (1 ft) in the pilecap). In addition, Sonic Echo/Impulse Response tests on the tops of the driven H-piles (0.3 m (1 ft) exposed) before construction of the pilecap and pier wall also did not show any echoes indicative of the pile tips. As shown by modeling, the damping of the compressional wave energy is very high per unit length for H-piles because of their large surface areas and small cross-sectional areas. Consequently, only shallower H-piles are likely to reflect sufficient energy back up the pile to predict their lengths with the low-

strain Sonic Echo/Impulse Response tests. No borings were drilled at this site.

<u>East and Center Piles, Bent 4, Alabama Bridge</u>. Sonic Echo/Impulse Response tests on the small protective concrete caps of the East (battered) and Center (vertical) steel BP piles were inconclusive. Ultraseismic VP compressional tests of the Center Pile tentatively showed a possible bottom echo at a depth of 10. 4-10.7 m (34-35 ft), but the data quality was poor. Ultraseismic VP flexural tests also tentatively showed a possible bottom reflection corresponding to 10.7 m (35 ft) deep, but it may also be due to a multiple reflection event from the column top. Because of the length of these piles, it is doubtful that the identified echoes are from the pile bottoms, but more likely they are from flexural wave reflections in the columns (from the top, or just below the ground surface). The predicted depths are short by up to 1.2 m (4 ft) to 1.5 m (5 ft) from the as-built driven depths of about 11.9 m (39 ft) below-grade for the piles.

Parallel Seismic results with the geophones predicted a depth of 9.1 m (30 ft) below-grade for the East pile and 9.6 m (31.6 ft) below-grade for the Center pile. The hydrophone Parallel Seismic results were unclear for the battered East pile, but predicted a center pile depth of 10.5 m (34.6 ft) which is closer to the as-built pile lengths of about 11.9 m (39 ft). It appears that the attenuation of the signal was sufficient to prevent the energy from propagating to the pile bottom with sufficient strength to be sensed in the nearby boreholes. If this is indeed the case, then stronger hammer blows might help to mitigate the problem in the future, which was found to be the case in Phase II research. Borehole Radar tests indicated depths of 9.4 m (31 ft) and 8.6 m (28.1 ft) for the East and Center Piles, respectively. This poor agreement with the as-built depths of about 11.9 m

(39 ft) below-grade may be due to severe attenuation of the radar signal in a more conductive clayey zone. The boring logs show that close to the bottom of the pile there is a layer of dense sand with some clay.

<u>Caisson, Old Bastrop Bridge, Texas</u>. A Sonic Echo test from the top of the column predicted a depth of 10.9 m (35.9 ft) to the top of the embedded bell of the caisson (about 0.3 m (1 ft) below the ground surface) which agrees fairly well with the actual dimension of 11.6 m (38.0 ft). The Impulse Response tests were inconclusive, probably because of complex vibrations of the column, wall and caisson substructure combination. Ultraseismic flexural results from the north side of the north column tentatively showed possible reflections from 5.7 m (18.6 ft) and 11.4 m (37.3 ft) deep below the top of the bell which are possibly the first and second echoes from the bottom to the top of the bell, or echoes from the bottom of the bell and bottom of caisson. Use of the Ultraseismic flexural wave method to test such large, massive substructures is complicated, so the confidence level in the above interpretation is low.

Parallel Seismic results with a geophone indicated the caisson bottom was at 10.5 m (34.3 ft) in Borehole 1 and 10.4 m (34 ft) below the bell top in Borehole 6. The hydrophone tests in Borehole 1 predicted a depth of 11.4 m (37.3 ft) below the bell top for the caisson bottom. These predictions show good to excellent agreement with the plan caisson depth of 10.4 m (34 ft) below the top of the bell. Borehole Sonic tests indicated caisson depths of 10.1 m (33.3 ft) and 10.0 m (33 ft) from the top of the bell for the north and east sides of the caisson, respectively. However, since the shale bedrock is at these same depths, it is not conclusive that the Borehole Sonic tests found the

bottom of the foundation. The Borehole Sonic test results would have been more definitive if the caisson had not extended into the shale bedrock. Borehole Radar tests were inconclusive, presumably because of the moist-to-saturated, likely conductive clay soils, and possibly also due to the bentonite-cement grouting of the PVC casings.

Concrete Pile Pier, Old Bastrop Bridge, Texas. Sonic Echo/Impulse Response tests were performed on the exposed top of the 0.9 m (3 ft) thick pilecap over the west concrete pile (14"x14") of the north column. The tests were inconclusive. The large impedance contrast between the thick, large pilecap and the small pile trapped the energy in the pilecap and no echoes from the pile bottom were apparent. Only unwanted, strong echoes from the column and beam above the pile cap were apparent in the results.

Parallel Seismic tests with hydrophones and geophones indicated pile bottom depths of 10.0 m (33 ft) and 9.8 m (32 ft) below-grade, respectively. These depths agree very well with the plan pile depth of 10.1 m (33.3) ft below-grade. Borehole Sonic tests were unsuccessful, most likely because of the smaller size of the concrete pile vs. the massive caisson. Borehole Radar tests were unsuccessful for the same reasons given above for the caisson.

Drilled Shaft, New Bastrop Bridge, Texas. Sonic Echo tests predicted a depth of 11.6 m (38 ft) below-grade. Impulse Response test results were inconclusive, presumably because of multiple resonance effects from the beam, column and shaft. Ultraseismic Vertical Profiling tests with flexural waves indicated a tentative, possible bottom echo, but again, multiple reflections from the

beam, column and shaft elements may have created the echo rather than the shaft bottom. The actual shaft plan depth is 11.6 m (38 ft) below-grade as shown in Table V.

Parallel Seismic tests with the hydrophones and geophones predicted depths of 11.7 m (38.3 ft) and 10.8 m (35.3 ft) below-grade. These depths show good to excellent agreement with the plan shaft depth of 11.6 m (38 ft) below-grade. Borehole Sonic tests were unsuccessful, most likely because of the smaller size and round shape of the shaft vs. the massive caisson. Borehole Radar tests were unsuccessful for the same reasons given above for the caisson.

3.2 APPRAISAL OF NDT METHODS CAPABILITIES AND LIMITATIONS.

NDT methods researched in this study, and other methods applicable to the unknown bridge foundation problem, are discussed below in terms of their capabilities and limitations. The six surface methods are discussed in Sections 3.2.1 through 3.2.6 (Sonic Echo/Impulse Response, Bending Wave, Ultraseismic, Spectral Analysis of Surface Waves, Ground Penetrating Radar and Dynamic Foundation Response). The four borehole methods are discussed in Sections 3.2.7 through 3.2.10 (Parallel Seismic, Borehole Radar, Borehole Sonic and Induced Electromagnetic Field). Most of the methods are primarily used for unknown foundation depth determination, but some of the methods can be used to provide additional information on foundation type, geometry and materials.

3.2.1 Sonic Echo/Impulse Response Method.

Capabilities - Unknown Depth. These two tests are best used for determining the depths of rod-like, columnar substructure shapes such as timber piles, concrete piles and drilled shafts that extend up above the ground or water surface. The Sonic Echo test can also be used to determine the depths of shallow, wall-shaped abutments and piers. If tests are performed on the side of a pile, then the Sonic Echo test should be used with 2 receivers to track upgoing vs. downgoing wave reflections. This also permits the measurement of compressional wave velocity, which is used to calculate reflector depths such as the pile bottom. Bottom echoes can be obtained for embedded length to diameter ratios of 20:1 or less in most soils, but results have been obtained for slenderness ratios greater than 50:1 in extremely weak, soft soils. Use of theoretical models allows one to evaluate the likelihood of success with the methods prior to going to the field, and modeling also aids in interpretation of results. Reflector depths can generally be predicted within 5% of actual values if the wave velocity is known, and within 10% of actual values if the velocity is estimated for a given foundation material. Impulse Response tests will generally only be useful when performed near the tops of piles and shafts as the resonant responses are clearest there. The two methods are fairly fast as both tests can be done by a two-person crew in 15 to 30 minutes depending on bridge access conditions and difficulties of mounting receivers.

Limitations. The Sonic Echo/Impulse Response methods will not work well when tests are attempted through more massive and complex members, such as beams and pilecaps, to detect echoes from smaller, columnar piles and shafts. Modeling and field tests have shown that the compressional wave energy is trapped in the larger element because of the impedance contrast

(larger cross-sectional area). Bottom echoes will likely not be measured for embedded length to diameter ratios much greater than 20:1 to 30:1. Reflections will also not be identified when the stiffness of soils and bedrock begins to approach that of the foundation element. Complex substructure shapes can cause multiple reflections that make interpretation of the data difficult, and even impossible. Generally, no reflections will be identified below the first major change in stiffness (impedance). Thus, if one is testing a column on a buried pilecap on piles, one would only typically be able to identify the pilecap, but would not know if the piles were even present, let alone how deep they were.

3.2.2 Bending Wave Method.

Capabilities - Unknown Depth. This method uses bending (flexural) waves instead of the compressional wave energy used in the Sonic Echo/Impulse Response methods. It is limited to applications on rod-like deep foundations such as timber piles, concrete piles and drilled shafts that extend above the surface and water. Experimentally, it has been applied to largely timber piles, but will also work on other, more slender members. The method has been used on timber piles of up to 60 ft in length in research by Douglas and Holt (7). The main advantage of the method is that only a horizontal blow is required, which is easy to apply to the side of a substructure.

Limitations. For a 12 m long, 1 m diameter concrete shaft, theoretical studies show that depth predictions may not be made for depths greater than 5 m due to the high attenuation associated with flexural waves as compared to compressional waves traveling down a rod. Stiff soil layers can also result in false apparent short pile lengths. More experimental and theoretical research is needed

to compare the capabilities of the Bending Wave method to the compressional wave based Sonic Echo/Impulse Response methods, but the BW method is most applicable to short piles in soft soils. Ground reflections also mask pile bottom echoes in stiffer soils in BW tests.

3.2.3 Ultraseismic Vertical Profiling Method.

Capabilities - Unknown Depth. This method has all the capabilities of the Sonic Echo/Impulse Response method with compressional waves, as well as the capabilities of the Bending Wave method with flexural waves. The method was found to be less affected by the presence of large beams on top of columnar substructure than the Sonic Echo/Impulse Response methods. The advantage of this method, as compared to the Sonic Echo/Impulse Response and Bending Wave methods, is that it uses multiple receiver (or source) locations to perform multi-channel, geophysical processing of the data. The additional data is processed to separate out upgoing and downgoing waves, to minimize noise from attached substructure reflections, and to permit "tracking" the wave travel to determine the reflector locations. The Ultraseismic method worked much better than the other methods for determining the depths of shallow abutment walls and pier walls, and also was much more useful for columnar substructure on footings with large beams on top. A typical dataset requires about 1 hour in the field to collect, depending on access.

Limitations. The method will not work well for foundations embedded in very stiff materials since little energy, if any, will be reflected. It also showed difficulties in identifying flexural wave reflections from more massive, deep foundations. It requires only a little more time to acquire and process data than the Sonic Echo/Impulse Response and Bending Wave tests, and data reliability is

typically increased. An accessible vertical surface of at least 4-5 ft or more in length is needed for testing. Like the Sonic Echo/Impulse Response and Bending Wave methods, Ultraseismic tests will not generate sufficient energy to penetrate below significant changes in stiffness (impedance). Thus, it will not detect piles below a buried pilecap.

3.2.4 Spectral Analysis of Surface Waves (SASW) Method.

Capabilities - Unknown Depth, Geometry, and Materials. The SASW method has been found to be capable of determining the depths of shallow abutments, pier walls and other solid substructures with a flat surface from which testing can be performed. Such flat surfaces could be the top of an abutment between girders, a ledge or step, or even the top of an exposed footing or pilecap. It can also be used to determine unknown thicknesses of abutment breastwalls and wingwalls, exposed footings and pilecaps, and indicate material properties in terms of stiffness (velocity) for substructures and surrounding soils and rock. It is estimated that SASW tests cannot penetrate depths that are much greater than the longest substructure dimension available for testing. Where substructure geometry has enabled the SASW test to be used, clear indications have been provided as to the depth (boundary) of the substructure abutments (faster velocity), and the, underlying soils and bedrock (slower velocity).

Limitations. The main limitations of the method are geometric. Flat access is required to generate the surface wave energy. Maximum foundation depths that can be determined are estimated to be not much deeper than longest test horizontal surface available on the tested substructure. The substructure must also be solid for the surface wave energy to travel down through it and interact

with the underlying soils.

3.2.5 Surface Ground Penetrating Radar (GPR) Method.

Capabilities - Geometry. The surface GPR method is mainly useful for determining the thicknesses of abutments from the wall surfaces, or the roadway. It can also be used to attempt to penetrate through the earth to detect the footprints of reinforced concrete footings or pilecaps, and perhaps the tops of steel piles themselves, but this will be highly site dependent in terms of its success and noise will occur from nearby exposed bridge substructure.

Limitations. The main limitations on the use of radar are environmental effects that attenuate or complicate the GPR signals. Radar is severely attenuated by salt water, brackish water, conductive clays and other soils, moisture in the ground, and man-made sources of noise such as buried power lines and adjacent bridge substructure. Consequently, the use of radar is best at sandy sites and others with low conductivity.

3.2.6 Dynamic Foundation Response Method.

Capabilities - None Confirmed, Some Potential. This test was proposed in an attempt to distinguish whether or not piles were present below a footing by vibrating a bridge substructure to measure its natural frequencies. Footings have lower resonant frequencies than pilecaps on piles in soils. It was hoped that this difference would be substantial enough to discriminate between shallow and deep foundations from the surface for substructures like walls (none of the other surface methods can do this). Although some promise was shown in the experimental, theoretical modeling, and

dynamic analysis results, the method is still considered unproven in its potential for indicating foundation type, i.e., shallow or deep.

Limitations. Some promise was shown in the research, but additional work is needed before the feasibility of this method is established in being able to be used to detect the presence of piles below a footing/pilecap. One problem encountered was the difficulty in exciting the bridges at their very low natural frequencies with 12 lb impulse sledgehammers. The use of large, truck-mounted geophysical vibrators (a Vibroseis) as vibration sources was researched in the FHWA study that came out of this work (31).

3.2.7 Parallel Seismic Method.

Capabilities - Unknown Depth. This borehole based method has the widest range of application of any of the methods for determining unknown foundation bottom depths regardless of depth, substructure type, geology and materials. The use of geophones was found to extend the range of the method to identify foundation bottoms under a wider range of conditions than with hydrophones alone. Both compressional and shear waves can be used with the geophone PS method as generated by vertical and horizontal impacts. The method can be used to depths of 100 ft, or more, if required.

Limitations. The main limitation of this method is that currently a borehole must be drilled and cased to protect the tools and keep the hole open. This can be quite costly in the river environments of bridges. Highly variable soil velocity conditions also complicate the results, but

this can be compensated for in the future (regardless, foundation bottoms were accurately identified at all bridges tested with this method in Phase I). Also, larger impact forces were investigated in Phase II so that greater energy is transmitted through pilecaps to piles.

3.2.8 Borehole Radar Method.

Capabilities - Unknown Depth, Type, and Geometry. Borehole Radar can provide a great deal of data if subsurface conditions are conducive to the radar signal, i.e., low conductivity (high resistivity) subsurface conditions which allow good penetration of soils by the electromagnetic wave energy. Radar also works well at detecting steel and reinforced concrete because the steel reflects the signal strongly. It can be used to estimate the thicknesses of toes and heels of footings, as well as indicate depths of unknown foundations. The testing is fast and data can be digitally recorded.

Limitations. The main limitation for the Borehole Radar test is that the radar transmission is highly environmentally dependent. Radar is severely attenuated by salt water, brackish water, conductive clays and other soils, and moisture in the ground. Consequently, the use of radar is best at sandy sites and others with low conductivity, and where the reflections targets include steel, which strongly reflects the signal. A borehole must be drilled and cased with a 4 inch diameter, PVC casing. Finally, since an omni-directional prototype radar transmitting and receiving antenna was used in this study, received signals were the average of all reflections coming from around the borehole. The use of a directional, focused radar antenna could potentially improve the results. Unfortunately, the only available directional radar system is currently quite expensive, on the order of $250,000.

3.2.9 Borehole Sonic Method.

Capabilities - None Confirmed, Some Potential. Only basic feasibility research was done on this method. Some promise was shown, but reflections were measured from only a very large caisson target. No reflections were measured from a 1.2 m (4 ft) diameter drilled shaft or a 36 cm (14 inch) square concrete pile.

Limitations. Some promise was shown in the research, but additional work is needed to evaluate possibilities of high frequency sources before the feasibility of this method in being able to determine depths of unknown foundations is established. For small targets, such as concrete or steel piles, it is difficult to obtain reflections without generating higher frequency, shorter wavelengths in the soils. This problem can be even more severe when one does not know where to drill to look for piles. The location and orientation of piles are generally not known in unknown foundation substructure investigations. The use of Borehole Sonic requires a PVC-cased boring. We are aware of a borehole sonic tool that is manufactured in Finland, but was not used in this research because of budget limitations.

3.2.10 Induction Field Method.

Capabilities - Unknown Depth. The Induction Field method is a proven technology for the determination of the depth of steel piles and reinforced concrete piles. One important consideration with respect to unknown bridge foundations is that the method requires a non-ferrous cased boring (typically plastic PVC pipe). The method could be performed in conjunction with the Parallel Seismic method or the Borehole Sonic method, which also require a borehole. The method works

best in soils of more uniform conductivity.

Limitations. Interpretation of data from the Induction Field method is complicated by the existence of ferrous materials in the bridge structure and soils. Also, these tests can only work for reinforced concrete or steel piles that are electrically connected to rebar or other metal which can be accessed at the surface. It will not work for unreinforced concrete, masonry, or timber. The boring must be drilled within 1 m (3 ft) of the foundation and should extend about 3 m (10 ft) below the bottom of the foundation.

3.3 RECOMMENDED NDT METHODS FOR UNKNOWN FOUNDATIONS.

In this section, general nondestructive testing approaches are outlined for NDT investigations of unknown bridge foundation conditions. There are two classes of nondestructive testing methods that can be utilized at a given bridge site: 1. those that require access from the exposed parts of the bridge substructure elements (surface methods); and, 2. those methods that require access from a nearby borehole (borehole methods). Discussions are presented below in Sections 3.3.1 for recommended surface NDT methods and in Section 3.3.2 for the recommended borehole NDT methods. The discussions are presented in terms of identifying unknown bridge foundation depths, types, geometry and materials. A discussion of NDT investigation approaches to determine unknown foundation conditions is presented in Section 3.3.3. The best technical approach would be to perform a borehole PS test and the surface US test (if applicable) on a substructure. If the PS and US tests agree, the cheaper surface US test could be used on more substructures to check for varying foundation depths.

3.3.1 Recommended Surface NDT Methods.

The recommended surface methods are Sonic Echo/Impulse Response, Bending Wave, Ultraseismic Vertical Profiling, Spectral Analysis of Surface Waves, and Ground Penetrating Radar. The recommended applications of these methods are discussed in terms of the NDT "target" of interest below.

3.3.1.1 Unknown Foundation Depths. Sonic Echo/Impulse Response and Bending Wave methods are both applicable to determining the depths of timber and concrete piles, concrete-filled steel pipe piles, and drilled shaft foundations that extend above the ground or water surface that are shorter and in softer soils (particularly so for the BW method). Since the tests can be complimentary, and use similar equipment, it is suggested that both tests be performed together using two vertical receivers and two horizontal receivers to better track wave travel up and down piles. It is also expected that the Sonic Echo/Impulse Response method with compressional waves will be able to identify greater pile bottom depths than the Bending Wave method. In addition, the Sonic Echo method is applicable for determining the depths of shallow, wall-shaped and more massive abutments and piers.

The processing of these two methods can be combined with the geophysical processing and display of the Ultraseismic Vertical Profiling method to provide detailed, accurate, and economical testing in one system. Ultraseismic testing could also be of use on piles, particularly if there is a need to track more complex reflections and arrivals than is possible with the SE/IR and BW methods. The determination of unknown depths of steel H-piles is likely to be limited to very

shallow depths at most sites. This is because of the high attenuation of stress wave energy over the large surface area of H-piles. Finite element modeling of wave propagation behavior in foundation substructure was found to be of real value in planning NDT programs and analyzing NDT results for compressional and flexural waves (7, 13).

Ultraseismic Vertical Profiling with compressional and flexural waves is recommended for determining the depths of the more complex columnar- and wall-shaped piers and abutments. Ultraseismic Horizontal Profiling was also found to be useful for these types of bridge substructures. Ultraseismic tests are also recommended for combined shallow/deep foundation substructures, e.g., columnar or wall substructures on pilecaps in order to determine the pilecap depth. However, for more massive and deep substructures, Ultraseismic tests are in need of more research.

For the special case of bridge substructures with exposed footing/pilecap tops, the two surface methods of surface radar and the spectral analysis of surface wave (SASW) methods can be employed in determining the depth of the footing/pilecap, using different physical principles (see Appendix E case histories in Phase I final report (7) for unknown foundations of Connecticut DOT abutment substructures). The SASW method uses the dispersive properties of Rayleigh surface waves to determine the concrete-soil depth (bottom of the footing). Spectral Analysis of Surface Waves (SASW) tests can work very well for foundation depth determination of concrete and possibly masonry bridge substructures that are more massive and wall-like in shape. A horizontal, flat access area is needed for testing, and the substructure should be of solid construction. For example, the SASW method may not work well for a bridge substructure that consisted of a layer

of masonry above earthfill, followed by a bottom layer of masonry. Also, the bottom depth of the substructure that can be detected with the SASW tests is likely to be not much greater than the longest accessible horizontal test surface of the substructure.

Surface ground penetrating radar records the reflection echoes from the concrete-soil boundary to determine the unknown depth. However, because of reflections from adjacent, exposed substructure, it can be difficult to determine the depths of even shallow footing foundations with radar.

To summarize, a good measure of the unknown depth of the foundation can be obtained in a large subset of bridges by utilizing the Sonic Echo/Impulse Response/Ultraseismic, SASW and Ground Penetrating Radar methods. <u>However, it should be noted that none of these methods are able to detect foundation elements below the first major change in stiffness, i.e. they will not detect the presence of piles underlying a buried footing/pilecap</u>.

<u>3.3.1.2 Foundation Type.</u> The next question concerns detecting the possible existence of piles underneath a footing/pilecap, in other words, determining whether the substructure foundation system is shallow (footing), deep (pile) or a combination (pilecap on piles). The dynamic foundation response (DFR) test was proposed to determine whether a substructure element is on a footing or on a pilecap supported by piles. The surface methods, discussed above in Section 3.3.1.1, can determine the unknown depth of shallow bridge foundations or exposed piles, but not of buried piles underneath a footing. These methods can, therefore, be used for the bridge substructures that do not

contain piles or have exposed piles and are susceptible to scour. For the other bridges with footing on pile substructure, the borehole methods must be used. The Dynamic Foundation Response test shows some potential, but the method is not yet conclusive in this regard. For a complex bridge structure, the foundation system typically exerted a relatively minor influence on the total dynamic response of the system (31).

None of the researched surface methods can directly determine whether piles exist underneath a footing at this time. However, there are other criteria that can aid in the selection of a subset of bridges that require subsequent borings. For example, the depth to bedrock can be obtained near a bridge abutment or pier using surface radar, SASW, or seismic refraction tests. This information, combined with the results of the Ultraseismic and/or SASW methods, can determine whether the footing rests on bedrock. A bridge engineer can then assess the susceptibility of the foundation to scouring using this and other hydraulics parameters. Obviously, if a footing is resting on competent bedrock, it is very unlikely that piles were driven into the bedrock below the footing.

3.3.1.3 Geometry. Several NDT methods are applicable to the problem of determining unknown subsurface bridge substructure geometry. The nondestructive testing investigation to determine unknown foundation conditions of the Connecticut bridges was performed to determine unknown foundation depths. The investigation was also performed to determine as much other unknown abutment geometry as possible, such as thicknesses of stem walls and the extent of a footing heel and toe, if present. For the Connecticut bridges, this was attempted with surface ground penetrating radar with some success. Surface radar test methods can be used to determine stem wall

thicknesses and inclinations. This can be done by running two or more test lines across the (exposed) front side of the stem wall of an abutment and recording the echoes from the (soil covered) back side. Similarly, the extent of the toe can be determined by running a surface radar line over the top of the bridge and also along the ground near the base of the existing substructure (abutments or piers). The signals are examined for the reflection event from the top of the toe, although interpretation of these results is a more challenging task because of reflections from the adjacent substructure. However, radar will not work well at sites with conductive soils, and its use on more reinforced abutments and piers, or around steel piles may produce data dominated by echoes from the steel.

Stress wave based tests such as Spectral Analysis of Surface Waves and Impact Echo (essentially high frequency Impulse Response testing (31)) can also be used to indicate the thicknesses of exposed substructure of abutments. Ultraseismic tests with high frequency impacts are also applicable to thickness measurements.

3.3.1.4 Materials. The Spectral Analysis of Surface Waves test can provide data on the change in the stiffness of foundation materials with depth without drilling a boring by measuring the dispersion curve (velocity versus wavelength). No other NDT methods provide such direct data on changes in subsurface substructure properties.

The other aspect of the unknown foundation problem concerns the local soil and bedrock geology conditions. The SASW method is useful in this regard as one can determine layer

thicknesses and the shear wave velocity profile vs. depth without drilling a boring. Shear wave velocity of the soil is a key input into the finite element modeling of compressional and flexural wave propagation behavior for bridge foundation substructure. The faster the shear wave velocity, the stiffer the soil and the greater is the attenuation of the stress wave energy. The SASW method is also uniquely capable of being able to measure the velocity of soft soils underlying stiffer soils. Surface Seismic Refraction surveys can also indicate the variation of velocity with depth, but only if the soil velocity increases with depth. Electromagnetic Induction surveys can be run to measure the ground water-table depth, bedrock and soil conductivity in order to predict the depth of radar penetration prior to radar measurements.

3.3.2 Recommended Borehole NDT Methods.

The recommended borehole methods are Parallel Seismic, Radar, and Induction Field. The recommended applications of these methods are discussed by the NDT "target" of interest below.

3.3.2.1 Unknown Foundation Depths. Of all the NDT methods, the Parallel Seismic method was found to most accurately indicate unknown foundation bottom depths for the broadest range of bridge substructures and subsurface geologic conditions. The borehole should be drilled within 1 to 2 m (3 to 6 ft) of the substructure and extend at least 3 to 4.5 m (10-15 ft) beyond the minimum acceptable foundation depth from a scour and capacity standpoint. Parallel Seismic tests with hydrophone or geophone receivers produced good to excellent depth predictions for every Phase I bridge foundation tested as shown in Table V. Hydrophone receivers will work well in saturated soils and other uniform velocity soil conditions where compressional wave first arrival

times are a linear function of the foundation depth. Geophone receivers should be used in grouted, cased holes in variable velocity soil conditions, or if the arrivals of both compressional and shear wave energy are to be determined. Under ideal circumstances of uniform seismic soil velocities, the Parallel Seismic test can provide an "image" of the shape and orientation of a foundation. However, such subsurface uniformity occurs rarely in nature for any significant depths. An image can be provided in non-uniform soils by correcting for varying soil velocities vs. depth based on the results of Downhole Seismic or SASW tests.

The Borehole Radar method produced results ranging from excellent to none in terms of foundation depth predictions at the study and case history bridge sites. The method will work best where some steel is present in the foundation; and soils, water and groundwater all have low conductivity. Attenuation of the radar signals is severe in salt water, conductive clays, and other conductive soils, to the point where the radar penetration may be only a foot or two. However, Borehole Radar worked well in 3 of 5 sites in Connecticut where local geology conditions were favorable. Electrical conductivity surveys of the ground can indicate whether a site is suitable for radar or not. Analogous to the Parallel Seismic method, Borehole Radar will work best in soils with near-constant electromagnetic wave velocities. At sites with such uniform conditions, Borehole Radar can even provide an "image" of subsurface foundation elements that vary in shape (i.e. footing heel beneath abutment wall) and orientation (i.e. battered steel BP-pile). With directional Borehole Radar (as opposed to the omni-directional tool used in this study), one might be able to check the depths of several piles from the same borehole under ideal test conditions. However, if the pile shape is such that it is not round, or square to the measurement, little energy may be reflected back

towards the tool.

The Induction Field test is the electromagnetic analog to the Parallel Seismic method. It is only applicable to steel piles or reinforced concrete piles that are able to be electrically connected to the surface. It also requires the boring to be PVC-cased and drilled within less than 1 m (3 ft) of the pile for the search coil. It should be noted that for embedded pilecaps on steel H-piles, the steel H-piles would not be detected by this method unless they were electrically connected to the reinforcing of the pilecap up to the exposed substructure. Consequently, this method is much more limited in its application.

3.3.2.2 Foundation Type. The next question concerns detecting the possible existence of piles underneath a footing/pilecap, i.e., determining whether the substructure foundation system is shallow (footing), deep (pile) or a combination (pilecap on piles). To answer this question, it is best that an appropriate surface NDT method be used in conjunction with Parallel Seismic tests (or Borehole Radar tests if subsurface conditions are favorable). However, in a simple sense, drilling the boring and performing the borehole NDT will indicate whether a foundation is deep or shallow simply by the indicated bottom depth.

3.3.2.3 Geometry. The nondestructive testing investigation to determine unknown foundation conditions of the Connecticut bridges also involved attempting to identify if a footing heel and toe were present. Borehole radar measurements gave a fair indication of the extent and thickness of the footing heel.

3.3.2.4 Materials. Both the Borehole Radar and Induction Field method are sensitive to steel in foundations. However, for that sensitivity to translate into differentiation between steel H-piles, reinforced concrete piles, and timber piles would require the performance of at least one of the two tests in conjunction with the Parallel Seismic method. Even then, local conditions may prevent a definitive answer.

3.3.3 NDT Investigation Approaches to Unknown Subsurface Foundations.

It is recommended that the interpretation of all the NDT and geophysical results be done by a specially trained engineer/geologist/geophysicist within a DOT or in conjunction with an independent consultant. Although many challenges still remain in determining all the unknown bridge parameters under all geological/hydraulic conditions, it is hoped with more research, these methods can be used routinely at many bridge sites.

Generally speaking, the most critical item to be determined is the unknown depth of the bridge foundation to compare with predicted scour depths in scour vulnerability analyses. Consequently, the selection of the NDT methods should reflect knowledge of the exposed substructure, local geology conditions, and the criticality of the bridge. Surface NDT methods will generally be more economical than boreholes because there are no drilling costs. Although the borehole methods are generally considered to be more expensive due to the associated cost of a boring, there are other means to mitigate their costs. For example, Parallel Seismic tests can now be run from 2 inch slim holes. Portable light-weight drilling units exist that can be mounted on the back of pick-up trucks. These units can be operated by DOT personnel in drilling slim holes

economically at large number of bridge sites. Jetting, drilling and/or driving of probes and/or casings may be possible in the future.

As an unknown foundation investigation is planned, one must first decide what needs to be learned. Is the most important parameter the: depth of an abutment/pier, pile depths, foundation type, subsurface substructure geometry, material types, geotechnical data on subsurface conditions, or geophysical surveys for soil velocity and bedrock depths. For the sake of economy, only the depth information may be needed for many bridges with unknown foundations, and engineers will be left to make reasonable assumptions about other variables in their scour vulnerability analyses. Critical bridges may require as much information as possible. Also, in general it is good practice in nondestructive testing to use two or more NDT methods to check the results to verify that they are consistent. This approach was taken for the unknown foundation conditions investigation of the Connecticut DOT bridges (Olson's independent consulting job presented in the final report of August, 1995). It is strongly recommended that the borehole PS test be done for at least one pier/abutment on a bridge for comparison and ground-truthing of any other NDT method.

3.4 NDT IMPLEMENTATION- EQUIPMENT, TRAINING, OPERATIONS.

In this section, the applicable surface and borehole NDT methods are appraised in terms of their system costs, training requirements, time required to complete a test (including the analysis time), and level of expertise needed. The implementation requirements for surface NDT methods are presented in Section 3.4.1 for the Sonic Echo / Impulse Response / Bending Wave / Ultraseismic, Spectral Analysis of Surface Waves (SASW), and Ground Penetrating Radar (GPR) tests. The

implementation requirements for the borehole NDT methods of Parallel Seismic, Borehole Radar and Induction Field are presented in Section 3.4.2. A distinction is made below in terms of "commercial" equipment, i.e., a commercial system which is available for purchase designed specifically for the method, versus "custom" equipment, i.e., a custom system which can be assembled by purchasing individual system components, and writing any software needed to process the data.

All the surface tests (Sonic Echo/Impulse Response, Bending Wave, Ultraseismic, and Spectral Analysis of Surface Wave methods) require accelerometers or geophones that are coupled or hand-held on the top or the side of the bridge substructure. Ordinary water-pump grease as a couplant can be used. In one Ultraseismic experiment on the Golden bridge, we bolted the triaxial accelerometers directly into the bridge column to study the effects of receiver coupling. When comparing the data from that test with another where only hand-held, grease-coupled accelerometers were used, the results were found to be comparable. Bolted accelerometers tend to pick the low frequencies below 80 Hz and high frequencies above 2 kHz better. Since most of our useful amplitudes lie between 80 Hz to 2 kHz, no clear advantage in bolting the receivers at each location was noted, and the use of hand-held accelerometers with good couplant was concluded to be adequate in most concrete structures—and probably the same in stone, timber or steel structures.

The surface tests require access to the top and the side of bridge substructures. If necessary, man-lifts are needed to lower the field engineer from the bridge deck to the side of the bridge abutment or column and boats and ladder to test from the water level. These support costs (where

necessary) for man-lift, boats, ladders, generator, and traffic control are not included in the cost estimates below. Drilling and coring through bridge decks may be the most economical means to install casings for the borehole methods.

3.4.1 Sonic Echo/Impulse Response/Bending Wave/Ultraseismic Tests.

The Sonic Echo/Impulse Response and Bending Wave tests are generally limited in their applications to determining unknown depths of timber and concrete piles, drilled shafts, and columnar substructure exposed above the ground or water surface. The Ultraseismic test will work on columnar substructure and piles as well, but will also indicate unknown depths of abutments and piers that are wall-like in shape. The Ultraseismic tests use compressional and flexural waves.

