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I am submitting herewith a thesis written by Anthony Kevin Bowling entitled "Seismic Modeling of Bridge Piers Supported by Piles in Loess." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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SEISMIC MODELING OF BRIDGE PIERS
SUPPORTED BY PILES IN LOESS

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Master of Science Degree
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Anthony Kevin Bowling
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DEDICATION

This thesis is dedicated to my parents, Mrs. Harriet Bowling and Mr. Lora Bowling. Their guidance provided a sound foundation on which I can model my entire life. The support and love provided the motivation to succeed throughout my academic career.

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ABSTRACT

The Tennessee Department of Transportation uses a program called SEISAB to determine the dynamic response of bridges. In addition to the properties of the bridge, SEISAB allows spring stiffnesses to be input to represent soil stiffnesses at the foundation. Since current modeling techniques by TNDOT assume fixed foundations, it was desired to determine the implications of adding spring stiffnesses to the bridges being modeled. These stiffnesses were calculated using results of pluck tests performed on a single pile. The tests were conducted in loess, a soil which is predominant in West Tennessee, which is an area of high seismic risk.

As a result of the pluck tests, a stiffness of 30 kips/inch was used to calculate stiffnesses which were applied to SEISAB for four different bridges. Each bridge was modeled with fixed bent foundations and with spring stiffnesses at the foundations. The bridges modeled were of two foundation types. Two of the bridges had bent foundations consisting of a pile cap supported by a pile group, while the other two bridge foundations consisted of a row of free-standing pile bents. Once the dynamic analyses were complete, it was evident that the bridges with the free-standing pile bents experienced a greater percent difference in forces and deflections than the bridges with the pile cap/pile group foundations. In all cases, the moments and axial forces in the bridge bents decreased, by a large amount in some cases, while the deflections of the structure increased. These results show that the assumptions of fixed conditions at the bent foundations are conservative for forces and unconservative for deflections.

TABLE OF CONTENTS

CHAPTER		PAGE
CHAPTER 1:	INTRODUCTION	1
CHAPTER 2:	REVIEW OF LITERATURE	5
CHAPTER 3:	DETERMINATION OF SPRING STIFFNESSES FROM PLUCK TESTS	9
	3.1 Introduction	9
	3.2 Conducting Pluck Tests	9
	3.2.1 Site Arrangement	10
	3.2.2 Set-Up of Pluck Test	10
	3.2.2.1 Hardware	11
	3.2.2.2 Instrumentation	13
	3.3 Reduction of Data	14
	3.3.1 Determination of Static Stiffness	15
	3.3.2 Determination of Natural Frequency and and Damping	16
	3.3.3 Determination of Dynamic Stiffness	19
	3.4 Results of pluck tests	19
	3.4.1 Test Group 1 Results	19
	3.4.1.1 Set 1	20
	3.4.1.2 Set 2	20
	3.4.2 Test Group 2 Results	21

	3.4.2.1 Set 1	21
	3.4.2.2 Set 2	22
	3.5 Selection of Stiffness to Use in SEISAB	22
CHAPTER 4:	DESCRIPTION OF SEISAB	23
	4.1 Introduction	23
	4.2 Structure Description	23
	4.3 Load Description	25
	4.4 SEISAB Output	26
CHAPTER 5:	APPLICATION OF SOIL STIFFNESSES TO SEISAB	28
	5.1 Introduction	28
	5.2 Determination of Soil Stiffnesses to Use in SEISAB	29
	5.2.1 Determination of Analytical Stiffnesses	29
	5.2.1.1 Abutment Stiffnesses	30
	5.2.1.2 Pile Cap Stiffness	31
	5.2.2 Determination of Pile-Soil Stiffnesses from Test Data	34
	5.3 Modeling of Bridges with Pile Group Foundations	35
	5.3.1 Modeling of the Nonconnah Parkway Bridge	35
	5.3.1.1 Description of Nonconnah Parkway Bridge	36
	5.3.1.2 Determination of SEISAB Foundation Stiffnesses	37
	5.3.2 Modeling of the Madison County Bridge	37

5.3.2.1	Description of the Madison County	
	Bridge	38
5.3.2.2	Determination of SEISAB Foundation	
	Stiffnesses	38
5.4	Modeling of Bridges With Pile Bent Foundations	39
5.4.1	Modeling of the Loosahatchie Bridge	39
5.4.1.1	Description of the Loosahatchie	
	Bridge	40
5.4.1.2	Determination of SEISAB Foundation	
	Stiffnesses	41
5.4.2	Modeling of the Haywood County Bridge	42
5.4.2.1	Description of the Haywood County	
	Bridge	42
5.4.2.2	Determination of SEISAB Foundation	
	Stiffnesses	42
CHAPTER 6:	RESULTS OF THE SEISAB ANALYSES	44
6.1	Introduction	44
6.2	The Nonconnah Parkway Bridge	44
6.3	The Madison County Bridge	45
6.4	The Loosahatchie Bridge	46
6.5	The Haywood County Bridge	47
6.6	Discussion of Results	48
CHAPTER 7:	SUMMARY AND CONCLUSIONS	50
7.1	Summary	50
7.2	Conclusions	51

LIST OF REFERENCES	53
APPENDICES	57
APPENDIX A: Tables	58
APPENDIX B: Figures	97
APPENDIX C: SEISAB Input Files	115
APPENDIX D: Sample Calculations	124
VITA	128

LIST OF TABLES

TABLE NO.	TITLE	PAGE
1.	Results of Test Group 1, Set 1 (Pile 1)	59
2.	Results of Test Group 1, Set 2 (Pile 2)	60
3.	Results of Test Group 2, Set 1 (Pile 2)	61
4.	Results of Test Group 2, Set 2 (Pile 2)	62
5.	Properties of Soil Used In Stiffness Calculations	63
6.	Nonconnah Parkway Bridge Bent Foundation Data	64
7.	Nonconnah Parkway Bridge Abutment Stiffnesses	65
8.	Nonconnah Parkway Bridge Lateral Bent Stiffnesses	66
9.	Madison County Bridge Abutment Stiffnesses	67
10.	Madison County Bridge Lateral Bent Stiffnesses	68
11.	Loosahatchie Bridge Abutment Stiffnesses	69
12.	Loosahatchie Bridge Bent Stiffnesses	70
13.	Haywood County Bridge Abutment Stiffnesses	71
14.	Haywood County Bridge Bent Stiffnesses	72
15.	Comparison of Natural Periods for the Nonconnah Parkway Bridge	73
16.	Maximum Longitudinal Displacements For the Nonconnah Parkway Bridge	74
17.	Maximum Transverse Displacements for the Nonconnah Parkway Bridge ...	75
18.	Maximum Longitudinal Column Moments for the Nonconnah Parkway Bridge	76
19.	Maximum Transverse Column Moments for the Nonconnah Parkway Bridge.....	77
20.	Maximum Axial Column Forces for the Nonconnah Parkway Bridge	78

21. Comparison of the Natural Period for the Madison County Bridge	79
22. Maximum Longitudinal Displacements for the Madison County Bridge	80
23. Maximum Transverse Displacements for the Madison County Bridge	81
24. Maximum Longitudinal Column Moments for the Madison County Bridge ..	82
25. Maximum Transverse Column Moments for the Madison County Bridge	83
26. Maximum Axial Column Forces for the Madison County Bridge	84
27. Comparison of the Natural Period for the Loosahatchie Bridge	85
28. Maximum Longitudinal Displacements for the Loosahatchie Bridge	86
29. Maximum Transverse Displacements for the Loosahatchie Bridge	87
30. Maximum Longitudinal Column Moments for the Loosahatchie Bridge	88
31. Maximum Transverse Column Moments for the Loosahatchie Bridge	89
32. Maximum Axial Column Forces for the Loosahatchie Bridge	90
33. Comparison of the Natural Period for the Haywood County Bridge	91
34. Maximum Longitudinal Displacements for the Haywood County Bridge	92
35. Maximum Transverse Displacements for the Haywood County Bridge	93
36. Maximum Longitudinal Column Moments for the Haywood County Bridge ..	94
37. Maximum Transverse Column Moments for the Haywood County Bridge ...	95
38. Maximum Axial Column Forces for the Haywood County Bridge	96

LIST OF FIGURES

FIGURE NO.	TITLE	PAGE
1.	Testing Site Arrangement	98
2.	Pluck Test Setup	99
3.	Typical Instrumentation Group	100
4.	Typical LVDT Trace for the Pluck Tests	101
5.	Typical LVDT Trace for the Dynamic Portion of the Response	102
6.	Ground Motion Spectra	103
7.	Typical Abutment Diagram	104
8.	Influence Factors for Surface Displacement	105
9.	Stiffness Reduction Factor	106
10.	Nonconnah Parkway Bridge Plan and Elevation	107
11.	Typical Nonconnah Parkway Bridge Pile Cap and Footing	108
12.	Madison County Bridge Plan and Elevation	109
13.	Madison County Bridge Pile Cap and Footing Plan	110
14.	Modeled Loosahatchie Bridge Plan and Elevation	111
15.	Actual Loosahatchie Bridge Bent vs. Modeled	112
16.	Haywood County Bridge Plan and Elevation	113
17.	Typical Haywood County Bridge Bent	114

CHAPTER 1

INTRODUCTION

With the ever increasing population, the number of structures being built is also increasing. With this increase in new structures has come a heightened awareness of safety concerns and changes in design procedures. One such change is the increased attention given to seismic activity in recent years. Most structures built in areas of high seismic activity must be designed in accordance with seismic provisions set forth in the applicable code. Of all the new structures being built, bridges are among the most important. The failure of a bridge can have catastrophic effects on urban traffic flow, not to mention the tragic loss of life that can occur. The recent California earthquakes, Loma Prieta in 1989 and Northridge in 1994, have increased concerns across the nation about the design of bridges for seismic activity.

Many variables can affect the response of a bridge during an earthquake; these include the geometry and structural characteristics of the superstructure as well as the properties of the foundation and supporting soil. All of these variables can be modeled with a high degree of confidence with the exception of the foundation-soil interface. The foundation response of a bridge is complex and is at least somewhat different for every site, soil type, and foundation type. Bridges can be founded on a number of different types of foundations including piles. For pile supported bridges, the interaction of the soil and pile under the action of lateral earthquake loads is of particular importance in attempting to determine the response of the bridge.

The magnitude of seismic forces to use in bridge design varies from region to region depending on the level of expected seismic activity. This level of activity

is dependent on the location at which the structure will be constructed. The United States is divided into five seismic zones ranging from zero to four, corresponding to the lowest risk and highest risk respectively. The western portion of Tennessee lies in zone two or three, which results in relatively high design forces (1). This high risk zone has been assigned to this region because of its proximity to the New Madrid Fault. This fault and the accompanying risk of a major earthquake have forced engineers to take a closer look at the dynamic response of bridges due to earthquakes in West Tennessee.

An important factor in the response of a structure to seismic motion is the type of soil on which the structure is founded. In the western portion of Tennessee, the prevalent soil type is loess. Loess is an alluvial soil whose characteristics include the ability to maintain a vertical slope and a tendency to collapse upon saturation. Very little research has been conducted on foundations in loess and even less dealing with the interaction of piles and loess under seismic loadings. Thus, due to the presence of loess and the fact that West Tennessee is a region of possible high seismic activity, the dynamic response of bridges to seismic loads in this region is particularly important in the design process.

Determining the response of a bridge to dynamic loads can be complicated and time consuming; however, there are computer programs available which greatly simplify this task. The Tennessee Department of Transportation (TNDOT) currently uses a program titled the SEISmic Analysis of Bridges (SEISAB), developed by Imbsen and Associates, to model their bridges (2). SEISAB provides information regarding the response of the structure under dead, live, and seismic loads. In modeling the bridges, all characteristics of the bridge must be input into the program including the boundary conditions at the bridge foundation. At the

foundation, SEISAB allows the interaction between the bridge pier and the pile to be modeled as either fixed or partially restrained using spring stiffnesses. The springs are oriented such that they limit lateral, torsional, and rotational movement. TNDOT currently assumes fixed conditions in modeling the bridge foundation which is the same as considering the springs to be infinitely stiff. This assumption can lead to either conservative or unconservative design forces (3). In an attempt to gain a better understanding of appropriate values to use for spring stiffnesses and how the response of bridges would be affected by considering these stiffnesses, TNDOT contracted with The University of Tennessee, in conjunction with the University of Memphis, to investigate this subject. The major thrust of the project was to determine stiffness coefficients from field tests. Then the question of how these coefficients affect the design forces became important. The results of the field tests are presented elsewhere (4). The questions of how these coefficients affect the design forces and how to apply the results of the field tests to the practical modeling of bridges for design purposes are addressed in this thesis.

A sensitivity study conducted as part of the research (5) considered the question of how varying the different spring coefficients would affect the design forces. From this study, it was concluded that varying the lateral springs had the most pronounced effect on the induced forces while the rotational and torsional springs made minimal contributions. At the time the sensitivity study was conducted, there had been no field tests to determine appropriate values to be input for the lateral stiffnesses. The stiffnesses used were based on analytical models including Beam on Elastic Foundation Theory and Novak's Method for Dynamic Analysis (3,5). As a result of the data from the study, it was evident that field tests needed to be conducted to determine values of lateral stiffness to

be used in SEISAB. These field tests would be composed entirely of pluck tests in which driven test piles are displaced and then released, setting them into free vibration. From the measured response of the pile, the dynamic stiffness could be determined. Tests were conducted on two pre-stressed concrete piles in December 1994 and March 1995. From these tests, the frequency, damping, and stiffness were determined (4).

Once all data from the tests had been reduced, analyzed, and the stiffnesses obtained, four bridges were analyzed using SEISAB. These four bridges were modeled considering both fixed conditions, as per current modeling techniques by TNDOT, and spring coefficients determined from the pluck tests. The bridges that were analyzed consisted of two pile/pile cap bridges and two free standing pile bent bridges.

The purpose of this thesis was to use the spring coefficients determined from the pluck tests in the analysis of bridges using SEISAB, and to determine how the forces and moments in the pier columns are affected in comparison to fixed conditions. Also, the sensitivity of the two different bridge foundation types to variation in foundation stiffness is also compared.

CHAPTER 2

REVIEW OF LITERATURE

Recent attention to seismic loading and advances in technology have increased the utilization of computer software in the seismic analysis of bridges. Software such as SEISAB lessens the burden of rigorous and time consuming computations; however, SEISAB will only produce reliable results if the information input is reliable. Because of the inherent uncertainty in evaluating soil parameters, this concept becomes especially important when dealing with the input of soil stiffnesses into SEISAB.

The effect of varying input of soil stiffnesses on results calculated by SEISAB was addressed in a sensitivity study conducted by Cook et al. (3, 4). The sensitivity study dealt with two bridges in West Tennessee. The bridges were modeled by Cook, first using fixed conditions at the foundation, as per current TNDOT methods, and then using analytical soil springs derived from Wilson, Novak, Poulos, and Scott (3). Translational, rotational, and torsional soil stiffnesses were considered, but it turned out that only the translational soil stiffnesses had a pronounced effect on the forces in the bridge bents analyzed. The purpose of that study was to determine how sensitive the bridge bent forces calculated by SEISAB were to variations in the foundation springs calculated by typical analytical methods.

The first bridge analyzed was a bridge located in Madison County, Tennessee. This bridge was a continuous, two span structure supporting two traffic lanes (4). The abutments of the Madison County bridge consisted of monolithic reinforced concrete walls supported by a single row of piles. The bent pier foundation was composed of a 12 pile group topped with a pile cap

providing continuity. In addition to the lateral resistance of the piles, this type of bent foundation has added lateral resistance provided by the pile cap which must be taken into consideration.

The other bridge which was analyzed was a bridge in Haywood County, Tennessee. This bridge consisted of a continuous, three span structure supporting two traffic lanes. The abutments of the bridge were constructed of concrete and served merely to provide a point of bearing for the prestressed box beams. The two bent piers consisted of free-standing piles topped with a cap on which the prestressed box beams rested (3, 4). A foundation of this type possesses no added lateral resistance from a pile cap; thus, lateral resistance is governed by the stiffness of the individual piles acting alone, less any spacing effect.

Cook considered different combinations of lateral soil springs in an attempt to determine the effect of various combinations of bent and abutment stiffnesses. The four lateral spring cases considered for each structure consisted of the following:

- All support fixed
- Bent and abutment springs
- Bent springs/10
- All springs/10

The sensitivity study showed that the largest variation in forces from those obtained assuming fixed condition occurred when the lateral springs at both the abutment and the bents were reduced by a factor of 10. The effect of varying spring stiffness values led to the conclusion that dynamic field tests needed to be conducted in order to obtain a more reliable set of values for the soil springs.

Extensive research has been conducted on the subject of laterally loaded piles. The spectra of tests are more narrow dealing with the dynamic response of piles, while tests dealing with piles founded in loess are virtually non-existent. This lack of test data underscored the need identified by Cook to conduct free vibration tests in loess.

Many of the dynamic tests conducted on driven piles consisted of forced vibration tests. Most forced vibration tests are similar to those conducted by Blaney and O'Neill (6). Their tests dealt with the dynamic response of a single pipe pile founded in stiff clay. To simulate the presence of a superstructure, a mass consisting of steel and concrete was rigidly attached to the top of the pile. A vibrator placed atop the mass induced forced oscillations through movement of an internal carriage. This type of forced vibration allows for increased control over the response of the system since a structure tends to oscillate at the excitation frequency. It is difficult to determine the dynamic stiffness from this type of test since the structure is not permitted to go into free vibration. In contrast, a pluck test is a more appropriate testing method for determining dynamic stiffness because, upon release of the load, the pile will vibrate at its natural frequency. From the natural frequency, the stiffness can be determined.

Information dealing with pluck tests is limited. A successful pluck test is dependent on certain factors which must be considered prior to testing, one of which is the selection of the mass to attach to the top of the pile. The selected mass should be that mass which would force the single pile frequency to imitate approximately the frequency of a superstructure (11). Another important factor to consider in conducting free vibration tests is the ability to achieve a true "quick" release. Pluck tests as described above were conducted by Blaney and O'Neill (6) on a single pile driven into stiff clay. Their pluck tests were conducted

as supplementary tests to their forced vibration tests and were not the main focus of work. Even though all work was conducted on laterally loaded piles driven in clay or sand, the same basic testing procedures used by Blaney and O'Neill can be applied to piles founded in all types of soil.

Upon completion of testing, one of the most important questions dealing with the behavior of driven piles is whether or not the response of a single pile can be related to the response of a pile group. Several groups of researchers have done work dealing with group efficiency (7, 8, 9). According to Das, the efficiency of a pile group is defined as the ratio of the total pile group capacity to the capacity of the pile group obtained without considering the group effect (10). The spacing of the piles in a group dictates the efficiency of the pile group. Since pluck tests by the University of Tennessee were conducted on a single pile, the resistance had to be reduced to take into consideration the effect of pile groups where applicable. This consideration is an important factor in the application of the soil springs to SEISAB since failure to consider group effects would overpredict the actual stiffness of the foundation.

The lack of information on the dynamic response of piles driven in loess was the main reason behind the research project sponsored by the Tennessee Department of Transportation. As a part of that research, a number of tests dealing with the dynamic response of piles have been conducted, the details of which are presented elsewhere (5). This thesis presents the effect of applying test-based stiffness coefficients to two different bridge foundation types via SEISAB and compares the resulting design forces to those determined using the current TNDOT modeling approach.

CHAPTER 3

DETERMINATION OF SPRING STIFFNESSES FROM PLUCK TESTS

3.1 INTRODUCTION

Fully instrumented pluck tests were conducted in December 1994 and March 1995 on two prestressed concrete piles driven near Memphis, Tennessee at the Nonconnah Parkway bridge no. 4. The pluck tests were deflection controlled, meaning a set deflection was desired before release, and once that deflection was reached, the load was released. Consecutive tests were conducted by increasing the deflection for each test. Typically, an attempt was made to conduct at least two consecutive tests at the same deflection to increase the confidence level of the tests. Throughout this thesis, the tests conducted in December are referred to as test group 1 while the tests conducted in March are referred to as test group 2. Each group consisted of two sets of tests. Each set consisted of tests on a particular pile, conducted from one direction. Test group 1 consisted of one set of tests on each pile and test group 2 consisted of two sets of tests conducted on only one pile with the directions of pull perpendicular to each other. This change in pull direction was to determine if pluck tests conducted in one direction affected the results of tests perpendicular to that direction. Soil stiffnesses were determined from the results of each quick release test. These stiffnesses, along with the details of the performance of the pluck tests, are presented in this chapter.

3.2 CONDUCTING PLUCK TESTS

In a pluck test, the test pile was deflected laterally under a load which was instantly released upon attaining the desired deflection, allowing the pile to oscillate. The load required to deflect the pile can be relatively large; thus, three

concrete slabs were constructed near the piles to provide lateral resistance. One slab was cast near pile 1 and two were cast near pile 2. The load application to the pile was achieved through a combination of components which are defined in this chapter. The instrumentation was also a very important part of the pluck test and is also described in this chapter.

3.2.1 SITE ARRANGEMENT

In order to understand the tests conducted by The University of Tennessee and the University of Memphis, all facets of the testing should be considered, beginning with the arrangement of the piles and slabs. The piles were driven in a line parallel to the bridge. The arrangement of pile 1 and pile 2 in relation to their respective slabs is shown in Figure 1* . The concrete slabs were constructed out of the influence zone between the slab and the piles. This was accomplished by spacing the slabs 25 feet away from the test piles. The following section describes the procedure and equipment used in conducting the pluck tests.

3.2.2 SET-UP OF PLUCK TEST

The equipment used in performing the pluck tests can be thought as being composed of two components: hardware and instrumentation. Only a brief description of the testing equipment is given since the equipment has been previously described in great detail in theses by David McAlister (11) and Anish Talati (12). The McAlister thesis contains information regarding qualification tests of the equipment as well as results of a preliminary pluck test conducted on a

* All tables and figures are in Appendices A and B.

steel H-pile, while the Talati thesis deals with the actual pluck tests whose results are presented in this thesis. A diagram of the test set-up is shown in Figure 2.