Commercial and custom equipment is available for the Sonic Echo/Impulse Response, Bending Wave and Ultraseismic systems. Most SE/IR systems have only 2 channels - one for the impulse hammer input and one accelerometer input. It is likely that the desired additional receiver channel(s) could be added by the manufacturers if the market demand warranted it. The costs for commercial Sonic Echo/Impulse Response systems are on the order of $10,000 to $20,000. The micro-processor based digital data acquisition systems typically include an impulse hammer and accelerometer, and may include an optional geophone, and analysis and modeling software for rod-like deep foundation shapes.

Custom equipment can be readily assembled for the Bending Wave method. The system includes impact sources, accelerometers, and data acquisition hardware plus some software

programming. The costs for the custom system are estimated to be on the order of \$15,000 to \$20,000. Commercial equipment is also available should the BW method prove to be more applicable than it currently appears to be.

Custom equipment and software was used in the research on the Sonic Echo/Impulse Response, Bending Wave and Ultraseismic Vertical and Horizontal Profiling methods with compressional and flexural waves. Ultraseismic testing requires the use of fast sampling, multi-channel digital data acquisition cards. These methods require impacts to the substructure to generate wave energy that travels down into a foundation and up into the superstructure. However, reflections may occur from the bottom of a foundation and the top of the superstructure. When this occurs, Sonic Echo/Impulse Response and Bending Wave records can be greatly complicated by the simultaneous reflection of energy up from the foundation and back down from the superstructure. Because of this and other problems, the multi-channel Ultraseismic method was proposed and developed as part of this research.

A commercial system has since been developed which would allow all three tests to be performed since they are all based on either compressional or flexural waves and use similar hardware to perform. The custom equipment used for all three tests in this research included 1.4 or 5.4 kg (3- or 12-lb) impulse hammers, 2- three component triaxial accelerometers (or more as needed) and conditioners, an analog filter/amplifier, and a dynamic signal analyzer card (4 channel) or digital oscilloscope card with processing software for each of the tests. The estimated commercial/custom system cost to perform all three tests is on the order of \$25,000 to \$30,000

including a portable PC with a card which acts as a digital oscilloscope. The analysis of the Ultraseismic data requires the use of commercially available, specialized geophysical software. A complete system for the three tests would therefore cost on the order of $30,000 to $35,000. The estimated custom system cost for the Ultraseismic equipment alone is $20,000.

Training on the equipment and tests is estimated to require from a few days to 2 weeks depending on the equipment and NDT methods. The training should also include field NDT of suitable bridge substructures. Training costs are not included in the above cost estimates. A trained technician with previous NDT, instrumentation and PC expertise within the owner's organization (DOT) is a suitable candidate for <u>collecting</u> the field data.

The cost of performing the NDT with one of these surface NDT methods by an outside consultant is estimated at $2,000 - $2,500 per bridge substructure units for ½ day of field testing and ½ day of analysis plus report time. These cost estimates assume that the DOT provides the necessary support personnel (1-2 persons depending on the bridge conditions). All consulting costs presented in this section assume that at least a few bridges would be tested in a single contract. Costs can likely be lowered significantly by testing a greater number of bridges per contract. At present, the interpretation of the Ultraseismic records requires a seismic geophysicist/geogolist, or engineer with appropriate NDT experience. The estimated training time for the basic processing of the Ultraseismic data is about one week.

3.4.2 Spectral Analysis of Surface Waves (SASW) Test.

The SASW test has applications for unknown foundation depth determination (where flat, wide substructure access is available), for geometry determination of abutment wall thicknesses and exposed footings/pilecaps, for determining substructure material properties vs. depth, and for measurement of the variation of stiffnesses (velocity) of soils and bedrock with depth. The custom equipment for the SASW method includes hammers ranging from 1 lb to 12 lb sledgehammers (vibrators can also be used), a dynamic signal analyzer, and two seismic accelerometers (or suitable geophones for greater depths and for testing of soils). The estimated cost for this commercial custom equipment with processing software (WINSASW, a windows program from the University of Texas at Austin) is about $20,000 and includes a portable PC.

With the exception of the seismic accelerometers and processing software, all of the other SASW system components are identical to those of the Sonic Echo/Impulse Response and Ultraseismic methods outlined above. Thus, if a combined commercial/custom system was developed for all four stress-wave based surface tests, its total cost might be on the order of $35,000 to $40,000 with all needed software.

The cost of performing this test by a consultant, where applicable, is estimated at $2,000 to $2,500 per bridge substructure unit for ½ day of field testing and ½ day of analysis by a consultant (assumes 1 DOT person for support at DOT cost) plus report time. Depending on access, from 1-2 bridges and 2-4 substructures could be tested per day. Training time is estimated to be about 1-2 weeks to train DOT technicians in proper field procedures and data collection. Data analysis must

be performed by a specialist consultant or a trained DOT engineer.

3.4.3 Surface Ground Penetrating Radar.

Surface radar is primarily useful for determining the thicknesses of the breast walls of abutments, exposed footings/pilecaps, and possibly the width of a footing toe or heel. It may also be used to determine the depth to bedrock if suitable subsurface conditions are present for the use of radar (low conductivity soils and water). Commercial surface GPR antennae are available in both monostatic (single transmitter/receiver) and bistatic (separate transmitter and receiver systems). The full system costs, including a graphical interface, range from $30,000-$80,000. A system with a 400 to 500 MegaHertz (MHz) monostatic antenna is adequate for the required testing at a cost of about $30,000. Radar processing software is available from the equipment manufacturer or other commercial software houses for about $1,000 to $5,000. With processing software, costs are typically on the order of $40,000 to $50,000 for a system.

The cost of performing a GPR test is estimated at $2,000 to $2,500 for ½ to 1 day of field testing in which 1-2 bridges and 2-4 substructures could be surveyed with 1 day of analysis. Data analysis must be done by a trained DOT engineer or geophysicist. Estimated training time is about one week for training of technicians and engineers in the performance and analysis of radar survey results.

3.4.4 Parallel Seismic Test.

The Parallel Seismic test requires the drilling and casing of a boring to determine unknown foundation depths. If a hydrophone receiver is used, a 2 inch ID schedule 40 PVC casing with saw-cuts every few feet is capped on its bottom and installed in the borehole to keep it open. If a 3-component geophone receiver is use, a 2 inch minimum ID schedule 40 PVC casing is capped on its bottom and grouted in the hole with a cement-bentonite mixture (this may kill the radar signal - so clean sand backfill may be needed for radar holes). Steel casings can also be used, but the faster wave of steel can complicate PS results, so steel casing is not recommended. No water is desired for geophone testing, but tests can be done below water with water-proof geophones.

The commercial/custom equipment required for this method includes 3 to 12 lb impulse hammers (or similar size hammers with a triggering device to start recording on impact to the substructure), a recording oscilloscope or digital data acquisition card in a PC or dynamic signal analyzer, differential amplifiers/filters, and a hydrophone receiver and/or a 3-component geophone receiver to go in the casings. The estimated sale price for a portable PC- based system for only Parallel Seismic tests is about $15,000 with 2- hydrophones to $25,000 with 2- three component geophones.

With the exception of the hydrophone and geophone receivers (with rods to orient the geophones or casing wheel guides for inclinometer type casing), most of the other system components are identical to the Sonic Echo/Impulse Response and Ultraseismic methods outlined above. A multi-purpose commercial/custom equipment system could be developed to perform Sonic

Echo/Impulse Response, Bending Wave, Ultraseismic, Spectral Analysis of Surface Waves and Parallel Seismic tests with an estimated sales price under $50,000.

The cost of performing Parallel Seismic tests by a consultant is estimated at $2,000 to $2,500 for ½ day of field testing in which 2-4 substructure units of 1 bridge could be tested (2 borings) with ½ day of analysis plus any required time for the report. This assumes 1 person is provided by the DOT to assist in the testing. The field testing can be done by a trained technician. It is recommended that the data analysis be performed by an outside consultant or a trained engineer within the DOT. Training time for DOT engineers and technicians is estimated at about 1-2 weeks. Drilling costs are estimated to be on the order of $1,000 to $2,000 per borehole if drilling can be done with a truck-mounted rig through the bridge deck. Drilling costs will be much higher if a barge must be used for drilling in water.

3.4.5 Borehole Radar Method.

At bridge sites with low conductivity soils and water, Borehole Radar may be used to determine unknown foundation depths and attempt to estimate the thickness and lateral dimensions of footings and pilecaps. Several custom Borehole Radar tools for geotechnical applications have been assembled by the USGS in Denver. The borehole system used in this study was an omni-directional 120 MHZ Borehole Radar system, after the USGS work, which sells for about $10,000 for the Borehole Radar antennae, plus $50,000 for the high-end two channel recording/display system ($23,000 for the single channel system). The monostatic system utilizes the same antenna acting as both transmitter and the receiver in a borehole.

Another borehole radar tool, one that is directional, is available from Sweden, but this system is currently expensive, about $250,000. The Borehole Radar (BHR) method can be very useful in discriminating vertical bridge members such as piles. It is, however, highly dependent on environmental factors such as the presence of conductive and clayey soils and saltwater.

The cost of performing borehole radar tests for a consultant is estimated at $2,000 to $2,500 for 2 hours per borehole per bridge substructure unit so that 2 bridges or more may be done in a field day with ½ day of analysis plus whatever report time is required. This assumes 1 person is provided by the DOT to assist in the testing. It is recommended that data acquisition and interpretation be performed by an outside consultant, or a trained geophysicist/engineer within a DOT. The required training time for a DOT specialist is estimated to be about one week.

3.4.6 Induction Field Test.

This test is analogous to the Parallel Seismic test, but is only useful for determining unknown depths of steel piles and reinforced concrete piles that are electrically continuous to the surface. Custom equipment must be developed and assembled. Costs for this system are estimated at about $20,000. The cost of performing Induction Field tests for a consultant is estimated at $2,000 to $2,500 for 2 hours per borehole per bridge substructure unit so that 2 bridges or more may be done in a field day with ½ day of analysis. This assumes 1 person is provided by the DOT to assist in the testing. Drilling costs are estimated to be on the order of $2,000 per borehole if drilling can be done with a truck-mounted rig. Drilling costs will be higher if a barge must be used. The required training time for a DOT engineer/geophysicist is estimated to be about 2 days.

CHAPTER 4

SUMMARY OF NCHRP 21-5 (2) PHASE II RESEARCH

UNKNOWN SUBSURFACE BRIDGE FOUNDATION TESTING

During Phase II research, 21 bridge sites were selected in the States of North Carolina, Minnesota, New Jersey, Michigan, Oregon, Massachusetts and Colorado (another bridge was selected in the State of Texas, but no bridge plans are available) to evaluate the accuracy of the NDT methods identified in Phase I research as potential methods for the unknown foundation problem. An initial blind investigation was undertaken wherein the bridge foundation depths were known to the DOT's, but not the researchers. This not only provided a check on the accuracy of the NDT methods, but also allowed for post blind depth prediction research on the NDT methods that significantly improved data interpretation and accuracy. The field work included the performance of the selected methods (where possible) from the research planning stage at each bridge site. They included the four *surface* techniques of Sonic Echo/Impulse Response, Bending (Flexural) Wave, Ultraseismic and Spectral Analysis of Surface Waves tests; and the two *borehole* techniques of Parallel Seismic and Induction Field tests. The surface techniques require access from the exposed parts of the bridge substructure as opposed to the borehole methods that require access from a nearby boring.

In addition to the initial blind testing of 21 bridges, Phase II research involved development of related hardware and software. The TFS software of Olson Engineering was improved for Sonic Echo/Impulse Response, Bending Waves, Spectral Analysis of Surface Waves and Parallel Seismic

test data analysis. The "Bridgix" software was developed by Interpex House. The "Bridgix" software is fully dedicated to the analysis of Parallel Seismic and Ultraseismic test data. In relation to hardware development, a prototype battery operated field computer was designed and built. A prototype 8-channel hydrophone string was developed for rapid field Parallel Seismic data collection. Two prototype systems for Ultraseismic and Induction Field tests were assembled on a custom basis.

Evaluation of several types of sources for the Parallel Seismic method was also performed as part of this research. Theoretical modeling for the Parallel Seismic and Ultraseismic methods was also conducted. Presented below is a summary of the results of Phase II research. Full descriptions of the NDT data and results for Phase II research were presented in two Interim Reports submitted in April, 1998 and February, 2000 (32, 33).

4.1 NDT RESULTS FROM A BLIND INVESTIGATION AT 21 BRIDGES.

A wide range of bridge types with varying geometry were investigated during the field research. The 21 bridges tested were selected by State DOT's of North Carolina, Minnesota, New Jersey, Michigan, Oregon, Massachusetts, Colorado and Texas and approved by the research team. Bridge substructures included timber pile foundations, piers and abutments with either spread footings or pilecaps supported by timber and concrete piles, steel pile foundations and concrete-filled steel pipe foundations. The field experimental research findings are summarized by bridge in the remainder of this chapter. The most employed NDT methods in this investigation included the Parallel Seismic (PS), Ultraseismic (US), Sonic Echo/Impulse Response (SE/IR) and Bending

Waves (BW) methods. A summary of the test results of the four methods mentioned above is presented in Table VI. The Spectral Analysis of Surface Waves (SASW) method was employed at two bridges. The Induction Field (IF) method was employed at two bridges with two piles, one in Colorado and one in Texas (with this bridge, 22 bridges were tested). The results from the Texas bridge are described separately and were not included in Table VI because no bridge plans are available. Detailed presentations of the field NDT results of Phase II research can be found in the interim report submitted in April, 1998 (32). The initial blind investigation of the NDT data foundation depths and the post processed evaluation are reported in Table VI and in the next two sections.

4.1.1 Accuracy of Initial Blind NDT Bridge Foundation Depth Predictions.

1- Parallel Seismic Method.

Depths of 12 out of the 19 bridge substructures tested with the Parallel Seismic method were predicted to within ± 13% of the actual bottom of foundation depths. Bottom foundation depths of the other 6 bridges were incorrectly predicted.

2- Ultraseismic Method.

Depths of 8 out of the 18 bridges were predicted within ± 14% of the actual depths. Depths of the other 4 bridges were incorrectly predicted. Ultraseismic test results were inconclusive on 6 out of the 18 bridges tested with the US method.

3- Sonic Echo/Impulse Response Method.

Only one bridge in North Carolina showed an echo which corresponded to the tip of the pile. Results were inconclusive on 14 out of the 15 bridges tested with the SE/IR method.

Table VI- Summary of Predicted Depths.

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Wilson County Bridge # 5, Bent 1, East Pile, North Carolina	7.7 (25.4)	8.1 (26.6) (Geophone) 8.1 (26.6) (Hydrophone)	7.6 (25) (upgoing waves)	Inconclusive, Possible echo from SE tests at a depth of 8.0 m (26.2 ft)	Inconclusive	Top of timber pile	+5 (-2, based on Ultraseismic results)	Excellent agreement between predicted and actual depth
Johnston County Bridge # 33, Bent 1, East Pile, North Carolina	10.8 (35.3)	8.0 (26.3) (Geophone) 8.0 (26.3) (Hydrophone)	7.9 (26) (upgoing waves)	Inconclusive	Inconclusive	Top of timber pile	-25 (-26, based on Ultraseismic results)	Surprising results: the measured velocity below a depth of 8.0 m (26.3 ft) was equal to 1,490 m/sec (4,900 ft/sec)
Wake County Bridge # 251, North Carolina	5.7 (18.7)	6.5 (21.2) (Geophone) 6.5 (21.2) (Hydrophone)	Inconclusive	Tests not performed	Tests not performed	Bottom of Pilecap (top of timber pile)	+13	Good agreement between predicted and actual depth, Pile not accessible, pilecap exposed through excavation
Bridge No. 5188, Minnesota Hwy 58, Zumbrota, Minnesota	9.4 (31)	Not performed	7 (23) first reflector 9.3 (30.5) second reflector	Inconclusive	Tests not performed	Top of Pier	-25 (blind, based on Ultraseismic results) *-2 (post processing, based on Ultraseismic results)**	During our first evaluation, the second reflector depth was not reported. Borehole not installed: No Parallel Seismic tests

** Post Processed Predictions*

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Bridge No. 9798 on Hwy 60, Minnesota	12.2 (40)	11.1 (36.5)	11.1 (36.6)	Inconclusive	Inconclusive	Top of pile/bottom of timber beam	-9 (-9, based on Ultraseismic results)	Good Agreement between predicted and actual depth; Geophone datasets not collected because of borehole diameter
Bridge # 55028, Hwy 52, North of Chatfield, Minnesota	10.1 (33.2)	Pile is deeper than borehole	Inconclusive	Inconclusive	Inconclusive	Top of concrete-filled steel pipe pile	Successful, predicted that pile is deeper than borehole	Excellent evaluation
North Abutment , Structure # 1103-151, Route 1 over Shipetaukin Creek, Lawrence Twp., Mercer County, New Jersey	3 (9.85)	5.9 (19.5) (Horizontal Hit, Hydrophone Recording) 3.2 (10.5) (Vertical Hit, Hydrophone Recording)	Inconclusive (very weak echoes were reported in quarterly report)	Inconclusive (echoes were reported in quarterly report, may be due to reflections of surface waves from boundaries	Tests not performed	Abutment Seat	+96 (blind) *+7 (post processing)**	Vertical hammer hit was not evaluated during the first round. Geophone datasets not collected because of borehole diameter

* *Post Processed Predictions*

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Structure # 1123-152, Route 130 over Rock's Brook, East Windsor Twp., Mercer County, New Jersey	5.24 (17.2)	5.49 (18) (Hydrophone)	5.72 (18.75) Reported at 6.63 m (21.75 ft) in quarterly report	Inconclusive	Tests not performed	Top of borehole, which is the same as top of roadway	+5 (If the predicted depth was equal to 6.7 m (22 ft), then the percentage difference would be equal to +28 (blind); +9 (based on Ultraseismic results) *+5 (post processing)**	Depth is mistakenly stated at 6.7 m (22 ft) in quarterly report, because the signals were not sufficiently amplified Geophone datasets not collected because of borehole diameter Excellent agreement between predicted and actual depth
Structure # 0324-158, Route 206 over the North Branch of the Rancocas Creek, Pemberton Twp., Burlington County, New Jersey	6.65 (21.8)	7.31 (24) (Hydrophone)	Tests not performed	Tests not performed	Tests not performed	Top of roadway	+10	Good agreement between predicted and actual depth Geophone datasets not collected because of borehole diameter
Structure # B01 of 33045A Bridge on I496 over Red Cedar River, Michigan	15.3 (50.2)	15.36 (50.4) (Hydrophone)	Inconclusive	Inconclusive	Tests not performed	Top of plint	0	Excellent agreement between predicted and actual depth Geophone datasets not collected because of borehole diameter

* *Post Processed Predictions*

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Structure # B04 of 64015E, Bridge on US31 over Pentwater River, Michigan	13.26 (43.5) Tip of pile 7.77 (25.5) bottom of pile cap	15.76 (51.7) (Hydrophone)	Inconclusive for pile depth determination 7.62 (25) for bottom of pile cap	Inconclusive	Tests not performed	Top of Pier Cap	+19 (for pile tip from PS tests) -2 (for bottom of pile cap based on Ultraseismic results)	Unsuccessful for pile tip determination (PS tests), excellent agreement between predicted and actual for bottom of pile cap (from US tests) Geophone datasets not collected because of borehole diameter
Structure # X01 of 25085B Bridge over Fenton Road, Thread Creek, Michigan	9.97 (32.7)	14.2 (46.5) (Hydrophone)	13.78 (45.2) 9.75 (32) not reported in quarterly report	Inconclusive	Tests not performed	Top of Pier Cap	+42 *-2 (based on an echo in Ultraseismic tests not reported in quarterly report)**	Unsuccessful, may be due to variable soil velocities and no saturation Geophone datasets not collected because of borehole diameter
Santiam River Overflow # 5 Bridge, Near Salem, Oregon	11.3 (37)	11.9 (39) (Geophone) 11.6 (38) (Hydrophone)	Tests not performed	Tests not performed	Tests not performed	Top of Pavement	+4	Excellent agreement between predicted and actual depth

** Post Processed Predictions*

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Cordon Road Overcrossing Hwy. 22 Bridge, Near Salem, Oregon	17.8 (58.4) tip of pile 8.35 (27.4) bottom of pilecap	13.99 (45.9) (Geophone, blind evaluation) 17.7 (57.9) Re-evaluated	13.3 (43.6) 8.6 (28.2) not reported in quarterly report, echo from bottom of pilecap	Tests not performed	Tests not performed	Top of Column	-21 (blind) *-0.8 (post processing)** -25 (blind, based on Ultraseismic results) *+3 (for bottom of pile cap based on Ultraseismic results)**	During our initial evaluation, we relied on the drop of amplitude of compression waves Excellent agreement after re-evaluation based on drop in amplitude of shear waves below tip of pile, clear indication of the bottom of the pile; An echo from the bottom of the pilecap was not reported in the quarterly report
Bridge on Dudley Road, Bent 1, Pile 1B, Structure # 0-6-11, Oxford Massachusetts	12.4 (40.8)	7.62 (25) (Geophone) Not very clear	Inconclusive Limited access	Tests not performed	Tests not performed	Top of pavement	-39	Unsuccessful Very weak signals borehole is about 8 ft away from the pile

** Post Processed Predictions*

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Bridge on Route 122, Structure # U-2-21, Uxbridge, Massachusetts	6.28 (20.59)	6.1 (20) (Geophone)	5.4 (17.7)	Tests not performed	Tests not performed	Top of sidewalk	-3 -14 (based on Ultraseismic results)	Excellent agreement between predicted and actual depth No indication of piles below abutment because of the massive nature of the abutment and the borehole being 12 ft away from the abutment face
Johnston County Bridge # 129, Bent 2, Pile 6, North Carolina	4.29 (14.1)	4.05 (13.4) Hydrophone 3.87 (12.7) Geophone	3.96 (13)	Inconclusive	Inconclusive	Bottom of beam/top of pile	-8 -8 (based on Ultraseismic results)	Good agreement between predicted and actual depth
Johnston County Bridge # 145, Bent 1, Pile 5, North Carolina	4.8 (15.7)	4.8 (15.7) Hydrophone 4.8 (15.7) Geophone	4.6 (15)	Inconclusive	Inconclusive	Bottom of beam/top of pile	0 -4 (based on Ultraseismic results	Excellent agreement between predicted and actual depth

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Wake County Bridge # 207, Bent 4, Pile 1, North Carolina	±7.6 (±25)	Tests not performed	7 (23)	7.22 (23.7) Sonic Echo 6.9 (22.6) Impulse Response	Inconclusive	Bottom of beam/top of pile	-8 (based on Ultraseismic results)	Good agreement between predicted and actual depth Actual depth is based on borehole drilling information Borehole collapsed, no Parallel Seismic tests were performed
Bridge on US 287, Structure # C-16-C, over Little Thompson River, near Longmont, Colorado	10.5 (34.5)	9.94 (32.6) Hydrophone 9.66 (31.7) Geophone	Not Performed	Inconclusive	Tests not performed	Bottom of beam/top of pile	-7	Good agreement between predicted and actual depth **Induction Field (IF)** tests were performed at this bridge; IF test results showed that the depth of the foundation is equal to 9.66 m (31.7 ft)

Table VI- Summary of Predicted Depths (Cont.).

Bridge	Actual Depth m (ft)	Parallel Seismic predicted depth, m (ft)	Ultraseismic predicted depth, m (ft)	Sonic Echo/Impulse Response predicted depth, m (ft)	Bending Wave predicted depth, m (ft)	Reference (zero depth)	Percentage Difference (%) (between actual and PS predicted)	Comments
Bridge on US 52, Structure # D-17-I over South Platte River, near Fort Lupton, Colorado	11.8 (38.6)	9.2 (30.2) Geophone	Inconclusive	Inconclusive	Tests not performed	Top of pier	-22 (blind) *0 (possible bottom based on drop of amplitude after receiving feedback from Colorado DOT**	Unsuccessful There is another indication for a drop in amplitude 8 ft below the predicted depth; Difficult to identify the bottom of the foundation, data is not very clear

** Post Processed Predictions*

4- Bending Waves Method.

Results were inconclusive on all 7 timber pile bridges tested with the BW method.

5- Induction Field Method.

Tests were performed on two bridges, one in Colorado and one in Texas. The IF results showed a drop in amplitude below the tip of the steel pile at the Colorado bridge with good agreement with the depth predicted from Parallel Seismic tests. The IF results at the steel-pile bridge in Texas were inconclusive.

6- SASW Method.

Test results performed at two bridges, one in New Jersey and one in Michigan, were inconclusive. Due to limited lateral access, wavelengths in SASW tests greater than the depth of the wall-shaped foundations were not generated, thus, the results are considered to be inconclusive.

4.1.2 Accuracy of NDT PS and US Bridge Foundation Depth Predictions after Receiving Feedback and Re-Evaluating the Data (Post Blind Analysis).

After receiving feedback from the State DOT's on our first evaluation and detail foundation depths, the test results were reviewed again to check if there were any indications of foundation bottoms that were mis-interpreted in the data during the initial evaluation. A summary of the post blind depth prediction results after the second evaluation is given below for the Parallel Seismic and Ultraseismic results where significant interpretation improvements were able to be made.

1- Parallel Seismic Method.

Depths of 16 out of the 19 bridges tested with the PS method were predicted to within ± 13% of the actual depths. There was no indication in the PS test results to support the actual depths for the remaining three bridges.

2- Ultraseismic Method.

Depths of 11 out of the 18 bridges were predicted within ± 14% of the actual depths. Test results were inconclusive on 6 out of the 18 bridges tested with the US method. There was no indication in the US data to support the actual depth reported for the Johnston County Bridge # 33.

Factors which may have contributed to the initial blind wrong PS and US predictions of depths included:

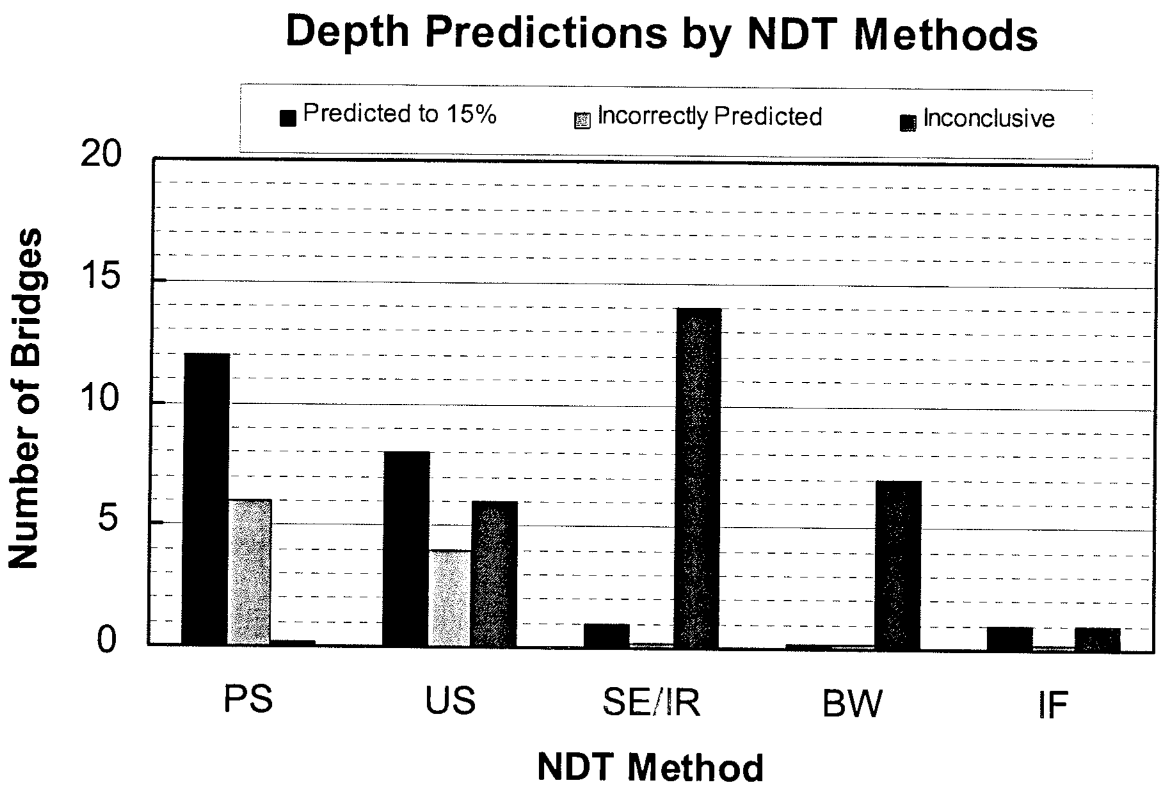
1- The PS boreholes being away from the foundations and not in alignment with the piles supporting a pier or abutment pilecap,

2- In some cases, the piles were inclined. This will add an extra variable as the distance between the foundation and the borehole is not constant. Also the verticality of the borehole can play an important role, especially in highly varying soil velocities,

3- The boreholes were not grouted properly. As a result, the Parallel Seismic test signals were extremely weak,

4- In some bridges, the analysis of the Parallel Seismic and Ultraseismic data analysis were performed in conjunction based on two weak indicators to produce the same depth. It is recommended that the data be analyzed separately for each test. For example, in a bridge abutment supported by a pilecap and piles, the PS tests may be used to obtain the depth of the supporting piles and the US tests may be used to obtain the bottom of the pilecap.

Descriptions of bridge sites, performed tests, and results are presented in the Interim Report submitted in April, 1998 along with graphical presentation of the results and the parameters used for data interpretation. The data was interpreted without knowledge of subsurface conditions from any borehole logging information or the knowledge of the surrounding soil velocities. Also the data was interpreted without any knowledge about the possible existence of piles underneath a bridge abutment or a bridge pier. If some of these parameters are available, they would assist in better interpreting the data from both US and PS tests.



Total number of bridges tested with the PS method = 19 bridges
Total number of bridges tested with the US method = 18 bridges
Total number of bridges tested with the SE/IR method = 15 bridges
Total number of bridges tested with the BW method = 7 bridges
Total number of bridges tested with the IF method = 2 bridges (including Texas bridge)

Figure 21- Graphical Summary of Accuracy of Foundation Depth Predictions Based on Blind Evaluations of the Field Data.

4.1.3 Graphical Presentation of NDT Initial Blind Results.

Figure 21 shows a graphical presentation of the accuracy of predicted depths based on initial blind evaluations of the NDT data for all of the methods except SASW. The Spectral Analysis of Surface Waves (SASW) test results were inconclusive due to the limited lateral extent of the two tested bridges relative to their depths where testing was performed. Wavelengths greater than the depth of foundations were not generated due to the limitations imposed by the bridge geometry, thus, the depth of the foundations could not be predicted. Consequently, the SASW results are not included in Fig. 21.

As shown in Fig. 21, the Parallel Seismic (PS) method was the most successful method with 12 foundation depth predictions accurate to within 15% and 6 incorrect depth predictions. The PS borehole test has the broadest application for predicting the depth of unknown bridge foundations.

The Ultraseismic (US) method was also effective in predicting the depth of specific types of foundations, including columnar-shaped piles and wall-shaped foundations such as bridge abutments and wall piers. However, the surface US method cannot detect the presence or predict the depth of piles underneath a buried pilecap, because the reflections from the bottom of the pilecap are dominant in the response of the foundations to impact loading. Thus, 8 foundation depths were accurately predicted within ±15% of actual depths, 4 foundations depths were incorrect and 6 foundations produced inconclusive results with the US method (see Fig. 21).

Sonic Echo/Impulse Response and Bending Waves tests were mostly inconclusive on all foundations tested with 1 accurate depth, no incorrect depths and 14 inconclusive results for the SE/IR tests and 7 inconclusive results for BW tests. This is largely due to the dominant reflections from the structural members of the superstructure /substructure and reflections from soil/foundation boundaries. Interpretation of SE/IR and BW data relies on only one or two traces which makes it extremely difficult to interpret the results in the common case of many reflecting boundaries in bridge substructures. It should be noted that the multiple receiver locations of the US test and geophysical processing resulted in much clearer identification of compression wave echoes than the SE/IR tests did as well as for bending (flexural) wave echoes than the BW tests did.

The Induction Field (IF) method was accurate on one bridge in Colorado. The IF results on a Texas bridge were inconclusive due to a possible direct electrical connection between the test pile and the reference pile. This direct connection resulted in an unusual magnetic field down the test borehole.

4.1.4 Graphical Presentation of NDT Post Blind Results.

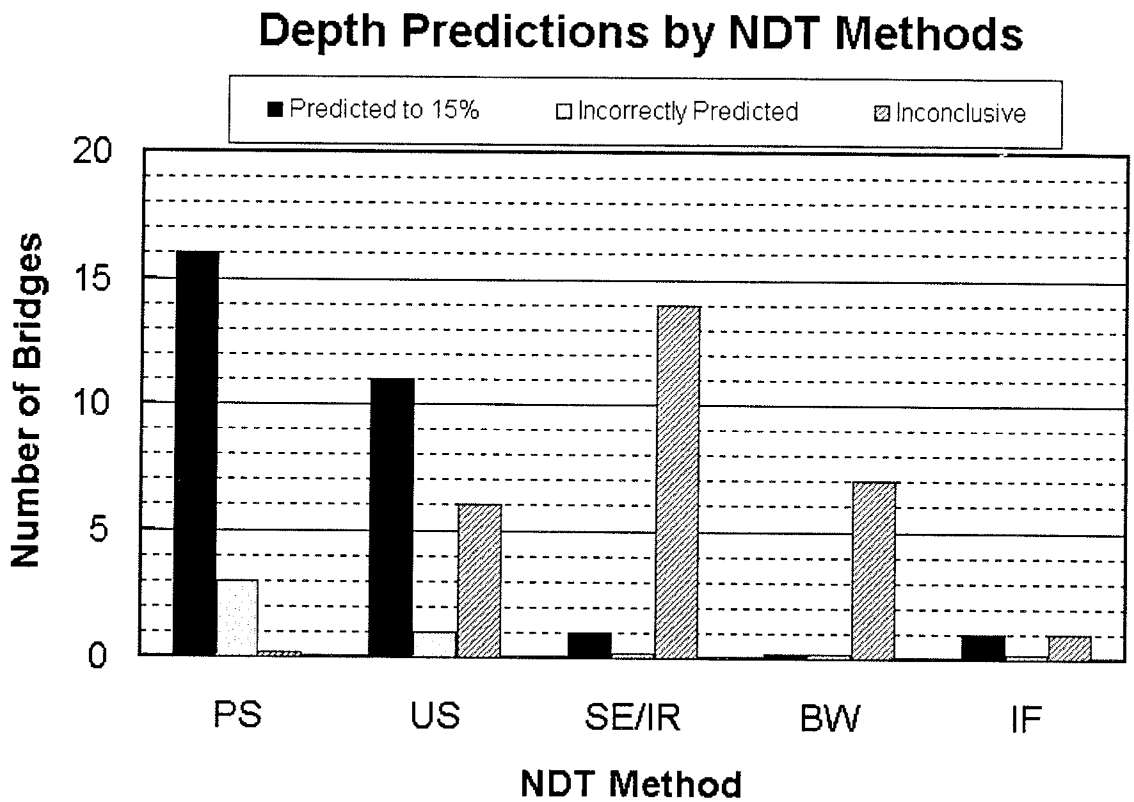
The field results were re-evaluated after receiving feedback from State DOT's. Figure 22 shows the foundation depth predictions based on the post-processed results. No improvement was made in the interpretation of the SE/IR and BW results as shown in Fig. 22. Reflections based on the actual known depth of the foundations were not identified in the SE/IR and BW field records.

Post blind analyses of the Parallel Seismic data showed that foundation bottom depths were apparent for 3 additional bridges which raised the total of accurate predictions to 16. The post blind

interpretation was based on additional amplification of the PS data for massive abutments at the New Jersey bridges and identification of drop of amplitudes in the shear wave arrivals at the Longmont Bridge in Colorado. However, foundation depths were still not clear in the Parallel Seismic results of 3 bridges. The criteria for interpreting PS results such as change in velocities or drop in amplitudes below the bottom of foundations were not satisfied at these 3 bridges at depths corresponding to the actual depths of the foundations.

Post blind analyses of the US data showed that foundation depths were apparent for 3 additional bridges with echoes not reported in the quarterly reports due to a combined analysis of the PS and US data. However, foundation depths were still not clear in US results of 1 bridge and were still inconclusive for 6 bridges. This is mainly due to dominant reflections from structural elements above ground and foundation-ground interfaces and in some cases were due to the limited access where US tests were performed (limited receiver locations).

Post blind analyses of the IF data did not show any more accurate identification of the depth of the steel BP piles in Texas. This was expected because of the poor data quality recorded at this site due to unexpected conductive paths which distorted the magnetic field.



Total number of bridges tested with the PS method = 19 bridges
Total number of bridges tested with the US method = 18 bridges
Total number of bridges tested with the SE/IR method = 15 bridges
Total number of bridges tested with the BW method = 7 bridges
Total number of bridges tested with the IF method = 2 bridges (including Texas bridge)

Figure 22- Graphical Summary of Accuracy of Foundation Depth Predictions Based on Evaluations after Receiving Foundation Details from State DOT's.

One of the most important pieces of information gained from the comparison of blind predictions and the post-processed predictions is that analysis of the Parallel Seismic and Ultraseismic data should be performed independently and based on dominant features in the records of each test so that neither data set unduly influences the initial interpretation of the data. An example of such an analysis approach is the case of a bridge with a massive pilecap supported by piles. In this case, the Parallel Seismic results would best be used to determine the length of the piles and the Ultraseismic results may be used to determine the bottom depth of the pilecap. It is also important to note that the initial blind data evaluation was performed without any knowledge of the foundation type and surrounding soil/rock properties. Because of the complexity of the unknown bridge foundation problem, knowledge of the expected foundation type and the surrounding soil/rock properties would assist in the evaluation of the field NDT data.

4.2 THEORETICAL MODELING RESULTS.

To better understand the experimental results, one bridge with a pilecap supported by timber piles (Wake County bridge # 251 in North Carolina) was selected for theoretical modeling.

One of the crucial parameters needed for the theoretical modeling is the shear wave velocities of the surrounding soil. Spectral Analysis of Surface Waves (SASW) tests were performed on the ground adjacent to the bridge bent tested with the NDT methods as discussed in the Section 4.2.1.

4.2.1 SASW for Soil Shear Wave Velocity Profile

Receiver spacings ranging from 0.6 m (2 ft) to 9.75 m (32 ft) were used at the site. A representative phase shift of the cross power spectrum between the two receivers spaced at 9.75 m (32 ft) at the Wake County bridge # 251 and the corresponding coherence function are shown in Fig. 23. The variation of surface wave velocity with wavelength, termed as a dispersion curve, can be easily determined from Fig. 23 and similar figures for other receiver spacings. The experimental dispersion curve determined from SASW measurements performed at the Wake County bridge # 251 is shown in Fig. 24. After forward modeling of the SASW data, a theoretical dispersion curve closely matching the experimental dispersion curve is determined based on an assumed shear wave velocity. Figure 25 shows a comparison between the experimental and theoretical dispersion curves from SASW tests at the Wake County bridge # 251. A summary of the shear wave velocity profiles determined at the Wake County bridge # 251 is presented in Table VII below. The shear wave velocity of 350 ft/sec corresponds to loose/soft soils while the velocity of 850 ft/sec is that of a medium dense/medium stiff soil.

Table VII- **Shear Wave Velocity Profiles Determined from SASW Measurements Performed at Wake County Bridge # 251, North Carolina.**

Bridge	Layer Thickness (ft)	Shear Wave Velocity (ft/sec)
Wake County Bridge # 251	3	500
	2	350
	10	650
	10	850

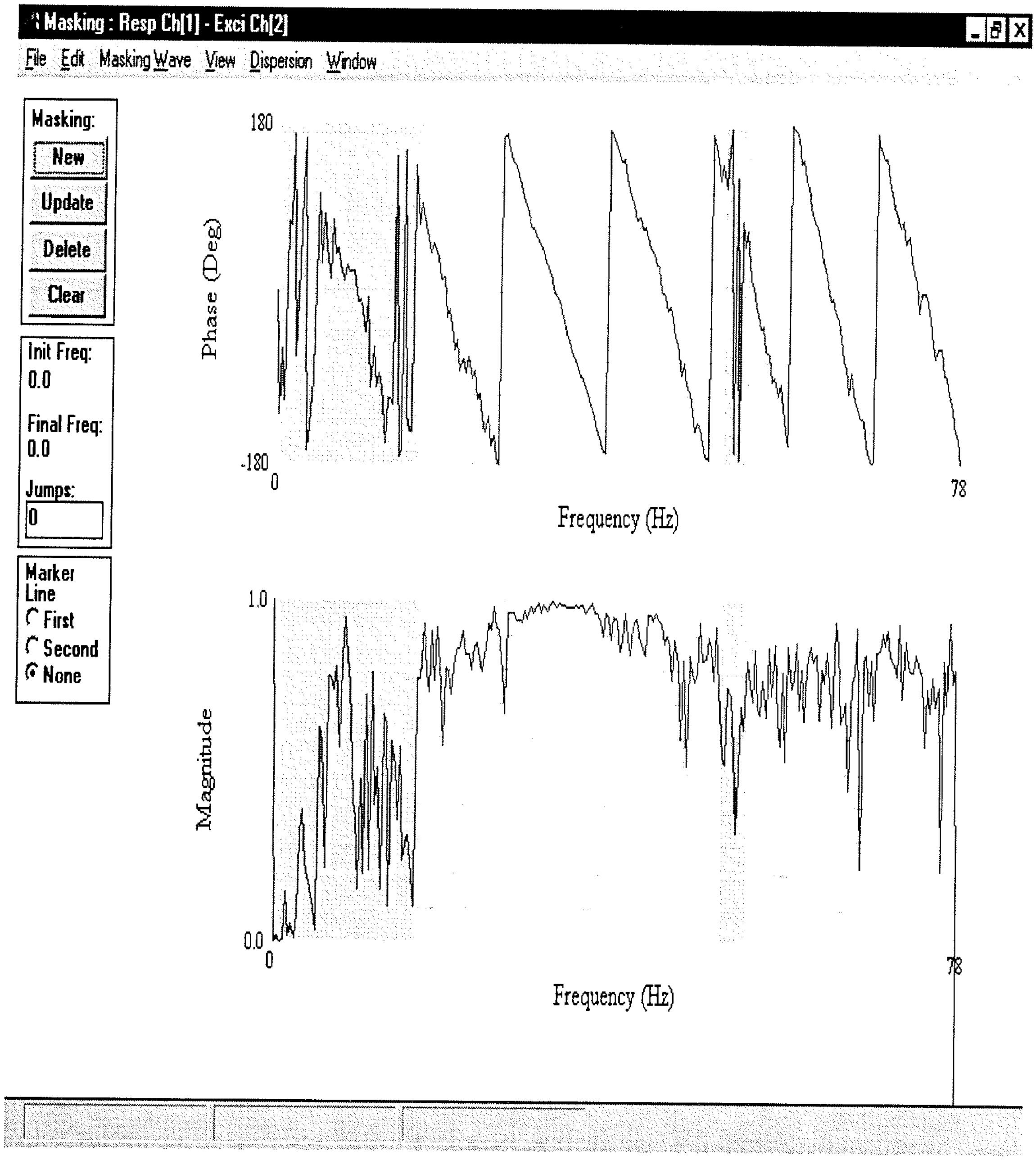


Figure 23- Phase Shift of the Cross Power Spectrum and Coherence Function, R1-R2 = 32 ft, Wake County Bridge # 251, North Carolina.

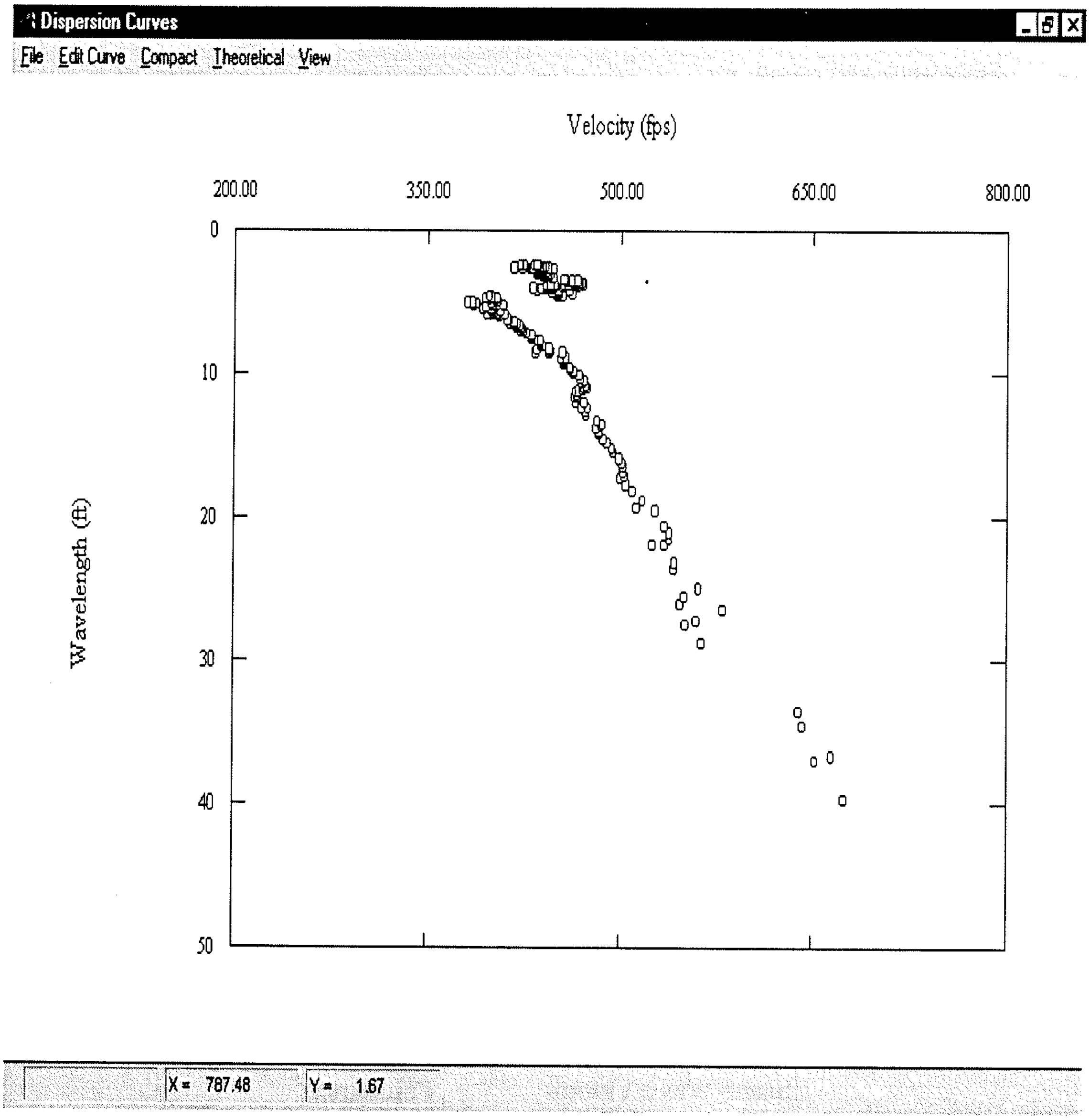
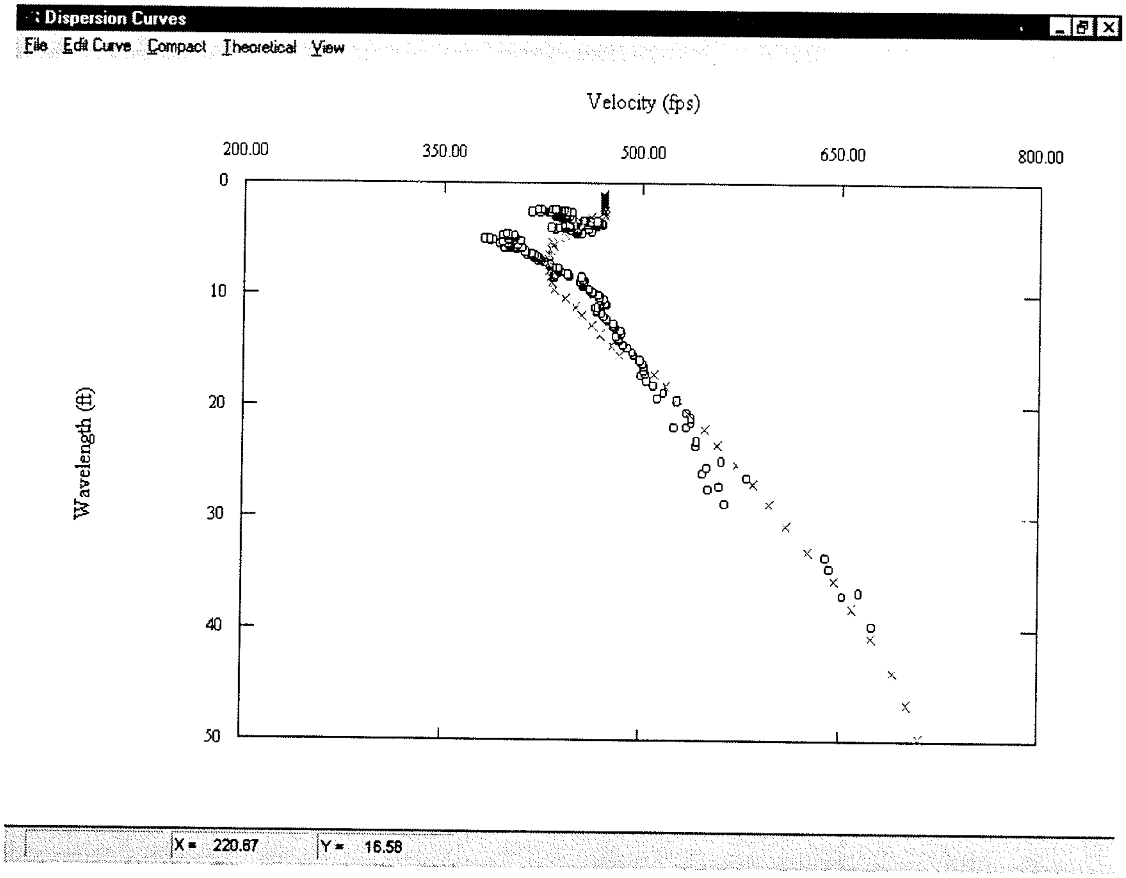


Figure 24- Experimental Dispersion Curve Determined from SASW Measurements Performed on the Surrounding Ground of Wake County Bridge # 251, North Carolina.



Shear Wave Velocity Profile Used in Determining the Theoretical Dispersion Curve:

Shear Wave Velocity (ft/sec)	Thickness (ft)
500	3
350	2
650	10

Figure 25- Comparison between Theoretical and Experimental Dispersion Curves Wake County Bridge # 251, North Carolina.

4.2.2 Geometry For Theoretical Modeling.

The Wake County # 251 North Carolina bridge was selected for modeling because its foundation consist of a pier column and a pilecap supported by timber piles. The modeling was done to further assess the capability of the Parallel Seismic and Ultraseismic methods in predicting unexposed pile depths existing underneath a pilecap. Figure 26 shows the geometry used to collect the field data.

4.2.3 Theoretical Modeling Results.

4.2.3.1 Parallel Seismic Modeling. Parallel Seismic tests at the Wake County bridge # 251 were simulated using a 3-D axi-symmetric program developed at the University of Texas at Austin by Drs. Jose Roesset and Shu-Tao Liao. Only the pilecap and the piles were considered in the modeling process. In the Parallel Seismic modeling, the geometry shown in Fig. 26 was modified to produce acceptable data input for the axisymmetric computer program. The rectangular pilecap and the 6 timber piles were represented by a circular footing and a circular pile rotating around the axis of symmetry. The pile foundation was represented by a single pile and the soil shear wave velocities and other properties were based on the SASW results and are summarized in Table VIII below. The superstructure was not considered in the modeling. The vertical impact was assumed to be on top of the pier column in the theoretical model with an assumed half cycle of a sine wave loading with a maximum force of 5,000 lbs for a duration of 3 milliseconds.

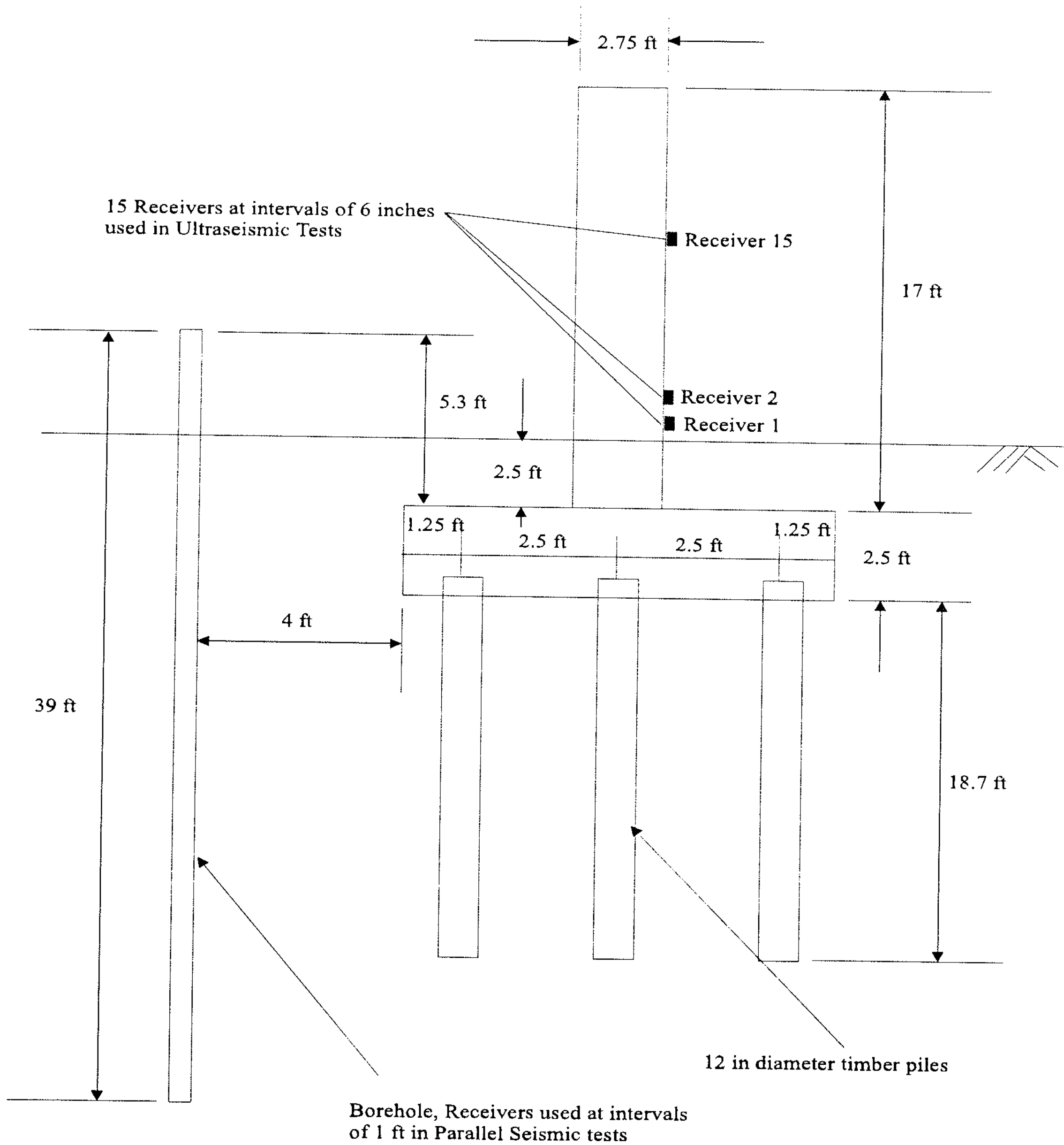


Figure 26- Geometry Used in the Parallel Seismic and Ultraseismic Field Testing Performed at the Wake County Bridge # 251 (Modified for Theoretical Modeling), North Carolina.

Table VIII- Summary of Material Properties Used in the Parallel Seismic Modeling.

Pilecap and Column Properties		Modulus (psi)	Unit Weight (pcf)	Poisson's Ratio	
		4.8 E6	144	0.2	
Soil Properties	Layer	Thickness (ft)	Modulus (psi)	Unit Weight (pcf)	Poisson's Ratio
	I	2	0.55E4	100	0.3
	II	10	2.08 E4	110	0.3
	III	25	3.57 E4	110	0.3

Figure 27 shows the theoretical results for Parallel Seismic (PS) modeling performed for the Wake County bridge # 251. Note that Trace 1 is for a receiver located in the borehole at the same elevation as the top of the pilecap. A pile length of 5.9 m (19.5 ft) was calculated based on the change of velocity below the tip of the pile to correspond to the compression wave velocity of soil of 550 m/sec (1,800 ft/sec). When compared to the experimental results, the theoretical results looked similar with the exception that dry soil conditions were assumed in the modeling and the experimental soils were saturated. A Poisson's ratio of 0.3 was assumed and a compression wave velocity was calculated from the measured shear wave velocity in the theoretical modeling (saturation conditions can also be modeled, but were not used). The theoretical modeling of PS tests showed that the modeling can help in understanding and interpreting the experimental data.

Comments:
Depth shown in Figure is depth below top of pilecap
Velocity of soil below a depth of 22 ft
Bottom depth = 22 ft (reference is top of pilecap)
Top of pilecap is 2.5 ft above top of pile
Pile length = 22 - 2.5 = 19.5 ft (reference is top of pile)

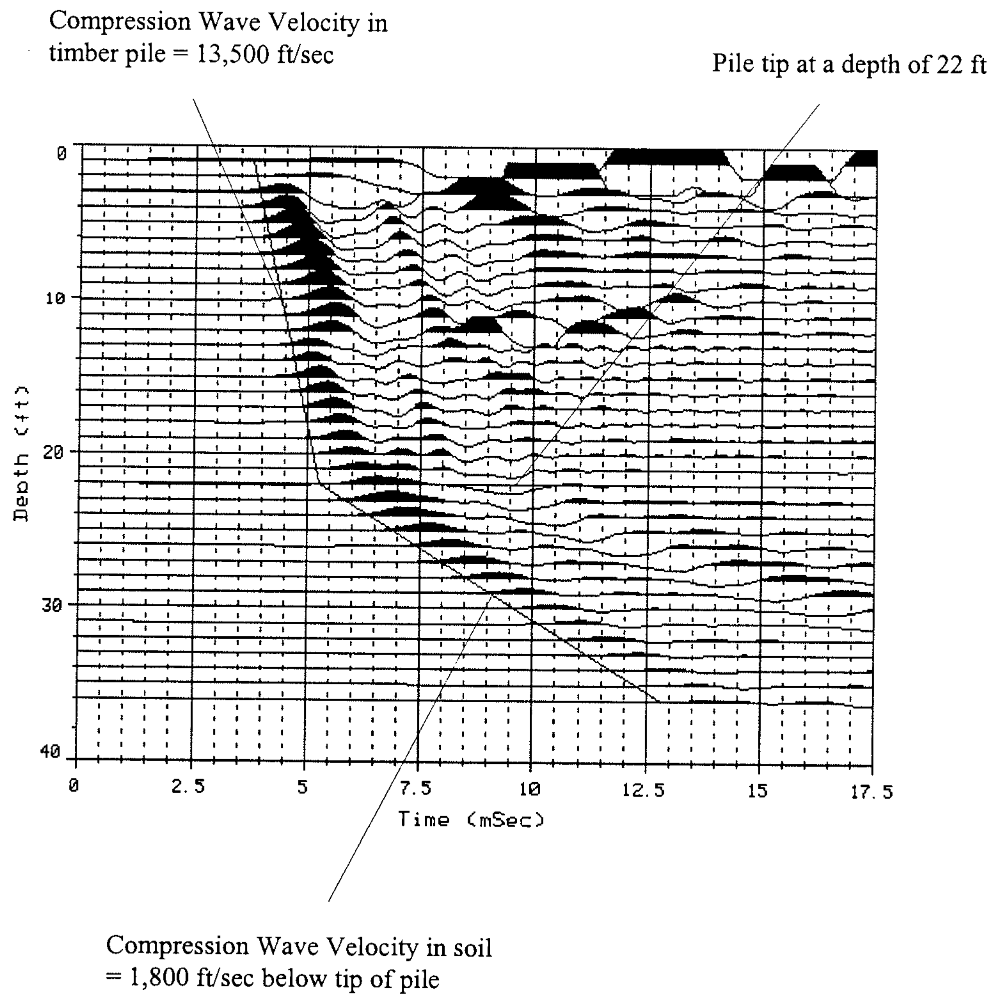


Figure 27- Parallel Seismic Data from a 12-lb Horizontal Hammer Hit and Vertical Geophone Component (Theoretical Modeling) Wake County Bridge # 251, North Carolina.

4.2.3.2 Ultraseismic Modeling. Ultraseismic tests at the Wake County bridge # 251 were simulated using a 2-D frequency domain program developed at the University of Texas at Austin by Drs. Jose Roesset and Chih-Peng Yu. The 2-D program was selected because it allows for horizontal hammer hits required to simulate the Ultraseismic tests. The geometry used for the Ultraseismic model is shown in Fig. 26 with Receiver 1 being at 15 cm (6 in.) above the ground surface and with receiver intervals of 15 cm (6 in.). The hammer hit was located horizontally at the ground level. Figure 28 shows the Ultraseismic results obtained from the theoretical model. A bending wave velocity from the initial slope of the bending wave arrival was calculated to be 1,370 m/sec (4,500 ft/sec). A reflection at a $\Delta t = 2.3$ ms was identified in the records which corresponds to a depth of 1.6 m (5.2 ft) below Receiver 1. This reflection was identified as a reflection from the bottom of the pilecap which is actually at a depth of 1.7 m (5.5 ft) below Receiver 1. Another strong reflection of downgoing waves from the top of the pile was identified at a $\Delta t = 6.5$ ms at Receiver 1 which corresponds to an elevation of 4.5 m (14.6 ft) above Receiver 1. The reflection was identified as a reflection from the top of the concrete column, which is actually 4.3 m (14.0 ft) above Receiver 1.

The Ultraseismic modeling results showed that the two reflections from the pilecap and the top of the concrete column were dominant and it is difficult to identify reflections from other boundaries such as the tip of the timber pile. This would be one of the limitations of the US method in identifying the existence or the length of the piles underneath a pilecap.

Comments:
Depth shown in Figure is height above ground surface
Two reflectors: one from the bottom of the pilecap at a $\Delta t = 2.3$ ms and another reflection from the top of the concrete column at $\Delta t = 6.5$ ms for Receiver 1

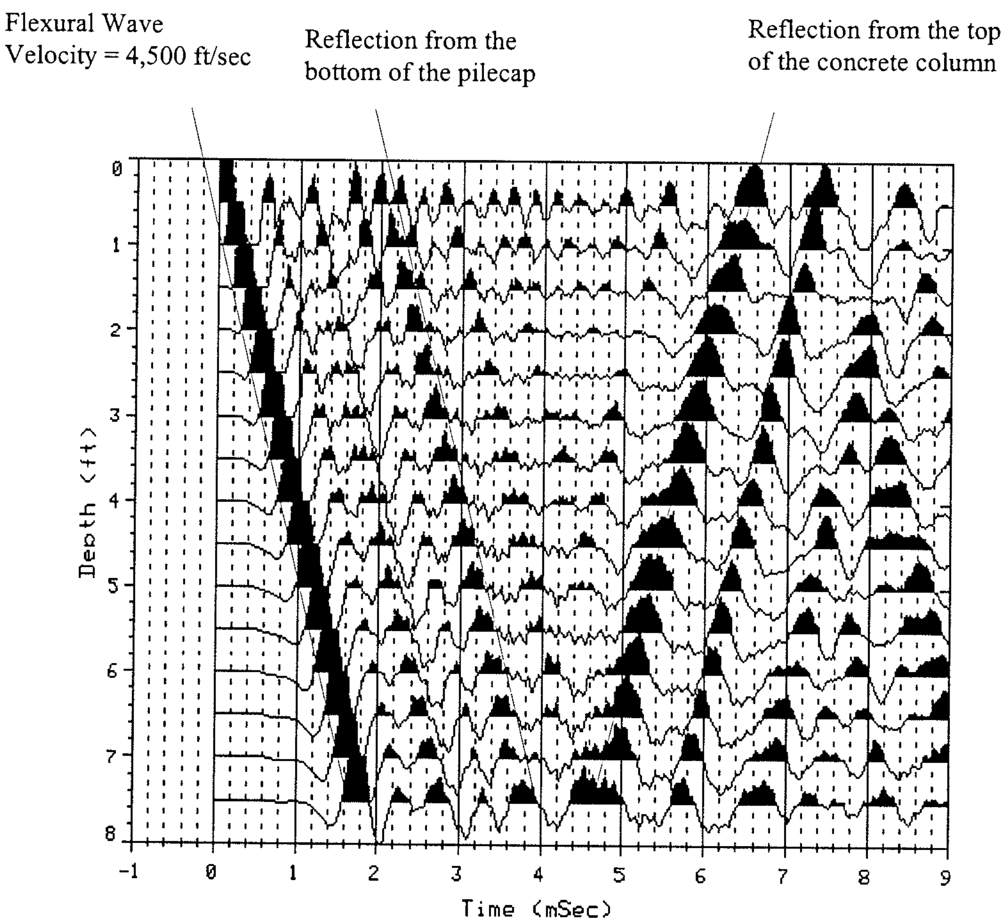


Figure 28- Ultraseismic Test Results Obtained from Theoretical Modeling Wake County Bridge # 251, North Carolina.

The experimental results indicated that the Bending Waves (BW) and the Sonic Echo/Impulse Response (SE/IR) methods were problematic at most of the sites where testing was performed. It should be noted that during BW and SE/IR tests, one is relying on only one trace to interpret the test results. Reflections from boundaries above the ground surface tend to obscure reflections coming from the bottom of the foundations.

4.3 SOFTWARE DEVELOPMENTS.

As part of this research project, software for the Parallel Seismic (PS), Ultraseismic (US), Sonic Echo/Impulse Response (SE/IR), Bending Waves (BW) and Spectral Analysis of Surface Waves (SASW) data analysis has been developed. The software included enhancements to the TFS software package previously developed by Olson Engineering for SE/IR, BW, SASW and PS data analysis for applications to unknown bridge foundation depth determination. The Bridgix software package was developed by Interpex House of Golden, Colorado under the direction of Olson Engineering and is devoted to PS and US data analysis with geophysical data processing techniques for multiple channel data.

4.3.1 Software Description.

All the data plots presented in this report for the above methods were produced using the TFS and the Bridgix software. To better understand how the software works, example results and user manuals for both the TFS and Bridgix software were presented in the Interim Report of April, 1998 (32). Discussions of the program capabilities are presented below.

4.3.1.1 TFS Software for SE/IR, BW, SASW and PS NDT Methods. The TFS software was first developed by Olson Engineering, Inc. for its own internal use and data analysis of stress wave based nondestructive testing data that was taken with a PC. In order better perform the NCHRP 21-5 (2) research, it was proposed to build on the TFS code for NDT methods that were applicable to determination of unknown bridge foundation depths. The TFS software runs under the Microsoft DOS 6.22 operating system, or from a DOS prompt in Microsoft's Windows software. Software modules include SE, SE Averaging, IR (compression and bending wave analysis), BW (short kernel method), SASW, and PS data acquisition and analysis capabilities. The software was written to analyze digital data (hexadecimal format) from 1 to 8 channels of source and/or receiver voltage inputs from the NDT equipment with time, frequency and transfer function analyses. The time-voltage data was gathered with an RC Electronics, Inc. (Santa Barbara, California) Computerscope data acquisition card with a maximum sampling rate of 1 microsecond per point and 12 bit Analog/Digital data conversion accuracy.

A digital oscilloscope program supplied for the RC card was used to obtain the raw data during the research, and then analyzed with the TFS software. Significant additional advancements include the following: data acquisition and analysis within the TFS software, semi-automatic testing modes, and adaptation of the TFS software to a National Instruments (Austin, Texas) data acquisition card.

4.3.1.2 Bridgix Software for PS and US NDT Methods. This software differs from the TFS software in that it is not used to take the data, but only for analysis. The raw data can be recorded by equipment that uses several different geophysical data formats, as well as the RC Computerscope or TFS data format. The software allows for plotting of multiple receiver locations of uni-axial, bi-axial and tri-axial receiver data to track upgoing and/or downgoing wave energy in PS and US data. Geophysical processing operations include lowpass, bandpass and highpass filtering, f-k processing, averaging, signal amplification, data integration from acceleration to velocity to displacement units, and establishment of elevation references to predict foundation depths. Bridgix runs under DOS and is best used on a 486 or faster microprocessor with a VGA or better color screen.