3.2.2.1 HARDWARE

The hardware necessary to perform the pluck tests consisted of a load frame, Dywidag bars, a bar coupler, a quick release, and the pile mass. Detailed descriptions of all above equipment, with the exception of the pile mass, can be found in the McAlister thesis (11). The pile mass was designed to be applied only to the 14" by 14" concrete piles and is described in detail in the Talati thesis. All components are briefly described here as they occur in a direction from the slab to the pile.

The first piece of equipment atop the slab was the load frame which transferred the applied load to the slab which provided the lateral load resistance. The load frame consisted of two side braces connected with a structural tube. Each of the side braces was constructed of three W12x40 steel beams connected flange to flange, and the base was 5 ft. long with 2, 2-ft. sections composing the rest. The tube which spanned between the braces had a hole drilled through the center of its four ft. span through which the bars which provided the path for the load passed.

The load was transferred from the load frame to the pile through Dywidag bars manufactured by Dywidag Systems International (13). These bars are "threaded" rebar which are formed with a continuous deformation which allows bars to be connected together. The one inch diameter bars used in the pluck tests have an ultimate strength of 127.5 kips which was far greater than the expected loads of around 20 kips. Dywidag couplers were used to connect the bars together and a nut placed near the end of the bar provided a seat for a plate which was

placed over the bar. A 100 kip center hole ram and jack manufactured by Enerpac, Inc. and the load cell were centered on the bars between the seat plate and the tube. As the ram was jacked, the piston applied pressure between the load frame, which was rigid and could not move, and the seat plate on the Dywidag bars. The load cell measured the load created by the ram as it was displacing the Dywidag bars.

At the opposite end of the Dywidag bars, a bar coupler was designed to connect the bars to the quick release. The quick release mechanism was a Release-A-Matic Hook model H44-9, manufactured by Peck and Hale, Inc. of New York City. The H44-9 is composed of a base, hook, and locking ring. The hook remains closed until the locking ring is manually pulled. This allows the hook to swing open, releasing the load. The Release-A-Matic Hook used had a working load of 18 kips with an ultimate load of 71.7 kips. The quick release was lab tested before the field tests to ensure the release of the load was instantaneous and did not interfere with the free vibration.

The final component in the performance of the pluck tests was the pile mass which was designed at the University of Tennessee. The purpose of the pile mass was to force the natural frequency of the test pile to imitate the natural frequency of a superstructure as well as provide a means of applying the load to the pile. The pile mass was designed with a 14" by 14" structural tube 31" long with a cap welded at one end which allowed it to be placed over the exposed end of the pile. Near the base of the tube, a square plate was welded around the tube. This plate resembled the brim of a hat but its purpose was to serve as a seat for 52 individual plates which could be manually loaded over the tube. The plates were 38" x 39" x 1/2" thick with a 16.5" by 16.5" hole cut in the center to allow an easy fit over the tube. These plates weighed 170 lbs. each and served to increase the

mass of the system to a total weight of 10,018 lbs. Threaded rods were used at each corner to connect the plates rigidly to the tube seat. The pile mass was attached to the pile using anchor bolts at the top and set screws at the bottom. For comparison purposes, two connectors were welded to the mass, one near the bottom and the other at the center of gravity of the pile mass. These connectors provided a place for the quick release to attach to the pile mass. Most of the instrumentation used was placed directly on the pile mass and is described in the next section.

3.2.2.2 INSTRUMENTATION

The instrumentation used during the pluck tests included linear variable displacement transducers (LVDT's), accelerometers, velocity transducers, and a load cell. The velocity transducers were only used on the first set of tests in group 1 because of unreliable data obtained from them. A typical diagram of the instrumentation for the two test groups is shown in Figure 3.

The LVDT's were the most crucial component of the instrumentation. Their output was the only means of recording the actual response of the pile. The LVDT's used were model DCS T1000 manufactured by Solatron Metrology, Inc. and had a range of ± 1 inch. The LVDT's were placed at the center of gravity of the mass system and at the same elevation as the bottom pull location. This allowed the static stiffness to be determined for each test regardless of whether the pull location was at the top or bottom.

Adjacent to each LVDT, an accelerometer was placed to determine the acceleration of the pile during the test. The accelerometers used were manufactured by Wilcoxon Research, Inc. and had a range of ± 1 g. The accelerations the piles experienced were beyond the range of the accelerometers

used. Thus, the data from the accelerometers are not presented in this thesis; however, measures have been taken to correct this situation in future tests.

Along with the misjudgment in selection of the accelerometer, the load cell also experienced some unforeseen problems. The load cell was a doughnut load cell manufactured by Interface, Inc. The output from the load cell showed a great amount of noise, and in some cases the noise band was greater than the applied load. Even though the output from the load cell could not be used directly, an approximate value of the load was obtained through the use of a Vishay digital strain box. The Vishay had been calibrated with the load cell which allowed the load to be recorded immediately prior to release.

All instrumentation was connected to a Dell 486 DX computer with a data acquisition board which allowed for the input of eight differential signals. This board by Data Translations, Inc. allowed for all instrumentation output to be put into a program titled DTVEE. With this program, the response, or trace, of the pile versus time could be viewed for all instrumentation. Through the use of DTVEE, it was possible to pull coordinates off the instrument trace in order to determine the state of the pile at any time during the test.

The LVDT's proved to be the most important and most reliable component of the instrumentation. The LVDT's provided the information necessary for the determination of soil stiffnesses. Even though the accelerometers and the load cell did not perform as well as expected, the tests were still a success since the desired output was obtained.

3.3 REDUCTION OF DATA

The instrumentation described in the previous section was important in determining the properties of the soil; however, the LVDT's and the load cell were

the only components considered in providing information regarding the stiffness of the soil. Of the two LVDT's, the one located at the center of gravity of the pile mass was chosen to determine the properties of the soil. This is because of the decision to model the system as a single degree of freedom (SDOF) system for the purposes of dynamic analysis. The SDOF system used was based on a concentrated mass located at the end of a vertical cantilever. All equations for the SDOF model were based on the response of the center of gravity of the pile mass.

A great amount of information can be deduced from the LVDT response. The LVDT's recorded the displacement response of the pile with respect to time which allowed for the determination of the static stiffness, natural frequency, damping, and most importantly, the dynamic stiffness. A typical trace of the LVDT response from DTVEE is given in Figure 4 and to a larger scale in Figure 5.. The methods used for determining these properties are discussed in the next section.

3.3.1 DETERMINATION OF STATIC STIFFNESS

One of the most important questions dealing with the stiffnesses of soil is whether or not the values of static stiffness and dynamic stiffness are approximately equal. This question is important because it would influence testing methods to determine stiffness. Since it is less expensive and less difficult to perform static tests, it would be advantageous to be able to use the stiffnesses determined from static methods in dynamic applications.

The static stiffness of the pluck tests is only an approximate stiffness since the output from the load cell was not "clean" as explained earlier in this chapter. The static stiffness was determined for each test by dividing the load at release by the deflection at release as expressed in the following formula:

$$k_{st} = \frac{P_{rel}}{\delta_{rel}} \quad (3.1)$$

The units of static stiffness are commonly given in kips per inch. While this is an important parameter in understanding the force required to deflect the structure through a certain distance, it is only a static property and does not necessarily give information on the dynamic response of the pile. The following sections discuss the method of determining dynamic properties from the LVDT trace.

3.3.2 DETERMINATION OF NATURAL FREQUENCY AND DAMPING

The response of the test pile remained static until the load was released. Upon release of the load, the dynamic response of the pile system was measured. The dynamic portion of the test can also be seen in Figure 5. The dynamic portion of the response is characterized by a series of peaks which closely follows a typical sine curve, with the exception of the decay in magnitude of the successive deflections. All dynamic parameters were determined from this portion of the trace.

The first dynamic characteristic determined was the natural period of the pile. The period of a structure is the time it takes for the completion of one cycle and is the time between consecutive positive peaks or consecutive negative peaks. Once the period was known, the natural frequency could be determined. The frequency determined from the LVDT is the damped frequency which is the frequency at which the pile actually vibrates. This value can be determined from the LVDT trace by determining the period of the structure. The natural frequency is determined as:

$$f_d = \frac{1}{T} \quad (3.2)$$

in which f_d is the natural frequency and T is the period of the system. The natural frequency can be expressed in either radians per second or, if multiplied by 2π , cycles per second (Hz.). The natural frequency is a good indicator of how stiff a structure is. An extremely stiff structure will have a very high frequency while a flexible structure will have low frequency. The damped frequency, however, is not the frequency used in determining the dynamic stiffness. The undamped frequency must be used in determining this parameter.

In determining the undamped frequency of the structure, the damping had to be calculated. The damping of a system is a measure of the amount of energy lost during free vibration and is usually expressed in terms of percent of critical damping. Critical damping is defined as

"the smallest amount of damping for which no oscillation occurs during free vibration response." (14)

The damping of the pile system was determined from the LVDT trace by looking at the magnitude of the peaks and the amount of decay for successive peaks. The following equation was used to calculate the damping of the pile from the pluck tests:

$$\xi = \frac{\sqrt{\left(\ln \frac{V_n}{V_{n+j}}\right)^2}}{\sqrt{(2\pi j^2) + \left(\ln \frac{V_n}{V_{n+j}}\right)^2}} \quad (3.3)$$

where ξ is the damping, V_n is the deflection at cycle n , V_{n+j} is the deflection at a peak j cycles away. The damping may be calculated for consecutive peaks or over a range of peaks. In an ideal case the damping would be the same for every peak or groups of peaks considered; however, this should not be the expected characteristic for the behavior of soil. Values for damping were calculated for all possible combinations on both the positive and negative cycles. An interesting observation was the fact that the positive cycle damping, which was the direction in which the pile was statically deflected, was smaller than the negative cycle damping for the same test. This phenomenon was probably due to the fact that the soil experienced some creep under the static load on one face and the fact that, upon release, the pile strikes the opposite soil face dynamically, causing the soil to act stiffer on that face. An average of the damping values was used in the calculation of the stiffness. The actual properties determined from the pluck tests are presented in Section 3.4.

Once the percent damping was known, the natural frequency could be determined from the following formula:

$$f_n = \frac{f_d}{\sqrt{1 - \xi^2}} \quad (3.4)$$

where f_d is the damped frequency and ξ is the damping. This value is the true natural frequency of the pile system and is key in the calculation of the dynamic stiffness.

3.3.3 DETERMINATION OF DYNAMIC STIFFNESS

The determination of the dynamic stiffness was the primary objective in the research project conducted by The University of Tennessee and the University of Memphis. This was calculated using an equation based on a SDOF system using the true natural frequency of the system as follows:

$$k_{dyn} = (2\pi f_n)^2 m \quad (3.5)$$

where f_n is the natural frequency and m is the mass atop the pile. The dynamic stiffness was calculated for each test performed. The dynamic stiffness calculated is valid only for horizontal translation and has units consistent with those of the static stiffness of kips/inch. The following section presents the actual values obtained from the pluck tests of test group 1 and test group 2.

3.4 RESULTS OF PLUCK TESTS

As stated earlier, each of the two test groups consisted of two sets of tests. Each individual test within the set was assigned an alphanumeric letter with a new beginning at the start of each test set. Both the static and dynamic parameters described in the above section were calculated for each pluck test performed. The results of the pluck tests are presented in this section.

3.4.1 TEST GROUP 1 RESULTS

Test group 1 was composed of two sets of tests; the first set was conducted on December 16, 1994 on pile 1 and the second set was conducted on December 17, 1994 on pile 2 (see Figure 1). The tests which compose set 2 were performed with the load applied from the northern pad. One of the purposes of test group 1

was to determine if there was any difference in the stiffnesses obtained from similar tests conducted on different piles near the same location. The results of set 1 and set 2 are presented in Tables 1 and 2.

3.4.1.1 SET 1

As can be seen from Table 1, the deflections for set 1 ranged from a low value of 0.042 inches to a maximum of 0.286 inches based on the bottom LVDT. The static stiffness ranged from a high of 125 kips/inch to a low of 24.14 kips/inch while the dynamic stiffness ranged from 62.77 kips/inch to 12.75 kips/inch. This extremely low value was believed to be caused by a crack in the pile. A general trend in each test was for the stiffness to decrease with increasing tests and increasing deflections; however, test group 1 set 1 showed the largest degradation. This probably can be attributed to the fact that the ground was becoming increasingly moist due to heavy rainfall. Both the static and dynamic stiffnesses show a decreasing magnitude with consecutive tests and increasing deflections. The average damping ratio ranged from 8.66% to 13.48% with the trend being a lower damping at lower deflections.

3.4.1.2 SET 2

The results of test set 2 are given in Table 2. The results of set 2 were quite different from those of test set 1. The deflections at release ranged from 0.038 inches to 0.106 inches with the static stiffness ranging from a high of 72.8 kips/inch to a low of 32.9 kips/inch. As can be seen, the degradation of the stiffness with increasing tests and deflections was not as pronounced as in set 1. The dynamic stiffnesses ranged from 39.1 kips/inch to 31 kips/inch. It should be noted that during this entire set of tests, water was standing around the base of

the pile. From experiences with loess during the pluck tests, it was observed that when the loess was dry it was very strong yet brittle, and upon saturation, had an entirely different consistency and was very soft. This change in consistency of the soil is also evident when considering the damping of the system. The damping of set 2 does not range above 7.2% which is much lower than the high value of 14 % from set 1.

3.4.2 TEST GROUP 2 RESULTS

Test group 2 was composed of two sets of tests conducted on March 20, 1995 and March 21, 1995. The first set of tests was conducted from the same pull direction as the tests from group 1 set 2. The purpose of this was to see if there was any repeatability in the tests or if there was any recovery of the soil conditions over time. Test set 2 was conducted on the same pile as set 1 with the pull direction moved 90 degrees to the eastern pad (see Figure 1).

3.4.2.1 SET 1

The results of test set 1 are given in Table 3. The results of this set of tests are comparable with the results of test group 1 set 2 since they were conducted on the same pile and from the same slab. The stiffnesses of test group 2 set 1 (March 1995) are slightly larger than those of test group 1 set 2 (December 1994). This can probably be attributed to the fact that the soil conditions were much drier for test group 2. The static stiffnesses ranged from 40.5 kips/inch to 36.4 kips/inch while the dynamic stiffness ranged from 44.1 kips/inch to 35 kips/inch. However, these stiffnesses did not vary greatly which shows some consistency in the tests. The average damping was also similar to those of group 1 set 2, ranging

from a high of 7 % to a low of 6.5 %. Upon completion of set 1, the pull direction was rotated 90 degrees to conduct test set 2.

3.4.2.2 SET 2

Table 4 shows the results of the tests which compose set 2. The results show the same general characteristics as those in test set 1. The static stiffness ranges from 56.4 kips/inch to 32 kips/inch while the dynamic stiffnesses went from 44.7 kips/inch to 33.5 kips/inch. The stiffnesses also decrease with consecutive tests with increasing deflections as those in group 1. The similarity between the two sets of tests on the same pile yet different directions indicates that the stiffnesses in one direction is independent of the movement in the other, a fact which allows for the performance of two independent tests on the same pile.

3.5 SELECTION OF STIFFNESS TO USE IN SEISAB

A number of stiffness values were presented in this chapter. The most important trend seen in the tests conducted was the decrease in the dynamic stiffness with the increased number of tests conducted and with increasing deflections. This trend is important because during an earthquake, a structure will experience many oscillations, some of which are larger than others. For this reason, a lower bound stiffness of 30 kips/inch was chosen to be input into SEISAB for determining the effects on bridge column design forces. The next chapter discusses the methods used as well as the output obtained when this soil stiffness is used to determine lateral spring coefficients for input into SEISAB.

CHAPTER 4

DESCRIPTION OF SEISAB

4.1 INTRODUCTION

In order to understand fully the results obtained from the application of bent foundation springs to SEISAB, an understanding of the program is necessary. Unlike most other dynamic analysis programs, SEISAB was specifically designed for determining the seismic response of bridges. SEISAB can be used to determine the response of continuous deck girder-type bridges or simply supported bridges (2). The program allows input of several different variables which describe the bridge structure and the loads to be imposed on the bridge. This input consists of the number of spans and corresponding lengths, the properties of the bridge elements, the height of the bridge, the response spectra, and the spring stiffnesses at the foundation (if desired). The output corresponding to the application of bent foundation stiffnesses is the focus of this thesis.

4.2 STRUCTURE DESCRIPTION

An important component in understanding the way SEISAB functions is the orientation of the axes. A bridge can be considered as composed of two major axes: the longitudinal, which is along the bridge centerline, and the transverse, which is perpendicular to the bridge centerline. In communicating the input data to SEISAB, the axes are assigned numbers to distinguish them apart. The number "1" corresponds to properties in the longitudinal direction while "3" corresponds to those properties in the transverse direction. The number "2" is reserved for those properties which pertain to the vertical axis. For example the term "KF1F1"

refers to the lateral spring stiffness in the longitudinal direction while "KM3M3" refers to the rotational stiffness in the transverse direction. An understanding of this convention will be helpful in understanding the input files for the four bridges which were analyzed. These output files are presented in Appendix 3.

The development of the input file is one of the most important aspects in the dynamic modeling of a bridge. Most errors in output are a direct result of incorrect information in the input file. The geometry of the bridge is modeled by inputting the number of spans and the corresponding lengths along with any alignment data. SEISAB also requires the input of the bridge's section properties such as the moment of inertia about each axis, the elastic modulus, and the weight per foot along the span of the bridge. Similar properties of the bent caps, bent columns, and abutments also must be input in order for an accurate model to be generated. The type of connection between each component is also very important and dictates how forces are transmitted between the bridge components. The connections input into SEISAB may be described as fixed, pinned, or transverse. A transverse connection allows free movement along the longitudinal axis of the bridge while providing a pinned connection along the transverse axis.

In addition to the properties of the structure, spring stiffnesses to represent the stiffness of the soil may be input into the program. The spring stiffness at the abutments and the bent columns can be input for the longitudinal, transverse, and vertical directions. If no stiffness is input, the default condition is fixed for all directions of motion. The following is a description of the stiffness parameters input into SEISAB:

KF1F1 longitudinal stiffness (force/distance)

KF2F2	vertical stiffness (force/distance)
KF3F3	transverse stiffness (force/distance)
KM1M1	longitudinal rotational stiffness (force*distance/radian)
KM2M2	torsional rotational stiffness (force*distance/radian)
KM3M3	transverse rotational stiffness (force*distance/radian)

The stiffnesses represent the total stiffness of the component being modeled. If a bridge bent is supported by two columns, the stiffness must be calculated for each column and input to SEISAB. The final item which composes the input file is the description of the loading.

4.3 LOAD DESCRIPTION

SEISAB will perform a frequency analysis or a response spectrum analysis, but will not perform a static analysis. This thesis is concerned with the forces generated from SEISAB as a result of the application of a response spectra. The response spectra allows SEISAB to determine the dynamic response of a bridge during an earthquake. In conducting a response spectra analysis, an acceleration response spectrum must be input. Several different options exist in the application of the response spectrum to SEISAB. The response spectrum may be input manually through coordinate values for a site specific spectra, or one of the pre-loaded response spectra in SEISAB may be utilized. The pre-loaded response spectra include curves from AASHTO and Caltrans for several different soil types. The AASHTO ground motion spectra for soil type II was used for all bridges analyzed in this thesis, consistent with TNDOT practices. The ground motion spectra, shown in Figure 6, shows the spectral acceleration on the vertical axis and

the period on the horizontal axis. The acceleration coefficient is also be input corresponding to the appropriate seismic zone. SEISAB applies the response spectrum in both the longitudinal and the transverse directions of the bridge independently. The following is a summary of the load cases considered in the calculation of forces by SEISAB:

Load Case 1	Longitudinal
Load Case 2	Transverse
Load case 3	$1.0 \times \text{Longitudinal} + 0.3 \times \text{Transverse}$
Load Case 4	$0.3 \times \text{Longitudinal} + 1.0 \times \text{Transverse}$

Upon completion of the analysis, SEISAB presents the forces generated as a result of the response spectrum. The forces are determined in each component for each of the four load cases.

4.4 SEISAB OUTPUT

In performing the response spectra analysis, SEISAB computes the period for each of the mode shapes along with the participation factors and the percentage of mass contribution for each of the three directions. It is recommended to use the number of modes to give at least 90 percent of the total mass contributing in both the longitudinal and transverse directions (2) which are the modes of greatest concern.

In addition to the mode shapes and characteristics of each mode, SEISAB also presents the resulting displacements at both the abutments and the bents for each load case. These displacements are with respect to the centerline of the bridge. Along with the displacements, the forces in the columns and abutments

are also given in the output. As with the displacements, the forces are given for each load case. The column forces presented include the shear, moment, axial, and torsional forces for the top and bottom of each column.

The column forces and deflections determined from SEISAB are the primary components in determining what effect the addition of springs stiffnesses had on the dynamic response of the bridges analyzed. The next section describes the methods used in determining the soil stiffnesses to be used on the bridges modeled.

CHAPTER 5

APPLICATION OF SOIL STIFFNESSES TO SEISAB

5.1 INTRODUCTION

This chapter deals with the results obtained in the application of the soil springs, determined from the pluck tests, to SEISAB. Four bridges were analyzed using SEISAB for both fixed foundations and foundations with soil stiffnesses. The bridges which were analyzed consisted of a bridge in Haywood County, Tennessee, a bridge on the Nonconnah Parkway near Memphis, Tennessee, a bridge in Madison County, Tennessee, and a bridge on the Paul W. Barret Parkway over the Loosahatchie River also near Memphis. All bridges were originally analyzed and designed by the Tennessee Department of Transportation using fixed conditions for the soil springs. The purpose of adding soil springs is to determine if any differences result in design forces.

The bridges were chosen to be analyzed because of their different foundation types. Both the Madison County and the Nonconnah Parkway bridge bents consist of columns supported by a pile group topped with a pile cap, while each of the Haywood County and Loosahatchie bridge bents consist only of a single row of free-standing piles without a pile group or pile cap. Upon inspection it is evident that the pile cap/pile group bridges will possess additional lateral resistance in comparison with the pile-bent bridges because the pile cap is below the ground surface.