4.4 HARDWARE DEVELOPMENTS.

Equipment research and development was proposed and performed during NCHRP 21-5 (2) because of the lack of commercially available equipment for some of the NDT methods applicable to determination of unknown foundation depths. Discussions are presented below of the prototype instrument PC, Parallel Seismic hydrophone receiver string, and Induction Field equipment, as well as evaluations of the PCB Piezotronics triaxial accelerometer and repeatable source.

4.4.1 Prototype Instrument PC.

Because of the desire for a more universal, field portable NDT system for unknown foundation depth determination, a portable instrument PC was researched and developed. A photograph of the prototype PC is shown in Fig. 29. The prototype PC was designed as a Single Board Computer (486DX2-66) plugged into an ISA backplane chassis where the data acquisition card was installed. The screen is an LCD monochrome VGA 640x480 backlit display that provides

for excellent viewing in sunlight to dark conditions. The PC includes a covered keyboard, internal harddrive and floppy drive, and can be operated by internal batteries, an external 12 VDC source such as the cigarette lighter of a vehicle, or a 110/220 VAC power source with an external power supply. The computer hardware is mounted in a ruggedized case which is water and air tight when closed. The PC can be connected to an external color monitor and also has serial and parallel ports. The operating system is Microsoft DOS 6.22, or it can be run with Microsoft's Windows software.

4.4.2 Prototype Parallel Seismic Hydrophone Receiver String.

The construction of an 8 channel hydrophone string was undertaken to provide a highly sensitive receiver string for detection of weak compression wave arrivals traveling down bridge substructures and foundations into the soils from surface impacts with 3 to 12 lb hammers.

Also, testing is faster and the data more consistent if 8 channels of data are acquired at a time, rather than measuring the response of a single hydrophone receiver. A photograph of the hydrophone receiver string is shown in Fig. 30 along with the power supply amplifier box. The hydrophone receiver string consists of 8 piezoceramic hydrophones which are about 10 cm (4 inch) long and 4.45 cm (1.75 inch) in diameter and are spaced 1 m (3.3 ft) apart on the shielded cable. The piezoceramic hydrophone elements are bonded with urethane to aluminum cases with downhole electronic amplification of received signals of x200. The hydrophone power supply/amplifier box has gains of x1, x10 and x100 for total gains of x200, x2000 and x20,000. Further amplification would only amplify noise. The hydrophone string is capable of clearly recording a foot stomp on the ground surface when the string is at a depth of 60 meters (200 ft) in a borehole. The 8 channel hydrophone string was successfully researched and developed to fit inside a 5 cm (2 inch) ID casing,

and it did provide for faster PS tests with greater sensitivity as desired.

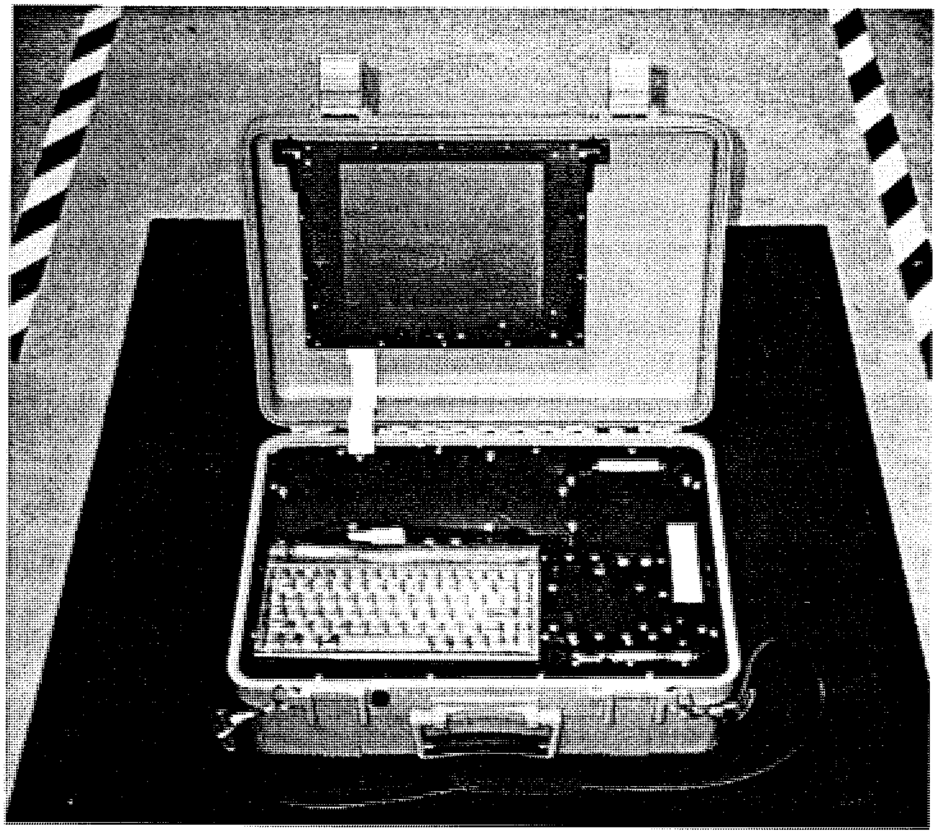


Figure 29- Photograph of the Prototype PC.

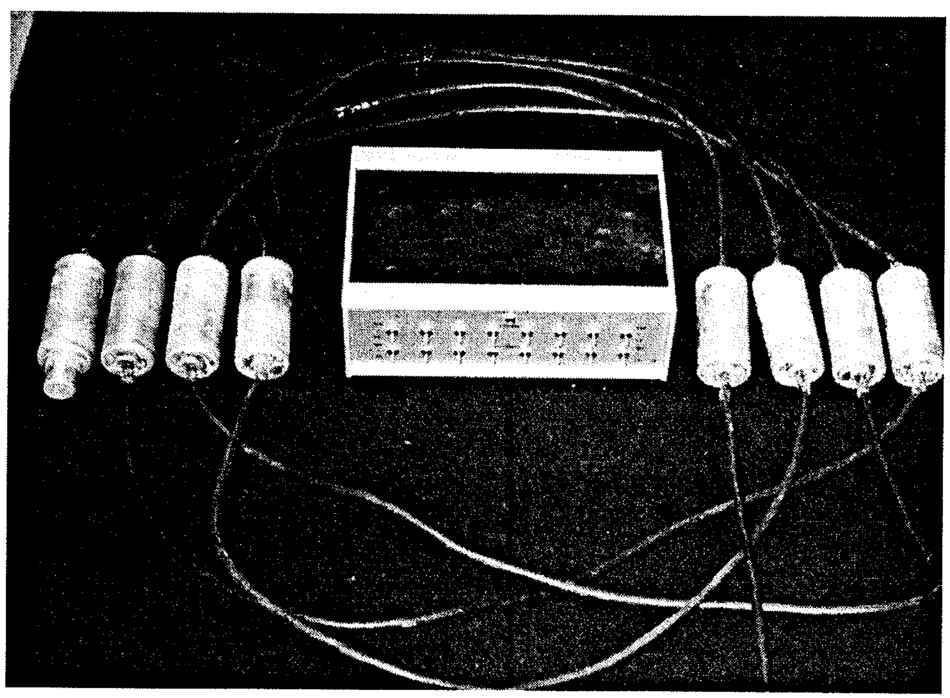


Figure 30- Photograph of the 8 Channel Hydrophone Receiver String.

4.4.3 Prototype Induction Field System.

A prototype Induction Field system was assembled using a commercially available triaxial magnetic field sensor and power supply unit as shown in Fig. 31 by Bartington Instruments Ltd. (Oxford, England) as the downhole search coil system. The magnetic coil sensor is nominally 30 cm (1 ft) long and 2.5 cm (1 inch) in diameter. The field was set up using a power amplifier (audio range) with a function generator and leads to connect to the test pile and another pile (or electrode driven in the ground). The data was recorded using the RC Computerscope card in a PC. The triaxial magnetic field sensor worked well. More work is needed to evaluate power amplifier requirements as it was heavily loaded at one site where the soils were likely saturated and very conductive.

4.4.4 Triaxial Accelerometer.

The PCB Piezotronics, Inc. triaxial accelerometer is shown in Fig. 32. This accelerometer attached easily to the bridge substructures, and performed well. It can be conditioned by a 27 volt DC power supply from battery powered units. The accelerometer was attached most often by grease coupling, particularly to concrete, or a screw/spike and magnetic mount on wood or steel substructures.

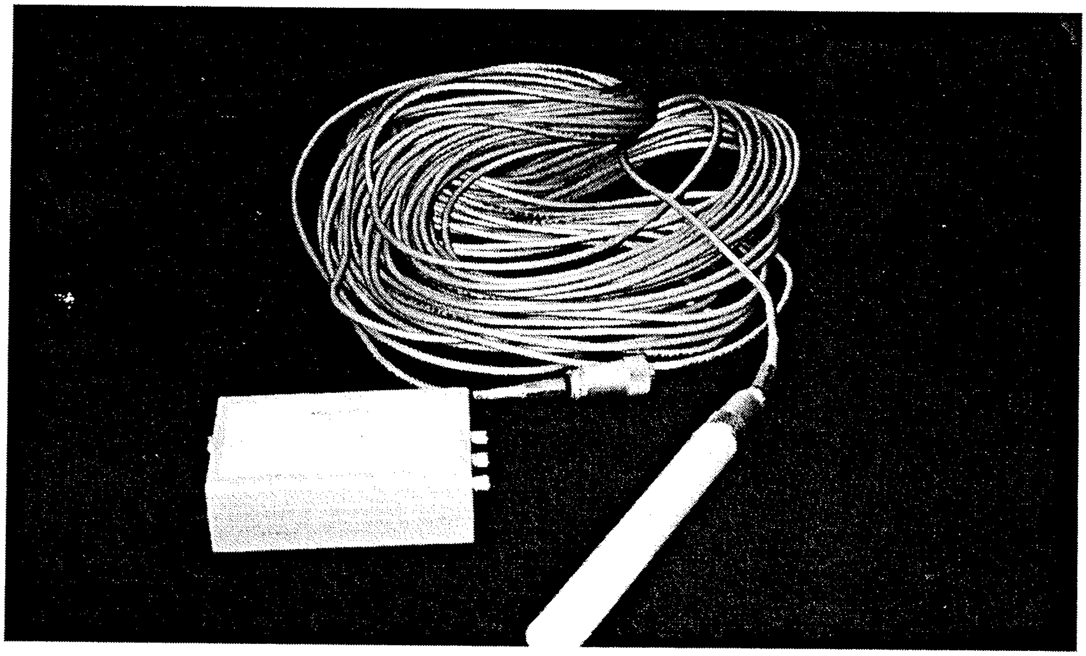


Figure 31- Photograph of the Induction Field Sensor and Power Units.

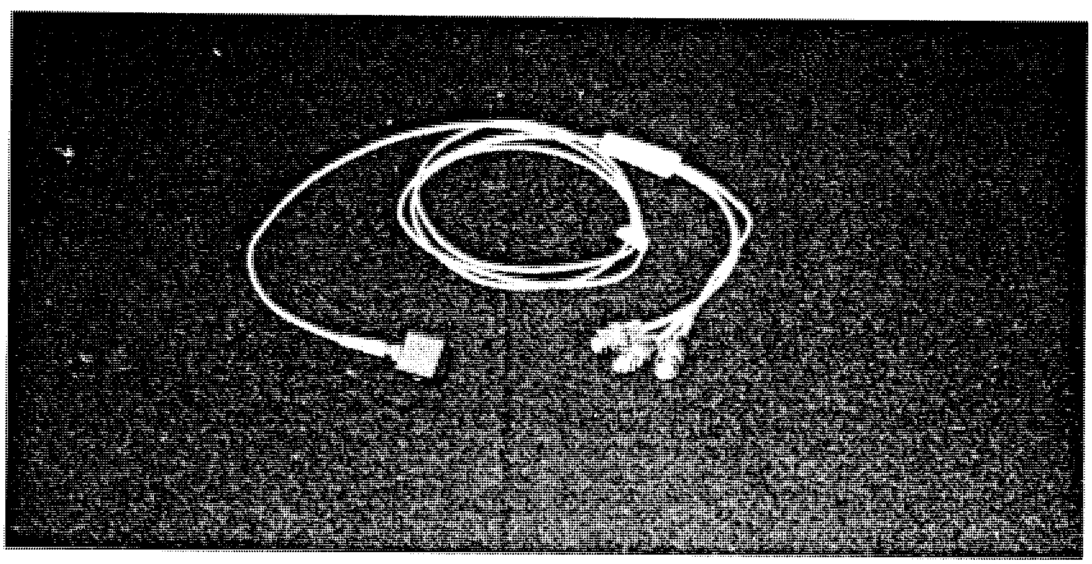


Figure 32- Photograph of the Triaxial Accelerometer Used in Ultraseismic Tests.

4.4.5 Repeatable Source for SE/IR, BW and US Tests.

The repeatable source was a solenoid operated hammer also manufactured by PCB. The data collected with the solenoid operated hammer at the Coors bridge in Colorado (tested in Phase I) did not differ from the hand held hammer after averaging. We concluded that 0.2, 1 and 3 lb instrumented and non-instrumented hand held hammers are adequate for Ultraseismic, Sonic Echo/Impulse Response and Bending Waves tests. The much greater expense of the automated, repeatable source did not result in a noticeable improvement in data quality. Also, it is important to remember that the use of an instrumented hammer (force transducer built-in) allows for normalization of the data to the impact force, and thus variation in the data is clearly canceled out.

4.5 PHASE IIB RESEARCH RESULTS.

The Phase IIB continuation of the NCHRP 21-5 (2) research effort was authorized to focus on research to improve the application of the Parallel Seismic and Ultraseismic methods for determination of unknown bridge foundation depths for scour safety evaluation purposes.

4.5.1 Evaluation of Energy Sources for Parallel Seismic Testing.

A previously tested bridge substructure consisting of steel H-piles below a concrete pilecap, and another previously tested bridge substructure with exposed timber piles were to be selected for Parallel Seismic and Ultraseismic testing, respectively. Arrangements were made with the Colorado Department of Transportation (CDOT) to test one bridge near Fort Lupton, Colorado for the PS research that had been previously tested. Unfortunately, the borehole casing was bent and the hydrophone string could not be inserted for falling weight deflectometer and Olson drop weight tests. Parallel Seismic (PS) tests were performed with jackhammer and fired stud sources on the 16th

Street bridge over the South Platte River in Denver, Colorado. This bridge had two concrete abutments underlain by footings on timber piles and had previously been tested by Olson Engineering.

Parallel Seismic (PS) tests were performed at the Coors bridge on Highway 58 at Golden, Colorado, using the Falling Weight Deflectometer (FWD) provided by the Colorado DOT pavement section. This is the same bridge that was tested in Phase I of this research project and its foundation consists of a concrete pilecap on steel H-piles. The PS results are discussed below.

Hilti Stud-Gun and Jackhammer Sources - 16th Street Bridge. The use of a Hilti concrete steel stud-gun and a Hilti jackhammer as possible Parallel Seismic sources were investigated at the 16th Street bridge in Denver, Colorado. This bridge was tested to evaluate potential new sources by firing studs and impacting the massive concrete abutment to send energy into the concrete pilecap and underlying timber piles. This was done as part of the evaluation of different sources in generating energy for PS tests on substructures that consist of massive concrete elements with piles underneath. In addition, 3-lb and 12-lb hammer sources had been used successfully to determine the timber pile depths with PS tests by Olson Engineering in a previous consulting investigation. Based on the analyzed PS data, neither the HILTI stud-gun or jackhammer sources generated sufficient energy to justify their use as more powerful sources than the typical 3-lb and 12-lb hammer sources used in the earlier Parallel Seismic tests.

Falling Weight Deflectometer and Large to Small Impact Studies - Coors Bridge. A Colorado DOT Falling Weight Deflectometer (FWD) was used to impact the bridge deck over the south

column of pier 4 at the Coors bridge. Impact forces to the deck ranged from 5,000 to 22,000 pounds. For comparison, a 12-lb impulse sledgehammer was also used to impact the deck. Unfortunately, due to the attenuation of the relatively high frequency energy through the bridge bearings, it was concluded that the FWD source is not a feasible source to excite the piles encountered below a massive bridge substructure because the FWD loading is always applied from the top of the bridge and the bearings will likely act as filters of the desired high frequency energy. The relatively high frequency energy is needed to obtain useful data in the piles underneath a massive bridge substructure.

Consequently, a swinging 70-lb weight with a plastic tipped head, a 12-lb impulse hammer and a 3-lb impulse hammer were used as sources with both horizontal and vertical hits applied below the bridge bearings to the pier column as close as possible to the ground surface on the Coors bridge. An 8-channel hydrophone string and a single hydrophone were used as receivers (triaxial geophone receivers were used in the original research and were not repeated). The 70-lb weight was swung in a pendulum fashion from a large tripod to impact the bridge column horizontally for evaluation of higher energy impacts as compared to the impulse hammers.

The single hydrophone receiver produced the clearest PS results for horizontal impacts to the bridge column as shown for the 3-, 12- and 70-lb sources in Figs. 33, 34 and 35, respectively. Examination of these figures shows the strong tube wave energy propagating in the cased borehole at a velocity of 490 m/sec (1,600 ft/sec). The faster traveling compression wave energy that propagates down the steel H-piles is not visible at shallow depths in any of the figures as it is dominated by the high-amplitude tube wave energy that came from the pilecap into the ground into

the PVC casing. The pilecap is so much larger in size than the steel H-piles, that most of the energy from the impacts stays within the pilecap and column above it. Very little of the energy is coupled into the comparatively small area of the steel H-piles because of the large difference in size and acoustic impedance (mass density * velocity * area) between the piles and the pilecap. Although not visible, some of the wave energy reached the bottom of the pile and was diffracted causing tube wave energy to be generated and measured by the hydrophone receiver. The diffracted energy is clearly apparent at the pile tip depth in Figs. 33-35 as this energy arrived at the hydrophone depths before the pilecap generated tube wave front arrived. Analysis of the bottom diffraction event indicates the pile tip to be at a depth of 8.8 m (29 ft) which agrees well with the actual depth of 8.78 m (28.8) ft from bridge plans. This diffraction is an important new finding that greatly improves the detection of pile depths below pilecaps.

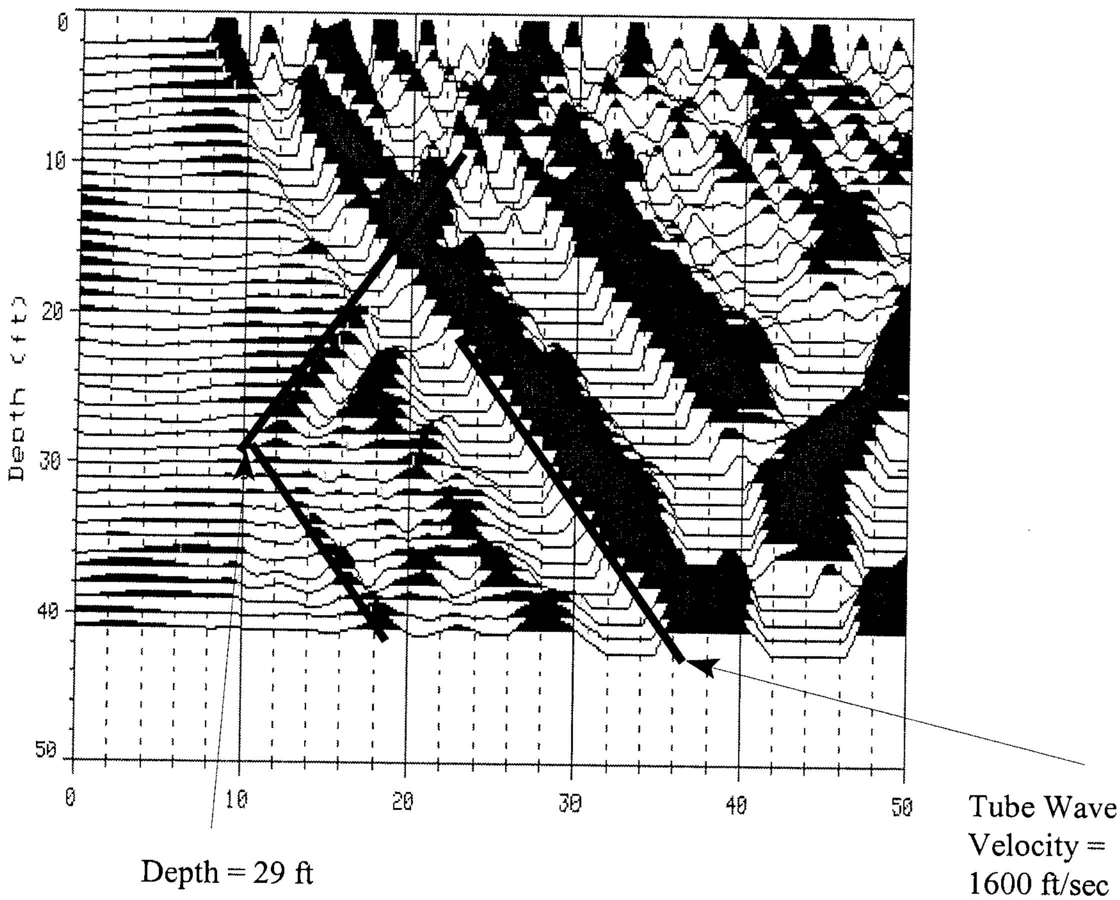


Figure 33- Parallel Seismic Data from a 3-lb Hammer Horizontal Hit and Single Hydrophone Recording,
Steel H-Pile Foundation with Concrete Pilecap on Top
Coors Bridge, Golden, Colorado.

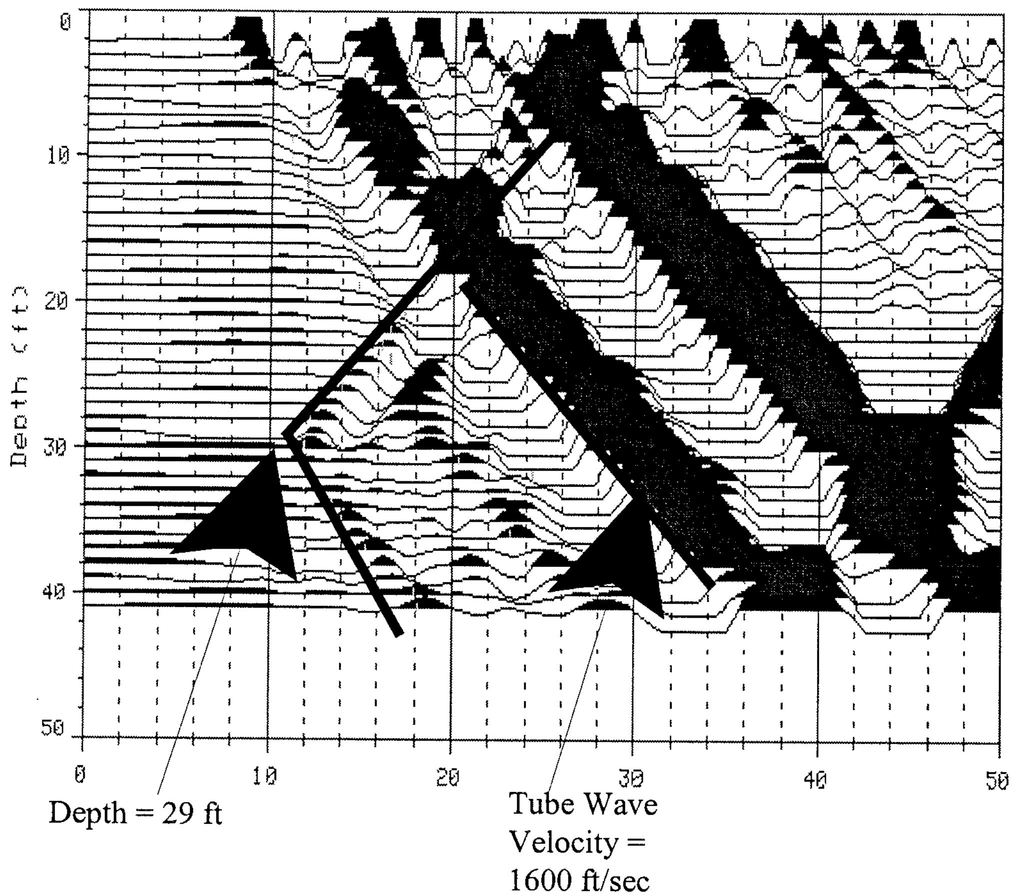


Figure 34- Parallel Seismic Data from a 12-lb Hammer Horizontal Hit and Single Hydrophone Recording,
Steel H-Pile Foundation with Concrete Pilecap on Top
Coors Bridge, Golden, Colorado.

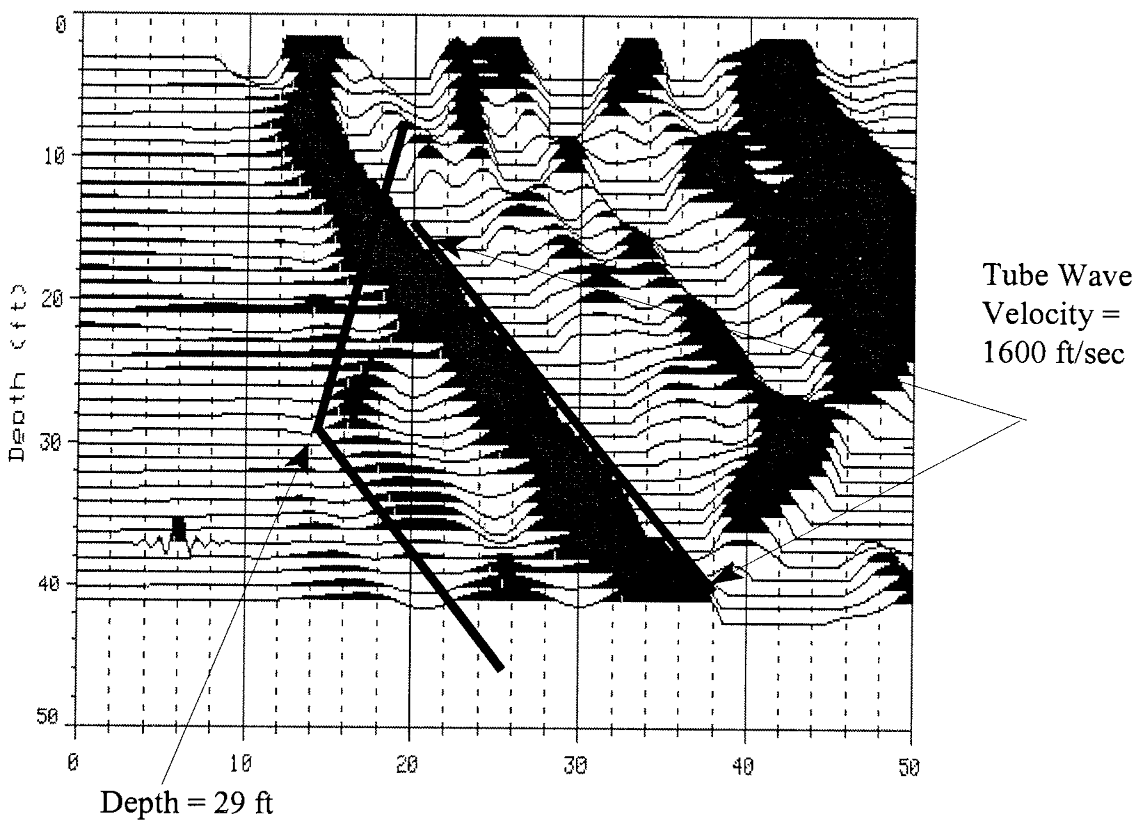


Figure 35- Parallel Seismic Data from a 70-lb Swinging Weight Horizontal Hit and Single Hydrophone Recording,
Steel H-Pile Foundation with Concrete Pilecap on Top
Coors Bridge, Golden, Colorado.

4.5.2 Additional Parallel Seismic Theoretical Modeling Results.

The theoretical modeling for the borehole based Parallel Seismic (PS) test was focused on a model that was designed to be similar to the South Column of Pier 4 of the Coors bridge on Colorado State Highway 58 over 44th Avenue in Golden, Colorado (CDOT Structure No. E-16-HI) where the experimental work was performed. The 3-D finite element modeling (FEM) program used in the research was first written by Dr. Shu-Tao-Liao during his Ph.D. research under Dr. Jose Roesset at the University of Texas at Austin. The program was modified to simulate an axisymmetric column-pilecap-pile section, instead of the single pile in the original version. Since the 3-D FEM program uses 4-node, isoparametric, axisymmetric elements to model both pile and soil materials, it can be used to simulate the Parallel Seismic tests. The impact load, however, is applied to the center of the pile top in the vertical direction only due to the axisymmetry. Therefore, the 3-D FEM program cannot be used to simulate the Bending Wave test. In contrast, the 2-D FEM program written by Dr. Chih-Peng Yu also under Dr. Roesset's direction (see April, 1995 Draft Final NCHRP 21-5 report) allows the impact loads at any nodes in both vertical and horizontal directions, but soil elements in the 2-D FEM program are simplified to be a spring with a damper at each node in the soil. As a result, the 2-D FEM program can be used to simulate the Ultraseismic horizontal impact and Bending Wave tests, but not the Parallel Seismic tests. A user-friendly interface computer program was developed to prepare the input data file for the 3-D FEM program and also to output the simulated pile and soil data for input into the Interpex Bridgix software as shown in Fig. 36. Also shown in Fig. 36 are the cylindrical dimensions and properties input for the bridge column (truly round), pilecap (truly square), and pile (truly 5 BP12x53 piles) for the South Column of Pier 4 of the Coors bridge.

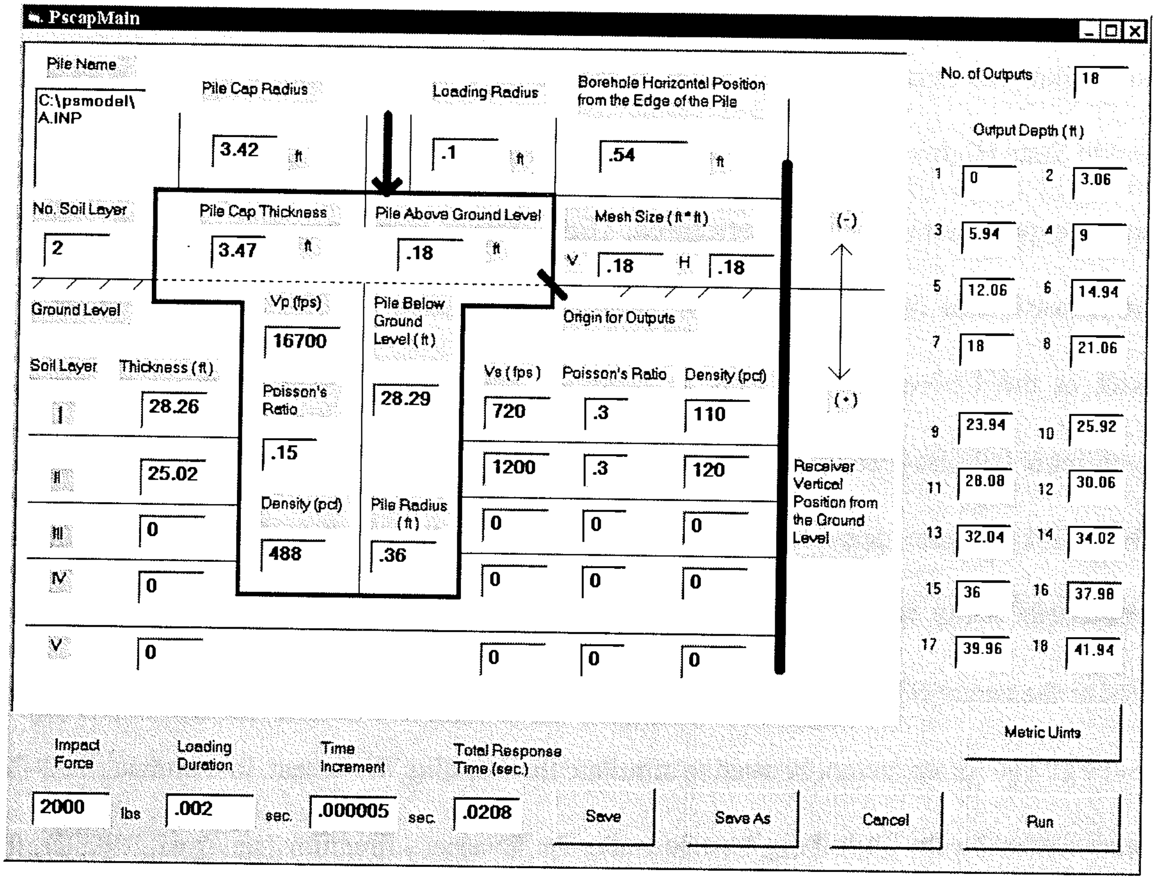
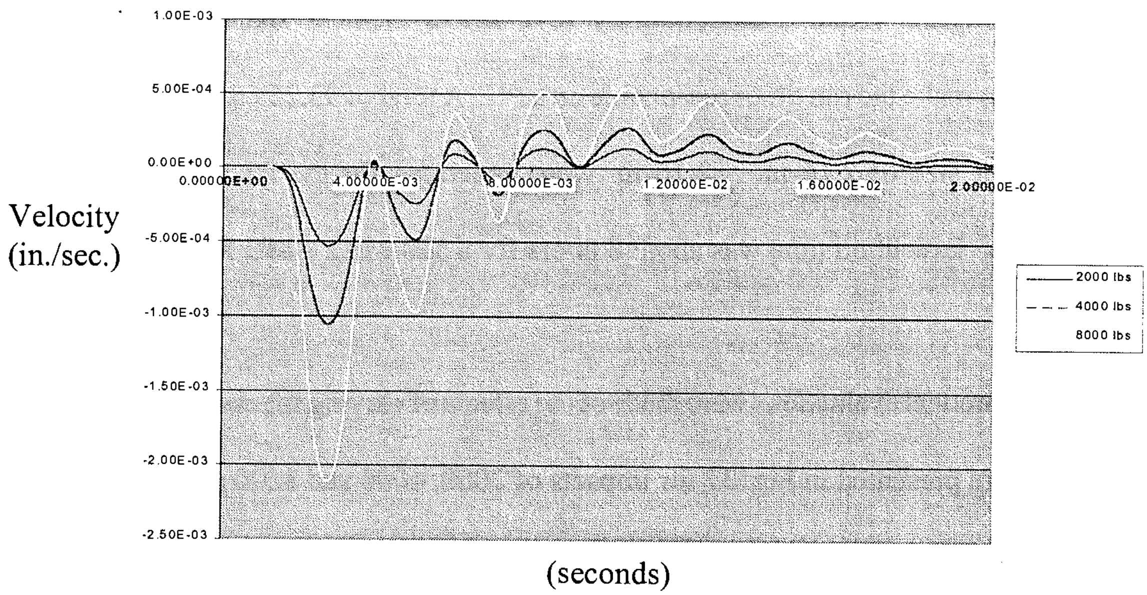


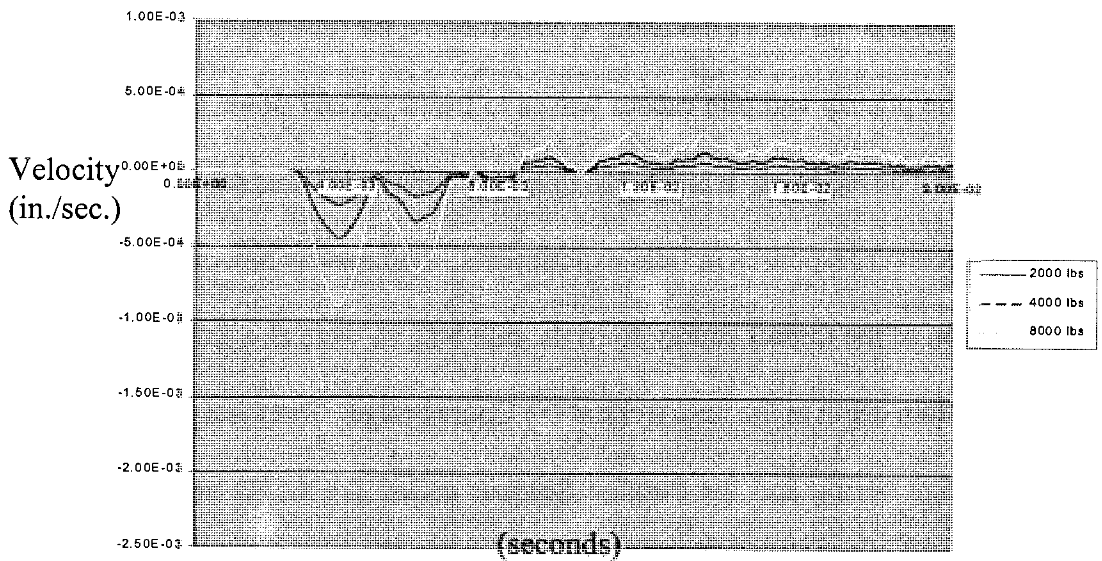
Figure 36- Soil Profile and Pile Geometry to Simulate Parallel Seismic Tests at Coors Bridge, Golden, Colorado.

The program requires axisymmetry (cylindrical element shapes about the pile-column centerline) to make the run times reasonable on a PC. A total of 93,259 axisymmetric elements with each element size of 2.16 in. × 2.16 in. were generated for the PS tests on the Coors bridge substructure. Execution time was about 8 hours for a 550 MHz AMD K6-2 processor.

Theoretical evaluations were made of the effects of varying the energy of source impacts, and the results are presented in Fig. 37 for impacts of 2000, 4000 and 8000 pounds force from a depth of 24 ft (Fig. 37a) and 32 ft (Fig. 37b) below the ground surface in the soils. These figures indicate that for a linearly elastic system, increasing impact force will generate a greater response in the soil, as expected. Unfortunately though, due to the axisymmetric limitation of the 3-D FEM program, the actual borehole could not be modeled. Consequently, the tube wave phenomena in which wave energy from the pilecap and pile causes waves to travel up and down the borehole/casing water column could not be modeled. It is these high energy tube waves that dominate the Parallel Seismic hydrophone receiver responses in field experiments. Fortunately however, tube waves generated from diffraction of the impact energy from the pile bottom were also found to clearly indicate the foundation bottom depth in the field work at the Coors bridge. Thres is a program that could model the Coors bridge boring and the Parallel Seismic called DYNA3D and it allows for true 3-D modeling of wave propagation in solids. This program is normally run on a mainframe or workstation computer to minimize lengthy execution times, but was not used in this research.



a) Vertical Velocity Responses at 24 ft below the Ground Surface (above the Pile Tip)



b) Vertical Velocity Responses at 32 ft below the Ground Surface (below the Pile Tip)

Figure 37- Comparison of Simulated Receiver Signals from Different Impact Force Levels in Parallel Seismic Tests at Coors Bridge, Golden, Colorado.

4.5.3 Additional Sonic Echo/Ultraseismic Theoretical Modeling Results.

The Ultraseismic (US) theoretical modeling was designed to be similar to pile 2 of the northeast wing wall of the Franktown, Colorado bridge. This actual timber pile is 30.5 cm (12 inches) in diameter and 8.5 m (28 ft) long with about 2.7 m (9 ft) of the pile above grade where a wood retaining wall is loosely attached to it with spikes. Because the pile is in a wing wall, there is no beam on its top, so it is the simplest case to analyze and test. Spectral Analysis of Surface Waves (SASW) tests were performed on the ground adjacent to the Franktown bridge. The soil profile at this bridge site consists of clay and streaks of sand and the water table was at about 5.2 m (17 ft) below the ground surface according to Colorado DOT information. The SASW results from Phase I results showed an average surface wave velocity of 104 m/sec (340 ft/sec), which is a low value representing soft material. Since the 3-D FEM program uses 4-noded, isoparametric, axisymmetric elements to model both pile and soil materials, it can be used to simulate the vertical impact US tests. The impact load, however, is applied to the center of the pile top in the vertical direction only due to the axisymmetry requirement. Therefore, the 3-D FEM program cannot be used to simulate the horizontal impact US and Bending Wave tests. In contrast, the 2-D FEM program allows the impact loads at any nodes in both vertical and horizontal directions, but soil elements in the 2-D FEM program are simplified to be a spring with a damper at each node in the soil. As a result, the 2-D FEM program is best used to simulate the horizontal impact US and Bending Wave tests, but is not as accurate for the vertical US and SE/IR tests, and is not suitable for the Parallel Seismic tests.

Vertical Impact and Response US Tests (Sonic Echo/Impulse Response). The 3-D axisymmetric finite element modeling (FEM) program from the University of Texas at Austin was used to simulate the US testing for vertical impacts to the top of a 8.5 m (28 ft) long pile with a velocity of 5,180 m/sec (17,000 ft/sec) and a soil shear wave velocity of 122 m/sec (400 ft/sec) (based on the surface wave velocity). The response on the top of the model pile corresponds to the Sonic Echo/Impulse Response test geometry, and additional responses were calculated down the pile side to simulate side mounted receivers for tracking wave travel down and up the pile. Sonic Echo results (topmost receiver in US test) are plotted in Figs. 38-40 for pile embedment of 0.3, 3 and 5.8 m (1, 10 and 19 ft), respectively. For the 8.5 m (28 ft) long timber pile with 0.3 m (1 ft) embedment, very clear multiple echos from the pile bottom appear at the pile top as shown in Fig. 38. For the 8.5 m (28 ft) long timber pile with 3 m (10 ft) embedment, Figure 39 shows that the velocity response at the pile top is dominated by the echo from the ground level and no pile bottom echo is clearly apparent as it is obscured by the ground echo. The simulation for the actual pile of 5.8 m (28 ft) long with 5.8 m (19 ft) embedment is presented in Figure 40, resulting in no clear echo identified. The pile was shortened so that its embedment would be 5.8 m (19 ft), but it would stick up only 0.15 m (0.5 ft) above ground, thereby minimizing any ground echo. Figure 41 shows that a 5.9 m (19.5 ft) long timber pile with 5.8 m (19 ft) embedment has no clear echoes at all. Consequently, it is concluded that the model pile case closest to the actual pile has no compression wave echo from the pile bottom when it is embedded 5.8 m (19 ft) deep.

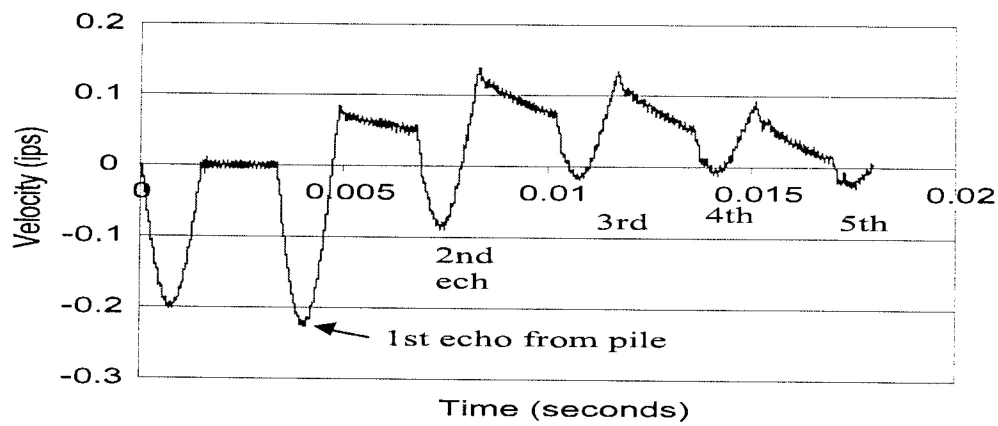


Figure 38- 3-D Finite Element Modeling Results for Vertical Impact, Top Response Ultraseismic (Sonic Echo) Tests at Franktown Bridge, Colorado (28 ft long timber pile with 1 ft embedment) (Timber Velocity = 17,000 fps).

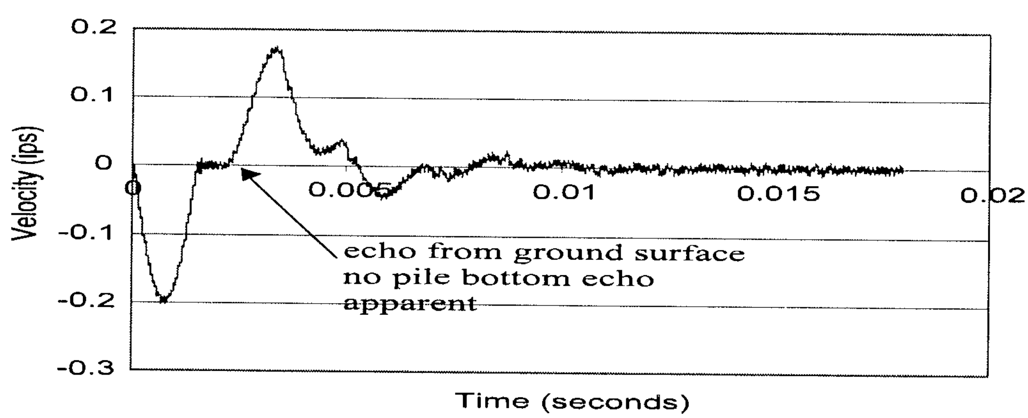


Figure 39- 3-D Finite Element Modeling Results for Vertical Impact, Top Response, Ultraseismic (Sonic Echo) Tests at Franktown Bridge, Colorado (28 ft long timber pile with 10 ft embedment) (Timber Velocity = 17,000 fps).

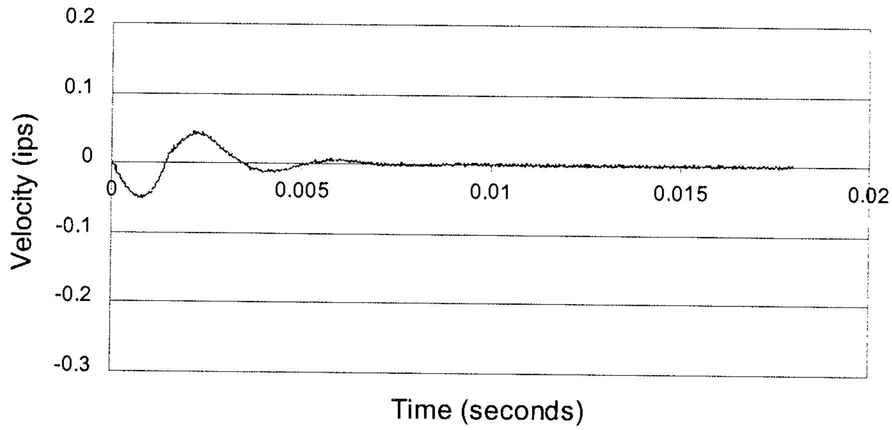


Figure 40- 3-D Finite Element Modeling Results for Vertical Impact, Top Response Ultraseismic (Sonic Echo) Tests at Franktown Bridge, Colorado (28 ft long timber pile with 19 ft embedment) (Timber Velocity = 17,000 fps) - no bottom echo apparent.

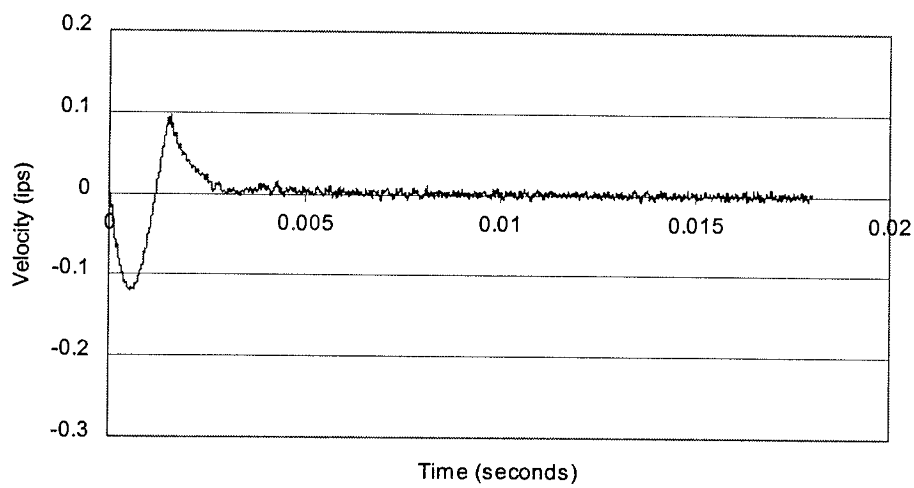


Figure 41- 3-D Finite Element Modeling Results for Vertical Impact, Top Response Ultraseismic (Sonic Echo) Tests at Franktown Bridge, Colorado (19.5 ft long timber pile with 19 ft embedment) (Timber Velocity = 17,000 fps) - no bottom echo apparent.

Horizontal Impact and Response Ultraseismic Modeling Results (Bending Wave). As shown in Figs. 42-44, three cases with embedment of 0.3, 3 and 5.8 m (1 ft, 10 and 19 ft) were modeled for a 8.5 m (28 ft) long timber pile 0.3 m (1 ft) in diameter. The impact was applied horizontally at the pile top and the receivers were placed horizontally at 0.3 m (1 ft) intervals from the pile top down to just above the ground surface. A bending wave velocity from the initial slope of the bending wave arrival was calculated to be 722 m/sec (2,370 ft/sec) using the Bridgix software. Examination of Fig. 42 shows the development of the compression, shear and bending wave energy traveling down the pile for the case of almost no soil embedment (0.3 m (1ft)). The spreading out of the energy (dispersion) is also apparent with increasing travel down the pile for the bending wave energy. Even in this ideal case, the reflected bending wave is not that apparent in Fig. 42. As the embedment increases, no clear identification of the bending wave reflection from the pile bottom is possible, only a ground surface reflection can be seen in Figs. 43 and 44. This problem of ground reflection and severe attenuation of bending wave energy at comparatively shallow depths and short length to diameter ratios was first discussed by Chih-Peng Yu under Dr. J. M. Roesset's supervision in the report "Determination of Pile Lengths Using Flexural Waves" that was presented in Section D.3 of the August, 1995 NCHRP 21-5 Final Report (7). More recent research that supports this finding on the limitations of the bending wave method was done by Mary Leigh Hughes in her Ph.D. thesis entitled "Nondestructive Determination of Unknown Pile Tip Elevations Using Modal Analysis" at the Georgia Institute of Technology, March, 1999.

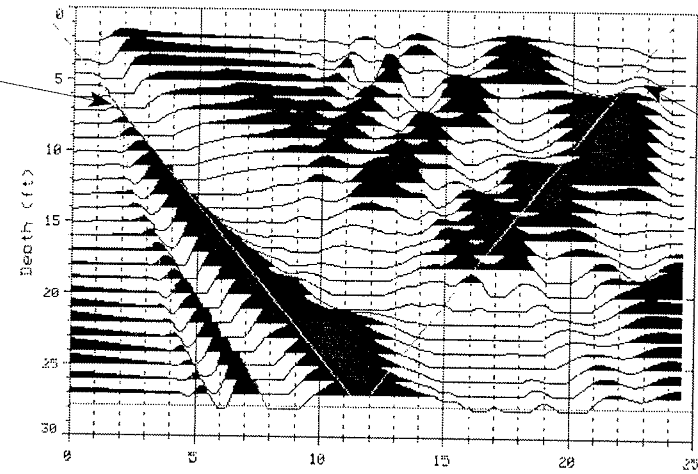


Figure 42- 2-D Finite Element Modeling Results for Bending Wave Tests at Franktown Bridge, Colorado (28 ft long timber pile with 1 ft embedment).

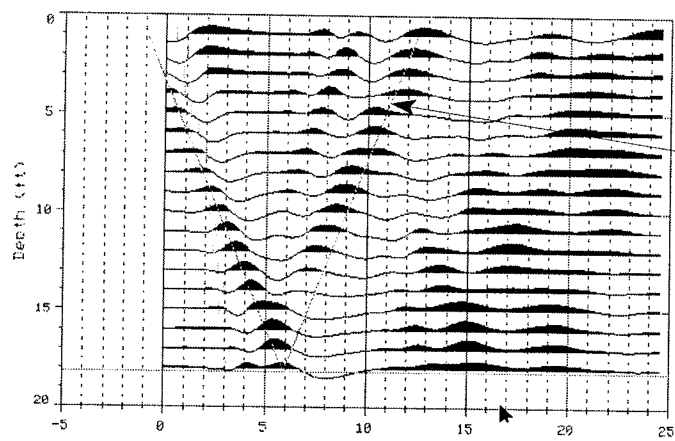


Figure 43- 2-D Finite Element Modeling Results for Bending Wave Tests at Franktown Bridge, Colorado (28 ft long timber pile with 10 ft embedment).

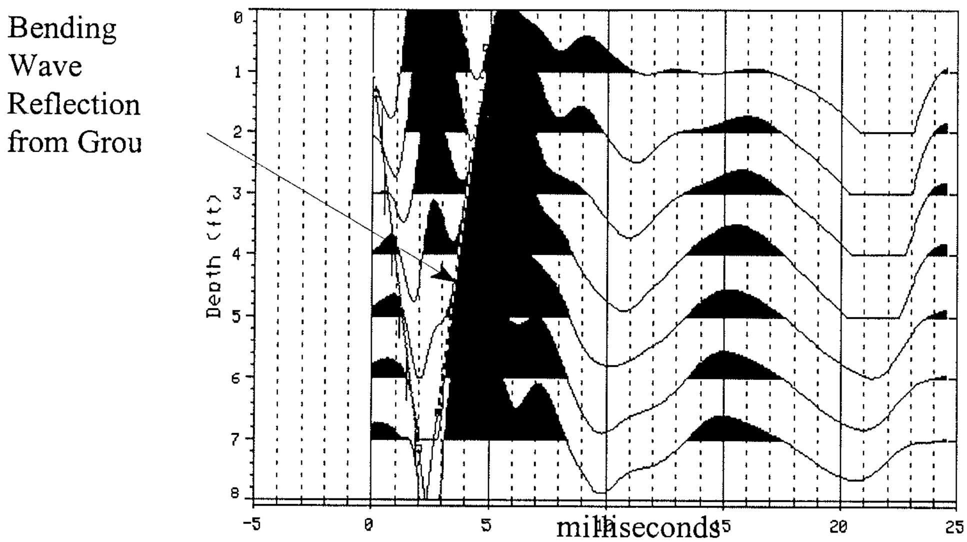


Figure 44- 2-D Finite Element Modeling Results for Bending Wave Tests at Franktown Bridge, Colorado (28 ft long timber pile with 19 ft embedment).

4.5.4 Ultraseismic Experimental Results.

Pile 2 of the northeast wing wall of the Franktown bridge was tested again to compare the experimental results with the theoretical model results. Both vertical and horizontal hits were applied at the pile top in the Ultraseismic (US) tests. The triaxial accelerometer used as the US receiver was installed at seven locations below the pile top at 0.3 m (1 ft) intervals to allow tracking of compression and bending wave energy up and down the pile. Both uniaxial and triaxial accelerometers were used to collect the acceleration response of the pile to the impacts in both vertical, radial and tangential (relative to the impact) orientations. Data was also collected using the Sonic Echo/Impulse Response (SE/IR) method (simple vertical impact/response US test) by impacting the pile top at its center and recording the vertical response of the pile with top mounted accelerometer and geophone receivers. Both 3-lb and 12-lb impulse hammers were used in the US and SE/IR tests to provide medium to large excitation forces. For the Bending Wave (BW) tests (simple horizontal impact/response US test), the pile was struck horizontally at its top with a 1 lb claw hammer, and the 3- and 12-lb impulse hammers. Uniaxial accelerometers were positioned at 1.1 and 2.2 m (3.5 and 7 ft) below the top of the pile on the opposite side of the pile from the impact. No water was present at the time of testing, but the ground was likely frozen near the surface for a foot or two as snow was present by the pile.

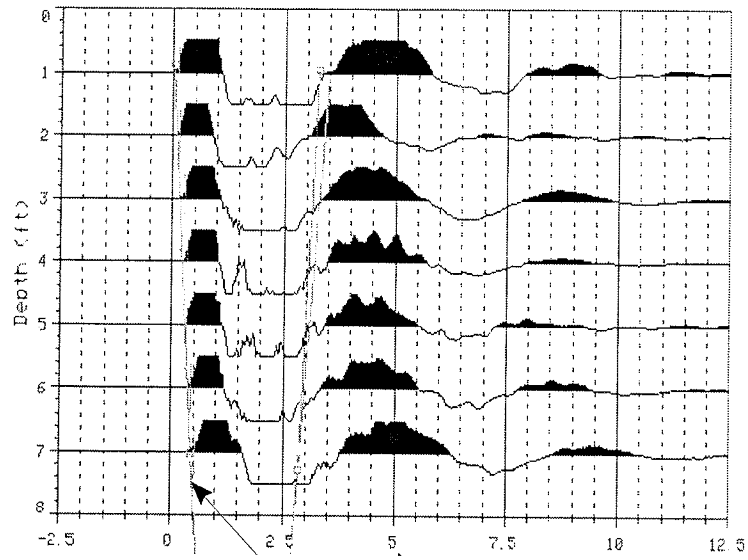
Vertical Impact/Vertical Response Ultraseismic Test Results. The US record presented in Fig. 45 plots the vertical velocity response of receivers from 0.3 to 2.2 m (1 to 7 ft) below the pile top versus time in milliseconds. While the propagation of the compression wave energy down the pile is clearly apparent, there is no clear reflection from the pile bottom. The use of the greater impacts from the 12-lb hammer also did not result in any clearly identifiable echo events

corresponding to the pile bottom. The earlier testing did show some apparent weak echoes. The lack of echoes in this dataset may be due to the frozen ground reflecting the compression wave energy such that less energy reached the bottom of the pile thereby preventing even weak reflections from being identified. Although not presented herein, detailed analyses of the SE/IR data likewise did not show any compression wave reflection events that corresponded to a clearly identifiable echo from the pile bottom. This result of not identifying an echo of the compression wave energy from the pile bottom is consistent with the theoretical modeling results of Section 4.5.3 (see Figs. 40 and 41).

Horizontal Impact/Horizontal Response Ultraseismic Test Results. The US record presented in Fig. 46 plots the horizontal acceleration responses of 7 receiver positions located at from 0.3 to 2.2 m (1 to 7 ft) below the pile top versus time in milliseconds. While the propagation of the bending wave energy down the pile is clearly apparent, there is no clear reflection from the pile bottom, but instead a strong ground reflection from the frozen ground. This was found to be the case in the earlier US horizontal/horizontal tests as well. The use of the greater impacts from the 12-lb hammer also did not result in any clearly identifiable echo events corresponding to the pile bottom.

The earlier testing did show an apparent echo using the Short Kernel Method (SKM) to analyze bending wave data in the time domain. The BW data collection with SKM analysis was repeated in this study with a long time record. Although not presented herein, careful analyses of the longer BW time records shows that possible reflections are noise, and do not track as having come from the pile tip, except again for the strong echo from the ground surface. The lack of echoes in this dataset may be due to the frozen ground reflecting the bending wave energy such that less energy reached the bottom of the pile thereby preventing even weak reflections from being identified. It is also

possible that the apparent echo in the earlier BW records was a misleading noise event. No resonances indicative of a BW echo were identified with Impulse Response analyses of the data either. This result of not identifying echoes from anything but the ground surface in the horizontal impact/horizontal response US and BW tests is consistent with the theoretical modeling results for bending waves as discussed in Section 4.5.3 which also only produced identifiable echoes from the ground surface (see Fig. 44).



Compression Wave Velocity = 17,000 ft/sec.

Figure 45- Ultraseismic Compression Wave Data from a 3-lb Hammer Vertical Hit and Velocity Integration of Vertical Response and Trace Max.

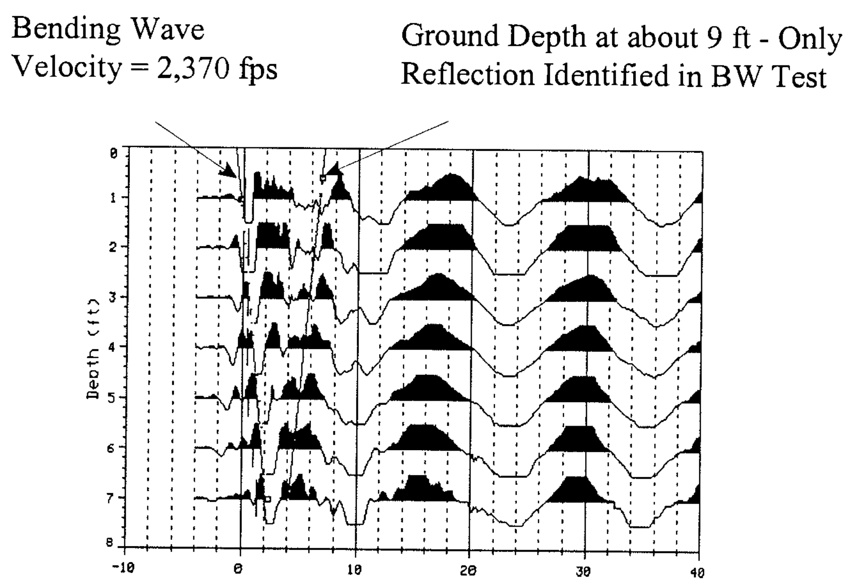


Figure 46- Ultraseismic Bending Wave Results from a 3-lb Hammer (Black Plastic Tip) Horizontal Hit and Radial (opposing horizontal) Accelerometer Receiver Response (4 hits averaged with F-K filtered and AGC amplification).

CHAPTER 5

RESEARCH CONCLUSIONS AND ASSESSMENT OF RESEARCH NEEDS

Prior to this research, only four NDT methods had been previously used to determine unknown foundation depths; the surface methods of Sonic Echo and Bending Wave, and the borehole methods of Parallel Seismic and Induction Field. Only the Sonic Echo and Parallel Seismic methods had been used much by engineers to determine the depths of foundations. The Bending Wave method had only been initially researched for application to exposed timber piles, and the Induction Field method was limited to piles with continuous steel, typically steel H-piles and steel pipe piles. No theoretical modeling of the NDT methods had been done either.

Accordingly, the NCHRP 21-5 Phase I project research was conceived and executed as a more basic research project. Phase I research was to first evaluate the capabilities and limitations of the existing methods, and second to research the potential for application of other nondestructive testing and evaluation (NDT&E) methods for determination of unknown bridge foundation depths. Following the successful completion of the Phase I research, the NCHRP 21-5 (2) Phase II project was funded to validate and advance the most promising NDT methods of Ultraseismic and Parallel Seismic with initial field evaluation of the Induction Field method. During this research, the four existing methods and several NDT methods not previously applied to the unknown foundation depth problem were investigated including stress wave, magnetic and electromagnetic methods. The findings and conclusions of the field research are discussed below in Section 5.1 with specific discussions of the Parallel Seismic and Ultraseismic methods. Theoretical modeling conclusions for the Parallel Seismic and Ultraseismic methods are discussed in Section 5.2. In conclusion, a

discussion of a proposed technical approach to conducting unknown foundation investigations is presented in Section 5.3.

5.1 FIELD TESTING RESEARCH FINDINGS AND CONCLUSIONS.

Several NDT methods were investigated on 7 bridges with known foundations in the Phase I research in relation to their application to depth determination of unknown bridge foundations. The surface methods that were evaluated included Sonic Echo/Impulse Response, Bending Wave, Ultraseismic, Spectral Analysis of Surface Waves, Ground Penetrating Radar and Dynamic Foundation Response. The borehole methods that were evaluated included Parallel Seismic, Borehole Radar, and Borehole Sonic. Summary evaluations of the applicable surface and borehole NDT methods are presented in Tables I and II of the Summary section at the beginning of this report. Based on the results of Phase I research, the Parallel Seismic and Ultraseismic methods were found to be the most promising methods for determining unknown foundation depths for the largest populations of bridge substructure types. Accordingly, it was recommended that these two methods, along with the bending wave and Sonic Echo/Impulse Response be further investigated in Phase II to evaluate their accuracy in determining depths of unknown bridge foundations. Field evaluation of the Induction Field method for steel piles was also performed for the first time in the Phase II research.

The Phase II evaluation of the accuracy of the Parallel Seismic, Ultraseismic, Sonic Echo/Impulse Response and Bending Wave methods was achieved by initial blind testing of 21 bridges in Phase II research and comparing the predicted depths from the NDT methods to the actual known depths after the predictions were reported to the NCHRP 21-5 (2) panel. The results of the

initial blind and post blind NDT predictions showed that the Parallel Seismic was the most accurate method for determining the depths of unknown bridge foundations. The Ultraseismic method was found to be accurate and applicable to finding the depths of the first foundation element of the substructure, but cannot see the depths of buried piles below pilecaps, abutments and piers. The results of the Sonic Echo/Impulse Response and Bending Waves tests were inconclusive in almost all cases due to the complexity of the problem with too many reflecting boundaries and the limited data that is acquired in these tests. A summary of the blind and post blind prediction results is presented in Table III of the Summary section at the beginning of this report. A discussion of the Parallel Seismic and Ultraseismic experimental field results is presented below.

5.1.1 Parallel Seismic (PS) Method.

The Parallel Seismic method provided the greatest accuracy blind (63%) and post-processed (84%) as shown in Table III for the widest range of bridge substructure types in terms of post blind foundation depth predictions in the Phase II research. The subsequent discovery of the diffracted tube wave phenomena from the pile tip greatly enhances the detection of pile bottom depths below pilecaps for cases when initial compression wave arrivals from the pile are not apparent. The use of greater impact energy can also help in identifying pile depths in PS tests. The use of larger impacts and the diffracted tube wave phenomena will further improve the accuracy and reliability of the PS method in interpreting unknown foundation depths. In conclusion, the research shows that the PS method is the single best method in terms of accuracy and applicability to determination of unknown bridge foundation depths.

The PS test and data interpretation are the most straightforward to apply when the ground is saturated. Data interpretation becomes increasingly complicated when soils are partially saturated. This is due to the resulting variation in wave velocity for partially saturated soils, unlike the case for saturated soils which have a constant compression wave velocity of 1,500 m/sec (4,900 ft/sec). Thus, PS test results in saturated soils of a vertical pile with a parallel and vertical, water-filled, plastic cased boring will provide the pile velocity since the wave velocity is constant in the soil between the foundation and the boring. PS data interpretation is still possible in partially saturated soils, but care should be taken, and if possible, downhole seismic tests should be performed to obtain the soil compression and shear wave velocity profiles to aid in interpretation of the PS data. Also, the use of geophones in PS tests at sites with unsaturated soil conditions can aid in the data interpretation. Massive abutments and piers on piles can also make it difficult to determine pile depths due to the large size (impedance) contrast between the massive substructure and small piles that results in very little energy propagating down the piles. However, as discussed above, the use of larger impacts and the diffracted tube wave phenomena have been found by the researchers to greatly aid in identifying pile bottoms in this case both in the research and consulting projects.

5.1.2 Ultraseismic (US) Method.

The US method was the second most accurate method in terms of foundation depth prediction with post blind predictions that showed 11 accurate (61%), 1 incorrect and 6 inconclusive predictions. The inconclusive predictions are attributed to the inability of the surface stress wave methods to detect piles below buried massive substructures and/or pilecaps. The US results are also dominated by reflections at the boundaries of pier/abutment/column substructure, pilecaps/footings, superstructure elements and foundation/soil interfaces that can mask any very weak echoes from

foundation bottoms in US test analyses. The use of 8 or more triaxial accelerometer receiver positions spaced 15 cm (6 inches) vertically apart from the ground/water surface up the exposed substructure was the most common receiver configuration. Impacts were applied with a 1.4 kg (3 lb) hammer horizontally near the top and bottom, and vertically to exposed beams and pilecaps to generate the wave energy. The use of a repeatable solenoid impact hammer did not show any noticeable improvement in data quality. It should be noted that the Ultraseismic test can be done as either a multiple vertical or horizontal receiver location test with either vertical or horizontal impacts to monitor the travel of compression (used in the Sonic Echo test) or flexural waves (used in the Bending Wave test) down and up exposed foundation substructures. Thus, the Ultraseismic method is a multiple receiver approach to both Sonic Echo and Bending Wave tests with geophysical processing to better track whether reflections are coming up from a foundation element boundary as desired, or down from overhead substructure/superstructure boundaries.

5.2 PARALLEL AND ULTRASEISMIC THEORETICAL MODELING CONCLUSIONS.

Theoretical modeling was performed to simulate Parallel Seismic and Ultraseismic tests. The theoretical modeling was performed using programs developed by Drs. Liao, Yu and Roesset at the University of Texas at Austin as documented in the Phase I final report (7).