As seen in Chapter 3, the stiffnesses obtained varied from test to test, and the stiffness decreased for increased deflections and the number of test repetitions. In an attempt to emphasize any difference in results between the fixed versus spring stiffness, a lower bound for the soil stiffness was used to

calculate the spring constants to be used in SEISAB. A stiffness of 30 kips/inch was used which represents a relatively low value from the pluck tests and does not correct for the fact that the stiffness at the ground surface would be greater than that at the center of mass obtained in the tests. The same stiffness is used for all 14" x 14" concrete piles and is adjusted accordingly to account for any piles which are of different size.

This chapter presents a description of each bridge and the method used to transform the stiffnesses obtained from the pluck tests into foundation stiffnesses for use in SEISAB. Also presented is a comparison of the SEISAB output for fixed foundation conditions and soil springs for each of the four bridges. The soil stiffnesses determined from the pluck tests are only for the piles; thus, the abutment stiffness as well as the stiffness of the pile cap had to be determined by analytical methods. These analytical methods are the same as those used by Cook (3) in a sensitivity study*. Both the analytical methods as well as the methods for determining stiffnesses from the tests are presented in the next section.

5.2 DETERMINATION OF THE SOIL STIFFNESSES TO USE IN SEISAB

The lateral abutment stiffnesses were based on analytical methods developed by Wilson (15), while the pile cap stiffnesses were developed using a model developed by Richart et. el. (20).

5.2.1 DETERMINATION OF ANALYTICAL STIFFNESSES

There are several different methods which could have been used in order to determine the stiffnesses, two of which were dealt with in the Cook sensitivity

* Sample calculations are provided in Appendix D for the analytical stiffnesses.

study (4). Cook considered methods developed by Novak (15) and the traditional Beam on Elastic Foundation Method to determine pile and pile cap stiffnesses and used a method developed by Wilson (16) to account for the abutment stiffnesses.

5.2.1.1 ABUTMENT STIFFNESSES

The stiffnesses determined for all bridges modeled were lateral soil springs. The model developed by Wilson (16) was based on several important assumptions. One of the most important is the assumption that the soil behind the abutment wall has a constant soil modulus E_s with respect to depth. This behavior is most prominent in clay backfills; however, is not completely irrational when dealing with loess. Another important assumption in the model is that the stiffness along the centerline of the bridge is the same regardless of direction, which means that there is no assumed difference between tension or compression because of sufficient bond between the back of the wall and the soil. Based on the above assumptions, the following equations were determined for the translational stiffness:

(longitudinal direction)

$$K_{11} = \frac{EsB}{(1 - \nu^2)I'} + K_1^p + 2K_1^w \quad (\text{Eq. 5.1})$$

(transverse direction)

$$K_{33} = \frac{2EsL}{(1 - \nu^2)I''} + K_3^p + 2K_3^w \quad (\text{Eq. 5.2})$$

where K_{11} and K_{33} represent the total abutment stiffness in the longitudinal and transverse directions of the bridge, respectively, while K^P and K^W represent the stiffness of the endwall piles and wingwall piles respectively. The variables B and L are characteristics of the abutment and are shown in Figure 7. The terms I^e and I^w represent influence factors for the endwall and the wingwalls respectively. These factors can be determined from Figure 8 and originate from Poulos and Davis (17). The terms for the soil properties were determined from a soil study on the dynamic properties of loess conducted by Chang et. al. (18) from the University of Memphis. The values used to determine the stiffness parameters can be found in Table 5. Even though other stiffnesses can be determined, only the translational stiffnesses will be used at the abutments. The abutment stiffnesses were used for both fixed bent models and spring bent models in the analysis each bridge.

5.2.1.2 PILE CAP STIFFNESS

The pile cap stiffness needed to be calculated for only two of the bridges being considered: the Nonconnah Parkway Bridge and the Madison County Bridge. As described by Cook (3), two common methods to determine the analytical lateral resistance supplied by the pile cap stiffness are those developed by Prakash and Puri (19) and those developed by Richart et. al. (20).

The method by Prakash and Puri treats the pile cap as embedded below the ground surface. Even though this is often the case, it may lead to stiffnesses which are too large due to the fact that, upon completion of the largest amplitude cycle during an earthquake, the soil against the face of the pile cap has been displaced and should not be considered to provide lateral resistance. For this

reason, the method by Richart et. al. was used to calculate the pile cap stiffness in this case.

The equations given by Richart et. al. were developed based on the elastic half-space theory. This theory assumes the pile to be resting on the ground surface. These formulas developed were based on a circular footing; however, they can be adjusted to account for a rectangular or square foundations. Only the translational soil stiffness is considered and is calculated from the following equation:

$$K_{xf} = \frac{8}{(2 - \nu_s)} G_s r_o \quad (\text{Eq. 5.3})$$

where ν_s is Poisson's Ratio for the soil being considered and G_s is the shear modulus of the soil beneath the footing. The term r_o is the equivalent circular radius for a rectangular footing which is given by:

$$r_o = \sqrt{\frac{bL}{\pi}} \quad (\text{Eq. 5.4})$$

The terms b and L correspond to the effective width and effective length of the pile cap. Since equation 5.4 relies on the shear modulus of the soil which acts over the equivalent radius, the portion of the length and width taken up by the piles underneath the pile cap must be subtracted from the actual length. Failure to do this can lead to overestimation of the stiffness of the pile cap since the stiffnesses of the piles are accounted for alone.

In addition to the analytical pile cap stiffnesses, an analytical value of the pile rotation was also calculated to be applied to the pile-bent bridge foundations.

This decision was made based on the behavior of a free-standing pile upon deflection. If only translational springs are used, the pile has zero rotation at the ground surface which is not very realistic. The actual behavior of the pile will allow the pile to rotate, and by modeling the pile with a rotational stiffness about the longitudinal and transverse directions, the point of zero rotation is actually below the ground surface. This rotational spring was not used for the bridges with pile caps since the pile cap tends to move translationally and have a great amount of rotational resistance and tends to prevent the piles from rotating at the ground surface. The rotational capacity was determined using the Beam on Elastic Foundation method also used by Cook (3). The rotational stiffness of the pile was calculated using the following formula:

$$K_{\phi 1} = \frac{K}{4\lambda^3} \quad (\text{Eq. 5.5})$$

where $K = k_o b$. The term k_o is the subgrade modulus and b is one-half the circumference of the pile. The term λ can be determined as follows:

$$\lambda = 4 \sqrt{\frac{K}{4E_p I_p}} \quad (\text{Eq. 5.6})$$

where the terms E_p and I_p correspond to the elastic modulus and moment of inertia of the pile respectively.

For the foundations where the rotation was not considered, the stiffness of the pile cap was combined with the stiffness of the pile group to produce the total bent foundation stiffness which is presented later in this chapter for each structure.

5.2.2 DETERMINATION OF PILE-SOIL STIFFNESSES FROM TEST DATA

The calculation of the pile stiffness to use in SEISAB was based on the stiffness of 30 kips/inch determined from the pluck tests. The process was slightly different for the two different bent foundation types. The pile cap foundations required the calculation of the lateral stiffness for the entire pile group under each column while the pile bent stiffness only needed to be reduced based on the spacing of the piles.

For the pile group stiffness, the unreduced stiffness of a single 14" x 14" concrete pile was multiplied by the number of piles in the particular group. This number was then reduced based on the spacing of the piles. The closer the piles are spaced, the larger the reduction. The formula used to calculate the stiffness of the pile group is given as:

$$K_{gr} = K_{gu}e_g \quad (\text{Eq. 5.7})$$

where K_{gu} is the unreduced group stiffness given as:

$$K_{gu} = (\text{individual pile stiffness}) \times (\text{number of piles}) \quad (\text{Eq. 5.8})$$

The term e_g is the stiffness reduction factor which can be determined from Figure 9, taken from a publication by Norris and Sack (21). As can be seen from Figure 9, this factor can greatly reduce the lateral capacity of a pile group and must be considered in attempting to determine appropriate soil stiffnesses.

The use of the reduction factor, e_g , was not limited to bridge foundations with pile groups but was also applied to the free-standing pile bent bridges. The

stiffness calculated was the stiffness of one pile multiplied by the number of piles and then by the appropriate reduction factor based on the spacing of the piles along the bent. The details of the stiffnesses used for each bridge is described in more detail later.

5.3 MODELING OF BRIDGES WITH PILE GROUP FOUNDATIONS

The modeling of the two different bridge types was handled separately in this thesis because of the different methods required to model their foundation. Both the Madison County bridge and the Nonconnah Parkway bridge have foundations which consist of rather large columns resting on a pile cap which is supported by a group of piles. These two structures were chosen based on their relative sizes. The Nonconnah Parkway is a massive, seven span structure with a total length of 1765 ft. The Madison County bridge, on the other hand, is composed of only two spans with a total length of 261 ft. By using these two structures, the effect of the mass and the size of the structures on the dynamic response could be considered.

5.3.1 MODELING OF THE NONCONNAH PARKWAY BRIDGE

In modeling the Nonconnah Parkway Bridge, the bridge was modeled to conform as closely as possible to the original analysis performed by the Tennessee Department of Transportation; however, the actual output from the TNDOT model was not used because an older version of SEISAB (version 1.6) was used by TNDOT. The output between the two versions differed by as much as 10 %. In order to obtain comparable values, the fixed condition model and soil spring

model were modeled on the same version of SEISAB (version 4.0) to obtain comparable values.

5.3.1.1 DESCRIPTION OF NONCONNAH PARKWAY BRIDGE

The Nonconnah Parkway Bridge (Figure 10) is a continuous 7 span structure with a total length of 1765 ft. The bridge is supported by prestressed pier caps which are supported by two 8 ft. diameter concrete columns spaced 70 ft. apart. The foundation for each bent is not the same; the number of piles and size of the pile cap under each column varies. Table 5 gives the dimensions of each pile cap along with the number of piles and their spacing. As can be seen, there is a large number of piles under each column, and they are spaced relatively close together. All pile groups have a 6 ft. thick pile cap with a 2 ft. pedestal on which the column rests. The abutments for the Nonconnah Parkway Bridge consist of monolithic reinforced concrete walls. The connection between the bridge and the abutments is very important in the response of the bridge during an earthquake since it dictates how the bridge will respond along its longitudinal axis. The Nonconnah Parkway bridge abutments have a slider plate as a connection between the bridge and the abutment. These slider plates allow movement along the longitudinal axis of the bridge while preventing movement in the transverse direction.

All components of the bridge foundations were assigned translational stiffnesses to represent the soil's ability to resist deformation. As discussed earlier the methods used to calculate the stiffnesses were based on both analytical formulas and test data. The following section describes the actual soil stiffnesses used in the SEISAB model of the Nonconnah Parkway bridge.

5.3.1.2 DETERMINATION OF SEISAB FOUNDATION STIFFNESSES

The stiffnesses used in SEISAB were calculated based on the methods described in Section 5.2. Analytical stiffnesses had to be calculated for both the abutments and the pile cap. A pile stiffness of 30 kips/inch was used to determine the stiffness of the pile group as well as the piles which are underneath the abutments. The abutment stiffnesses were calculated using equations 5.1 and 5.2. A summary of the abutment stiffnesses is presented in Table 7.

The foundation stiffnesses were calculated using equation 5.3 for the pile cap stiffness and equation 5.7 for the stiffness of the pile group. A summary of the stiffnesses used is presented in Table 8 along with the individual contribution of each component. As can be seen, the contribution from the pile cap is significantly larger than the contribution of the piles, a fact which can be attributed to the small reduction factor (e_g) which had to be used. A factor of 0.354 was used for the reduction factor since the spacing of the piles was closer than 3 pile diameters. The lowest value from Figure 9 had to be used since it was thought best not to extrapolate values from the table. The stiffnesses calculated were the actual stiffnesses used in the SEISAB analysis.

5.3.2 MODELING OF THE MADISON COUNTY BRIDGE

As with the Nonconnah Parkway bridge, the Madison County bridge was modeled as closely to the original by TNDOT as possible. The Madison county bridge properties were the same as those used by TNDOT. The following sections describe the structure of the bridge as well as the spring stiffnesses used in the SEISAB model.

5.3.2.1 DESCRIPTION OF THE MADISON COUNTY BRIDGE

The Madison County bridge is a continuous, two span structure with a total length of 261 ft. Plan view and elevation views of the bridge are shown in Figure 12. The bridge is supported by a central bent supported by three 3 ft. by 3 ft. columns spaced 19 ft. center to center and which rest on a pile cap. A plan and elevation of a typical pile cap can be seen in Figure 13. This configuration is the same for all three columns along the bent. The pile caps and the number of piles are much smaller than those of the Nonconnah Parkway bridge and thus provide less resistance to movement. The one major difference in the two bridges is the abutment connection. The Madison County bridge has pinned connections at the abutments, unlike the Nonconnah Parkway bridge which was modeled with a transverse connection. This pinned connection allows the abutments to restrict the movement of the bridge in the longitudinal axis of the bridge and also reduces the forces generated in the columns longitudinally since they will be restrained from deflecting by the abutments.

The determination of the stiffnesses used in SEISAB was made using the same methods as those used in the Nonconnah Parkway bridge calculation. The following section discusses the stiffnesses calculated which were used in the SEISAB model.

5.3.2.2 DETERMINATION OF SEISAB FOUNDATION STIFFNESSES

The stiffnesses used in the modeling of the Madison County bridge were calculated using the methods discussed earlier in this chapter. The analytical abutment stiffnesses were calculated using equations 5.1 and 5.2 with a summary

presented in Table 9. While these stiffnesses were smaller than those calculated for the Nonconnah Parkway bridge, they are still large enough to provide a substantial amount of resistance. The results of the application of these spring stiffnesses to SEISAB is discussed later in this chapter. The following section describes the differences and methods used in modeling the bridges with free-standing pile bents.

5.4 MODELING OF BRIDGES WITH PILE BENT FOUNDATIONS

The foundations of the Loosahatchie and Haywood County bridges differ substantially from those of the Nonconnah Parkway and Madison County bridges. The Loosahatchie and Haywood County bridge bents consist of single rows of free standing piles which also serve as the sub-surface foundation of the bridge. As with the pile cap bridges, these two bridges differ greatly with respect to their size. The Loosahatchie bridge is actually a 30 span structure; however, for purposes of modeling, the number of spans had to be reduced to 15 due to limitations of computer memory. The Haywood County bridge is a three span structure with a total length of 94 ft. The primary difference in modeling the pile bent bridges was the addition of rotational springs as well as translation. The rotational springs were calculated based on equation 5.5. The details of modeling each of the bridges follows.

5.4.1 MODELING OF THE LOOSAHATCHIE BRIDGE

The modeled version of the Loosahatchie bridge was nearly half the span of the actual bridge. The bridge was modeled using the first fifteen spans of the actual bridge along with the original section properties used in the TNDOT SEISAB model. This was done to keep the response of the structure as near the actual as

possible, but the difference in lengths between the actual structure and the model prevents an exact comparison. In an attempt to correct for the difference in lengths, the average height of the actual bridge was increased and applied to the model. This was not considered to be a problem since both the fixed foundation model and the foundation springs model were modeled identically with the exception of the bent spring stiffnesses.

5.4.1.1 DESCRIPTION OF THE LOOSAHATCHIE BRIDGE

The Loosahatchie bridge model has a total length of 1130 ft. as indicated in Figure 14. The model was taken to have a constant height along the span in order to simplify any discussions of reasons for differences in output between fixed bents and bent springs. The bridge was also assumed to be straight for the same reason. The actual bridge bents consist of 13-18" x 18" precast concrete piles driven into the ground with the exposed portion tied into the pier cap. For the same reason the bridge spans had to be reduced, the number of columns modeled at each bent also had to be reduced. The number of columns modeled was reduced to 7 piles per bent, yet the properties of the original 18" x 18" piles were adjusted to account for the reduction in the number of piles. Thus, the piles had to be modeled as 24" x 24" piles as can be seen in Figure 15. This procedure was the same as used in the original TNDOT SEISAB model of the Loosahatchie bridge.

The abutment connection of the Loosahatchie bridge is the same as the Nonconnah Parkway bridge. At each abutment there is a slider which allows movement in the longitudinal direction but serves as a pinned connection in the transverse direction. This design allows the bridge to freely deflect in the longitudinal direction without any resistance from the bridge abutments. The

soil stiffnesses input into SEISAB reflect another difference in the two bridge foundation types and is described in the next section.

5.4.1.2 DETERMINATION OF SEISAB FOUNDATION STIFFNESSES

The calculation of the abutment stiffnesses for the Loosahatchie bridge was accomplished using equations 5.1 and 5.2 described in section 5.2. A summary of the abutment stiffnesses used in the SEISAB model are given in Table 11. The abutment stiffnesses were used in both fixed bents and bents with spring constants in order to determine the effect of changing the bent stiffness.

In determining the bent stiffnesses to be used, spring constants had to be calculated for each of the piles which are embedded in the ground. The same stiffness was used for each pile throughout the entire structure. This stiffness which was input into the program was calculated by first adjusting the standard stiffness for a 14" x 14" pile to represent the stiffness of an 18" x 18" pile using a linear increase. The resulting stiffness of 38.4 kips/inch was then multiplied by 13 (the number of piles per bent as constructed) to obtain a total stiffness per bent. This total stiffness was then divided by seven (the number of piles per bent as modeled) to obtain a stiffness per pile to be used in the SEISAB model. This stiffness was then multiplied by a reduction factor (eg) of 0.354 (Figure 9) based on the actual pile spacing. The bent stiffness used in the model can be seen in Table 12. The value of 304 kips/ft is the stiffness for each individual pile and has been reduced based on the spacing of the piles.

The rotational stiffnesses which were input in the model along with the translational stiffnesses were determined using equation 5.5. This rotational stiffness was calculated based entirely on analytical methods and reflects no influence from the pluck test data. The rotational stiffnesses used can be seen

along with the translational stiffnesses in Table 12. The methods used in the modeling of the Loosahatchie bridge were also applied to the Haywood County bridge.

5.4.2 MODELING OF THE HAYWOOD COUNTY BRIDGE

The Haywood County bridge structure is similar to that of the Loosahatchie bridge in that the bents consist of free-standing piles. The Haywood County bridge was able to be modeled as it was originally designed without any reductions in spans or the number of piles.

5.4.2.1 DESCRIPTION OF THE HAYWOOD COUNTY BRIDGE

The Haywood County bridge is a three span structure with a total length of 94 ft. as shown in Figure 16. The bridge is supported by two bents composed of 6 - 14" x 14" concrete piles. A typical elevation of the bridge bents is shown in Figure 17. This bridge was the shortest structure analyzed using SEISAB, with the Nonconnah and Loosahatchie bridge being more than ten times that length.

Though the bent foundation most nearly resembles the Loosahatchie bridge, the abutment connection is the same as that of the Madison county bridge. A pinned connection is used to connect the superstructure with the abutments. This decision allows forces to be transmitted to the abutments in both the longitudinal and transverse directions which tends to reduce the deflections of the superstructure in the longitudinal direction.

5.4.2.2 DETERMINATION OF SEISAB FOUNDATION STIFFNESSES

The stiffnesses which were input into SEISAB for modeling the Haywood County bridge were calculated using methods consistent with the other bridges

modeled. The abutment stiffnesses used in the program are given in Table 13. The determination of the soil stiffnesses to be used on the pile bents was similar to that used on the Loosahatchie bridge. A stiffness of 30 kips/inch was used for the 14" x 14" piles and was adjusted only by reducing the stiffness based on the spacing of the piles along the bent. Table 14 gives the stiffness per pile used in the SEISAB model. This stiffness is very low when compared to the stiffnesses used for the Nonconnah Parkway bridge because of the lack of an actual pile group or pile cap. The stiffness of each individual pile was put into SEISAB separately as opposed to being input as a pile group.

CHAPTER 6

RESULTS OF THE SEISAB ANALYSES

6.1 INTRODUCTION

A total of eight SEISAB models were analyzed, two for each bridge. Each bridge was first modeled using translational abutment springs and fixed foundation conditions at the bents. The original model was then adjusted only by adding translational springs except for the pile bent bridges which received both translational and rotational springs. The output from a typical SEISAB run consists of all mode shapes considered in the solution, along with the corresponding period, as well as forces induced in the columns and the displacements in both the columns and abutments. The two mode shapes which were considered to be critical were the longitudinal and the transverse directions.

6.2 THE NONCONNAH PARKWAY BRIDGE

The difference in natural periods between the two analyses was not significant in the Nonconnah Parkway bridge. As can be seen in Table 15, the natural period varied very little between fixed bents and springs. The highest difference was 1.6 percent which is insignificant. The same trend seen in the natural period was also evident in the maximum longitudinal and transverse displacements which can be seen in Tables 16 and 17 respectively. The longitudinal direction only showed a maximum difference of 2.3 percent while the transverse displacement showed a maximum 8.79 percent difference at bent 1. The 8.79 percent also becomes insignificant when the maximum deflection that occurred at bent 1 was 0.091 ft. The results were as expected in that more

deflection was attained upon addition of the soil springs even though the increase was slight.

The difference in the longitudinal and transverse moments was also insignificant. Tables 18 and 19 present the maximum column moments experienced per bent for the longitudinal and transverse direction respectively. The same results can be seen in Table 20 which compares the axial forces induced in the columns. The lack of substantial difference in any of the variables considered reflects the effect of the pile group and pile cap and their significant contribution to the stiffness of the structure.

6.3 THE MADISON COUNTY BRIDGE

The Madison County bridge experienced no significant change in the natural period in the two modes considered. Table 21 shows a maximum difference of 1.24 percent in the transverse mode and zero difference in the longitudinal mode. The same trend can be seen in Tables 22 and 23 which presents the resulting displacements in the longitudinal and transverse directions respectively. The longitudinal direction again showed no difference in comparison of the two runs with only minimal difference in the transverse displacements. This trend continues through the resulting moments as presented in Tables 24 and 25.

The lack of difference in the longitudinal direction can be attributed to the pinned connection between the abutment and the superstructure. This pinned connection allows all forces generated in the longitudinal direction to be directly transferred into the abutment which has a high longitudinal stiffness. The transverse parameters did not vary much because of the near fixity of the pile

group and pile cap. These components provide enough lateral resistance to simulate nearly fixed conditions.