5.2.1 Parallel Seismic Modeling Results.

The theoretical modeling for the borehole based Parallel Seismic (PS) test was focused on a model that was designed to be similar to the South Column of Pier 4 of the Coors bridge on Colorado State Highway 58 over 44^{th} Avenue in Golden, Colorado (CDOT Structure No. E-16-HI) where the experimental work was performed. The 3-D finite element modeling (FEM) program used

in the research was first written by Dr. Shu-Tao-Liao during his Ph.D. research under Dr. Jose Roesset at the University of Texas at Austin. Theoretical evaluations were made of the effects of varying the source energy for impacts of 900, 1800, and 3600 (2,000, 4,000 and 8,000) pounds force. The PS results indicate that for a linearly elastic system, increasing impact force will generate a greater response in the soil, as expected. Unfortunately though, due to the axisymmetric limitation of the 3-D FEM program, the actual borehole could not be modeled. Consequently, the tube wave phenomena in which wave energy from the pilecap and pile causes waves to travel up and down the borehole/casing water column could not be modeled. It is these high energy tube waves that dominate the Parallel Seismic hydrophone receiver responses in field experiments. Fortunately, tube waves generated from diffraction of the impact energy from the pile bottom were also found to clearly indicate the foundation bottom depth in the field work at the Coors bridge.

In addition, theoretical modeling was performed for Wake County Bridge # 251 in North Carolina. The theoretical results looked similar to the experimental results with the exception that dry soils were assumed in the modeling and the experimental soils were saturated. An easy to use interface was made for the 3-D axisymmetric finite element program for modeling compression wave propagation in the Parallel Seismic test. Overall, the theoretical modeling of PS tests showed that the modeling can help in understanding and interpreting the experimental data.

5.2.2 Ultraseismic Modeling Results.

Ultraseismic theoretical modeling was performed at Wake County Bridge # 251 in North Carolina and at the Franktown bridge in Colorado for both compression waves (3-D axisymmetric finite element program by Liao) and bending waves (2-D finite element program by Yu). The US

theoretical modeling showed that dominant reflections from structural elements connections above ground and substructure/soil interfaces make it difficult to identify weak reflections from the bottom of small size foundation elements such as piles for both compression and bending waves. In addition, the US modeling showed that the US method cannot identify piles underneath a pilecap, a wall pier or a massive abutment. The attenuation of the bending wave energy was found to be particularly severe with increasing embedment of piles and stiffer soils as compared to compression waves in this research. Similar work on bending waves by Dr. Hughes at Georgia Tech also supports this finding as discussed in Section 4.5.3. As piles become long and slender, and soils stiffer, the strength of any reflections of compression wave or bending wave energy from their bottoms will be attenuated to the point where the reflections are not identifiable from the background noise of the tests. The Ultraseismic, Sonic Echo and Bending wave tests are most applicable to piles and other testable foundation elements that are comparatively short and/or in comparatively soft soils.

5.3 TECHNICAL APPROACH TO UNKNOWN FOUNDATION INVESTIGATIONS.

The main advantages of any surface method is that the cost of a boring is eliminated. However, the research has shown that even the best surface method, Ultraseismic, is not applicable to many bridge substructures, particularly for any substructure where there is a major change in cross-section below exposed portions of a bridge such as piles below a pilecap of a bridge, pier or abutment. At best in these cases, the Ultraseismic test will indicate the depth to the first major change in cross-sectional properties. Even in the simplest case of exposed concrete, steel or timber piles, the research has shown that there are many combinations of foundation materials, geometry and subsurface conditions where the Ultraseismic test was not successful.

The borehole method of Parallel Seismic was found to be the most accurate method for unknown foundation depth determination in the research, but there is the additional cost of drilling a boring to drilled and casing it with PVC plastic. However, the boring does provide an opportunity for sampling of the soils which further aids in scour safety evaluations of unknown bridge foundations.

Considering the above, the researchers recommend that at least one pier or abutment substructure of an unknown foundation bridge with exposed piles be initially tested with both the Parallel Seismic and Ultraseismic methods. If the two tests agree, then the less costly Ultraseismic method can be used to check for variations in pile depths of other piers/abutments on the bridge. Otherwise, the Parallel Seismic test should be used to determine unknown foundation depths of other piers/abutments. For bridge substructures with unknown foundations other than exposed piles, the use of both methods can be helpful to determine the depth of the first change in cross-section of a bridge substructure with the Ultraseismic method, and to check for deeper foundation elements such as piles with the Parallel Seismic method.

Other nondestructive testing methods evaluated in this study such as Spectral Analysis of Surface Waves, Ground Penetrating Radar from the surface and borings, Borehole Sonic and Induction Field, may also be useful in evaluating unknown foundation conditions in special cases as discussed herein. In order to provide specifications for equipment and test procedures, the researchers have prepared a document entitled "Guideline Document for Unknown Subsurface Bridge Foundation Testing". A more complete discussion of the specification and performance of Parallel Seismic and Ultraseismic tests is presented in the guideline document. The guideline

document is intended to provide sufficient information for State Departments of Transporation to both specify and/or perform unknown bridge foundation studies.

REFERENCES

1. Pagan-Ortiz, J.E., "Scour Evaluation Program, National Status Report", Eastern/Southern Resource Centers, Hydraulic Engineers Conference, July, 2000.

2. Brooks, R., Engineering Data Management, Inc., "Determining In-Situ Timber Pile Length Using Stress Wave", Timber Bridge Initiation Special Projects Program, USDA Forest Service, Morgantown, West Virginia, (1992).

3. Beattie, G. J. "Pile Length Determination - Induction Field Method," Technical Note 5-3, Works Consultancy Services, Central Laboratories, Gracefield, P.O. Box 30-845, Lower Hutt, New Zealand, (1982).

4. Wright P.D., "Rangitaiki River Bridge, Te Teko, Pile Length Investigations", Ministry of Works and Development, Central Laboratories Report 5-79/9, Lower Hutt, New Zealand, (1979).

5. Douglas, R. A., and Holt J. D., "Determining Length of Installed Timber Pilings by Dispersive Wave Propagation Methods", Report: Center for Transportation Engineering Studies, North Carolina State University, June, 1993.

6. Holt, J.D. and Douglas, R.A., "A Field Test Procedure for Finding the Overall Lengths of Installed Timber Piles by Dispersive Wave Propagation Methods, Technical Report No. NC/R&D/94-001, ITRE, North Carolina State University, March, 1994.

7. Olson, L.D., Jalinoos, F. and Aouad, M.F., "Determination of Unknown Subsurface Bridge Foundations," NCHRP 21-5, Final Report, August, 1995.

8. Koten, H. van and Middendorp, P., "Testing of Foundation Piles", HERON, joint publication of the Department of Civil Engineering of Delft University of Technology, Delft, The Netherlands, and Institute TNO for Building Materials and Sciences, Rigswijk (ZH), The Netherlands, vol. 26, no. 4, (1981).

9. Davis, A.G. and Dunn, C.S., "From Theory to Field Experience with the Non-Destructive Vibration Testing of Piles," Proceedings, Institute of Civil Engineers, Part 2, (Dec 1975), pp. 571-593.

10. Chen, S., "Comparing Three Methods for Finding the Overall Lengths of Installed Timber Piles", 77th Annual Meeting, Transportation Research Board, Washington, D.C., 1998

11. Kim, Y.R., and Qian, Y., "Prediction of Remaining Cross-Sectional Area of Timber Piles Using Stress Wave Techniques", 77th Annual Meeting, Transportation Research Board, Washington, D.C., 1998.

12. Hughes, M.L., Rix, G.J. and Jacobs, L.J., "Nondestructive Determination of Pile Tip Elevation

Using Modal Analysis", Proceedings, Structural Materials Technology III, San Antonio, Texas, April, 1998.

13. Liao, S. T., "Nondestructive Testing of Piles", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1994.

14. Kolsky, H., Stress waves in solids: Dover Publ., Inc., 1963.

15. Heisey, J. S., Stokoe, K. H., Hudson, W. R., and Meyer A. H., "Determination of In Situ Shear Wave Velocity from Spectral Analysis of Surface Waves", Research Report 256-2, Center for Transportation Research, The University of Texas at Austin, Austin, 1982.

16. Nazarian, S., and Stokoe, K. H., "Nondestructive Testing of Pavements Using Surface Waves", Transportation Research Board, Record 993, Washington, D. C., 1984.

17. Nazarian, S., "In Situ Determination of Elastic Moduli of Soil Deposits and Pavement Systems by Spectral Analysis of Surface Waves Method", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1984.

18. Sheu, J. C., "Applications and Limitations of the Spectral Analysis of Surface Waves Method", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1987.

19. Rix, G. J., "Experimental Study of Factors Affecting the Spectral Analysis of Surface Waves Method", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1988.

20. Stokoe, K. H., II, Nazarian, S., Rix, G. J., Sanchez-Salinero, I., Sheu, J.C., and Mok, Y.J., "In Situ Seismic testing of Hard-to-Sample Soils by Surface Wave Methods", Proceedings, Earthquake Engineering and Soil Dynamics II, ASCE Specialty Conference, Utah, (1988), pp. 264-278.

21. Aouad, M. F., "Evaluation of Flexible Pavements and Subgrades using the Spectral Analysis of Surface Waves Method", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1993.

22. Stokoe, K. H., II, Wright, S. G., Roesset, J. M., Gauer, R. C., and Sedighi-Manesh, M., "In-Situ Measurement of Stiffness Profiles in Ocean Bottom Materials Using the SASW Method", 22nd Annual Offshore Technology Conference, Houston, Texas, (1990), pp. 299-305.

23. Chang, D. W., "Nonlinear Effects on Dynamic Response of Pavements Using the Nondestructive Testing Techniques", Dissertation Submitted in Partial Fulfillment of the Doctor of Philosophy Degree, The University of Texas at Austin, Austin, 1991.

24. Foinquinos, R. M., "Analytical Study and Inversion for the Spectral Analysis of Surface Waves Method", Thesis Submitted in Partial Fulfillment of the Master of Science Degree, The University of Texas at Austin, Austin, 1991.

25. Stain, R.T., "Integrity Testing", Civil Engineering, (April, 1982), pp. 53-73.

26. Olson, L.D., Wright, C.C., "Nondestructive Testing of Deep Foundations with Sonic Methods", Proceedings, ASCE Geotechnical and Construction Divisions Specialty Conference, Northwestern University, Evanston, Illinois, (1989).

27. Novak, M., "Dynamic Stiffness and Damping of Piles", Canadian Geotechnical Journal, Vol II, (1976), p. 574.

28. Duke, S. K., "Calibration of Ground Penetrating Radar and Calculation of Attenuation and Dielectric Permittivity Versus Depth", Mater's Thesis# T-3920, Colorado School of Mines, Golden, Co, (1990), 236 pp.

29. Haeni, F.P., Placzek, G., Trent, R.E., "Use of Ground Penetrating Radar to Investigate Refilled Scour Holes at Bridge Foundations", Geophysical Survey of Finland, Special Paper 16, (1992), pp. 285-292.

30. Olson, L.D., "Nondestructive Testing of Unknown Subsurface Bridge Foundations - Results of NCHRP Project 21-5", Research Results Digest, Transportation Research Board, NCHRP, Number 213, December, 1996.

31. Olson, L.D., and Liu, M., "Dynamic Bridge Substructure Evaluation and Monitoring System", FHWA Report, December, 2000 (in progress).

32. Olson, L.D., and Aouad, M.F., "Unknown Subsurface Bridge Foundation Testing," NCHRP 21-5 (2), Phase IIA Research, Interim Report, April, 1998.

33. Olson, L.D., and Aouad, M.F., "Unknown Subsurface Bridge Foundation Testing," NCHRP 21-5 (2), Phase IIB Research, Interim Report, February, 2000.

APPENDIX A

BRIDGE DESCRIPTIONS

APPENDIX A

BRIDGE DESCRIPTIONS

A.1 NCHRP 21-5, PHASE I BRIDGES

A wide range of bridge types with varying geology and hydrology conditions were investigated during the field research of Phase I. The seven bridges selected for field nondestructive testing were proposed by the research team and approved by the NCHRP 21-5 Panel. Testing was performed of 7 bridges for the NCHRP 21-5 field research: 4 in Colorado (Golden, Coors, Franktown, and Weld County), 2 in Texas (old and new Bastrop bridges) and 1 near Prattmont, Alabama. A listing of the nondestructive test methods applied at each of the bridge sites is presented in Table A-I.

In terms of foundation substructures and materials, concrete substructure bridges which were nondestructively tested included spread footing (Golden), wall concrete pier and stub abutment supported by steel piles (Weld), shallow spread footing and pilecap supported by steel BP piles (Coors), a concrete pile foundation with a pile cap (old Bastrop), a concrete caisson foundation (old Bastrop), and a concrete drilled shaft foundation (new Bastrop). A timber bridge with a timber pile pier and abutment (Franktown) and a steel BP pile bridge (Alabama) were also tested. Descriptions of the 7 tested bridges are given below.

Table A-I. **Applied NDT Methods on Seven Bridges in Colorado, Alabama and Texas.**

Bridge Location	Tested Unit	Applied NDT Methods									
		Substructure NDT								Soil NDT	
		US	SE/IR	BW	SR	DFR	PS	BHR	BHS	CH	SASW
Golden (Colorado)	North Pier	X	X			X					
Coors (Colorado)	Pier 4	X	X		X	X	X	X		X	
	Pier 2	X	X		X	X		X			
Franktown (Colorado)	Northeast Wingwall	X	X	X							X
	Middle Pier	X	X								
Weld (Colorado)	West Abutment		X			X					
	West Pier	X									
Alabama	Bent 4	X	X				X	X			
Old Bastrop (Texas)	Caisson	X	X			X	X	X	X	X	
	Piles		X			X	X	X	X		
New Bastrop (Texas)	Drilled Shaft	X	X			X	X	X	X		

US = Ultraseismic; SE/IR = Sonic Echo/Impulse Response; DFR = Dynamic Foundation Response; PS = Parallel Seismic; BHR = Borehole Radar; BHS = Borehole Sonic; CH = Crosshole Seismic of Soils; SASW = Spectral Analysis of Surface Waves of Soils; BW = Bending Wave Method; SR = Surface Radar.

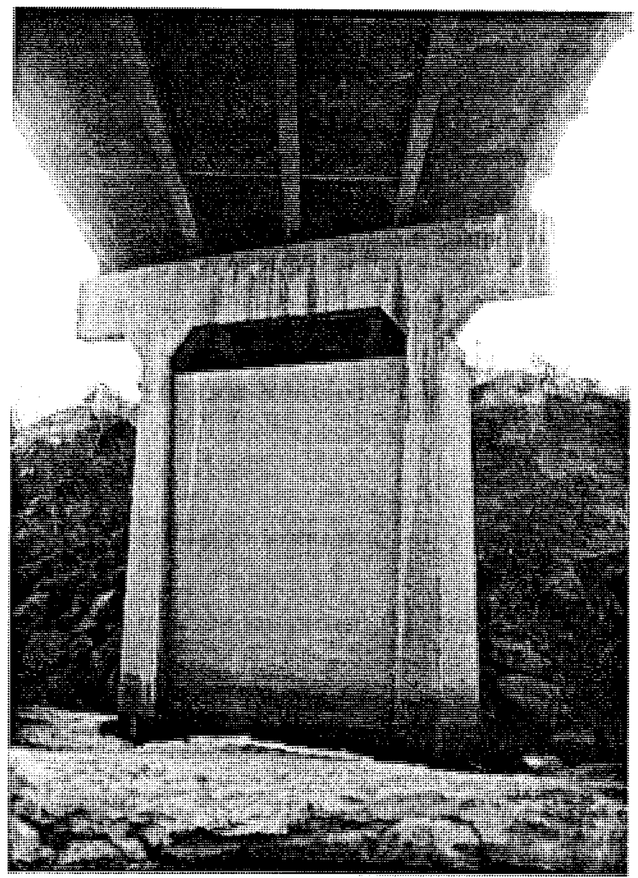
A.1.1 Golden Bridge, Colorado.

This bridge is located on Colorado State Highway 6 over Clear Creek near Golden, Colorado (Structure No. E-16-EF, Colorado Department of Transportation). The bridge consists of a 3-span (62'x76'x62') continuous concrete slab 30' wide on steel I-beams. The superstructure is supported by two concrete abutments and two concrete piers resting on embedded spread footings. The following summary substructure dimensions were extracted from the engineering drawings:

Abutments. Cap: 4' 2 7/8" high, 2' 11 ½" wide, 34' 9" long. Columns: 2 columns (three vertical sides and one battered), 29' long, 2' thick, 10' 2 ½" wide at bottom and 2' 11 ½" wide at top. The two columns are connected by a transverse beam. The beam is 4' thick, 2' wide and 21' 6" long. The top of the beam is at 16' 6" from the bottom of the columns. Footings: 2 spread footings 15'x7'x2'. Part of the columns and all of the footings are buried in sand and gravel.

Piers. Cap: 4' 8 11/16" high, 2 3/4", 37' wide, 34' 9" long. Columns: 2 columns (battered on sides), 35' long, square cross-section (3' 9 2/8" x 3' 9 2/8" at bottom, 2' 3 3/4" x 2' 3 3/4" at top). Footings: 2 spread footings: 10' x 8' 6" x 2'. Two photographs of the Golden Bridge are shown in Fig. A.1.

a. North Pier (South Face)



b. South Pier (North Face)

Figure A.1- Photographs of Golden Bridge, Colorado State Highway 6 over Clear Creek, Colorado.

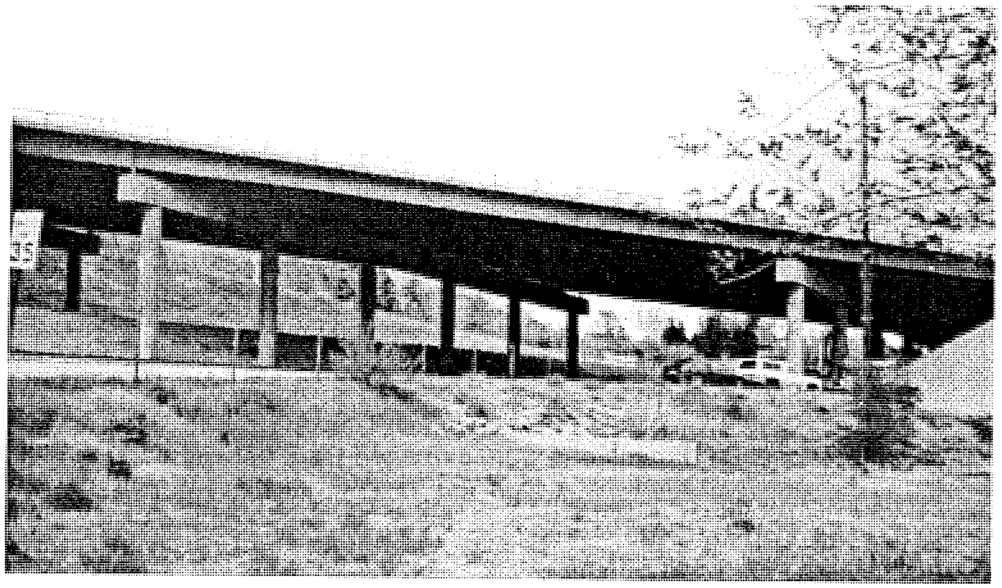
A.1.2 Coors Bridge, Colorado.

This bridge is located on State Highway 58 and crosses W. 44th Avenue near the Coors brewery in Golden, Colorado (Structure No. E-16-HI, Colorado Department of Transportation). An irrigation canal is located immediately to the south of this bridge. The bridge consists of a 4-span (74'x117'x117'x66' 6") continuous concrete slab 82' 6" wide on steel I-beams. The superstructure is supported by two abutments and three piers resting on either spread footings or piles. The following summary substructure dimensions were extracted from the engineering drawings:

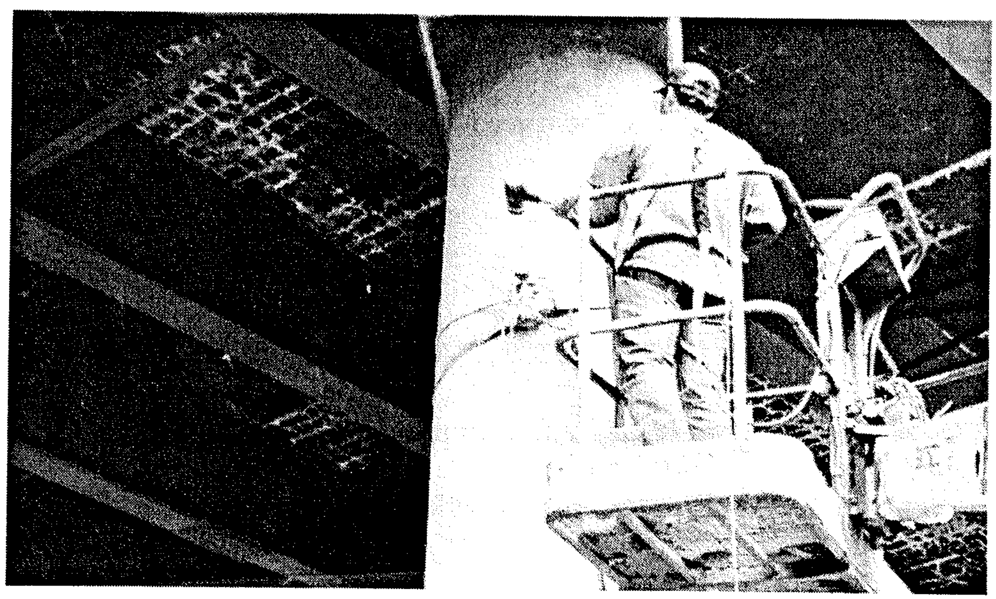
Abutments. 12BP53 bearing piles of lengths varying from 39 to 49 ft.

Piers 2&3. Cap: 4' 3" high, 3' 4" wide, 133' 10" long. Columns: 6 circular columns 3' 0" in diameter, 19' to 31' long. Footings: buried spread footings 8' x 8' x 2' 6". Tests were performed on the north column of Pier 2.

Pier 4. Cap: 4' 3" high, 3' 4" wide, 135' 6" long. Columns: 6 circular columns 3' 0" in diameter, 16' to 24' long. Footings (under each column): pile cap 6'x6'x3' 4", five 12BP53 piles 25' long. Tests were performed on the south column of Pier 4. Two photographs of the Coors Bridge are shown in Fig. A.2.



a. Views of Pier 3 (Left) and Pier 4 (Right



b. Testing at Pier 4 with 12 lb Impulse Hammer and bi-axial accelerometers
Dynamic Foundation Response Testing

Figure A.2- Photographs of Coors Bridge, Colorado State Highway 58 over West 44th Avenue in Golden by the Coors Brewery, Colorado.

A.1.3 Franktown Bridge, Colorado.

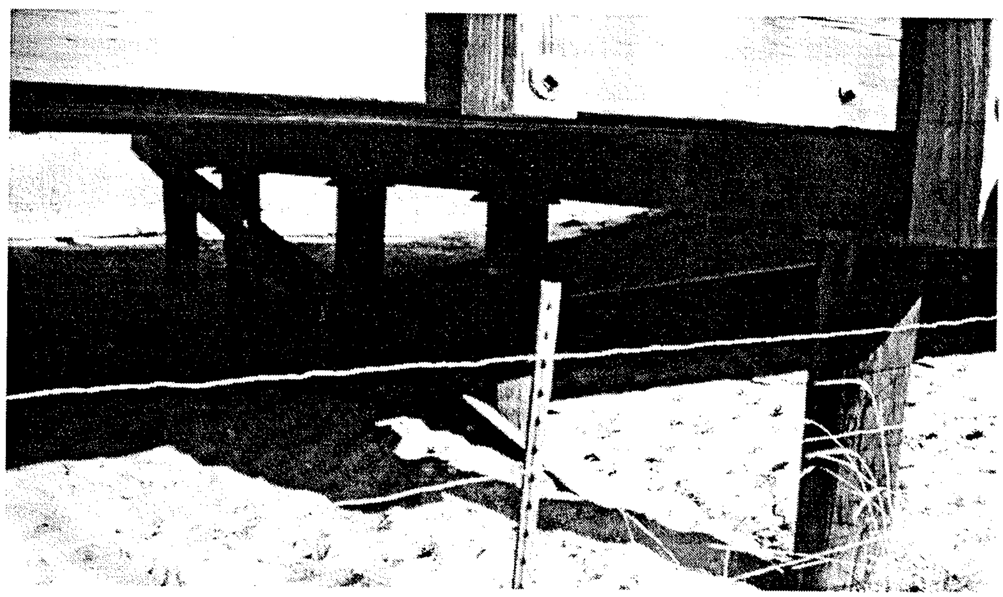
This timber bridge is located on State Highway 86 across a dry wash near Franktown, Colorado (Structure No. G-17-W, Colorado Department of Transportation). Water was observed flowing in the wash in Spring, 1994. This bridge consists of a 2-span continuous slab 25' wide. The superstructure is supported by 2 abutments and 1 pier. The following summary dimensions were extracted from the engineering drawings:

Abutments. 2 similar abutments consisting of treated timber piles 12" in diameter driven to a depth of 28 ft (14 piles at each abutment: 6 piles for the breast wall of the abutment, 4 piles at the left wing wall and 4 piles at the right wing wall).

Pier. 5 treated timber piles 12" in diameter driven to a depth of 28 ft. It should be mentioned that the upper 4 to 5 feet of the piles are exposed.

Subsurface Conditions. The materials encountered at this bridge site consist of clay and streaks of sand. The water table was at 17 ft below the ground surface. Spectral Analysis of Surface Waves (SASW) measurements at this bridge site showed an average surface wave velocity of 340 ft/sec, which is a low value representing soft material. Two photographs of the Franktown Bridge are shown in Fig. A.3.

a. Sonic Echo/Impulse Response and Bending Wave Testing at Pile 2 of the Northeast Wing With 3 lb Impulse Hammer



b. Center Pier Supported by 5 timber piles

Figure A.3- Photographs of the Franktown Bridge, State Highway 86, Colorado.

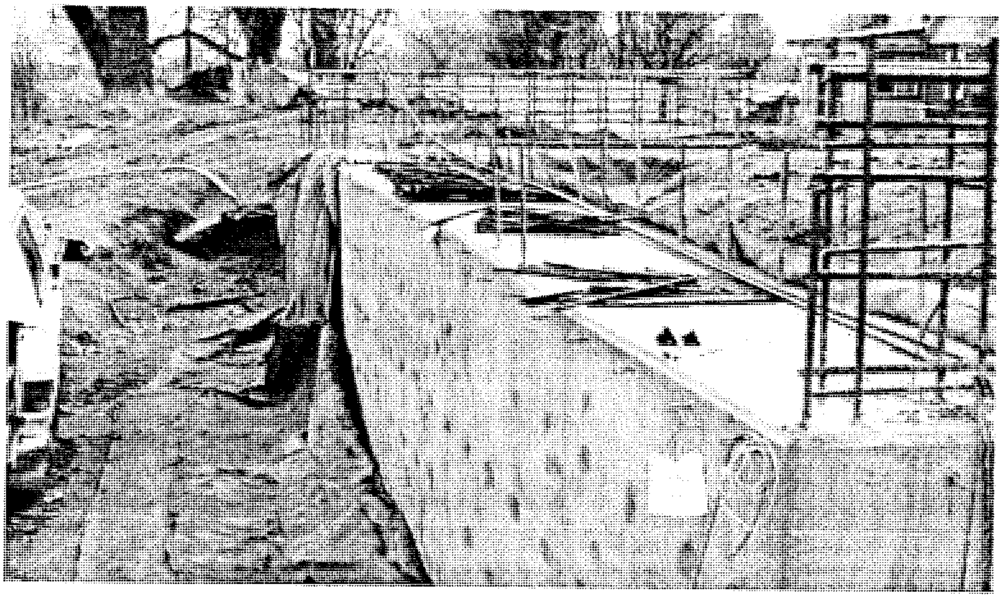
A.1.4 Weld County Bridge, Colorado.

This bridge was constructed during this project on Weld County Road 8 over the South Platte River in Weld County, Colorado north of Brighton. The bridge consists of a 3-span (66' 2"'x66' 2"x66' 2") continuous concrete slab 34' 6" wide on precast concrete box girder. The superstructure is supported by two abutments and two piers resting on steel piles. The following summary dimensions were extracted from the engineering drawings:

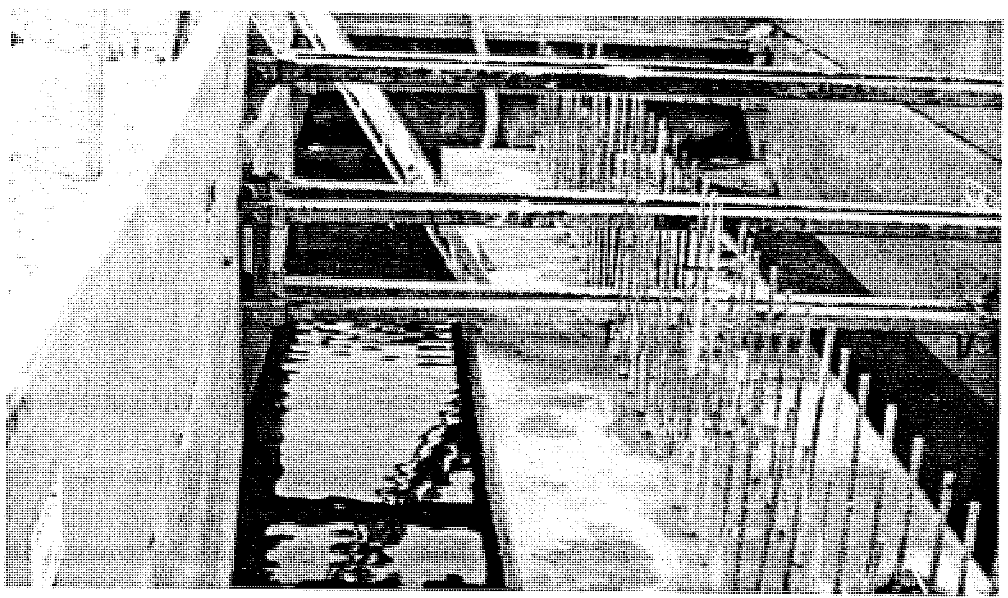
Abutments 1&4. 34' 6" long, 2' 6" wide and approximately 6 ft thick; Four 12HP53 steel piles approximately 35 ft long. Tests were performed on Abutment 1 which was referred to as the west abutment in this report.

Piers 2&3. Wall: 35' long, 3' wide and 16' thick; Cap: 36' long, 6' 6" wide and 3' thick.; Twelve 12HP53 steel piles approximately 22 ft long. Tests were performed on Pier 2 which was referred to as the west pier in this report.

Subsurface Conditions. The materials encountered at this bridge site consist of brown silty sand with scattered gravel followed by grey silty claystone bedrock. The materials encountered in one of the test holes were as follows: 7 ft of fill, primarily a brown to dark brown, silty sand with scattered gravel; 23 ft of brown silty sand with scattered gravel (blow counts ranging from 7 to 21); 6 ft of brown silty sand and gravel (blow counts of 29) underlain by grey silty claystone bedrock. Two photographs of the Weld County bridge are shown in Fig. A.4.



a. West Abutment



b. West Pier During Construction (Pilecap is Shown)

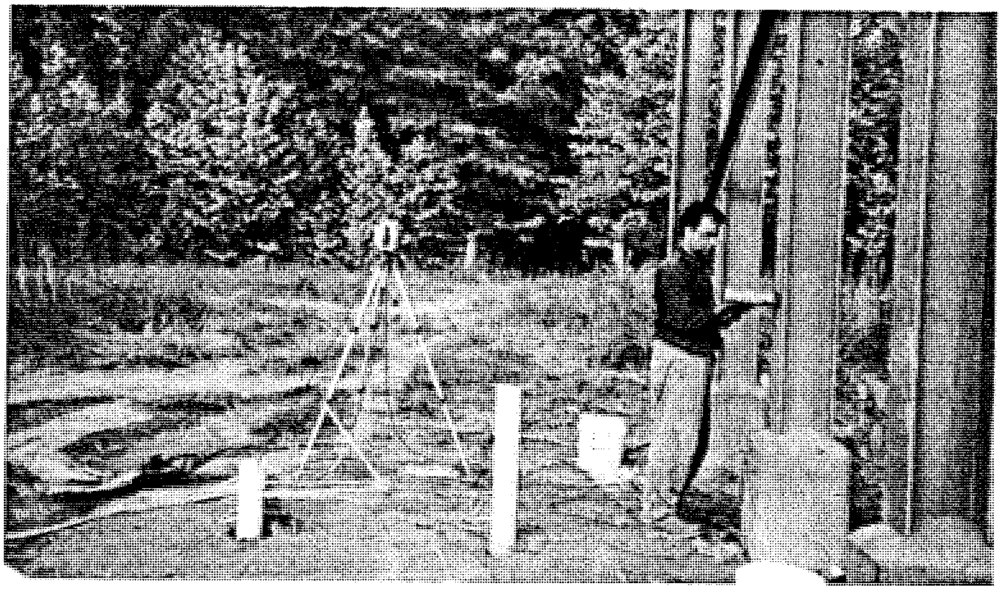
Figure 1.4- Photographs of the New Weld County Bridge, Weld County Road 8 over the Platte River bear Brighton, Colorado.

A.1.5 Alabama River Bridge, Alabama.

This bridge is located near Montgomery by Prattmont, over the Alabama river on Alabama State Highway 31 connecting Hunter Station to Prattmont, Alabama (Structure No. FI-117 (2), Alabama Department of Transportation). This bridge consists of a 38-span concrete slab deck that is 28 ft wide and is supported by 10BP42 steel H-piles. Some of the spans are supported by concrete piers resting on steel piles. The majority of the spans are supported by steel piles which extend to the bottom of the supporting beams. The embedded depths of the piles varied from 38.5 to 39 ft as shown on the as-built plans with splices at 4 to 5 ft below the ground surface.