6.4 THE LOOSAHATCHIE BRIDGE

In contrast to the Nonconnah Parkway bridge, the Loosahatchie bridge showed some significant differences in the dynamic response of the two models. Table 27 shows a comparison of the natural period for the longitudinal and transverse modes. A 14.1 percent difference was experienced along the axis of the bridge with a 24.5 percent difference in the transverse direction. The addition of the soil springs made the structure more flexible which increased the period in each mode. The corresponding displacements also showed some significant differences. The longitudinal displacement increased by 13 percent as shown in Table 28, while the transverse displacement increased by 31 percent at mid-span. This increase in deflections can be critical in the behavior of a structure. The deflections presented are in feet, and close inspection of Table 28 shows a maximum deflection of 1.6 ft. occurred in the model with the bent springs. An equally alarming deflection of 1.4 ft. occurred for the analysis with fixed bent foundations. These large deflections are the result of the transverse connections between the abutments and the superstructure. In actuality, the sliders have keepers at the ends to prevent excessive movement but it is not possible to model this behavior in SEISAB. Deflections of the magnitude shown in Table 28 could result in serious damage to the structure of the bridge.

The moments experienced by the model with the bent springs showed a 14 to 16 percent reduction from the fixed bent model as can be seen in Tables 30 and 31. This decrease is consistent throughout the span of the structure and results because the piles supporting the bridge were able to rotate as the

superstructure oscillated. The same behavior can be seen when the axial forces between the two models are examined in Table 32. The axial forces experienced more than a 50 percent drop upon addition of soil springs. This decrease can be attributed to the translational soil springs which allow the foundation to move somewhat as the superstructure moves. The largest axial forces occur at the midspan of the bridge as expected for the earthquake movement in the transverse direction.

6.5 THE HAYWOOD COUNTY BRIDGE

An assumption based on the structure type and components would suggest that the Haywood county bridge would behave as the Loosahatchie bridge in the transverse direction and as the Madison County bridge in the longitudinal direction due to the pinned abutments. Table 33 shows the assumption to be somewhat correct. The longitudinal period did not increase while the transverse mode period showed a slight increase of 5 percent. The longitudinal displacements shown in Table 34 only experienced a decrease of 0.001 ft. in deflection of the abutments while the difference in the bent deflection was zero. The transverse displacements showed an increased deflection of only 0.002 ft.

The moment comparison shown in Tables 36 and 37 shows a decrease of 86 percent; however, to put the difference into perspective, the magnitudes of the moments only differ by 6 kip-ft. in the longitudinal direction. This low moment value can be attributed to the pinned abutment connection. The transverse moments showed a substantial decrease of 47 percent upon addition of the soil springs. This difference is significant as is the difference in the axial forces shown in Table 38. The axial forces decreased nearly 73 percent; however, since these forces are relatively small, the difference is not as meaningful.

6.6 DISCUSSION OF RESULTS

The results presented in this chapter show that the bridges which were most sensitive to the addition of foundation springs were the pile bent bridges. The Loosahatchie and Haywood County bridges both showed a larger percent difference than the Nonconnah Parkway bridge and the Madison County bridges. The addition of the foundation springs resulted in decreased moments and forces while increasing the deflections. The Loosahatchie bridge showed the largest differences in results from SEISAB because of its foundation type and large span.

In conducting these analyses, the aspect of the output of most concern was the deflections experienced in the longitudinal direction of the Loosahatchie bridge. The 1.6 ft. of deflection raises questions as to the validity of SEISAB. This large deflection is allowed through the transverse connection between the abutments and the superstructure. In SEISAB, this type of connection allows complete free movement in the longitudinal direction while providing a pinned connection in the transverse direction. The actual slider on the bridge has keepers on the sides to prevent transverse movement but nothing to prevent movement in the longitudinal. The distance from the center line of the bearing device to the end of the abutment is 1'-6" for the Loosahatchie bridge. Any movement greater than this will result in the bearing device coming off the end of the abutment. For this reason, the output from SEISAB should be viewed with some skepticism.

Upon inspection of the results from the SEISAB models, it is evident that the structures which are most sensitive to the bent springs are the structures whose bents consist of free-standing pile bents. These structures are sensitive to

both translational and rotational springs. The structures with the pile groups topped with a pile cap are less sensitive to the stiffnesses with no significant increase or decrease in deflections or moments.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 SUMMARY

Current bridge analysis by the Tennessee Department of Transportation (TNDOT) uses the program SEISAB to determine forces resulting from dynamic loadings. SEISAB allows for the input of spring stiffnesses to represent the stiffness of the foundation at the bridge abutments and bents. Since the current modeling techniques used by TNDOT assume fixed conditions at the bents, it was desired to determine, through conducting dynamic field tests, actual values for the stiffnesses of piles founded in loess. Tests were deemed necessary because no test data existed on the dynamic characteristics of piles in loess, a soil which is predominant in West Tennessee.

The majority of the dynamic stiffnesses determined from the pluck tests ranged from 30 to 40 kips/inch. A value of 30 kips/inch was used in calculating the stiffnesses of the bridge components for application in SEISAB. This value represented a lower bound for the tests conducted. The 30 kips/inch was used to calculate the individual pile stiffnesses as well as the stiffnesses of the pile groups. Analytical methods were used to determine the abutment stiffnesses and the pile cap stiffnesses. Four bridges, originally modeled by TNDOT, were used for analysis in this thesis. The bridges modeled consisted of two bridges with pile group foundations and two bridges with free-standing pile bent foundations. The Nonconnah Parkway bridge and the Madison County bridge have pile bents which rest on a pile group supported by a pile cap, while the Loosahatchie and the Haywood County bridges are supported by free standing pile bents. The bridges

with the pile cap/pile group foundations are much stiffer laterally than the bridges with pile bents.

Each of the bridges was modeled with both fixed foundations and soil springs at the foundations. The resulting output was then compared for the two cases. The forces, moments, and deflections from the dynamic analysis showed no significant change between the fixed foundation bridge models and the spring stiffness foundation models for the bridges with the pile cap/pile group foundations. These parameters showed a significant change in the bridges with the pile bent foundations. The largest percent difference experienced was in the Loosahatchie bridge because of its long span and foundation type.

7.2 CONCLUSIONS

Upon inspection of the output presented in Chapter 6, the largest change in the dynamic response occurred in the Loosahatchie bridge models. The forces in the Haywood County bridge also experienced a significant percent difference as a result of the addition of the foundation spring stiffnesses, but the magnitudes of the forces in the Loosahatchie bridge identified it to be the most critical. This difference in effects of adding springs can be attributed to the free standing pile bent foundation which is common to both the Loosahatchie and the Haywood County bridges. The pile cap/pile group foundations of the Nonconnah Parkway and Madison County bridges provide a great amount of resistance to lateral movement, even when springs are added. This high resistance is the reason for the relatively small percent differences in moments and displacements upon addition of the foundation springs.

The displacements which were determined from the SEISAB analyses also raised some important concerns. These concerns are evident when looking at the

displacements of the Loosahatchie bridge. The original model of the Loosahatchie bridge by the Tennessee Department of Transportation (TNDOT) showed deflections of over 1.2 ft. in the longitudinal direction. These deflections increased to 1.4 ft. and 1.6 ft. for the University of Tennessee models of fixed bents and spring bents respectively. The University of Tennessee models represent the output from the SEISAB version 4.0 runs described earlier in this thesis. These deflections are a direct result of the type of transverse connection at the abutments which puts no limit on the amount of deflection in the longitudinal direction. These values of longitudinal deflection push the limit of possible movement the abutments can accommodate. If the allowable travel were exceeded, the bridge would either slip off the edge of the abutment or be driven into the opposite endwall. This behavior cannot be modeled in SEISAB.

From the results presented in Chapter 5, the TNDOT modeling procedure of using fixed foundations for stiffnesses in SEISAB is sufficiently accurate for those bridges with pile cap/pile group foundations. The bridges with the free standing pile bents showed a significant decrease in the forces when modeled with spring stiffnesses which indicates the TNDOT modeling assumptions tend to be conservative with respect to forces and moments. However, the deflections increased in each bridge modeled with the bent spring stiffnesses. These increased deflections should be considered, particularly for long bridges with free-standing pile bents and which have "transverse" connections at the abutments. The deflections merit concern because the transverse connection at the abutments, which serves only as a means of allowing the bridge to expand and contract due to thermal changes, must be able to accommodate the movement which may occur in an earthquake.

LIST OF REFERENCES

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1. Guide Specification For Seismic Design of Highway Bridges. American Association of State Highway & Transportation Officials, 1983.
2. Imbsen and Associates. SEISmic Analysis of Bridges (SEISAB), Users Manual, 1993.
3. Cook, Tommy L. "Modeling of Soil-Pile Interaction for Bridge Foundations Under Seismic Loading." Thesis submitted in partial fulfillment for the requirements for the degree of Master of Science. The University of Tennessee, Knoxville, Tennessee, 1994.
4. Cook, Tommy L.; Burdette, Edwin G.; Graves, Richard L.; Goodpasture, David W.; Deatherage, J. Harold. "Effect of Varying Foundation Stiffnesses on Seismically Induced Loads in Bridge Bents: A Sensitivity Study." Transportation Research Board, 1994.
5. Graves, Richard L. "Dynamic Stiffness and Damping of Single Piles in Loess." Thesis submitted in partial fulfillment for the requirements for the degree of Master of Science. The University of Tennessee, Knoxville, Tennessee, 1995.
6. Blaney, Geoffrey W. and O'Neill, Michael W. "Measured Lateral Response of Mass on Single Pile In Clay," Journal of Geotechnical Engineering. Vol. 112, No. 4, April, 1986, pp. 443-457.
7. Blaney, Geoffrey W. and O'Neill, Michael W. "Procedures for prediction of Dynamic Lateral Pile Group Response in Clay From Single Pile Tests," ASTM Geotechnical Testing Journal. Vol. 14, March, 1991, pp. 3-12.

8. Brown, Dan A., and Reese, Lymon C., and O'Neill, Michael W. "Cyclic Lateral Loading of A large-Scale Pile Group," Journal of Geotechnical Engineering. ASCE, Vol. 113, No. 11, November, 1987, pp. 1326-1343.
9. Poulos, H. G. "Behavior of Laterally Loaded Piles: II-Pile Groups," Journal of The Soil Mechanics and Foundations Division. ASCE, Vol. 97, No. SM5, May, 1971, pp. 733-751.
10. Das, Braja M. Principles of Foundation Engineering. PWS-Kent. Boston, Mass., 1990.
11. McAlister, David H. "Development of Dynamic Test for Analyzing Free Vibrations of Bridge Piers." Thesis submitted in partial fulfillment for the requirements for the degree of Master of Science. The University of Tennessee, Knoxville, Tennessee, 1994.
12. Tatali, Anish K. "Dynamic Testing of Single Piles In Loess Soil." Thesis submitted in partial fulfillment for the requirements for the degree of Master of Science. University of Memphis, Memphis, Tennessee, 1995.
13. Dywidag Systems International , "Dywidag Threadbar Post-Tensioning System." Bolingbrook, Illinois (1994).
14. Clough, Ray W. and Penzien, Joseph. Dynamics of Structures. 2nd Edition. New York, New York. McGraw-Hill, Inc. 1993.
15. Novak, M. "Dynamic Stiffness and Damping of Piles," Canadian Geotechnical Journal. Vol. ii, pp. 574-598 1974.

16. Wilson, John C. "Stiffness of Non-Skew Monolithic Bridge Abutments for Seismic Analysis," *Earthquake Engineering and Structural Dynamics*. Vol. 16, January, 1988, pp. 867-883.
17. Poulos, H. G. and Davis, E. H. Elastic Solutions for Soil and Rock Mechanics. Joch Wiley and sons, 1974.
18. Chang, Tzyy-Shiou, Teh, Lin Kee, Yigong, Zhang. "Seismic Characteristics of Sediments in the New Madrid Seismic Zone," Memphis State University, Memphis, Tennessee, 1992.
19. Prakash, Shamsheer, and Puri, Vijay Kumar. Foundations For machines: Analysis and Design. John Wiley and Sons, 1988.
20. Richart, F. E. Jr., and Hall, J. R., and Woods, R. D. *Vibrations of Soils and Foundations*. Prentice-Hall, 1970.
21. Norris, G. M. and Sack, R. L., "Evaluation of the Lateral Stiffnes of Pile Groups for Seismic Analysis of Highway Bridges" *Proceedings, 22nd Engineering Geology and Soils Engineering Symposium*, Boise, Feb. 1986, pp. 323-363.

APPENDICES

APPENDIX A:

TABLES

TABLE 1: RESULTS OF TEST GROUP 1, SET 1 (PILE 1)

TEST	DEFLECTION AT RELEASE (inches)		LOAD (kips)	STATIC STIFFNESS (kips/inch)	DAMPED FREQUENCY (cyc/sec)	DAMPING RATIO (%)	UNDAMPED FREQUENCY (cyc/sec)	DYNAMIC STIFFNESS (kips/inch)
	TOP	BOTTOM						
D	0.05281	0.04187	4.49	-	7.8	9.3	7.83	62.77
E	0.0547	0.04319	5.41	-	7.4	8.66	7.44	56.58
F	0.07017	0.04205	3.91	55.58	7.13	9.35	7.16	52.49
G	0.2081	0.09614	7.78	37.48	6.5	11.99	6.55	43.93
H	0.2213	0.1134	9.08	41.13	6.13	11.73	6.17	39
J	-	-	-	-	5.66	12.9	5.71	33.39
K	-	-	-	-	5.71	13.1	5.76	33.91
L	0.124	0.1039	10.2	-	5.6	11.74	5.64	32.57
M	0.167	0.1348	13.2	-	5.17	13.48	5.22	27.84
N	0.1618	0.1342	12.7	-	4.92	12.35	4.96	25.16
O	0.2206	0.1373	10.8	48.97	4.94	12.5	4.98	25.36
P	0.287	0.1326	9.66	33.45	4.92	12.1	4.95	25.12
Q	0.4064	0.2138	14.9	36.91	4.31	13.3	4.35	19.36
R	0.2576	0.2053	17.2	-	4.32	13.46	4.36	19.45
S	0.6092	0.286	19.5	32.01	3.98	13.6	4.02	16.52
T	0.8076	0.247	21.7	26.86	3.72	12.87	3.75	14.38
U	0.7885	0.2487	19.8	25.11	3.8	12.32	3.82	14.97
V	0.8533	0.1882	20.6	24.14	3.51	11.46	3.53	12.75

TABLE 2: RESULTS OF TEST GROUP 1, SET 2 (PILE 2)

TEST	DEFLECTION AT RELEASE (inches)		LOAD (kips)	STATIC STIFFNESS (kips/inch)	DAMPED FREQUENCY (cyc/sec)	DAMPING RATIO (%)	UNDAMPED FREQUENCY (cyc/sec)	DYNAMIC STIFFNESS (kips/inch)
	TOP	BOTTOM						
B	0.0553	0.0434	2.17	39.2	6.17	5.07	6.18	39.1
D	0.0485	0.0378	2.75	-	6.08	7.24	6.1	38
E	0.0606	0.0523	3.33	-	6.15	5.29	6.16	38.9
F	0.0684	0.0562	3.67	-	6.26	5.67	6.17	38.9
G	0.0761	0.0581	2.66	35	6.08	5.77	6.09	37.9
H	0.0789	0.058	2.9	36.8	6.08	5.71	6.09	38
I	0.0683	0.0535	2.61	38.1	6.06	5.74	6.07	37.7
J	0.131	0.101	4.39	33.7	5.85	6.25	5.87	35.2
K	0.137	0.106	4.49	32.9	5.71	6.95	5.72	33.5
M	0.128	0.106	5.6	-	5.7	6.63	5.71	33.4
N	0.126	0.105	5.31	-	5.61	6.94	5.62	32.4
O	0.122	0.102	5.07	-	5.61	6.81	5.62	32.4
P	0.114	0.0956	4.93	-	5.55	6.44	5.56	31.7
Q	0.126	0.105	5.36	-	5.49	6.76	5.51	31
R	0.126	0.105	5.36	-	5.46	6.66	5.47	31.6

TABLE 3: RESULTS OF TEST GROUP 2, SET 1 (PILE 2)

TEST	DEFLECTION AT RELEASE (inches)		LOAD (kips)	STATIC STIFFNESS (kips/inch)	DAMPED FREQUENCY (cyc/sec)	DAMPING RATIO (%)	UNDAMPED FREQUENCY (cyc/sec)	DYNAMIC STIFFNESS (kips/inch)
	TOP	BOTTOM						
B	0.06264	0.0479	2.29	36.6	6.36	6.53	6.37	41.6
D	0.06076	0.04599	2.46	40.5	6.35	6.97	6.36	44.1
E	0.06085	0.04734	2.36	38.8	6.3	6.99	6.31	40.8
F	0.12312	0.09687	4.56	37	6	6.94	6.01	37
G	0.12309	0.09648	4.48	36.4	5.88	7.02	5.9	35.6
H	0.12138	0.09618	4.41	36.3	5.83	6.89	5.85	35

TABLE 4: RESULTS OF TEST GROUP 2, SET 2 (PILE 2)

TEST	DEFLECTION AT RELEASE (inches)		LOAD (kips)	STATIC STIFFNESS (kips/inch)	DAMPED FREQUENCY (cyc/sec)	DAMPING RATIO (%)	UNDAMPED FREQUENCY (cyc/sec)	DYNAMIC STIFFNESS (kips/inch)
	TOP	BOTTOM						
B	0.0346	0.05025	1.95	56.4	6.6	5.35	6.61	44.7
D	0.03475	0.02451	1.73	49.8	6.52	3.38	6.53	43.6
E	0.03319	0.02337	1.57	47.3	6.42	6.02	6.43	42.4
F	0.0328	0.02337	1.69	51.5	6.46	5.93	6.4	42.9
G	0.06755	0.04854	2.46	36.4	6.23	6.57	6.24	39.9
H	0.06494	0.04698	2.27	35	6.17	6.41	6.18	39.1
I	0.06884	0.04971	2.47	35.9	6.11	6.44	6.13	38.4
J	0.09849	0.0714	3.38	34.3	5.97	6.84	5.98	36.6
K	0.09744	0.07107	3.29	33.8	5.94	6.75	5.95	36.3
L	0.09705	0.07125	3.24	33.4	5.89	7.16	5.9	35.7
M	0.09636	0.07035	3.23	33.5	5.86	6.87	5.88	35.3
N	0.13066	0.09621	4.43	33.9	5.78	7.49	5.8	34.4
P	0.12871	0.09453	4.31	33.5	5.75	7.26	5.77	34.1
Q	0.12688	0.09363	4.18	32.9	5.71	7.22	5.72	33.5
R	0.09239	0.06842	2.96	32	5.77	7.22	5.78	34.2
S	0.05911	0.04364	1.96	33.2	5.84	6.49	5.86	35.1
T	0.02622	0.01898	1.15	43.9	6.07	6.16	6.08	37.9
V	0.0127	0.09351	4.22	33.2	5.74	7.21	5.76	33.9

TABLE 5: PROPERTIES OF SOIL USED IN STIFFNESS CALCULATIONS

SAMPLE 101507	
LOCATION: MEMPHIS	
PROPERTY	MAGNITUDE
POISSON'S RATIO	0.4
SHEAR MODULUS (psi)	14650

SOURCE : Chang, Tzyy-Shiou; Teh, Lin Kee; Yigong, Zhang. "Seismic Characteristics of Sediments in the New Madrid Seismic Zone," Memphis State University. Memphis, Tennessee. 1992.