Subsurface Conditions. The materials encountered at this bridge site consist of 9.4 ft of stiff brown clay underlain by 24.6 ft of brown tan clay with sand underlain by 6 ft of dense sand with some silty clay underlain by 20 ft of very dense sand. Two photographs of the Alabama Bridge are shown in Fig. A.5.

a. Parallel Seismic Testing at the Center Pile of Bent 4 with the 12 lb Impulse Hammer



b. Ultraseismic Testing at the Center Pile of Bent 4

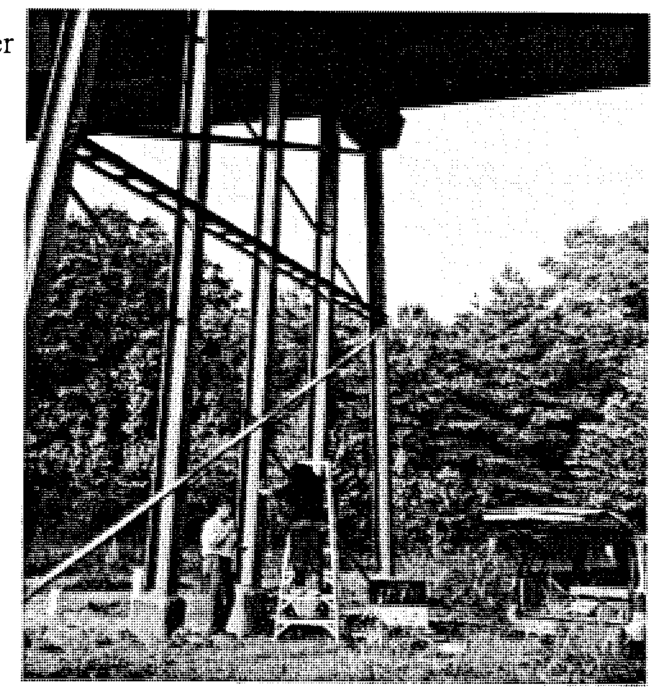


Figure A.5- Photographs of the Alabama Bridge, Highway 31 over the Alabama River Near Prattmont, Alabama.

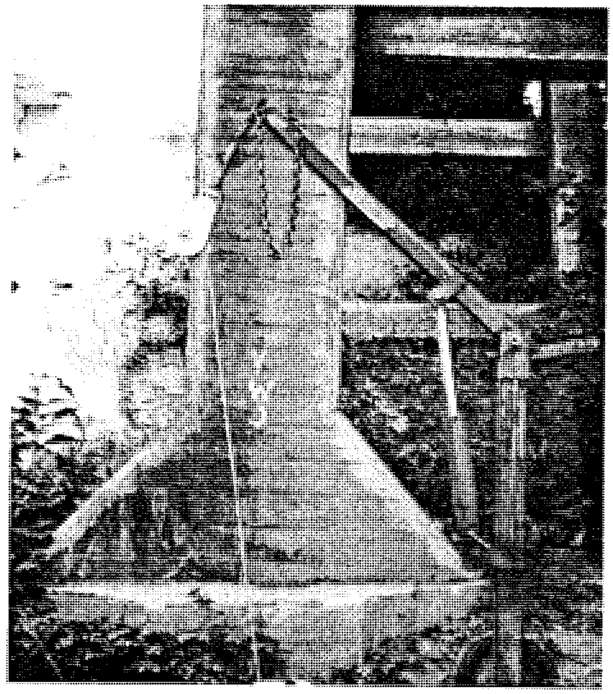
A.1.6 Bastrop Bridges, Texas.

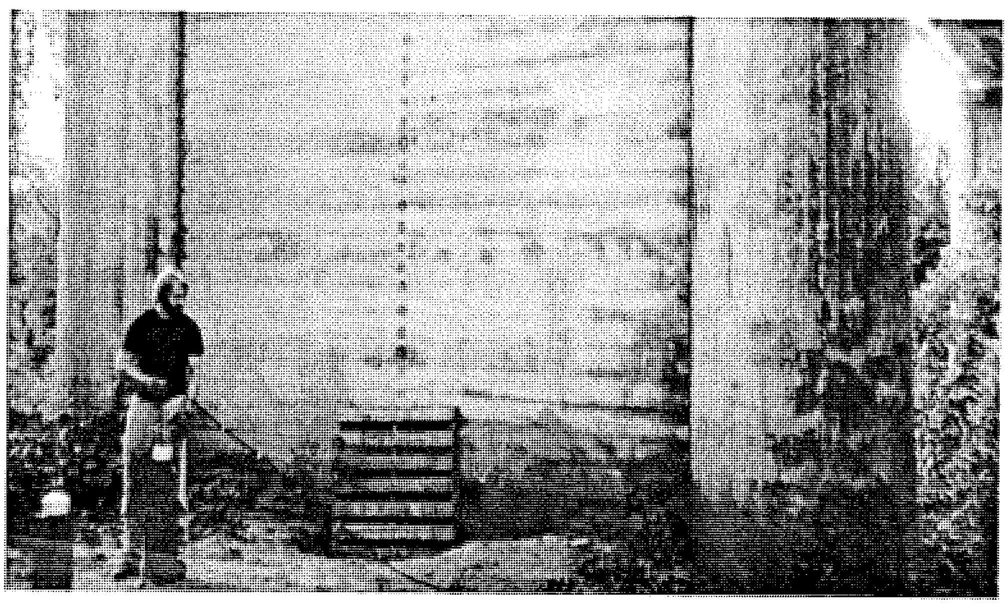
A historic old bridge is located over the Colorado River on Texas State Highway 3A (now Loop 150) in Bastrop, Texas. The bridge consists of 6 bents each 39 ft long and 3 steel truss spans each 192 ft long. The old Bastrop Bridge is now owned and used for pedestrian traffic by the City of Bastrop. A new concrete substructure and steel girder bridge was recently built to the north of the historic bridge by the Texas DOT on Loop 150.

Three types of foundations were tested at the Bastrop bridge. A caisson foundation and a foundation with concrete piles were tested at the old Bastrop bridge. Two photographs of the two types of foundations of the old Bastrop bridge are shown in Fig. A.6. A drilled shaft foundation which is a part of the new Bastrop bridge was also tested. A photograph of the new Bastrop bridge is shown in Fig. A.7.

Subsurface Conditions. The materials encountered at this bridge site consist of sand, loam and clay in the upper 10 ft underlain by 10 ft of sand underlain by 10 ft of sand and gravel underlain by 8 ft of packed sand underlain by shale. Crosshole Seismic (CS) tests were performed between Boreholes 2 and 3 near the caisson foundation. The CS results showed an increase in the shear wave velocity (decrease in the time arrival) from the surface to a depth of 35 ft. Below a depth of 35 ft, which is considered to represent the beginning of the shale layer, the shear wave velocity remained constant. The depth of 35 ft is close to the 38 ft depth of the shale layer obtained from the boring shown on the plans. The boring on the plans is located about 150 ft away from the caisson foundation.

a. Pile Pier, Old Bastrop Bridge





b. Caisson Pier, Old Bastrop Bridge

Figure A.6- Photographs of the Old Bastrop Bridge Caisson and Pile Piers, Old Texas Highway Loop 150, Now a National Historic Landmark Serving as a Pedestrian Bridge over the Colorado River for the City of Bastrop, Texas.

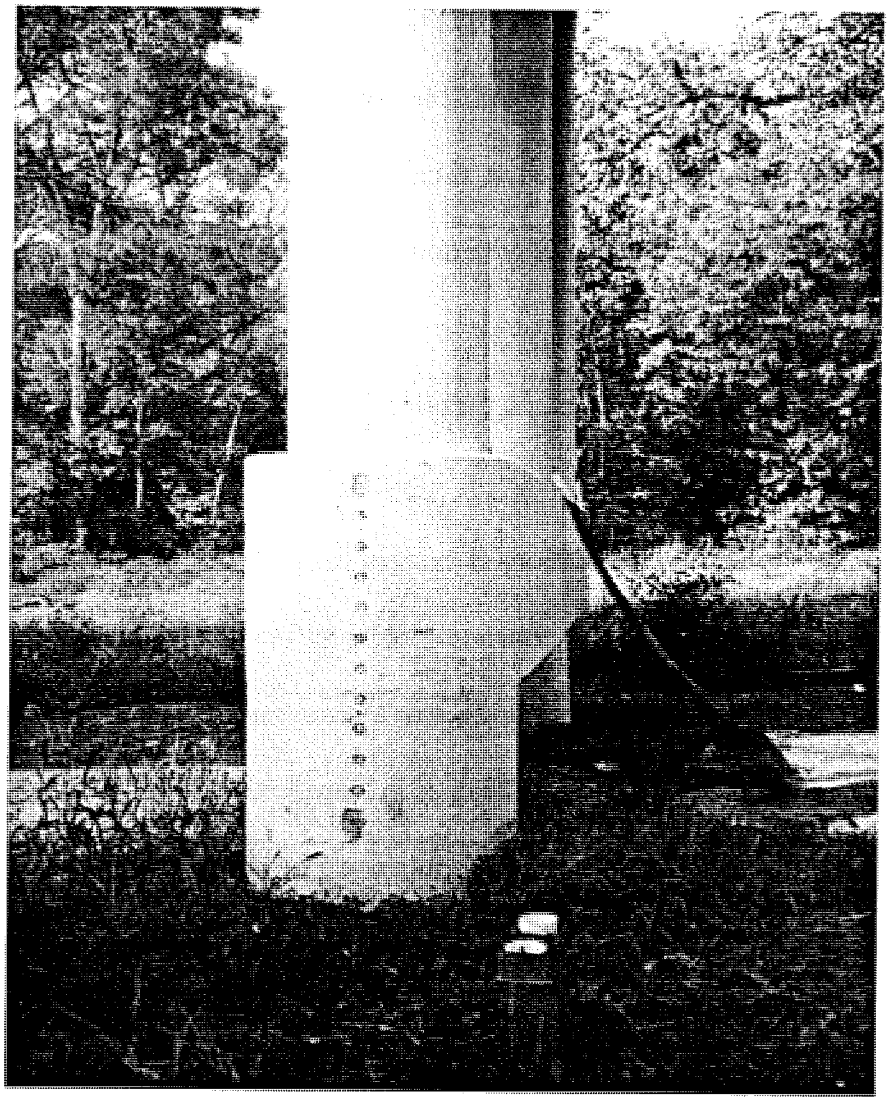


Figure A.7- Photograph of the New Bastrop Bridge, Texas Highway Loop 150 over the Colorado River, Texas.

A.2 NCHRP 21-5 (2), PHASE II BRIDGES

A wide range of bridge types with varying geology and hydrology conditions were investigated during the field research of Phase II. The 21 bridges selected for field nondestructive testing were proposed by the research team and approved by the NCHRP 21-5 Panel. Testing was performed of 21 bridges for the NCHRP 21-5 (2) field research: 6 in North Carolina, 3 in Minnesota, 3 in New Jersey, 3 in Michigan, 2 in Oregon, 2 in Massachusetts and 2 in Colorado. A listing of the nondestructive test methods applied at each of the bridge sites is presented in Table A-II including the additional bridge in Texas with no available plans.

In terms of foundation substructures, the bridges which were nondestructively tested included the following:

1. Exposed timber pile foundations (5 bridges in North Carolina, 1 bridge in Minnesota),
2. Timber piles buried below concrete piercap or concrete abutments (1 bridge in North Carolina, 2 bridges in Oregon, 2 bridges in Massachusetts, and 1 bridge in Colorado,
3. Massive concrete abutments (3 bridges in New Jersey),
4. Shallow footings (1 bridge in Minnesota and 1 bridge in Michigan),
5. Cast in place or driven concrete piles below massive concrete pilecaps (2 bridges in Michigan),
6. Concrete filled steel piles (1 bridge in Minnesota), and
7. Steel H-piles below concrete pilecaps (1 bridge in Colorado and 1 bridge in Texas).

Descriptions of the 7 tested bridges are given below.

Table A-II. Applied NDT Methods on Twenty one Bridges, Phase II Research, NCHRP 21-5 (2).

Bridge Location	Tested Unit	Applied NDT Methods			
		PS	US	SE/ IR	BW
Wilson County, Bridge # 5, North Carolina	Bent 1, Timber Piles	X	X	X	X
Johnston County, Bridge # 33, North Carolina	Bent 1, Timber Piles	X	X	X	X
Wake County, Bridge #251, North Carolina	Pier, Buried Concrete Pilecap Over Timber Piles	X	X		
Bridge No. 5188, Hwy 58, Zumbrota, Minnesota	Pier, Concrete Columns Supported by a Concrete Footing		X	X	
Bridge No. 9798, Highway 60, Minnesota	Pier, Timber Piles	X	X	X	X
Bridge # 55028, Hwy 52, North of Chatfield, Minnesota	Pier, Concrete filled Steel Pipe	X	X	X	X
Bridge # 1103-151, Route 1 over Shipetaukin Creek, Mercer County, New Jersey	Concrete Abutment	X	X	X	
Bridge # 1123-152, Route 130 over Rock's Brook, Mercer County, New Jersey	Concrete Abutment	X	X	X	
Bridge # 0324-158, Route 206 over Rancocas Creek, Burlington County, New Jersey	Concrete Abutment	X			
Bridge # B01 of 33045A, I496 over Red Cedar River, Michigan	Concrete Plint Supported by Cast in Place Concrete Piles	X	X	X	
Bridge # B04 of 64015E, US31 over Pentwater River, Michigan	Concrete Pier # 2 Supported by Concrete Driven Piles	X	X	X	
Bridge # X01 of 25085B, Fenton Road, Thread Creek, Michigan	Concrete Pier Supported by a Shallow Strip Footing	X	X	X	
Santiam River Overflow # 5 Bridge, Near Salem, Oregon	Abutment, Concrete Pilecap Supported by Timber Piles	X			
Cordon Road Overcrossing Hwy 22 Bridge, Near Salem, Oregon	Concrete Abutment Supported by Treated Timber Piles	X	X		
Bridge on Dudley Road, Structure # 0-6-11, Oxford, Massachusetts	Concrete Pilecap Supported By Timber Piles	X	X		
Bridge on Route 122, Structure # U-2-21, Uxbridge, Massachusetts	Concrete Abutment Supported By Timber Piles	X	X		

Table A-II. Applied NDT Methods on Twenty one Bridges, Phase II Research, NCHRP 21-5 (2), Cont.

Bridge Location	Tested Unit	Applied NDT Methods			
		PS	US	SE/ IR	BW
Johnston County, Bridge # 129, North Carolina	Bent 2, Timber Piles	X	X	X	X
Johnston County, Bridge # 145, North Carolina	Bent 1, Timber Piles	X	X	X	X
Wake County, Bridge # 207, North Carolina	Bent 4, Timber Piles		X	X	X
Bridge on US 287, Structure No. C-16-C, Over** Little Thompson River, Longmont, Colorado	Concrete Pilecap Supported by Steel H-Piles	X		X	
Bridge on US 52, Structure No. D-17-I, Over South Platte River, Fort Lupton, Colorado	Concrete Pier Supported by Timber Piles	X	X	X	
Additional Bridge: Over Bethel Creek, Snook** Texas, No plans available	Abutment Consisting of a Concrete Beam Supported by Steel H-Piles	X			

** These two bridges were tested with the Induction Field Method

PS = Parallel Seismic; US = Ultraseismic; SE/IR = Sonic Echo/Impulse Response; BW = Bending Wave;

A.2.1 Wilson County Bridge # 5, North Carolina.

The bridge substructure consists of steel I-beams resting on timber pilecaps supported by timber piles. Figure A.8 shows a photograph of Bent 1 of the bridge which is supported by a 6 timber pile foundation. Shown in the photo also is the borehole used for Parallel Seismic tests on the East Pile of Bent 1. This is a maintenance bridge, only pile driving records were available with bridge plans not available. The driving records are presented in the interim report submitted in April, 1998.

Figure A.8- Photograph of Bent 1, Wilson County Bridge # 5, North Carolina.

A.2.2 Johnston County Bridge # 33, North Carolina.

The bridge substructure consists of steel I-beams resting on timber pilecaps supported by timber piles. Figure A.9 shows a photograph of Bent 1 of the bridge which is supported by a 6 timber pile foundation. Shown in the photo also is the setup for Bending Waves tests on the East Pile. This is a maintenance bridge, only pile driving records were available with bridge plans not available. The driving records are presented in the interim report submitted in April, 1998.

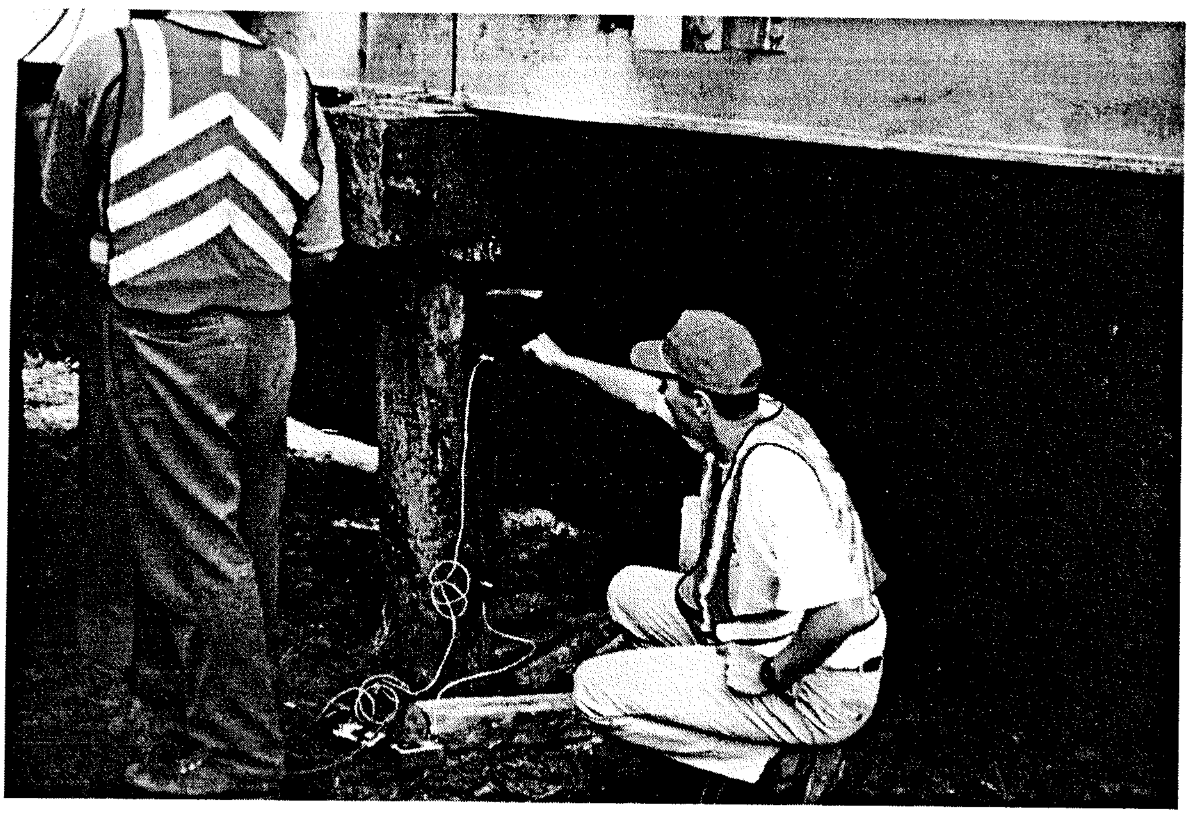


Figure A.9- Photograph of Bent 1, Johnston County Bridge # 33, North Carolina.

A.2.3 Wake County Bridge # 251, North Carolina.

This bridge is supported by two abutments and four piers. Figure A.10 shows a photograph of the bridge. Shown in the left of the photo also is the pier where testing was performed. The substructure of the pier consists of concrete beams supported by concrete columns. The columns are supported by concrete pilecaps and timber piles. The top of the pilecap was exposed to allow vertical and horizontal hammer hits for Parallel Seismic tests. The available bridge plans are presented in the interim report submitted in April, 1998.

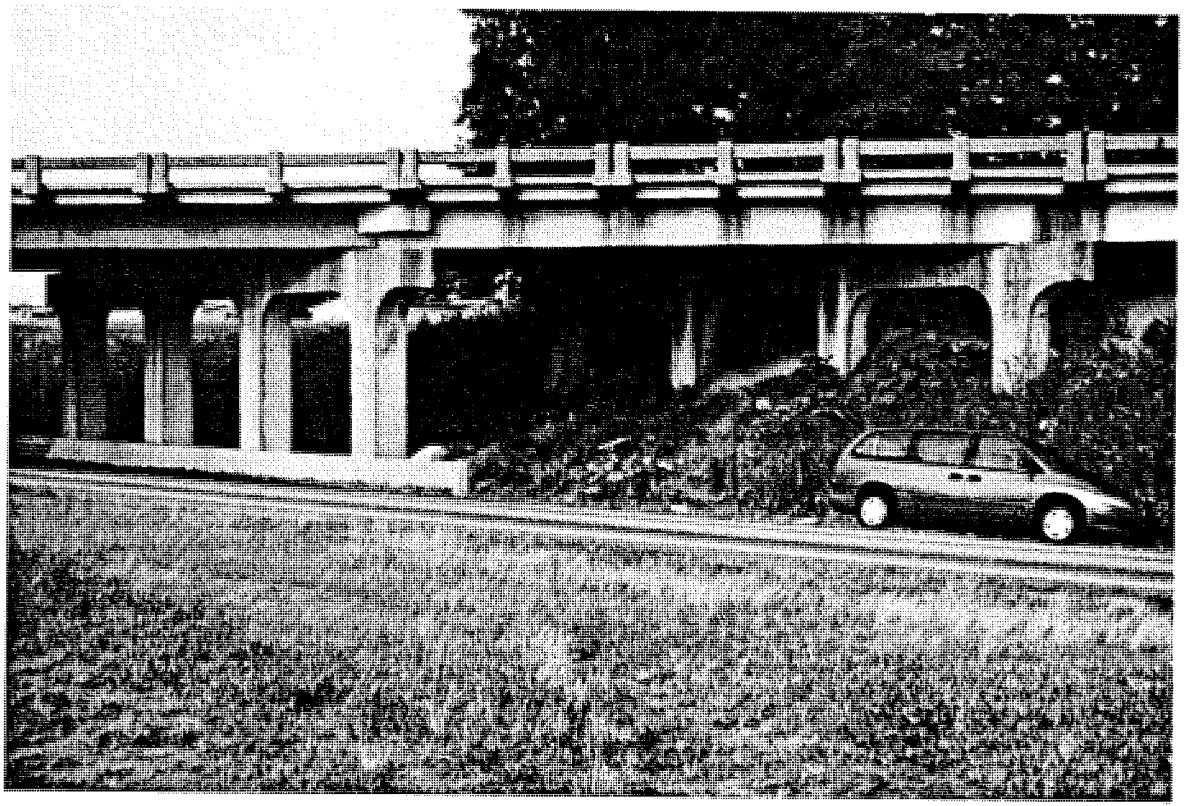


Figure A.10- Photograph of Wake County Bridge # 251, North Carolina.

A.2.4 Bridge No. 5188, Minnesota Hwy. 58, Zumbrota, Minnesota.

This bridge is supported by two abutments and one pier. Figure A.11 shows a photograph of the pier of the bridge. The substructure of the pier consists of a concrete beam supported by two concrete columns. The columns are supported by spread footings. The available bridge plans are presented in the interim report submitted in April, 1998.

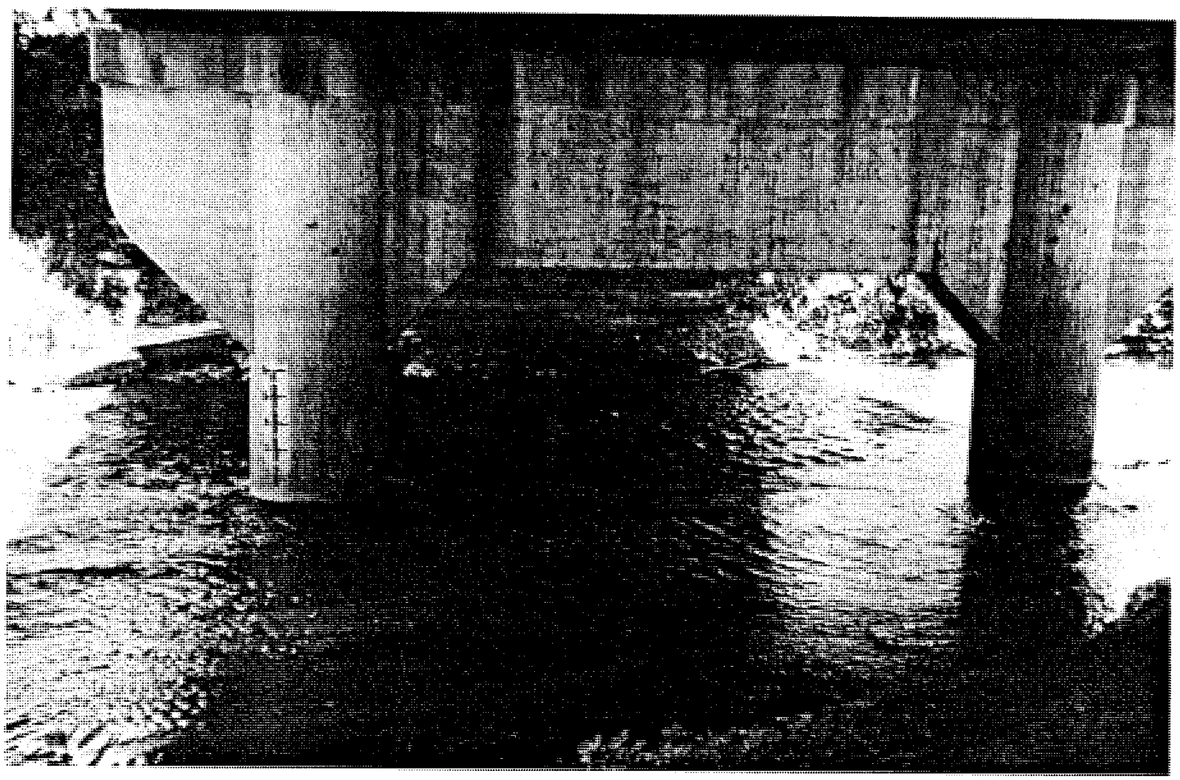


Figure A.11- Photograph of Bridge No. 5188, Minnesota Highway 58, Zumbrota, Minnesota.

A.2.5 Bridge No. 9798, Hwy. 60, Minnesota.

This bridge is supported by two abutments and two piers. Figure A.12 shows a photograph of the pier of the bridge where testing was performed. The substructure of the pier consists of a timber beam supported by ten timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

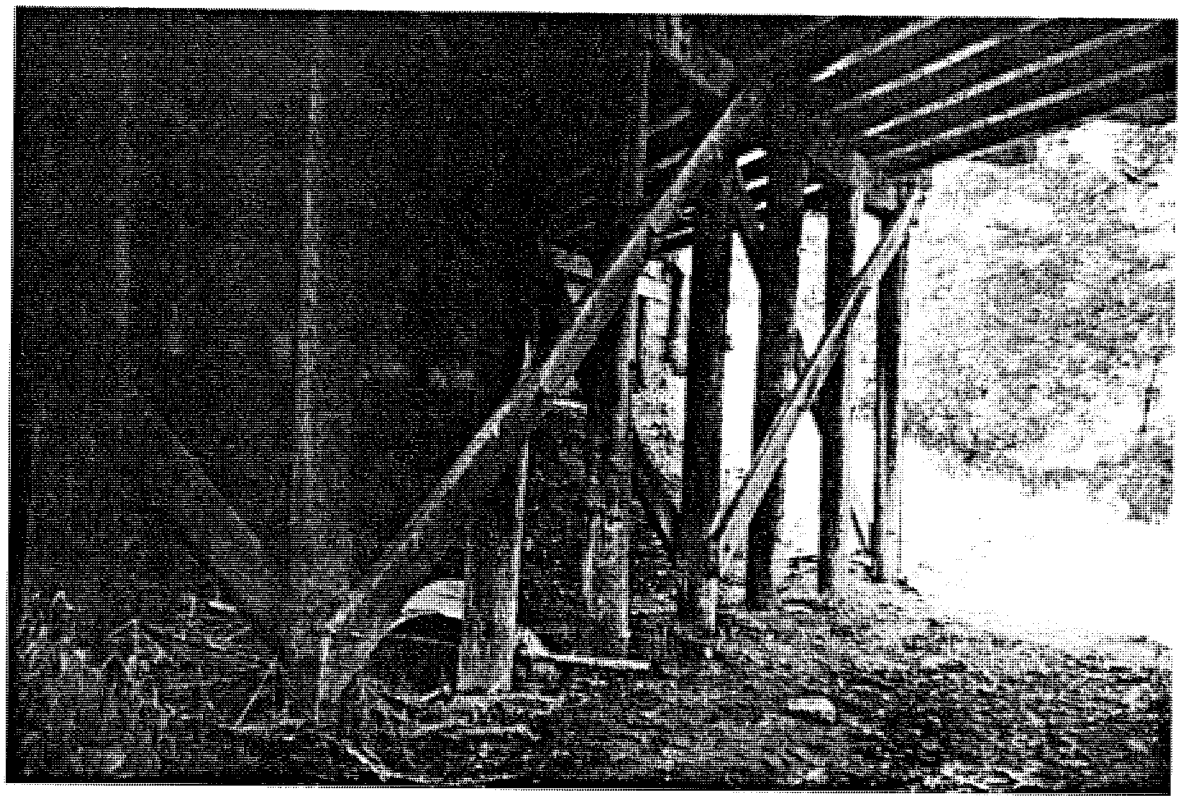


Figure A.12- Photograph of Bridge No. 9798, Hwy 60, Minnesota.

A.2.6 Bridge No. 55028, Hwy. 52, North of Chatfield, Minnesota.

This bridge is supported by two abutments and two piers. Figure A.13 shows a photograph of the pier of the bridge where testing was performed. The substructure of the pier consists of a concrete beam supported by 5 concrete-filled steel pipes. The available bridge plans are presented in the interim report submitted in April, 1998.

Figure A.13- Photograph of Bridge No. 55028, Hwy 52, North of Chatfield, Minnesota.

A.2.7 Bridge No. 1103-151, Route 1 over Shipetaukin Creek, Lawrence Twp., Mercer County, New Jersey.

This bridge is supported by two abutments and one pier. Figure A.14 shows a photograph of the massive concrete abutment of the bridge where testing was performed. The available bridge plans are presented in the interim report submitted in April, 1998.

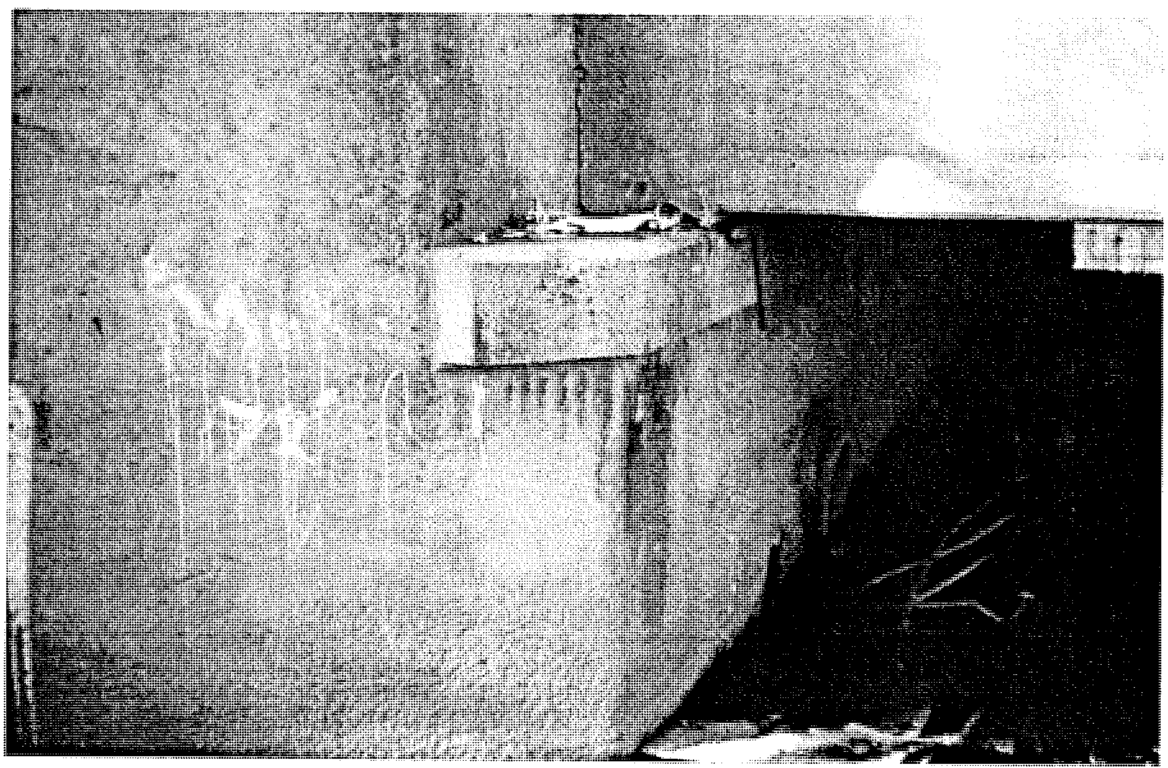


Figure A.14- Photograph of Bridge No. 1103-151, Route 1 over Shipetaukin Creek, Lawrence Twp., Mercer County, New Jersey.

A.2.8 Bridge No. 1123-152, Route 130 over Rock's Brook, East Windsor Twp., Mercer County, New Jersey.

This bridge is supported by two abutments. Figure A.15 shows a photograph of the massive concrete abutment of the bridge where testing was performed. The available bridge plans are presented in the interim report submitted in April, 1998.

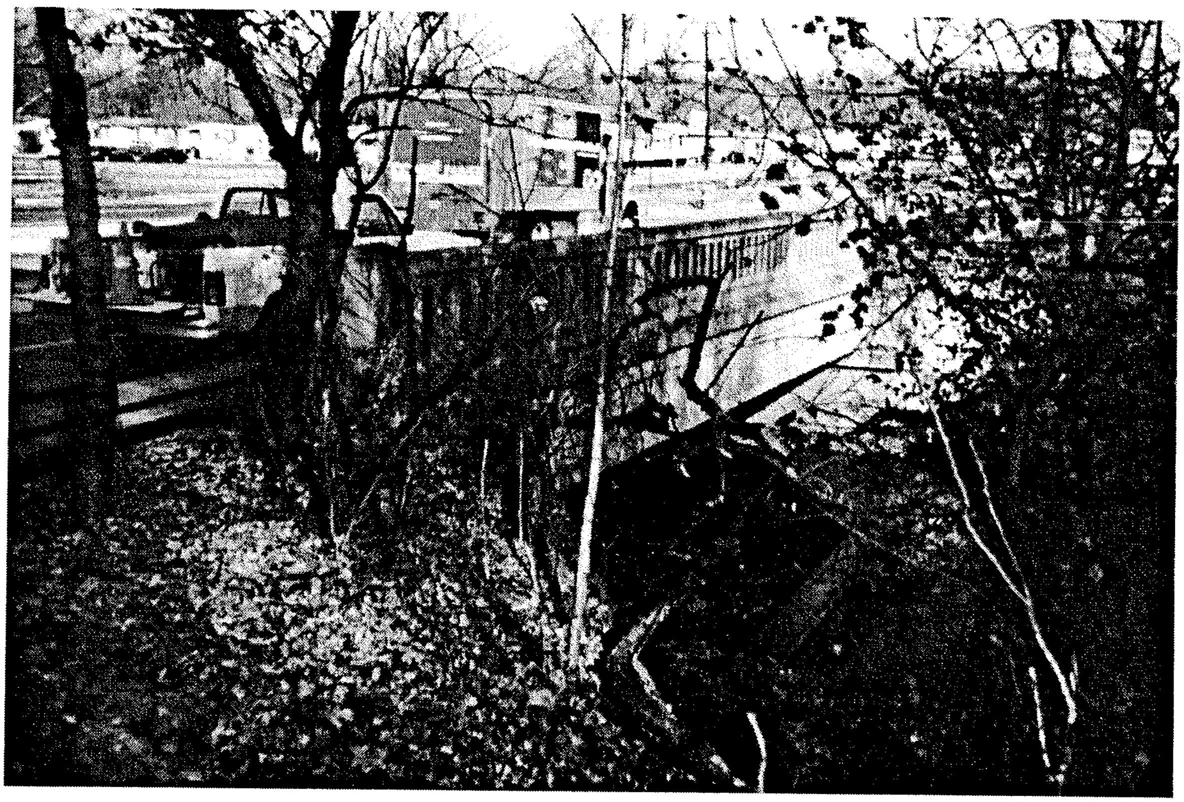


Figure A.15- Photograph of Bridge No. 1123-152, Route 130 over Rock's Brook, East Windsor Twp., Mercer County, New Jersey.

A.2.9 Bridge No. 0324-158, Route 130 over Rancocas Creek, Pemberton Twp., Burlington County, New Jersey.

This bridge is supported by two abutments. Figure A.16 shows a photograph of the massive concrete abutment of the bridge where testing was performed. The available bridge plans are presented in the interim report submitted in April, 1998.

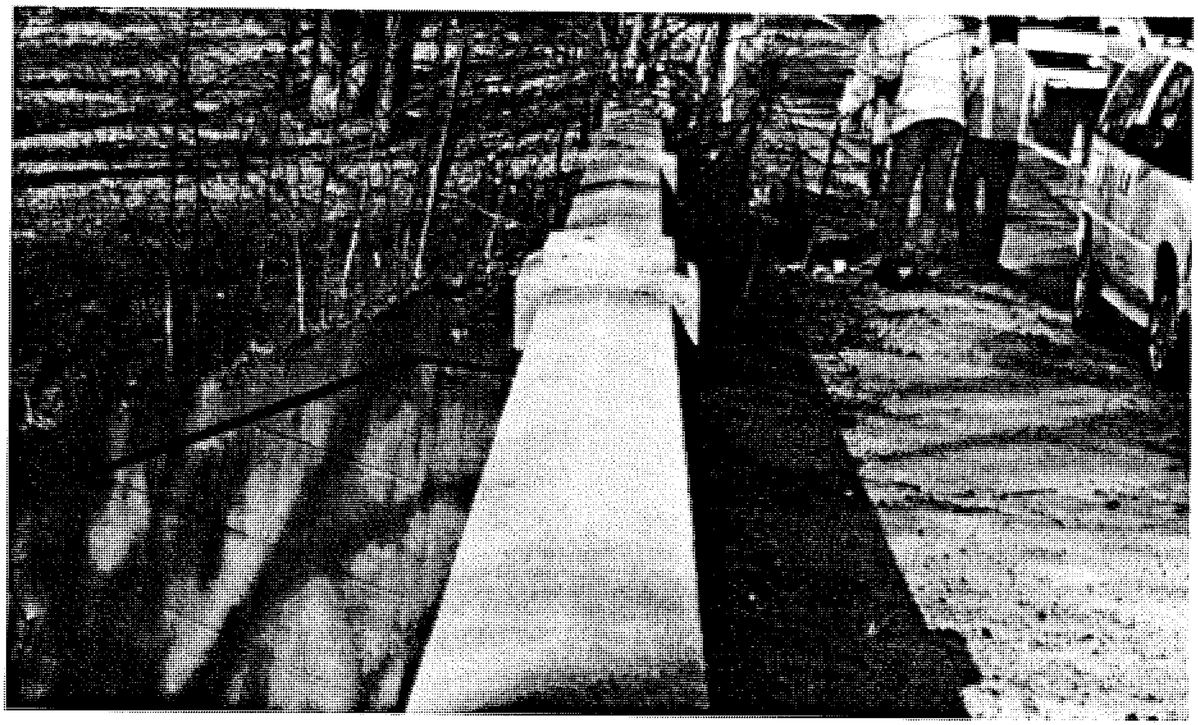


Figure A.16- Photograph of Bridge No. 0324-158, Route 206 over Rancocas Creek, Pemberton Twp., Burlington County, New Jersey.

A.2.10 Bridge No. B01 of 33045A, Bridge on I496 over Red Cedar River, Michigan.

This bridge is supported by two abutments and 4 piers. Figure A.17 shows a photograph of Pier # 3 of the bridge where testing was performed. The substructure of Pier 3 consists of concrete beams supported by concrete columns. The columns are resting on a concrete plint supported by a concrete pilecap and cast-in-place concrete piles. The available bridge plans are presented in the interim report submitted in April, 1998.

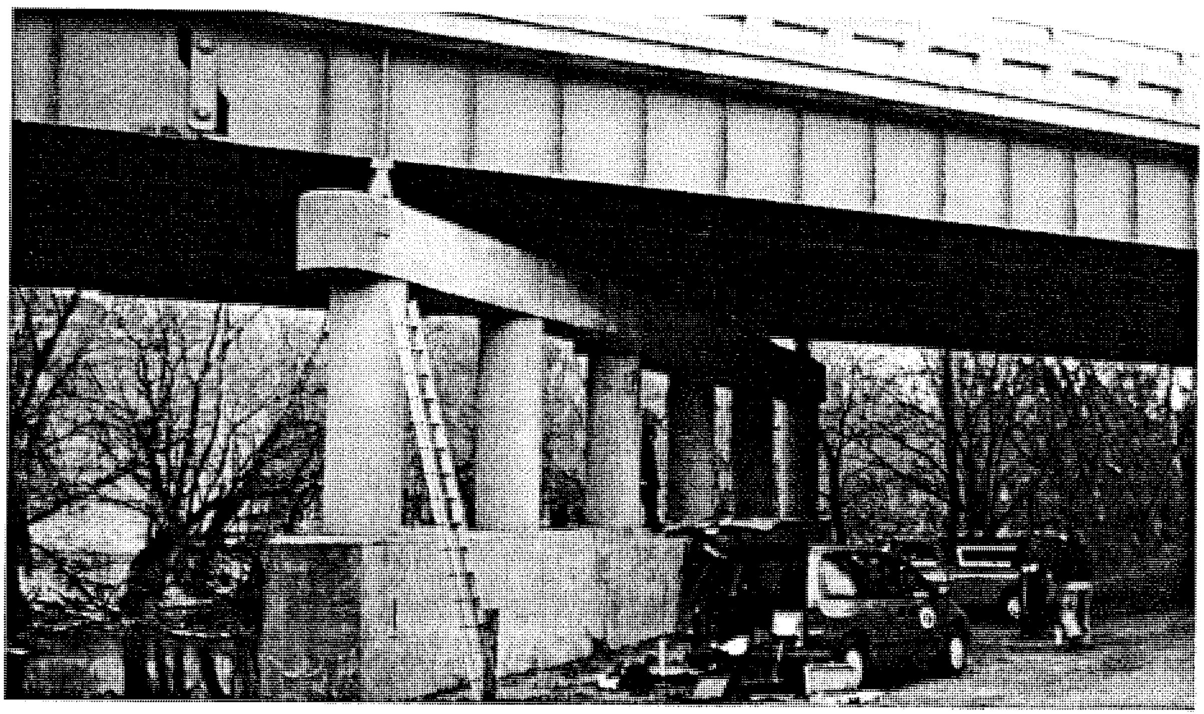


Figure A.17- Photograph of Bridge No. B01 of 33045A, Bridge on I496 over Red Cedar River, Michigan.

A.2.11 Bridge No. B04 of 64015E, Bridge on US31 over Pentwater River, Michigan.

This bridge is supported by 2 abutments and 2 piers. Figure A.18 shows a photograph of Pier # 2 of the bridge where testing was performed. The substructure of Pier 2 consists of concrete beams supported by concrete columns. The columns are resting on a concrete pilecap supported by driven concrete piles. The available bridge plans are presented in the interim report submitted in April, 1998.

Figure A.18- Photograph of Bridge No. B04 of 64015E, Bridge on US31 over Pentwater River, Michigan.

A.2.12 Bridge No. X01 of 25085B, Bridge over Fenton Road, Thread Creek, Michigan.

This bridge is supported by 2 abutments and 6 piers. Figure A.19 shows a photograph of Pier # 3 of the bridge where testing was performed. The substructure of Pier 3 consists of concrete beams supported by concrete columns. The columns are resting on strip footings. The available bridge plans are presented in the interim report submitted in April, 1998.

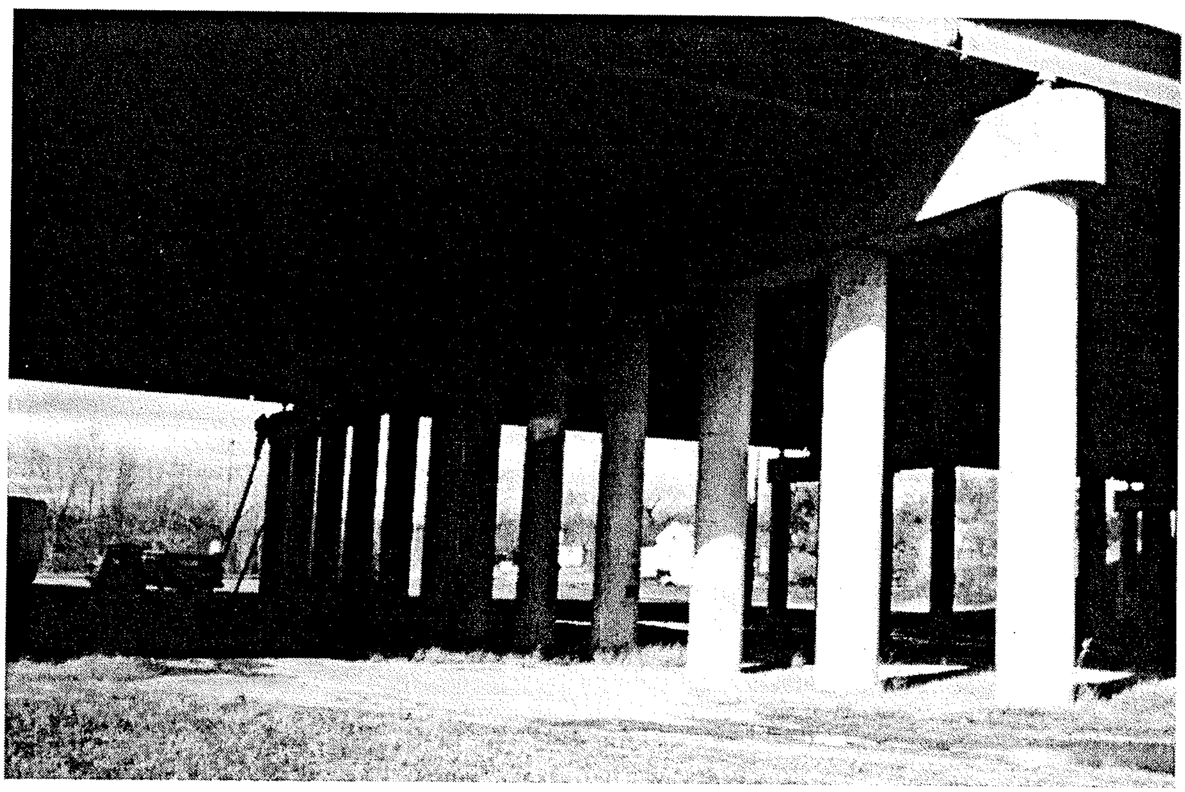


Figure A.19- Photograph of Bridge No. X01 of 25085B, Bridge over Fenton Road, Thread Creek, Michigan.

A.2.13 Santiam River Overflow # 5 Bridge, Near Salem, Oregon.

This bridge is supported by 2 abutments and 4 piers. Figure A.20 shows a photograph of one of bridge abutment where testing was performed. The substructure of the abutment consists of a concrete pilecap supported by timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

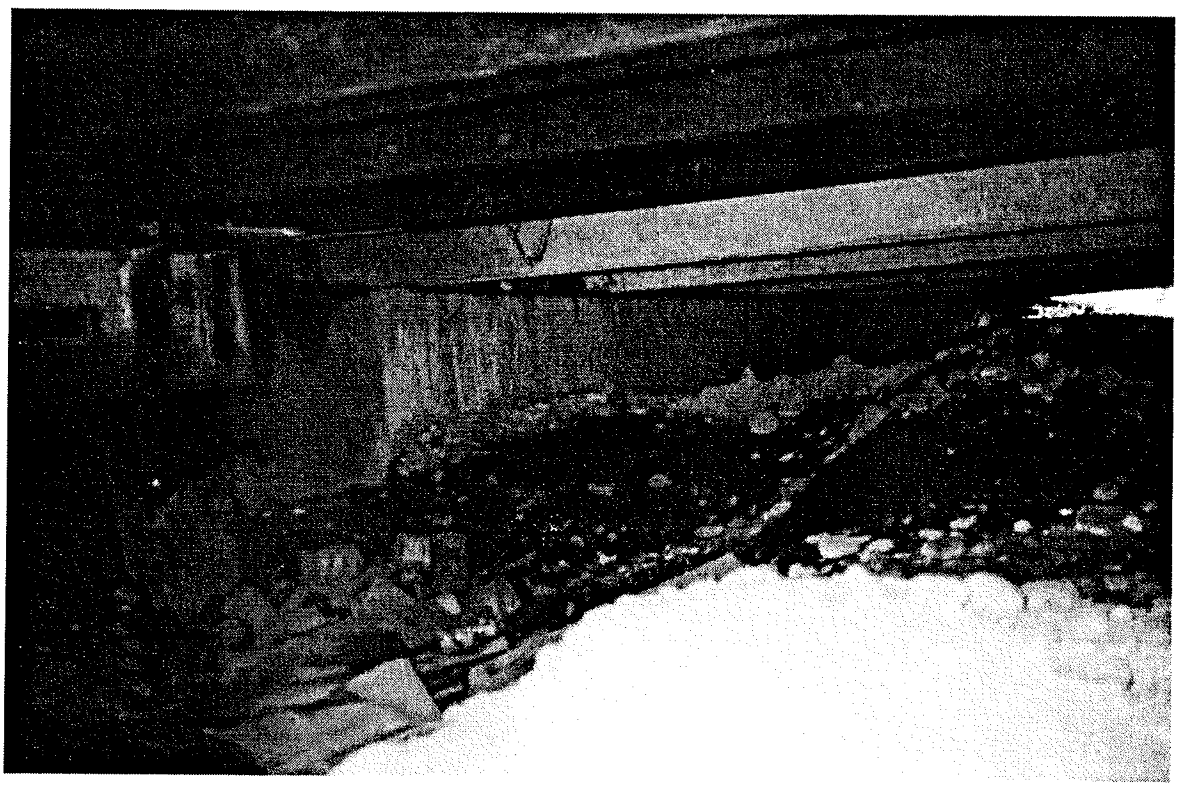


Figure A.20- Photograph of Santiam River Overflow # 5 Bridge, Near Salem, Oregon.

A.2.14 Cordon Road Overcrossing Highway 22 Bridge, Near Salem, Oregon.

This bridge is supported by 2 abutments and 2 piers. Figure A.21 shows a photograph of one of the bridge piers where testing was performed. The substructure of the pier consists of a concrete beam supported by concrete columns. The concrete columns are resting on a pilecap supported by 16 treated timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

Figure A.21- Photograph of Cordon Road Overcrossing Hwy. 22 Bridge, Near Salem, Oregon.

A.2.15 Dudley Road Bridge, Structure No. 0-6-11, Oxford, Massachusetts.

This bridge is supported by 2 abutments and 4 piers. Figure A.22 shows a photograph of Bent 1 where testing was performed. The substructure Bent 1 consists of a concrete pilecap supported by timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

Figure A.22- Photograph of Dudley Road Bridge, Structure No. 0-6-11, Oxford, Massachusetts.

A.2.16 Bridge on Route 122, Structure No. U-2-21, Uxbridge, Massachusetts.

This bridge is supported by 2 abutments. Figure A.23 shows a photograph of one of the bridge abutments where testing was performed. The substructure of the abutment consists of a massive sloping concrete wall with concrete wings supported by 80 timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

Figure A.23- Photograph of Bridge on Route 122, Structure No. U-2-21, Uxbridge, Massachusetts.

A.2.17 Johnston County Bridge # 129, North Carolina.

The bridge substructure consists of concrete beams resting on concrete pilecaps supported by timber piles. Figure A.24 shows a photograph of Bent 2 of the bridge which is supported by a 6 timber pile foundation. Shown in the photo also is the borehole used for Parallel Seismic tests on Pile 6 of Bent 2. This is a maintenance bridge, only pile driving records were available with bridge plans not available. The pile driving records are presented in the interim report submitted in April, 1998.

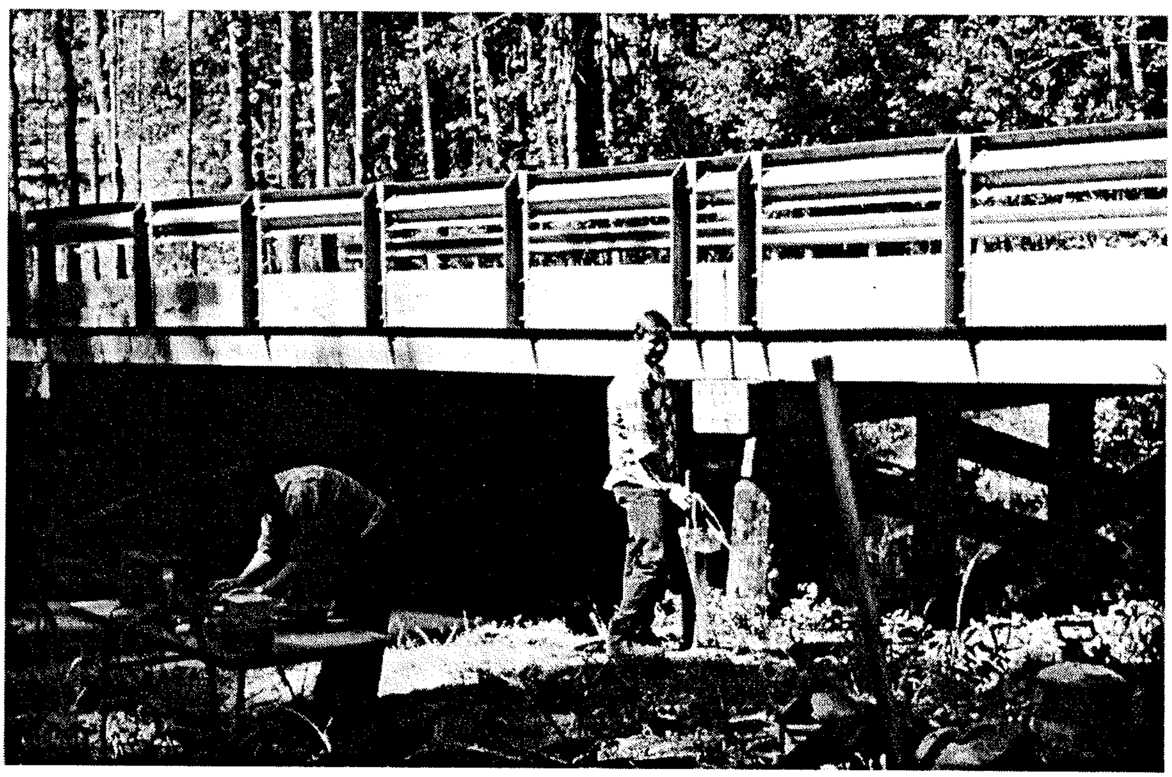


Figure A.24- Photograph of Bent 2, Johnston County Bridge # 129, North Carolina.

A.2.18 Johnston County Bridge # 145, North Carolina.

The bridge substructure consists of concrete beams resting on concrete pilecaps supported by timber piles. Figure A.25 shows a photograph of Bent 1 of the bridge which is supported by a 5 timber pile foundation. Shown in the photo also is the borehole used for Parallel Seismic tests on Pile 5 of Bent 1. The pile driving records and bridge plans are presented in the interim report submitted in April, 1998.

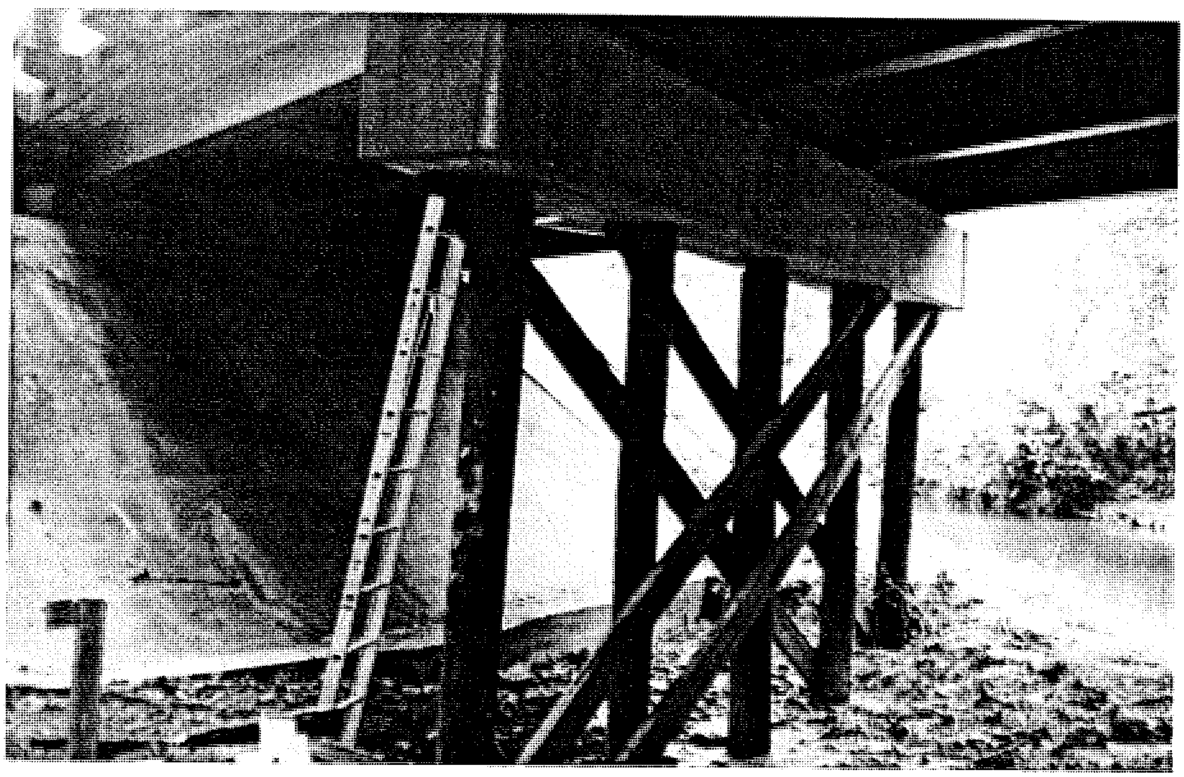


Figure A.25- Photograph of Bent 1, Johnston County Bridge # 145, North Carolina.

A.2.19 Wake County Bridge # 207, North Carolina.

The bridge substructure consists of timber beams supported by timber piles. Figure A.26 shows a photograph of Bent 4 of the bridge which is supported by a 3 timber pile foundation. The pile driving records and bridge plans are not available.

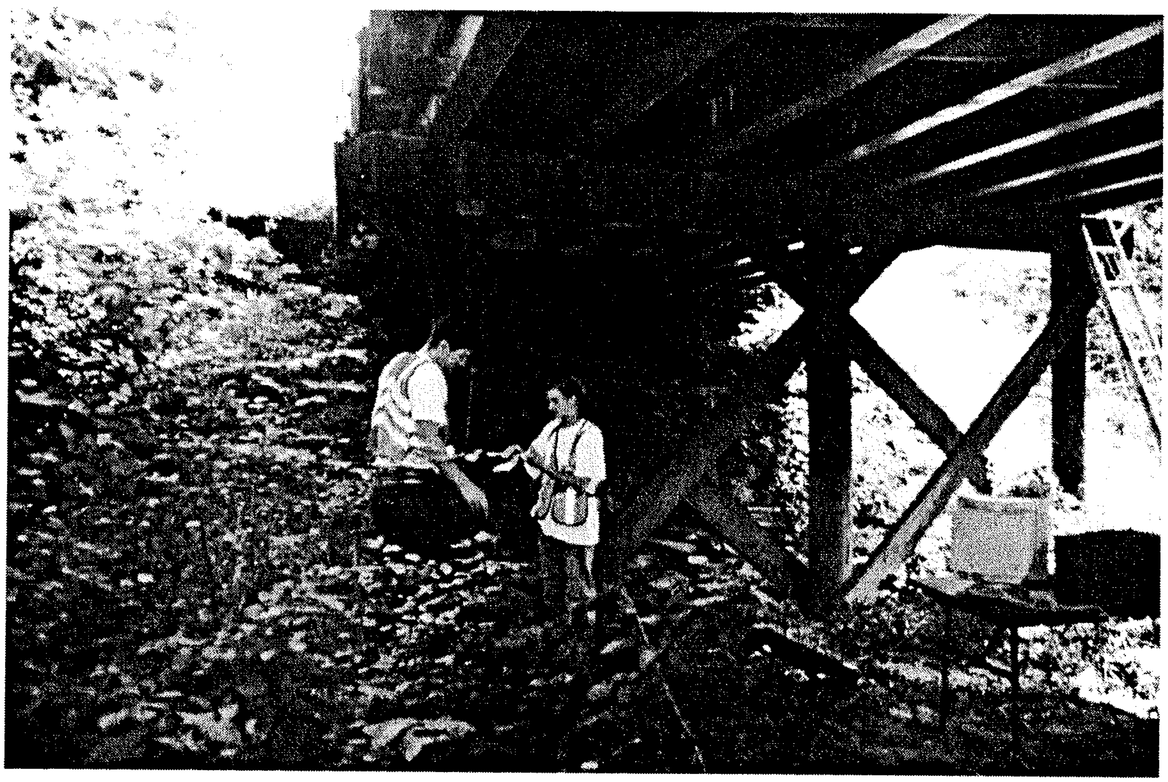


Figure A.26- Photograph of Bent 4, Wake County Bridge # 207, North Carolina.

A.2.20 Bridge on US 287, Structure No. C-16-C, over Little Thompson River, near Longmont, Colorado.

This bridge is supported by 2 abutments and 5 piers. Figure A.27 shows a photograph of one of the bridge piers where testing was performed. The substructure of the pier consists of a concrete beam supported by 7 steel H-piles. The H-piles are encased in concrete above the ground level. The available bridge plans are presented in the interim report submitted in April, 1998.

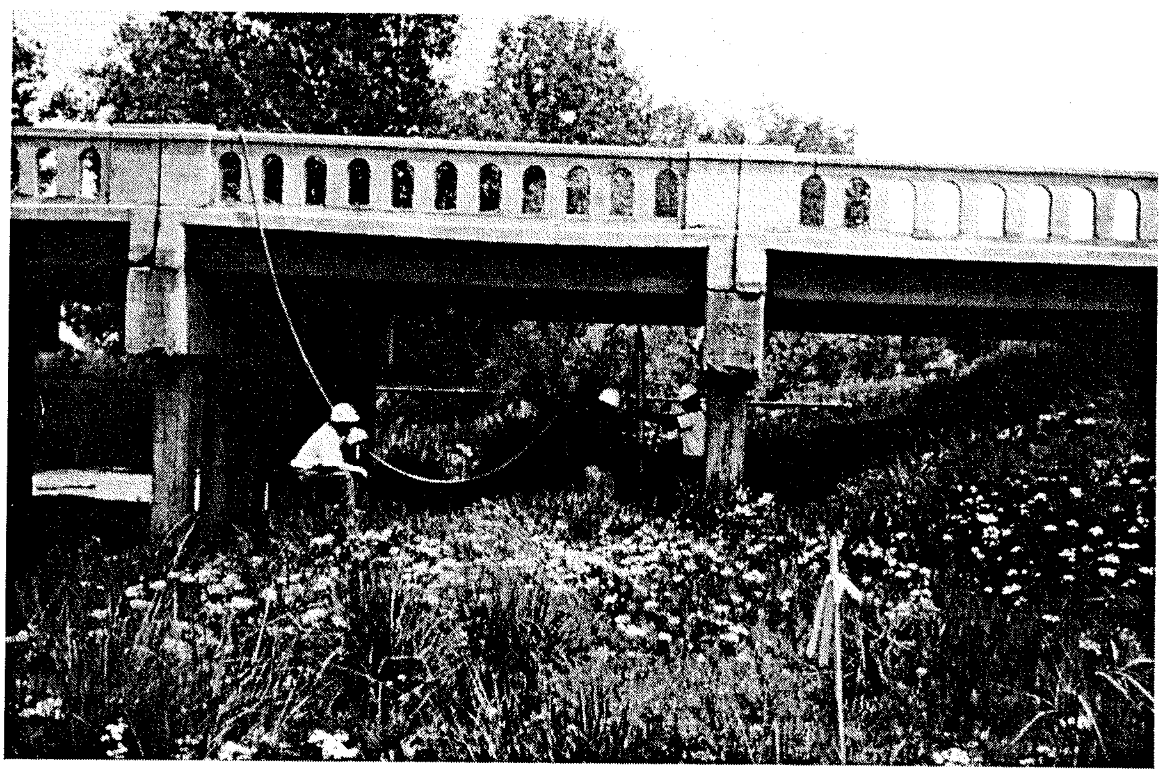


Figure A.27- Photograph of Bridge on US 287, Structure No. C-16-C, over Little Thompson River, near Longmont, Colorado.

A.2.21 Bridge on US 52, Structure No. D-17-I, over South Platte River, near Fort Lupton, Colorado.

This bridge is supported by 2 abutments and 8 piers. Figure A.28 shows a photograph of one of the bridge piers where testing was performed. The substructure of the pier consists of a concrete wall supported by timber piles. The available bridge plans are presented in the interim report submitted in April, 1998.

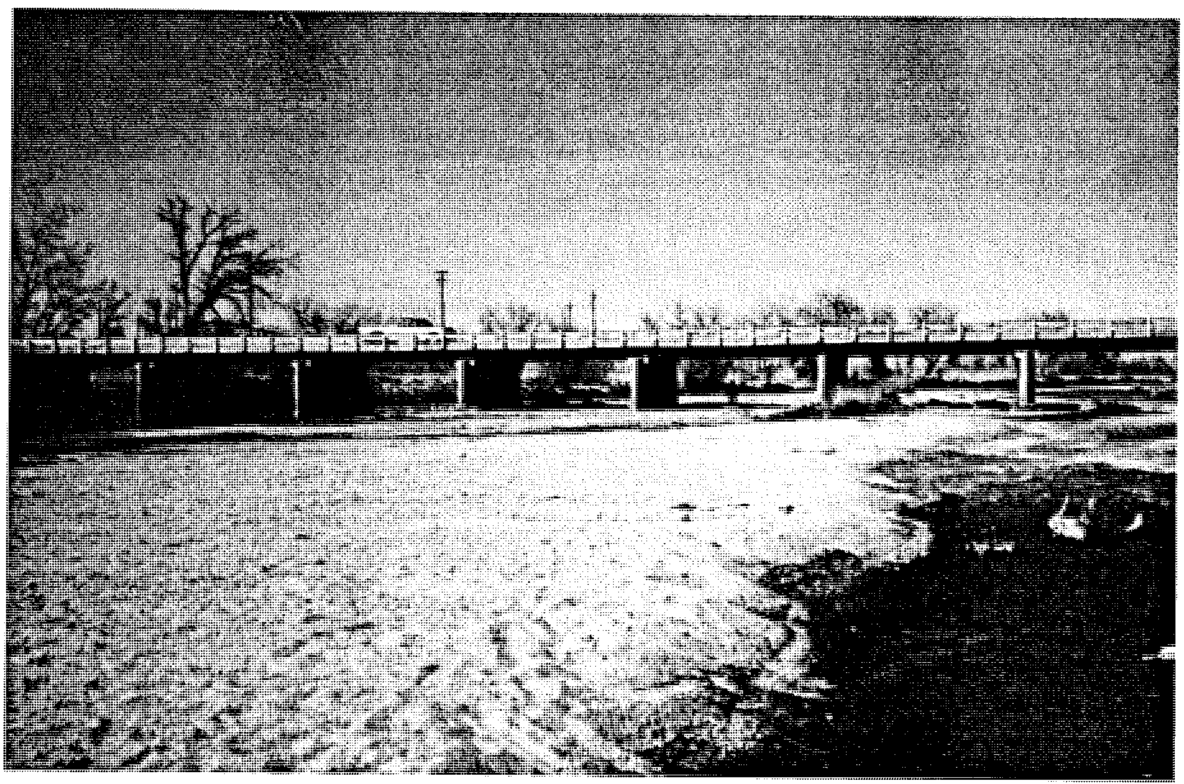


Figure A.28- Photograph of Bridge on US 52, Structure No. D-17-I, over South Platte River, near Fort Lupton, Colorado.

A.2.22 Bridge Over Bethel Creek, Near Snook, Texas.

This bridge is supported by 2 abutments and 1 pier. The tested abutment substructures consist of a concrete beam supported by steel H piles. Bridge plans are not available, but this bridge was tested to try the Induction Field method at a second site.