TABLE 6: NONCONNAH PARKWAY BRIDGE BENT FOUNDATION DATA

LOCATION	PILE CAP DIMENSIONS	NUMBER OF PILES	PILE SPACING
BENT 1	36' X 36'	156	2' - 9"
BENT 2	33' X 33'	132	3'
BENT 3	33' X 33'	121	3'
BENT 4	33' X 33'	121	3'
BENT 5	33' X 33'	121	3'
BENT 6	33' X 33'	132	3'

TABLE 7: NONCONNAH PARKWAY BRIDGE ABUTMENT STIFFNESSES

ABUMENT 1	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	462,189
TRANSVERSE Kzz (KIPS/FT)	166,833

ABUMENT 2	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	372,093
TRANSVERSE Kzz (KIPS/FT)	63,638

**TABLE 8: NONCONNAH PARKWAY BRIDGE LATERAL BENT
STIFFNESSES**

LOCATION	PILE GROUP STIFFNESS (KIPS/FT)	PILE CAP STIFFNESS (KIPS/FT)	TOTAL STIFFNESS (KIPS/FT)
BENT 1	19,881	127,279	147,160
BENT 2	16,822	125,453	142,275
BENT 3	15,420	120,247	135,667
BENT 4	15,420	120,247	135,667
BENT 5	15,420	120,247	135,667
BENT 6	16,822	125,453	142,275

TABLE 9: MADISON COUNTY BRIDGE ABUTMENT STIFFNESSES

ABUMENT 1	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	71,240
TRANSVERSE Kzz (KIPS/FT)	6,240

ABUMENT 2	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	71,240
TRANSVERSE Kzz (KIPS/FT)	6,240

**TABLE 10: MADISON COUNTY BRIDGE LATERAL BENT
STIFFNESSES**

LOCATION	PILE GROUP STIFFNESS (KIPS/FT)	PILE CAP STIFFNESS (KIPS/FT)	TOTAL STIFFNESS (KIPS/FT)
BENT 1	44,134	1,530	45,664

TABLE 11: LOOSAHATCHIE BRIDGE ABUTMENT STIFFNESSES

ABUMENT 1	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	149,430
TRANSVERSE Kzz (KIPS/FT)	89,824

ABUMENT 2	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	145,280
TRANSVERSE Kzz (KIPS/FT)	89,792

TABLE 12: LOOSAHATCHIE BRIDGE BENT STIFFNESSES

LOCATION	INDIVIDUAL PILE TRANSLATION STIFFNESS (KIPS/FT)	INDIVIDUAL PILE ROTATION STIFFNESS (KIP*FT/RAD)
ALL BENTS	304	93,843

TABLE 13: HAYWOOD COUNTY BRIDGE ABUTMENT STIFFNESSES

ABUMENT 1	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	34,560
TRANSVERSE Kzz (KIPS/FT)	7,650

ABUMENT 2	
DIRECTION	TOTAL ABUTMENT STIFFNESS
LONGITUDINAL Kxx (KIPS/FT)	34,560
TRANSVERSE Kzz (KIPS/FT)	7,650

TABLE 14: HAYWOOD COUNTY BRIDGE BENT STIFFNESSES

LOCATION	INDIVIDUAL PILE TRANSLATION STIFFNESS (KIPS/FT)	INDIVIDUAL PILE ROTATION STIFFNESS (KIP*FT/RAD)
ALL BENTS	162	34,342

**TABLE 15: COMPARISON OF NATURAL PERIODS FOR THE NONCONNAH
PARKWAY BRIDGE**

RESPONSE DIRECTION	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
MODE 1 (LONGITUDINAL) PERIOD (sec.)	1.26	1.277	1.33
MODE 2 (TRANSVERSE) PERIOD (sec.)	0.858	0.875	1.94

TABLE 16: MAXIMUM LONGTUDINAL DISPLACEMENTS FOR THE
NONCONNAH PARKWAY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL DISPLACEMENT (FEET)	LONGITUDINAL DISPLACEMENT (FEET)	
ABUTMENT 1	0.412	0.42	1.90
ABUTMENT 2	0.387	0.395	2.03
BENT 1	0.407	0.415	1.93
BENT 2	0.411	0.418	1.67
BENT 3	0.414	0.421	1.66
BENT 4	0.41	0.418	1.91
BENT 5	0.399	0.407	1.97
BENT 6	0.383	0.392	2.30

TABLE 17: MAXIMUM TRANSVERSE DISPLACEMENTS FOR THE
NONCONNAH PARKWAY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE DISPLACEMENT (FEET)	TRANSVERSE DISPLACEMENT (FEET)	
ABUTMENT 1	0.003	0.003	0.00
ABUTMENT 2	0.005	0.005	0.00
BENT 1	0.083	0.091	8.79
BENT 2	0.133	0.14	5.00
BENT 3	0.274	0.284	3.52
BENT 4	0.322	0.332	3.01
BENT 5	0.189	0.199	5.03
BENT 6	0.047	0.057	17.54

TABLE 18: MAXIMUM LONGITUDINAL COLUMN MOMENTS FOR THE
NONCONNAH PARKWAY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL MOMENT (KIP-FEET)	LONGITUDINAL MOMENT (KIP-FEET)	
BENT 1	80,106	79,084	1.28
BENT 2	55,856	55,827	0.05
BENT 3	37,894	38,135	-0.63
BENT 4	36,245	36,544	-0.82
BENT 5	42,563	42,826	-0.61
BENT 6	94,837	92,695	2.26

TABLE 19: MAXIMUM TRANSVERSE COLUMN MOMENTS FOR THE
NONCONNAH PARKWAY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE MOMENT (KIP-FEET)	TRANSVERSE MOMENT (KIP-FEET)	
BENT 1	29,718	29,292	1.43
BENT 2	32,568	32,297	0.83
BENT 3	45,613	45,609	0.01
BENT 4	52,242	52,011	0.44
BENT 5	36,443	36,626	-0.50
BENT 6	20,914	21,807	-4.27

TABLE 20: MAXIMUM AXIAL COLUMN FORCES FOR THE
NONCONNAH PARKWAY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	AXIAL FORCE (KIPS)	AXIAL FORCE (KIPS)	
BENT 1	787	756	3.89
BENT 2	850	842	0.94
BENT 3	1,197	1,196	0.11
BENT 4	1,371	1,364	0.53
BENT 5	954	958	-0.40
BENT 6	538	560	-4.07

TABLE 21: COMPARISON OF NATURAL PERIOD FOR MADISON COUNTY BRIDGE

RESPONSE DIRECTION	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
MODE 1 (LONGITUDINAL) PERIOD (sec.)	0.18	0.18	0.00
MODE 2 (TRANSVERSE) PERIOD (sec.)	0.476	0.482	1.24

TABLE 22: MAXIMUM LONGITUDINAL DISPLACEMENTS FOR THE
MADISON COUNTY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL DISPLACEMENT (INCHES)	LONGITUDINAL DISPLACEMENT (INCHES)	
ABUTMENT 1	0.007	0.007	0.00
ABUTMENT 2	0.007	0.007	0.00
BENT 1	0.009	0.009	0.00

**TABLE 23: MAXIMUM TRANSVERSE DISPLACEMENTS FOR THE
MADISON COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE DISPLACEMENT (FEET)	TRANSVERSE DISPLACEMENT (FEET)	
ABUTMENT 1	0.043	0.044	2.27
ABUTMENT 2	0.043	0.044	2.27
BENT 1	0.055	0.056	1.79

**TABLE 24: MAXIMUM LONGITUDINAL COLUMN MOMENTS FOR THE
MADISON COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL MOMENT (KIP-FEET)	LONGITUDINAL MOMENT (KIP-FEET)	
BENT 1	127	126	0.79

**TABLE 25: MAXIMUM TRANSVERSE COLUMN MOMENTS FOR THE
MADISON COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE MOMENT (KIP-FEET)	TRANSVERSE MOMENT (KIP-FEET)	
BENT 1	1,685	1,630	3.26

TABLE 26: MAXIMUM AXIAL COLUMN FORCES FOR THE
MADISON COUNTY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	AXIAL FORCE (KIPS)	AXIAL FORCE (KIPS)	
BENT 1	163.5	157.5	3.67

TABLE 27: COMPARISON OF NATURAL PERIOD FOR THE LOOSAHATCHIE
BRIDGE

RESPONSE DIRECTION	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
MODE 1 (LONGITUDINAL) PERIOD (sec.)	3.758	4.376	14.12
MODE 2 (TRANSVERSE) PERIOD (sec.)	1.801	2.386	24.52

**TABLE 28: MAXIMUM LONGITUDINAL DISPLACEMENTS FOR THE
LOOSAHATCHIE BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL DISPLACEMENT (FEET)	LONGITUDINAL DISPLACEMENT (FEET)	
ABUTMENT 1	1.412	1.623	13.00
ABUTMENT 2	1.412	1.623	13.00
BENT 1-14	1.412	1.623	13.00

**TABLE 29: MAXIMUM TRANSVERSE DISPLACEMENTS FOR THE
LOOSAHATCHIE BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE DISPLACEMENT (FEET)	TRANSVERSE DISPLACEMENT (FEET)	
ABUTMENT 1	0	0	0.00
ABUTMENT 2	0	0	0.00
BENT 1	0.141	0.207	31.88
BENT 2	0.275	0.4	31.25
BENT 3	0.396	0.576	31.25
BENT 4	0.501	0.727	31.09
BENT 5	0.584	0.847	31.05
BENT 6	0.641	0.93	31.08
BENT 7	0.671	0.972	30.97
BENT 8	0.671	0.972	30.97
BENT 9	0.642	0.93	30.97
BENT 10	0.585	0.848	31.01
BENT 11	0.503	0.729	31.00
BENT 12	0.399	0.579	31.09
BENT 13	0.278	0.403	31.02
BENT 14	0.144	0.209	31.10

TABLE 30: MAXIMUM LONGITUDINAL COLUMN MOMENTS FOR THE
LOOSAHATCHIE BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL MOMENT (KIP-FEET)	LONGITUDINAL MOMENT (KIP-FEET)	
BENT 1	846	720	14.89
BENT 2	845	719	14.91
BENT 3	843	718	14.83
BENT 4	842	717	14.85
BENT 5	839	714	14.90
BENT 6	837	712	14.93
BENT 7	834	710	14.87
BENT 8	834	710	14.87
BENT 9	837	712	14.93
BENT 10	839	714	14.90
BENT 11	842	716	14.96
BENT 12	843	718	14.83
BENT 13	845	719	14.91
BENT 14	846	720	14.89

TABLE 31: MAXIMUM TRANSVERSE COLUMN MOMENTS FOR THE
LOOSAHATCHIE BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE MOMENT (KIP-FEET)	TRANSVERSE MOMENT (KIP-FEET)	
BENT 1	167	139	16.77
BENT 2	326	275	15.64
BENT 3	470	396	15.74
BENT 4	595	500	15.97
BENT 5	693	582	16.02
BENT 6	761	639	16.03
BENT 7	796	668	16.08
BENT 8	796	668	16.08
BENT 9	762	639	16.14
BENT 10	695	583	16.12
BENT 11	597	501	16.08
BENT 12	474	398	16.03
BENT 13	330	277	16.06
BENT 14	171	143	16.37

TABLE 32: MAXIMUM AXIAL COLUMN FORCES FOR THE
LOOSAHATCHIE BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	AXIAL FORCE (KIPS)	AXIAL FORCE (KIPS)	
BENT 1	29.5	14.9	49.49
BENT 2	58.1	26.9	53.70
BENT 3	83.9	38.7	53.87
BENT 4	106.1	48.9	53.91
BENT 5	123.6	56.9	53.96
BENT 6	135.7	62.5	53.94
BENT 7	142	65.3	54.01
BENT 8	142.1	65.3	54.05
BENT 9	136	62.5	54.04
BENT 10	124	57	54.03
BENT 11	106.6	49	54.03
BENT 12	84.5	38.9	53.96
BENT 13	58.8	27.1	53.91
BENT 14	30.4	14.4	52.63

TABLE 33: COMPARISON OF NATURAL PERIOD FOR HAYWOOD COUNTY BRIDGE

RESPONSE DIRECTION	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
MODE 1 (LONGITUDINAL) PERIOD (sec.)	0.118	0.119	0.84
MODE 2 (TRANSVERSE) PERIOD (sec.)	0.237	0.249	4.82

**TABLE 34: MAXIMUM LONGITUDINAL DISPLACEMENTS FOR THE
HAYWOOD COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL DISPLACEMENT (FEET)	LONGITUDINAL DISPLACEMENT (FEET)	
ABUTMENT 1	0.006	0.005	16.67
ABUTMENT 2	0.006	0.005	16.67
BENT 1	0.006	0.006	0.00
BENT 2	0.006	0.006	0.00

**TABLE 35: MAXIMUM TRANSVERSE DISPLACEMENTS FOR THE
HAYWOOD COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE DISPLACEMENT (FEET)	TRANSVERSE DISPLACEMENT (FEET)	
ABUTMENT 1	0.021	0.023	8.70
ABUTMENT 2	0.021	0.023	8.70
BENT 1	0.024	0.026	7.69
BENT 2	0.024	0.026	7.69

TABLE 36: MAXIMUM LONGITUDINAL COLUMN MOMENTS FOR THE
HAYWOOD COUNTY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATION STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	LONGITUDINAL MOMENT (KIP-FEET)	LONGITUDINAL MOMENT (KIP-FEET)	
BENT 1	7	1	85.71
BENT 2	7	1	85.71

**TABLE 37: MAXIMUM TRANSVERSE COLUMN MOMENTS FOR THE
HAYWOOD COUNTY BRIDGE**

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	TRANSVERSE MOMENT (KIP-FEET)	TRANSVERSE MOMENT (KIP-FEET)	
BENT 1	45	24	46.67
BENT 2	45	24	46.67

TABLE 38: MAXIMUM AXIAL COLUMN FORCES FOR THE
HAYWOOD COUNTY BRIDGE

BRIDGE COMPONENT	BENT FOUNDATON STATE		PERCENT DIFFERENCE
	FIXED	SPRINGS	
	AXIAL FORCE (KIPS)	AXIAL FORCE (KIPS)	
BENT 1	8.2	2.2	73.17
BENT 2	8.2	2.2	73.17

APPENDIX B:

FIGURES

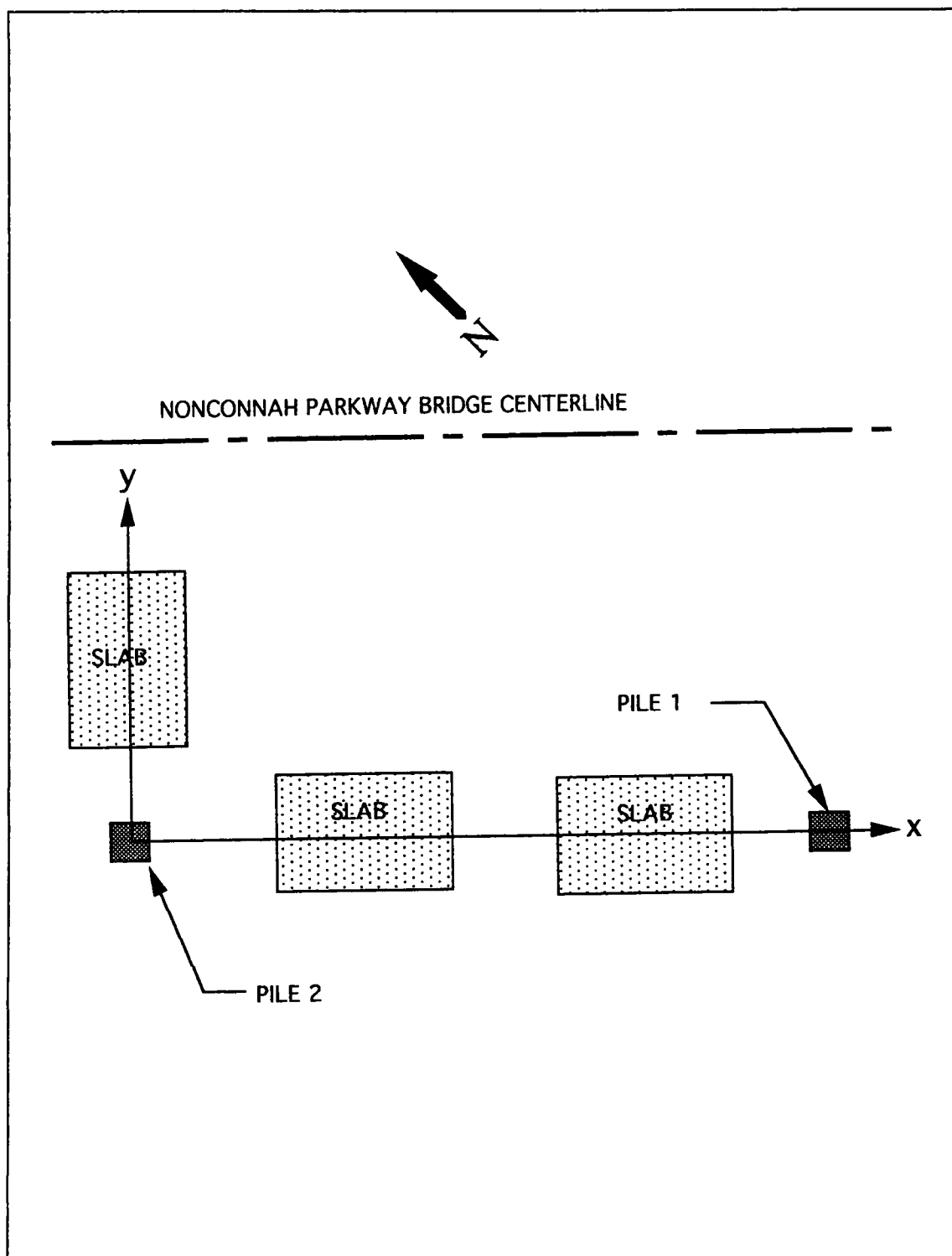


FIGURE 1: TESTING SITE ARRANGEMENT

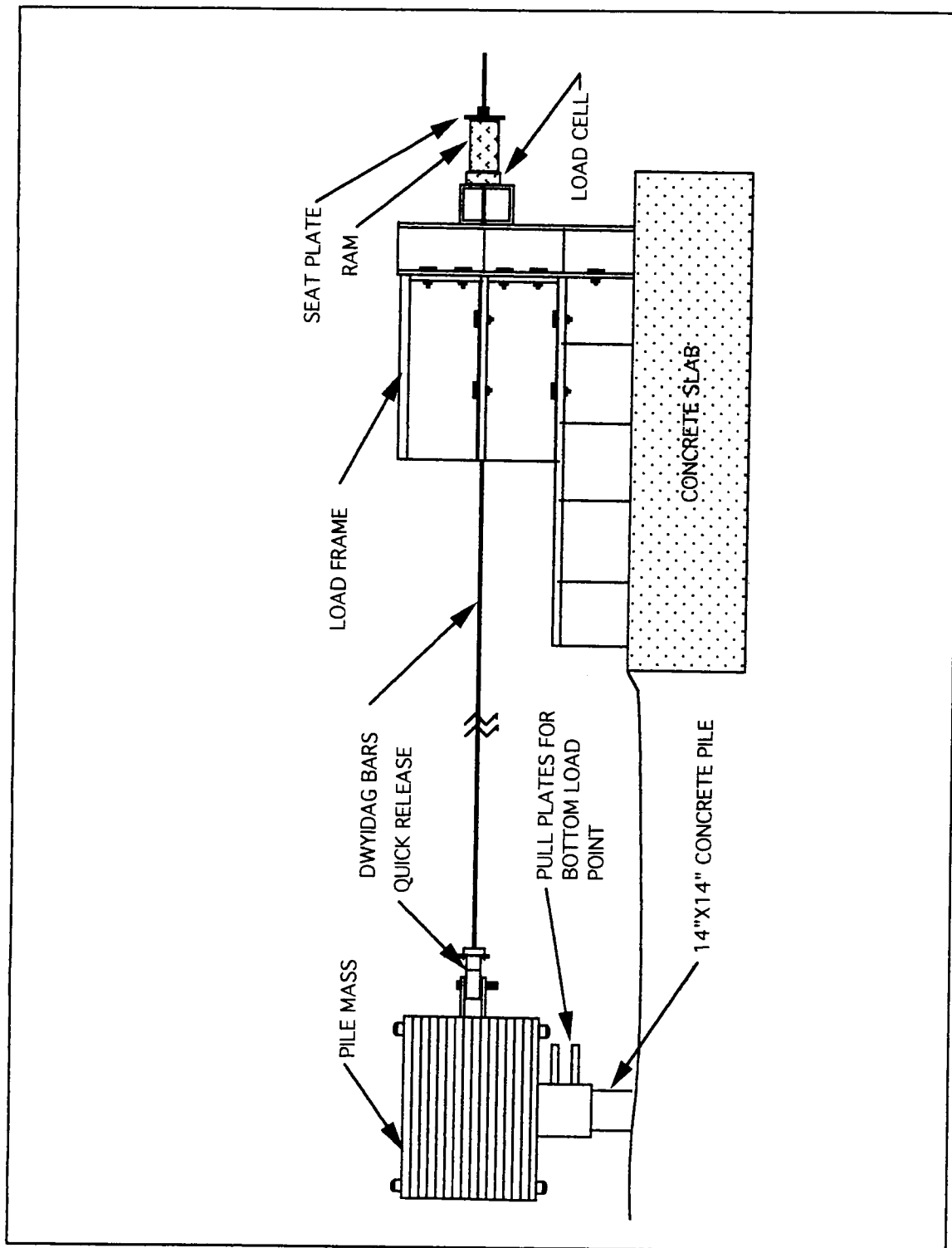


FIGURE 2: PLUCK TEST SET-UP

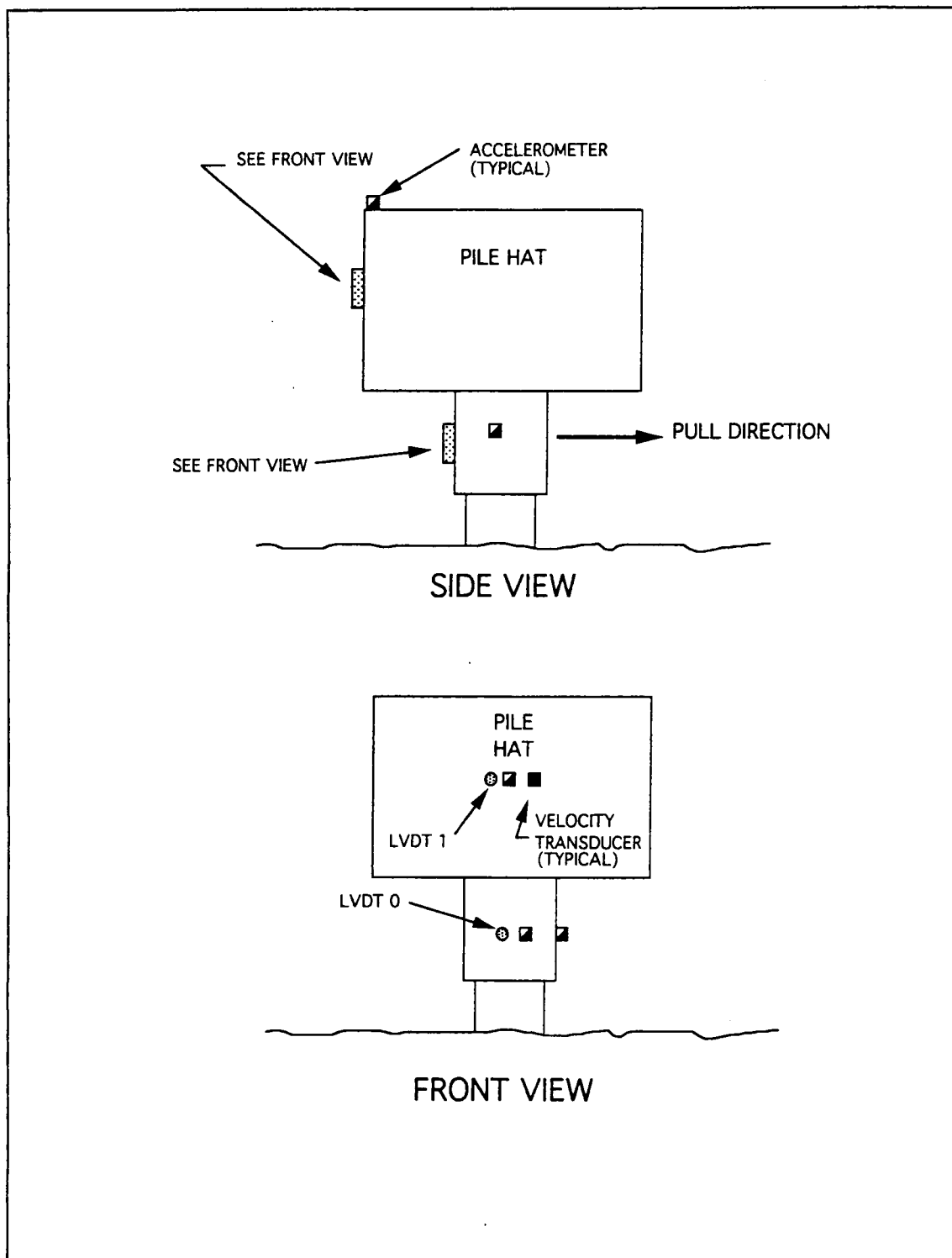


FIGURE 3: TYPICAL INSTRUMENTATION GROUP

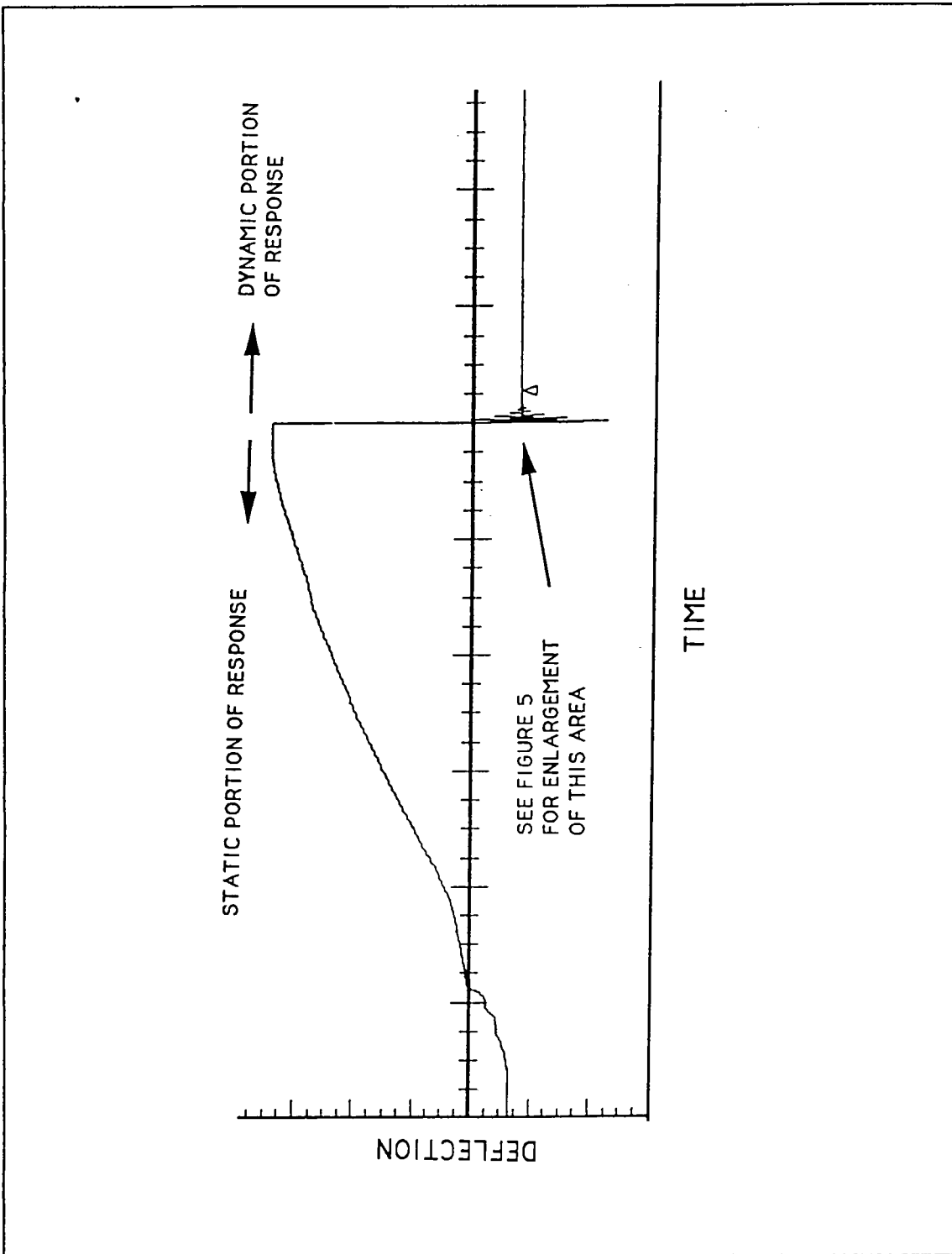


FIGURE 4: TYPICAL LVDT TRACE FOR THE PLUCK TEST

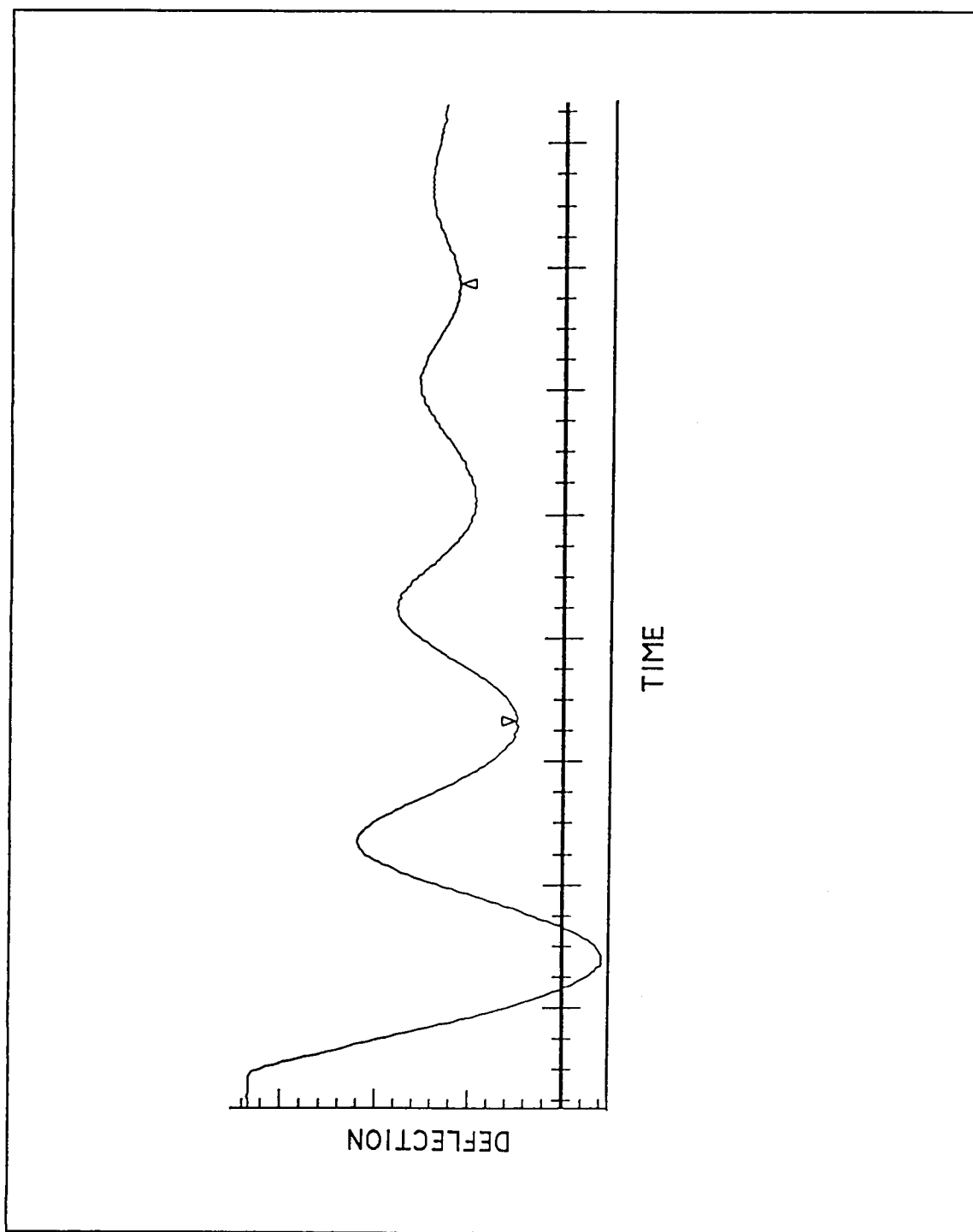


FIGURE 5: TYPICAL LVDT TRACE FOR THE DYNAMIC PORTION OF THE RESPONSE

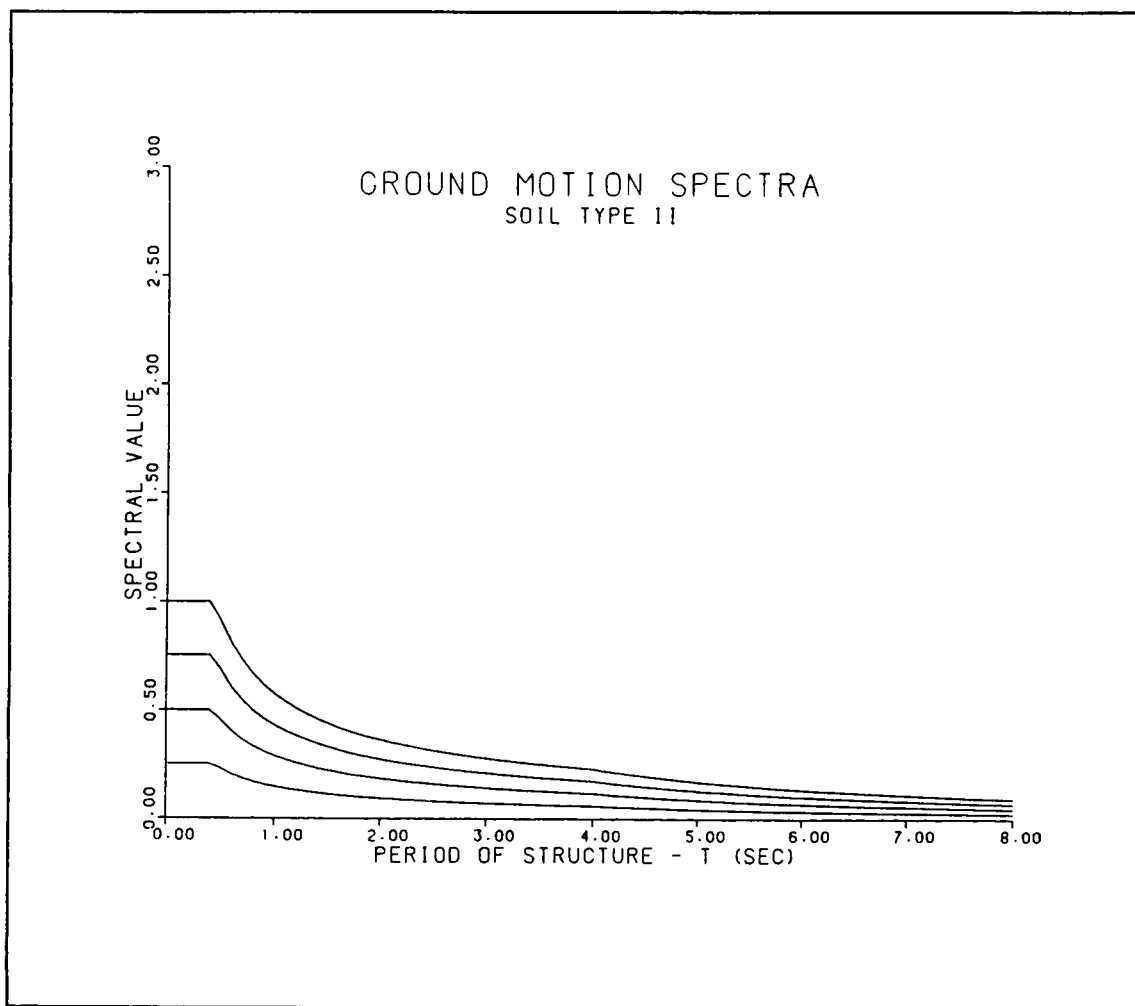


FIGURE 6: GROUND MOTION SPECTRA

SOURCE : Imbsen and Associates. *SEISmic Analysis of Bridges*
(SEISAB) Users Manual. 1993

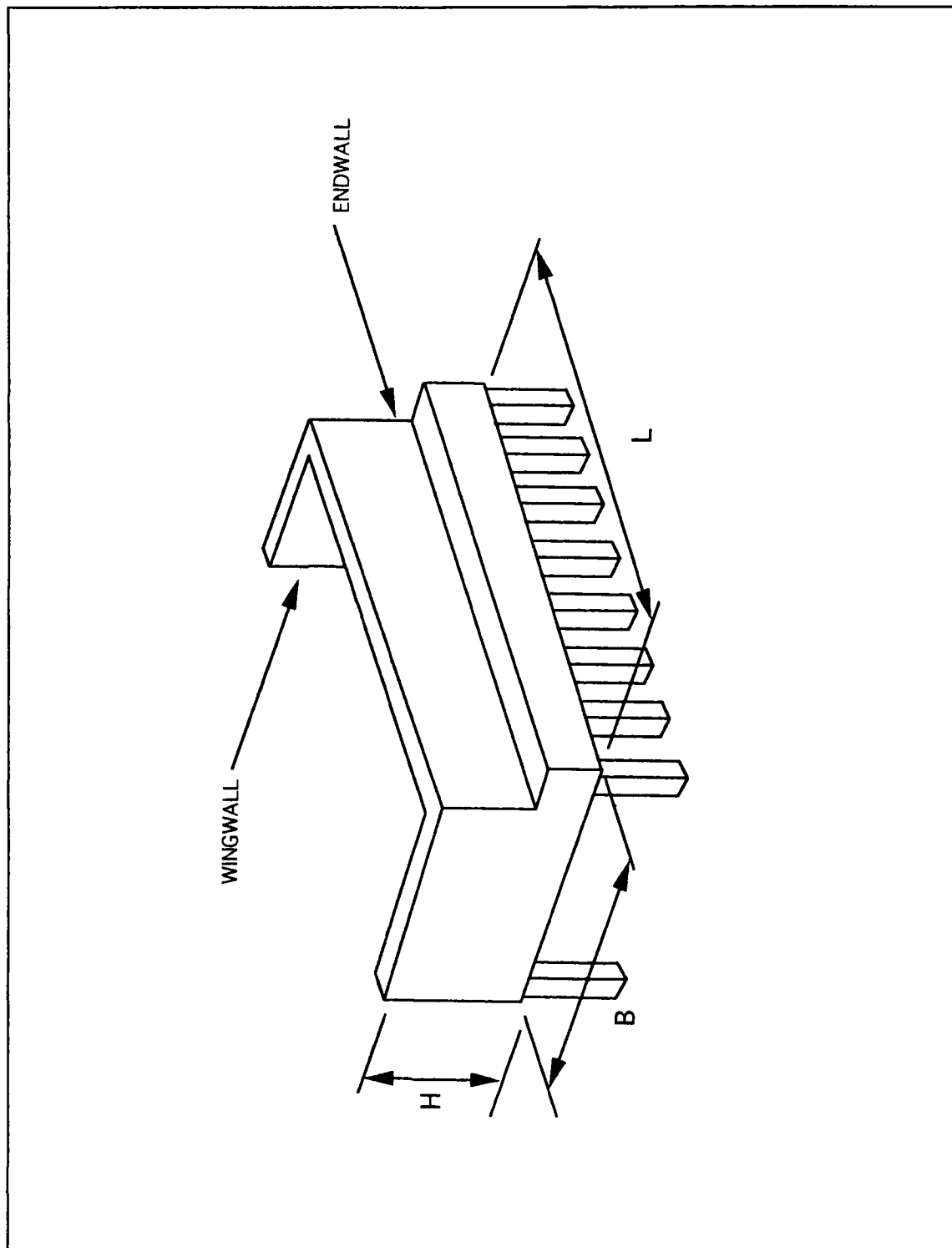
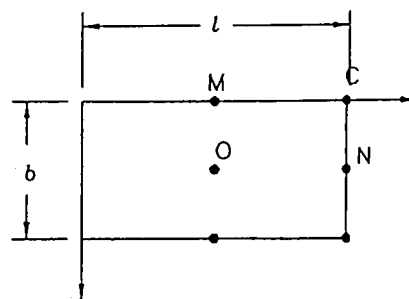


FIGURE 7: TYPICAL ABUTMENT DIAGRAM

$\alpha = \frac{l}{b}$	I_C	I_M	I_N	I_O	I_m	$\alpha = \frac{l}{b}$	I_C	I_M	I_N	I_O	I_m
1	0.561	0.766	0.766	1.122	0.946	15	1.401	2.362	1.621	2.802	2.498
1.1	0.588	0.810	0.795	1.176	0.992	20	1.493	2.544	1.713	2.985	2.677
1.2	0.613	0.852	0.822	1.226	1.035	25	1.564	2.686	1.784	3.127	2.817
1.3	0.636	0.892	0.847	1.273	1.075	30	1.622	2.802	1.842	3.243	2.932
1.4	0.658	0.930	0.870	1.317	1.112	40	1.713	2.985	1.934	3.426	3.113
1.5	0.679	0.966	0.892	1.358	1.148	50	1.784	3.127	2.005	3.568	3.254
1.6	0.698	1.000	0.912	1.396	1.181	60	1.842	3.243	2.063	3.684	3.370
1.7	0.716	1.033	0.931	1.433	1.213	70	1.891	3.341	2.112	3.783	3.467
1.8	0.734	1.064	0.949	1.467	1.244	80	1.934	3.426	2.154	3.868	3.552
1.9	0.750	1.094	0.966	1.500	1.273	90	1.971	3.501	2.192	3.943	3.627
2	0.766	1.122	0.982	1.532	1.300	100	2.005	3.568	2.225	4.010	3.693
2.2	0.795	1.176	1.012	1.590	1.353	200	2.225	4.010	2.446	4.451	4.134
2.4	0.822	1.226	1.039	1.644	1.401	300	2.355	4.268	2.575	4.709	4.391
2.5	0.835	1.250	1.052	1.669	1.424	400	2.446	4.451	2.667	4.892	4.574
3	0.892	1.358	1.110	1.783	1.527	500	2.517	4.593	2.738	5.034	4.717
3.5	0.940	1.450	1.159	1.880	1.616	600	2.575	4.709	2.796	5.150	4.833
4	0.982	1.532	1.201	1.964	1.694	700	2.624	4.807	2.845	5.248	4.931
4.5	1.019	1.604	1.239	2.038	1.763	800	2.667	4.892	2.887	5.333	5.015
5	1.052	1.669	1.272	2.105	1.826	900	2.704	4.967	2.925	5.408	5.092
6	1.110	1.783	1.330	2.220	1.935	10 ³	2.738	5.034	2.958	5.476	5.158
7	1.159	1.880	1.379	2.318	2.028	10 ⁴	3.471	6.500	3.691	6.941	6.623
8	1.201	1.964	1.422	2.403	2.110	10 ⁵	4.204	7.966	4.424	8.407	8.089
9	1.239	2.038	1.459	2.477	2.182	10 ⁶	4.937	9.432	5.157	9.874	9.555
10	1.272	2.105	1.493	2.544	2.246	∞	∞	∞	∞	∞	∞



b - length of the shorter side
M - center of the longer side
N - center of the shorter side
C - corner
O - center of the rectangle
m - mean value
l - length of the longer side

FIGURE 8: INFLUENCE FACTORS FOR SURFACE DISPLACEMENT

SOURCE: Poulos, H. G. and Davis, E. H. Elastic Solutions
For Soil and Rock Mechanics. John Wiley and
 Sons. 1974.

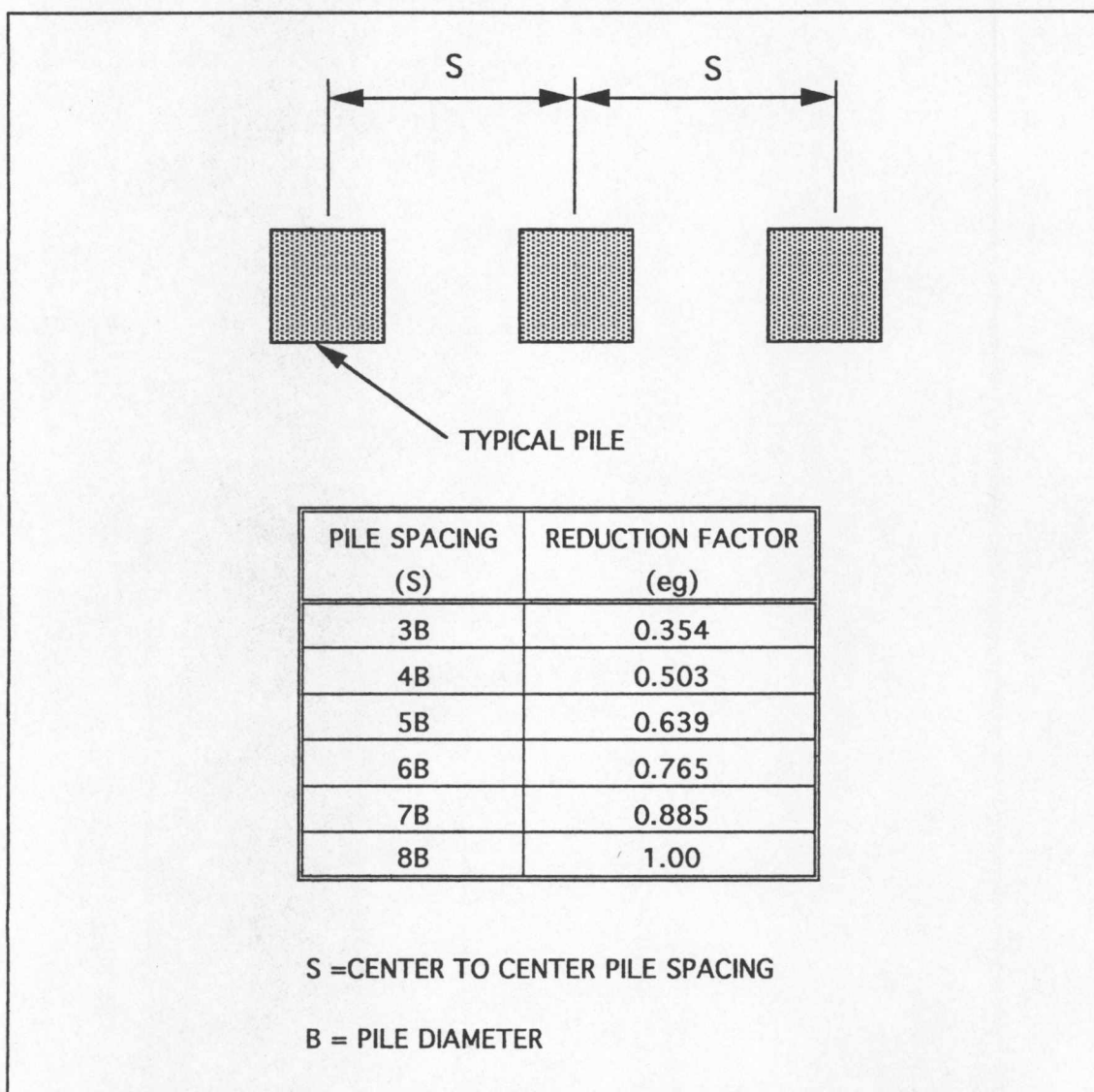


FIGURE 9: STIFFNESS REDUCTION FACTOR

SOURCE: Norris, G. M. and Sack, R. L., "Evaluation of the Lateral Stiffness of Pile Groups for Seismic Analysis of Highway Bridges." Proceedings, 22nd Engineering Geology and Soils Engineering Symposium. Boise, Idaho. Feb. 1986, pp. 323-363.

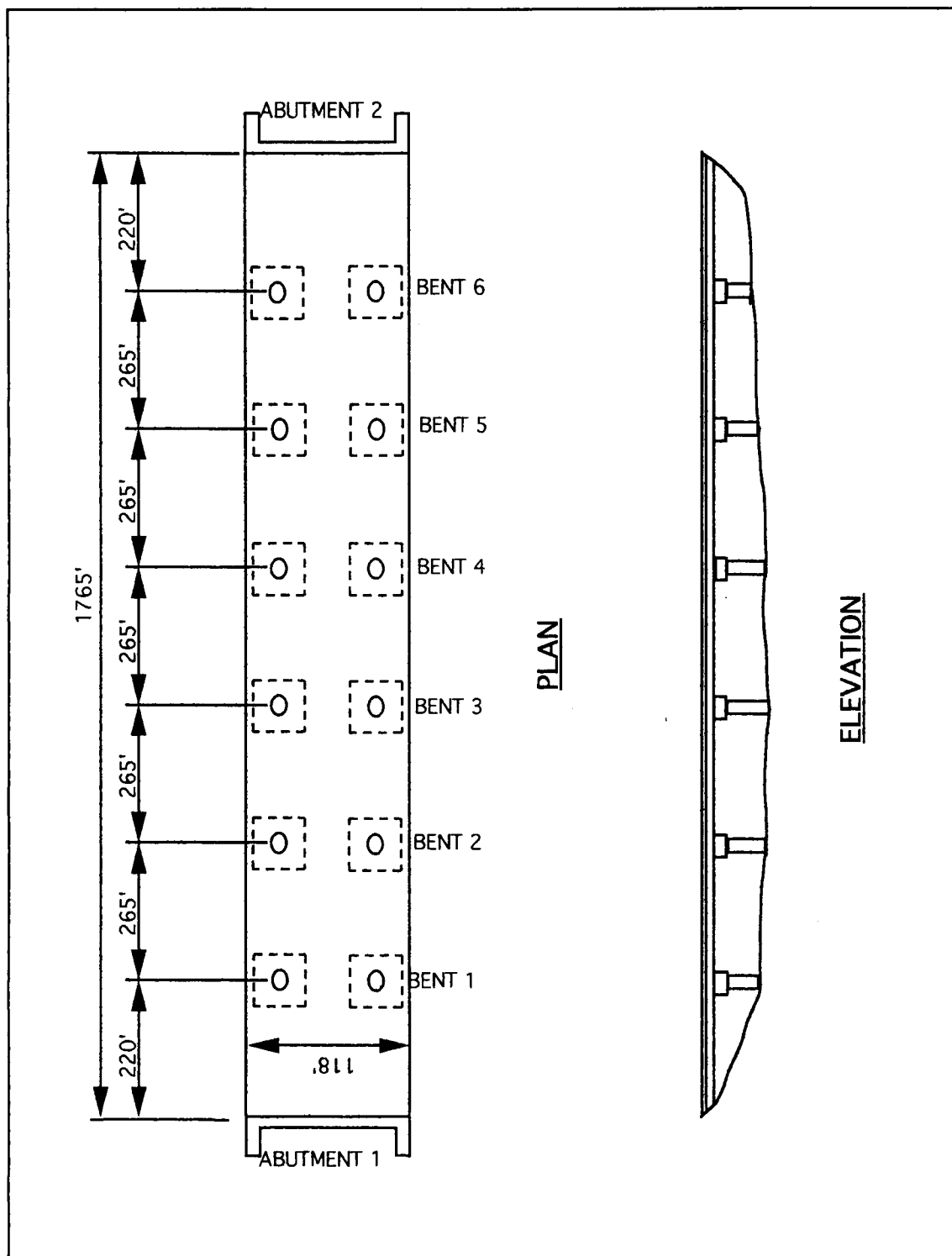


FIGURE 10: NONCONNAH PARKWAY BRIDGE PLAN AND ELEVATION

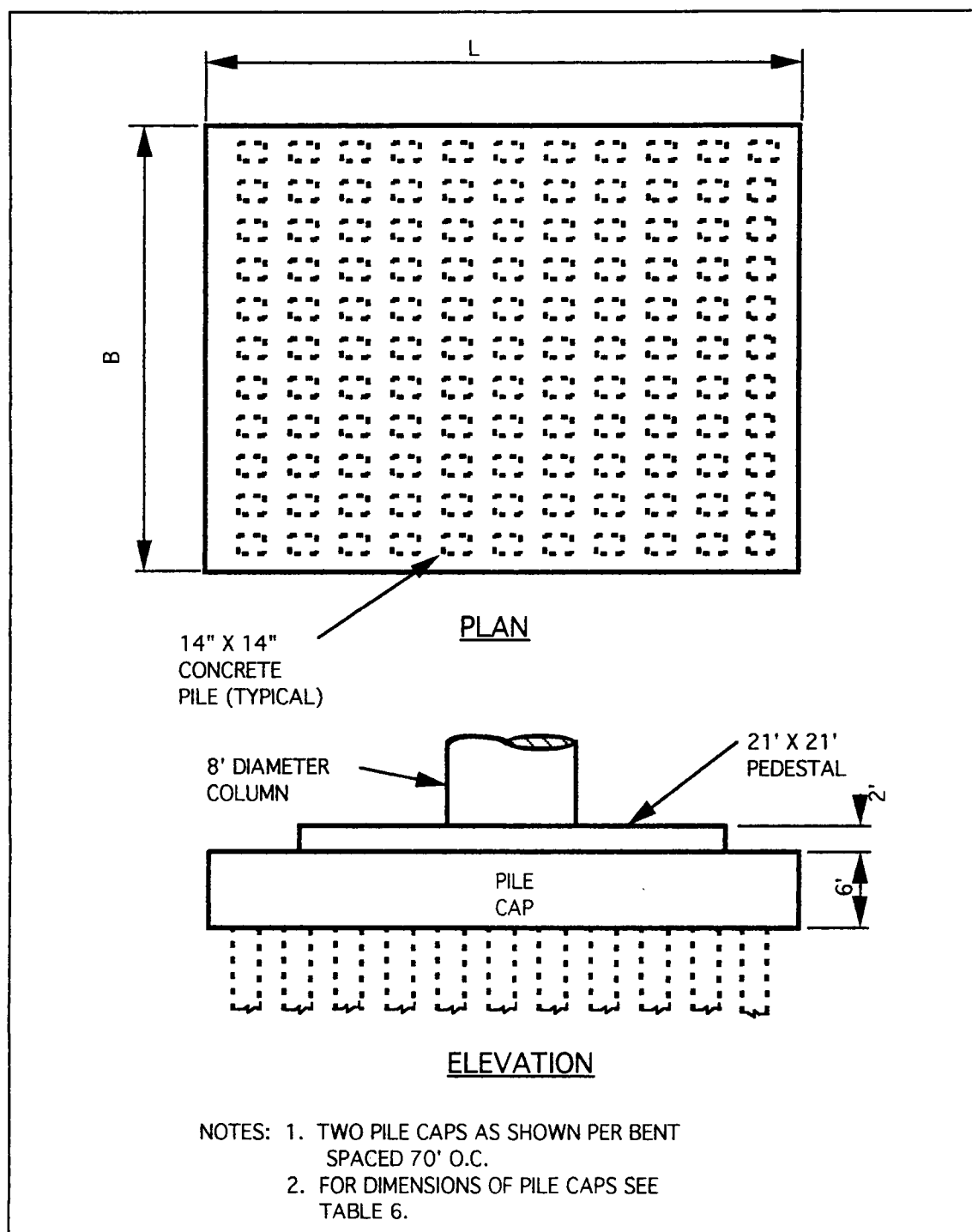
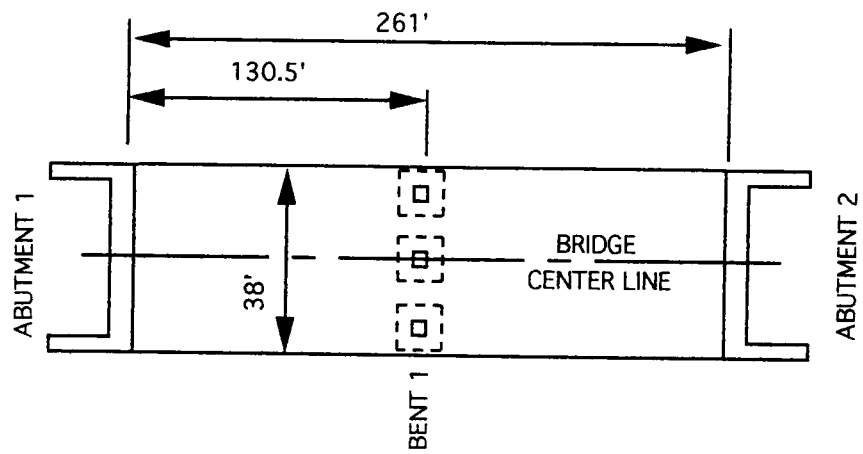
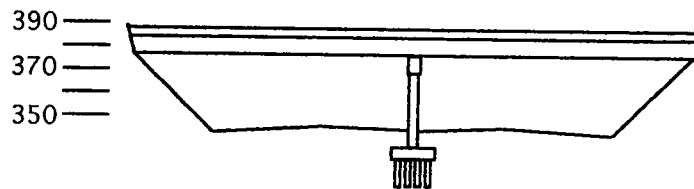


FIGURE 11: TYPICAL NONCONNAH PARKWAY BRIDGE PILE CAP AND FOOTING

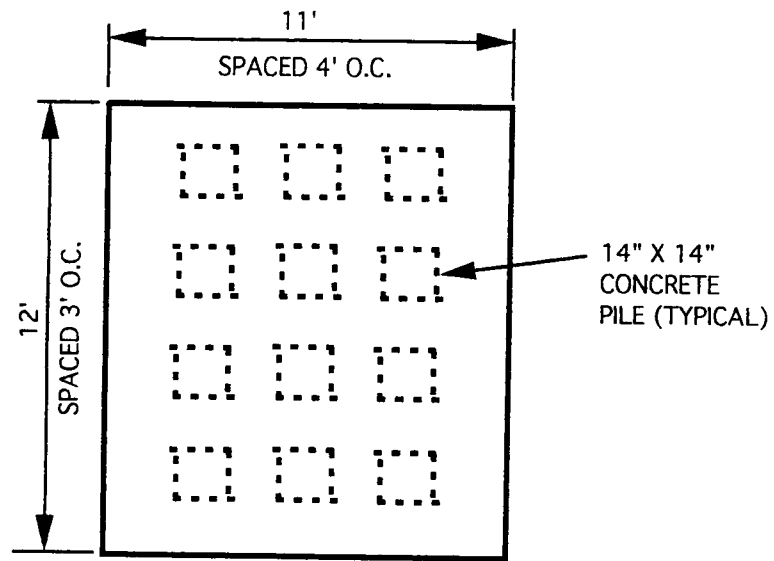


PLAN

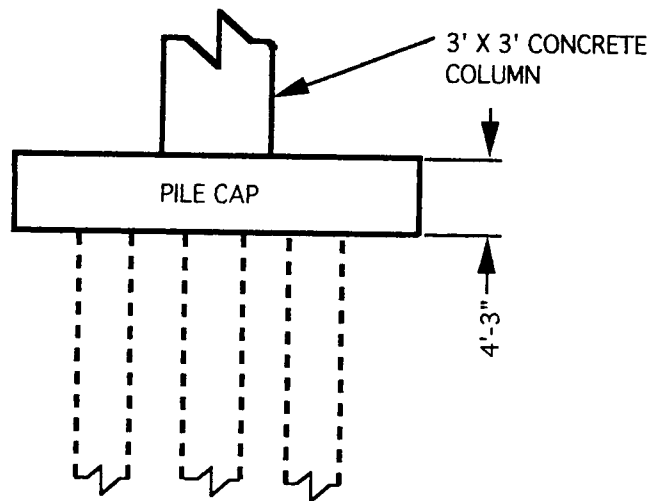


ELEVATION

FIGURE 12: MADISON COUNTY BRIDGE PLAN AND ELEVATION



PLAN



ELEVATION

NOTE: THREE PILE CAPS AS SHOWN PER BENT
SPACED 19' O.C.

FIGURE 13: MADISON COUNTY BRIDGE PILE CAP AND FOOTING PLAN

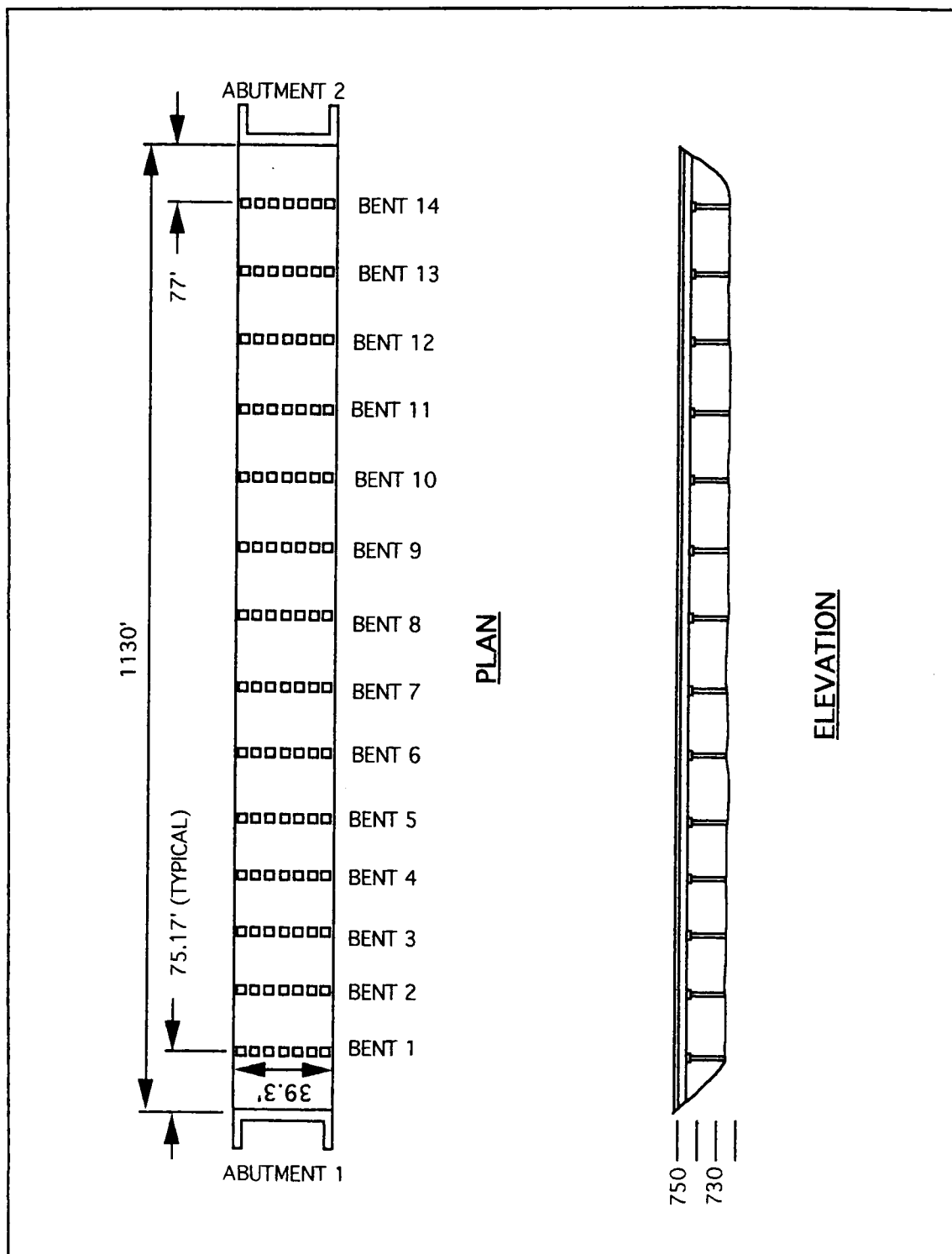
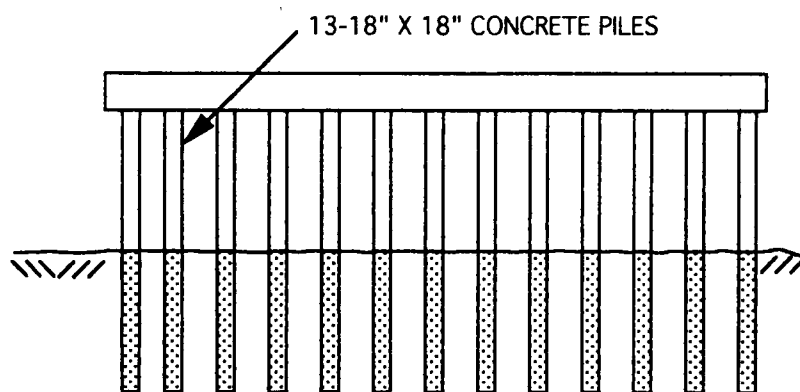
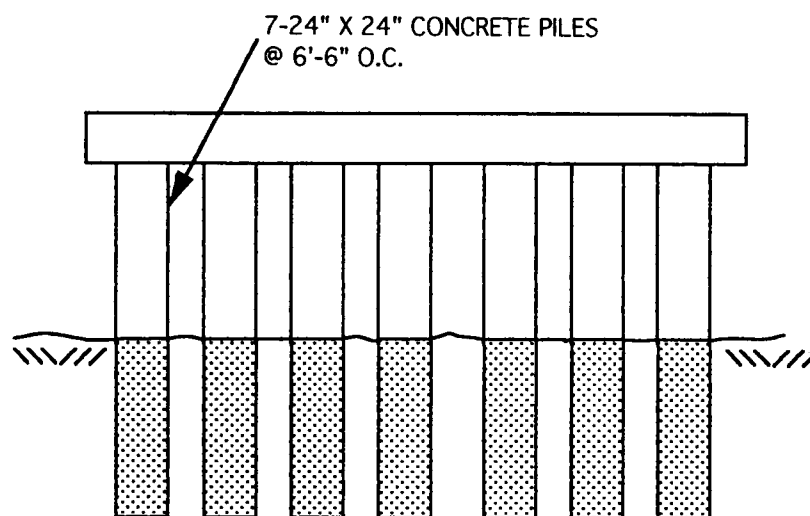


FIGURE 14: MODELED LOOSAHATCHIE BRIDGE PLAN AND ELEVATION



TYPICAL LOOSAHATCHIE BRIDGE BENT
AS CONSTRUCTED



TYPICAL LOOSAHATCHIE BRIDGE BENT
AS MODELED IN SEISAB

FIGURE 15: ACTUAL LOOSAHATCHIE BRIDGE BENT VS. MODELED

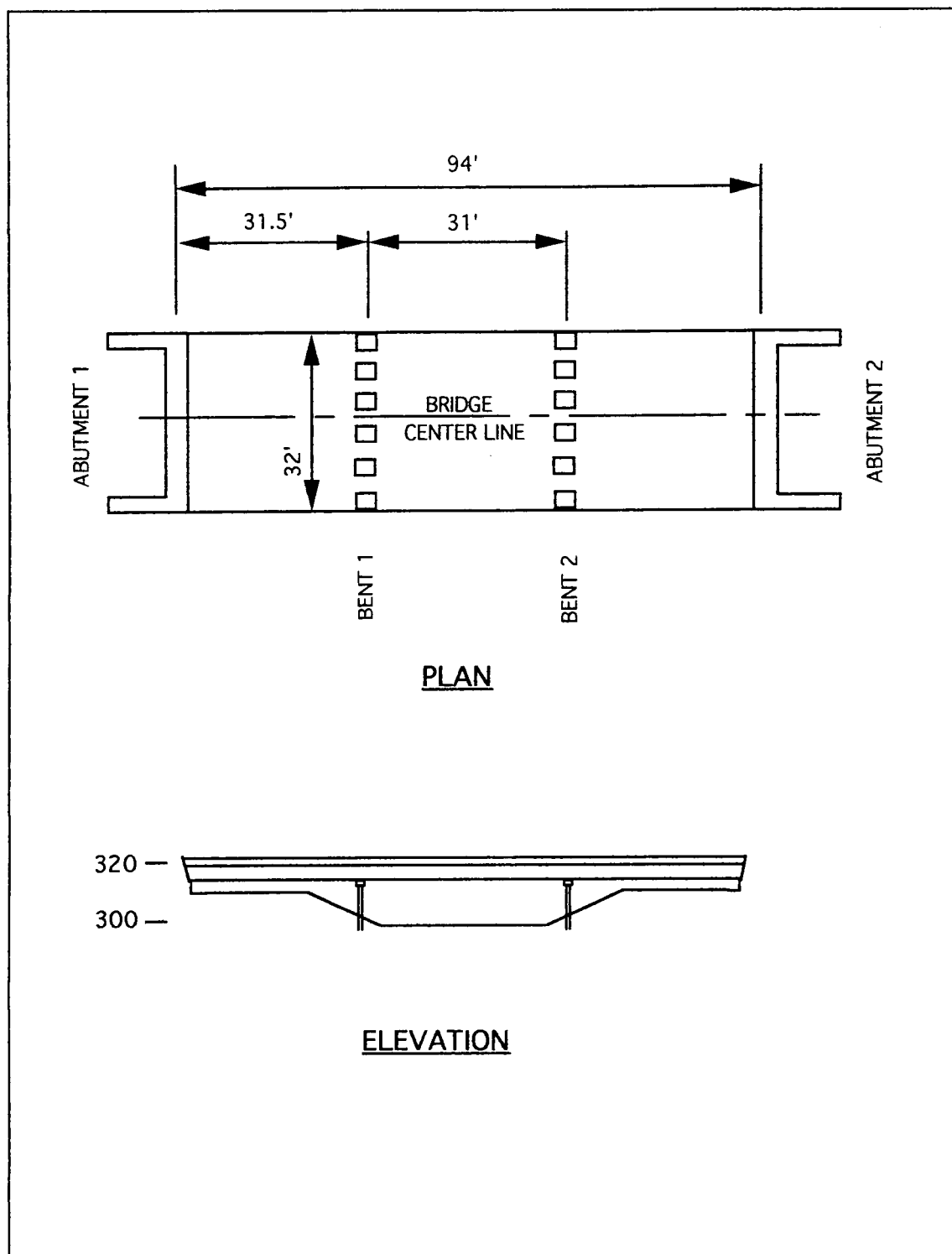


FIGURE 16 HAYWOOD COUNTY BRIDGE PLAN AND ELEVATION

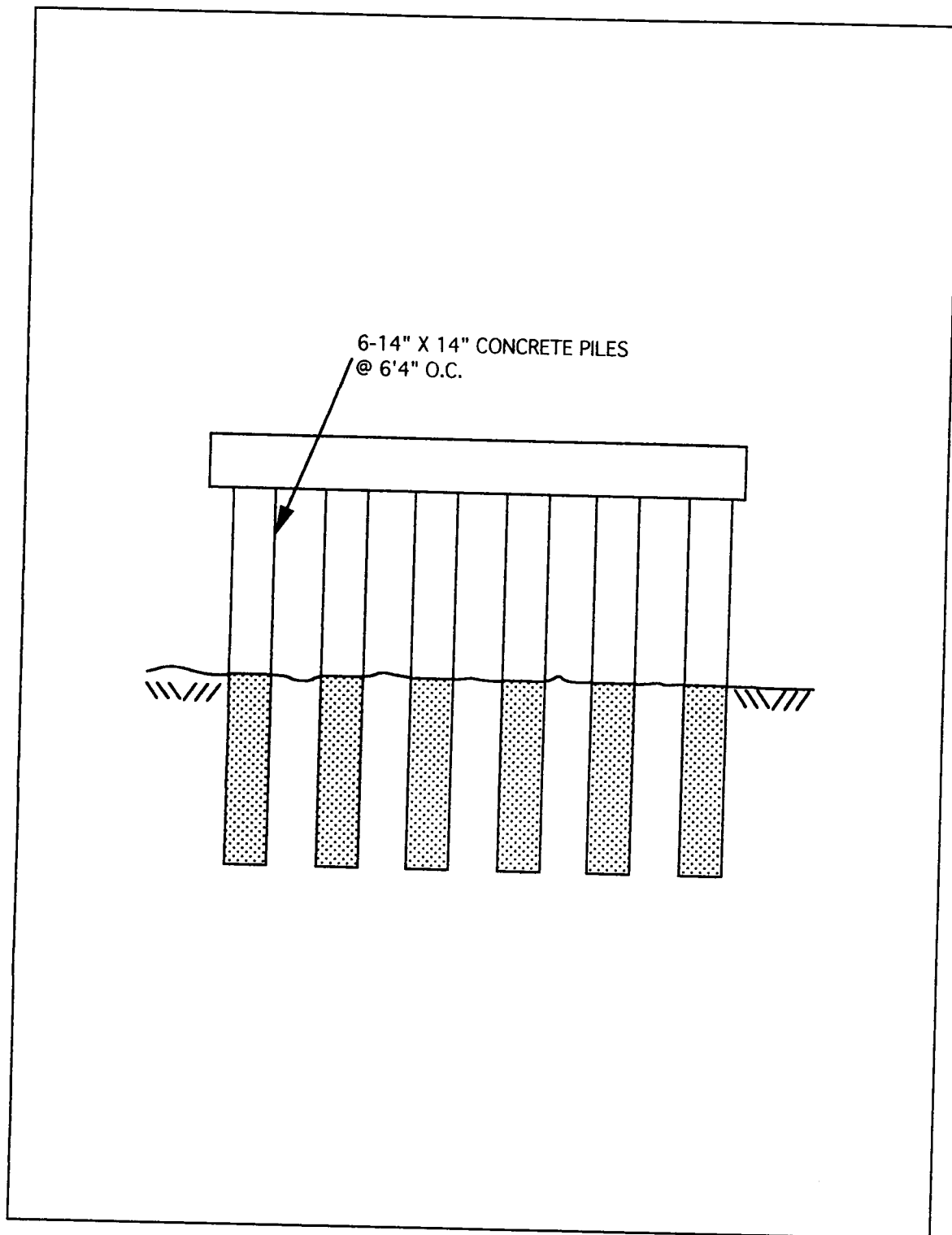


FIGURE 17: TYPICAL HAYWOOD COUNTY BRIDGE BENT

APPENDIX C:

SEISAB INPUT FILES

SEISAB
 C NONCONNAH PRKWY BRDGE #4 STATE ANALYSIS WITH FIXED FOUNDATIONS
 RES SPEC ANAL
 BLO FAC 50
 C UNITS KIP, FT, RAD, SEC
 C ALIGNMENT DATA OMITTED FOR AN ASSUMED STRAIGHT BRIDGE
 SPA
 LEN 220.0, 5*265.0, 220.0
 ARE 9.3, 3*10.25, 3*9.4
 I11 0.03, 6*.04, \$ J (TORSIONAL VALUES)
 I22 9357, 3*7662, 3*5894
 I33 95.9, 6*104.3
 DEN .490 \$STEEL UNIT WEIGHT
 WEI 18.75, 14.6, 5*13.5, \$ TO ACCOUNT FOR DECK WEIGHTS (PER UNIT LENGTH)
 E 4176000
 C
 DESCRIBE
 COL 'ROUND'
 ARE 50.3
 I11 402.1
 I22 201.1
 I33 201.1
 DEN .150
 E 580000
 SPE CAP 'TYP CAP'
 I33 1224.0
 E 580000
 WAL 'TYP WALL'
 ARE 16.625
 I11 15.0
 I22 4.24
 I33 125.0
 DEN .150
 E 450000
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 ELEV TOP 100 AT ABUT 1,8
 ELEV WALL BOT 90.5 AT ABUT 1,8
 WAL 'TYP WALL' AT ABUT 1,8
 CON TRAN AT ABUT 1,8
 BENT
 ELEV TOP 100
 HEI 42.55,51.71,63.17,64.01,58.62,38.02
 WEI 1775 \$ WEIGHT FOR EACH PIER CAP(8.5 X 12 X 116)
 SPE CAP 'TYP CAP' AT BENT 2,3,4,5,6,7
 COLUMN TOP FIXED AT BENT 2,3,4,5,6,7
 COLUMN BOTTOM FIXED AT BENT 2,3,4,5,6,7
 COL NOR LAY 'ROUND' 70. 'ROUND' AT BENT 2,3,4,5,6,7
 CON PIN AT BENT 2,3,4,5,6,7
 CON BEARING ELEMENT NOR LAY 'BEAR' 70 'BEAR' AT BENT 2,3,4,5,6,7
 FOUNDATION
 AT ABUT 1
 SPRING CONSTANTS
 KF1F1 462189
 KF3F3 166833
 AT ABUT 8
 SPRING CONSTANTS
 KF1F1 372093
 KF3F3 63638
 LOADS
 RESPONSE SPECTRUM
 MODE SHAPES 20
 ATC6 CURVE
 SOIL TYPE III
 ACC COEFF 0.2
 FINISH

```

SEISAB
C NONCONNAH PRKWY BRDGE #4 ANALYSIS WITH TRANSLATION SPRINGS
  RES SPEC ANAL
  BLO FAC 50
C UNITS KIP, FT, RAD, SEC
C ALIGNMENT DATA OMITTED FOR AN ASSUMED STRAIGHT BRIDGE
SPA
  LEN 220.0, 5*265.0, 220.0
  ARE 9.3, 3*10.25, 3*9.4
  I11 0.03, 6*.04, $ J (TORSIONAL VALUES)
  I22 9357, 3*7662, 3*5894
  I33 95.9, 6*104.3
  DEN .490 SSTEEL UNIT WEIGHT
  WEI 18.75, 14.6, 5*13.5, $ TO ACCOUNT FOR DECK WEIGHTS (PER UNIT LENGTH)
  E 4176000
DESCRIBE
  COL 'ROUND'
    ARE 50.3
    I11 402.1
    I22 201.1
    I33 201.1
    DEN .150
    E 580000
  SPE CAP 'TYP CAP'
    I33 1224.0
    E 580000
  WAL 'TYP WALL'
    ARE 16.625
    I11 15.0
    I22 4.24
    I33 125.0
    DEN .150
    E 450000
  BEARING ELEMENT 'BEAR'
    KF1F1 1.0E10
    KF2F2 1.0E10
    KF3F3 1.0E10
  ABUTMENT
    ELEV TOP 100 AT ABUT 1,8
    ELEV WALL BOT 90.5 AT ABUT 1,8
    WAL 'TYP WALL' AT ABUT 1,8
    CON TRAN AT ABUT 1,8
  BENT
    ELEV TOP 100
    HEI 42.55,51.71,63.17,64.01,58.62,38.02
    WEI 1775 $ WEIGHT FOR EACH PIER CAP(8.5 X 12 X 116)
    SPE CAP 'TYP CAP' AT BENT 2,3,4,5,6,7
    COLUMN TOP FIXED AT BENT 2,3,4,5,6,7
    COLUMN BOTTOM FIXED AT BENT 2,3,4,5,6,7
    COL NOR LAY 'ROUND' 70. 'ROUND' AT BENT 2,3,4,5,6,7
    CON PIN AT BENT 2,3,4,5,6,7
    CON BEARING ELEMENT NOR LAY 'BEAR' 70 'BEAR' AT BENT 2,3,4,5,6,7
  FOUNDATION
    AT ABUT 1
      SPRING CONSTANTS
      KF1F1 462189
      KF3F3 166833
    AT ABUT 8
      SPRING CONSTANTS
      KF1F1 372093
      KF3F3 63638
    AT BENT 2
      SPRING CONSTANTS
      KF1F1 147160
      KF3F3 147160
    AT BENT 3,7
      SPRING CONSTANTS
      KF1F1 142275
      KF3F3 142275
    AT BENT 4,5,6
      SPRING CONSTANTS
      KF1F1 135667
      KF3F3 135667
  LOADS
    RESPONSE SPECTRUM
    MODE SHAPES 20
    ATC6 CURVE
    SOIL TYPE III
    ACC COEFF 0.2
    DAMP COEFF .02
FINISH

```

SEISAB
 C MADISON COUNTY BRIDGE ANALYSIS WITH FIXED FOUNDATIONS
 RESPONSE SPECTRUM ANALYSIS
 SPANS
 LENGTHS 130.5,130.5
 AREA 66.33
 I11 388.92
 I22 13516.5
 I33 352.86
 DENSITY 0.150
 WEIGHT 2.4405
 DESCRIBE
 COLUMN 'P1' STYPICAL BENT COLUMN
 AREA 9.00
 I11 11.39
 I22 6.75
 I33 6.75
 DENSITY 0.15
 SPECIAL CAP 'CAP'
 I33 26.67
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 BEARING NORMAL
 CONNECTION PINNED AT ABUTMENT 1,3
 ELEVATION 372.34
 BENT
 BEARING NORMAL
 ELEVATION TOP 372.34
 ELEVATION BEARING 367.52
 ELEVATION CAP 365.52
 ELEVATION BOT 342.65
 WEIGHT 150
 SPECIAL CAP 'CAP' AT BENT 2
 COL NOR LAY 'P1' 19.0 'P1' 19.0 'P1' AT BENT 2
 CONNECTION BEARING ELEM NOR LAY 'BEAR' 38 'BEAR' AT BENT 2
 FOUNDATION
 AT ABUTMENT 1,3
 SPRING CONSTANTS
 KF1F1 71240
 KF3F3 6240
 LOADS
 USE FIXED NOTATION FOR FORCES
 RESPONSE SPECTRUM
 ATC6 CURVE
 SOIL TYPE II
 ACCELERATION COEFFICIENT 0.12
 FINISH

SEISAB
 C MADISON COUNTY BRIDGE ANALYSIS WITH TRANSLATION SPRINGS
 RESPONSE SPECTRUM ANALYSIS
 SPANS
 LENGTHS 130.5,130.5
 AREA 66.33
 I11 388.92
 I22 13516.5
 I33 352.86
 DENSITY 0.150
 WEIGHT 2.4405
 DESCRIBE
 COLUMN 'P1' STYPICAL BENT COLUMN
 AREA 9.00
 I11 11.39
 I22 6.75
 I33 6.75
 DENSITY 0.15
 SPECIAL CAP 'CAP'
 I33 26.67
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 BEARING NORMAL
 CONNECTION PINNED AT ABUTMENT 1,3
 ELEVATION 372.34
 BENT
 BEARING NORMAL
 ELEVATION TOP 372.34
 ELEVATION BEARING 367.52
 ELEVATION CAP 365.52
 ELEVATION BOT 342.65
 WEIGHT 150
 SPECIAL CAP 'CAP' AT BENT 2
 COL NOR LAY 'P1' 19.0 'P1' 19.0 'P1' AT BENT 2
 CONNECTION BEARING ELEM NOR LAY 'BEAR' 38 'BEAR' AT BENT 2
 FOUNDATION
 AT ABUTMENT 1,3
 SPRING CONSTANTS
 KF1F1 71240
 KF3F3 6240
 AT BENT 2
 SPRING CONSTANTS
 KF1F1 45663
 KF3F3 45663
 LOADS
 USE FIXED NOTATION FOR FORCES
 RESPONSE SPECTRUM
 ATC6 CURVE
 SOIL TYPE II
 ACCELERATION COEFFICIENT 0.12
 FINISH

SEISAB
 C LOOSAHATCHIE BRIDGE ANALYSIS WITH FIXED FOUNDATIONS
 RES SPEC ANAL
 BLOCKING FACTOR 0
 SPAN
 LEN 14*75.17 77.0
 ARE 15*53.58
 I11 15*5.763
 I22 15*7443.44
 I33 15*100.15
 E 15*617303
 WEI 15*2.243
 DESCRIBE
 COLUMN 'T2'
 AREA 4.18
 I11 1.32
 I22 0.78
 I33 0.78
 E 617303
 SPE CAP 'TYPE C'
 I33 6.75
 E 478161
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 ELEV 274.12 274.96
 CON TRAN AT ABUT 1,16
 BENT
 ELEV TOP 275.35
 ELEV BOT 225.67
 WEI 62.8
 SPE CAP 'TYPE C' AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 COL NOR LAY 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2'-
 6.547 'T2' AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 CON PIN AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 CON BEAR ELEM NOR LAY 'BEAR' 39.3 'BEAR' AT BENT 2,3,4,5,6,7,8,9,10,11,12-
 13,14,15
 FOUNDATION
 AT ABUTMENT 1
 SPRING CONSTANTS
 KF1F1 149430
 KF3F3 89824
 AT ABUTMENT 16
 SPRING CONSTANTS
 KF1F1 145280
 KF3F3 89792
 LOADS
 RES SPEC
 MODE SHAPES 3
 ATC6 CURVE
 SOIL TYPE II
 ACC COEFF 0.2
 FINISH

SEISAB
 C
 C LOOSAHATCHIE BRIDGE WITH TRANSLATION AND ROTATION SPRINGS
 C
 RES SPEC ANAL
 BLOCKING FACTOR 1400
 C
 SPAN
 LEN 14*75.17 77.0
 ARE 15*53.58
 I11 15*5.763
 I22 15*7443.44
 I33 15*100.15
 E 15*617303
 WEI 15*2.243
 C
 DESCRIBE
 COLUMN 'T2'
 AREA 4.18
 I11 1.32
 I22 0.78
 I33 0.78
 E 617303
 SPE CAP 'TYPE C'
 I33 6.75
 E 478161
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 C
 ABUTMENT
 ELEV 274.12 274.96
 CON TRAN AT ABUT 1,16
 C
 BENT
 ELEV TOP 275.35
 ELEV BOT 225.67
 WEI 62.8
 SPE CAP 'TYPE C' AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 COL NOR LAY 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2' 6.547 'T2'-
 6.547 'T2' AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 CON PIN AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 COL TOP FIXED AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 CON BEAR ELEM NOR LAY 'BEAR' 39.3 'BEAR' AT BENT 2,3,4,5,6,7,8,9,10,11,12-
 13,14,15
 C
 C
 FOUNDATION
 AT ABUTMENT 1
 SPRING CONSTANTS
 KF1F1 149430
 KF3F3 89824
 AT ABUTMENT 16
 SPRING CONSTANTS
 KF1F1 145280
 KF3F3 89792
 AT BENT 2,3,4,5,6,7,8,9,10,11,12,13,14,15
 SPRING CONSTANTS
 KF1F1 304
 KF2F2 304
 KM1M1 93843
 KM3M3 93843
 LOADS
 USE FIXED NOTATION FOR FORCES
 RES SPEC
 MODE SHAPES 3
 ATC6 CURVE
 SOIL TYPE II
 ACC COEFF 0.2
 DAMP COEFF 0.02
 C
 FINISH

SEISAB
 C HAYWOOD COUNTY ANALYSIS WITH FIXED FOUNDATIONS
 RESPONSE SPECTRUM ANALYSIS
 SPANS
 LENGTHS 31.5,31.0,31.5
 AREA 39.99
 I11 11.33
 I22 2699.4
 I33 14.99
 DENSITY 0.15
 WEIGHT 1.9875
 E 432000
 DESCRIBE
 COLUMN 'P1' \$14"X14" PILE
 AREA 1.36
 I11 0.26
 I22 0.154
 I33 0.154
 DENSITY 0.15
 E 617303
 SPECIAL CAP 'CAP'
 I33 5.625
 E 479161
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 BEARING NORMAL
 CONNECTION PINNED AT ABUTMENT 1,4
 BENT
 BEARING NORMAL
 HEIGHT 17.5,17.5
 SPECIAL CAP 'CAP' AT BENT 2,3
 COL NOR LAY 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' AT BENT 2,3
 COL TOP FIXED AT BENT 2,3
 COL BOT FIXED AT BENT 2,3
 CONN BEAR ELEM NOR LAY 'BEAR' 32 'BEAR' AT BENT 2,3
 FOUNDATION
 AT ABUT 1,4
 SPRING CONSTANTS
 KF1F1 34560
 KF3F3 7650
 LOADS
 USE FIXED NOTATION FOR FORCES
 RESPONSE SPECTRUM
 MODE SHAPES 6
 ATC6 CURVE
 SOIL TYPE II
 ACCELERATION COEFFICIENT 0.2
 FINISH

SEISAB
 C HAYWOOD COUNTY BRIDGE ANALYSIS WITH TRANSLATION AND ROTATION SPRINGS
 RESPONSE SPECTRUM ANALYSIS
 SPANS
 LENGTHS 31.5,31.0,31.5
 AREA 39.99
 I11 11.33
 I22 2699.4
 I33 14.99
 DENSITY 0.15
 WEIGHT 1.9875
 E 432000
 DESCRIBE
 COLUMN 'P1' S14"X14" PILE
 AREA 1.36
 I11 0.26
 I22 0.154
 I33 0.154
 DENSITY 0.15
 E 617303
 SPECIAL CAP 'CAP'
 I33 5.625
 E 479161
 BEARING ELEMENT 'BEAR'
 KF1F1 1.0E10
 KF2F2 1.0E10
 KF3F3 1.0E10
 ABUTMENT
 BEARING NORMAL
 CONNECTION PINNED AT ABUTMENT 1,4
 BENT
 BEARING NORMAL
 HEIGHT 17.5,17.5
 SPECIAL CAP 'CAP' AT BENT 2,3
 COL NOR LAY 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' 6.33 'P1' AT BENT 2,3
 COL TOP FIXED AT BENT 2,3
 COL BOT FIXED AT BENT 2,3
 CONN BEAR ELEM NOR LAY 'BEAR' 32 'BEAR' AT BENT 2,3
 FOUNDATION
 AT ABUT 1,4
 SPRING CONSTANTS
 KF1F1 34560
 KF3F3 7650
 AT BENT 2,3
 SPRING CONSTANTS
 KF1F1 162
 KF2F2 162
 KM1M1 34342
 KM3M3 34342
 LOADS
 USE FIXED NOTATION FOR FORCES
 RESPONSE SPECTRUM
 MODE SHAPES 6
 ATC6 CURVE
 SOIL TYPE II
 ACCELERATION COEFFICIENT 0.2
 FINISH

APPENDIX D:

SAMPLE CALCULATIONS

I. ABUTMENT STIFFNESS (NONCONNAH PARKWAY BRIDGE)

Constants:

$$\nu = 0.4$$

$$I' = 2.51$$

$$E_s = 41.02 \text{ kips/inch}$$

$$\text{stiffness per pile} = 30 \text{ kips/inch}$$

number of piles:

$$\text{row 1} = 47 \text{ piles}$$

$$\text{row 2} = 27 \text{ piles}$$

pile reduction factor:

$$\text{row 1} = 0.354$$

$$\text{row 2} = 0.765$$

$$\text{endwall length} = 1962 \text{ inches}$$

Lateral abutment stiffness (Equation 5.1)

$$K_{11} = \frac{(41.02 \frac{\text{kips}}{\text{in.}^2}) * (1962 \text{ in.})}{(1 - (0.4)^2) * (2.562)} + 47 \text{ piles} * (0.354) * (30 \frac{\text{kips}}{\text{in.}}) + 27 \text{ piles} * (0.765) * (30 \frac{\text{kips}}{\text{in.}})$$

$$= 38,516 \frac{\text{kips}}{\text{in.}} * (12 \frac{\text{in.}}{\text{ft.}})$$

$$= 462,188 \frac{\text{kips}}{\text{ft.}}$$

II. COLUMN FOUNDATION STIFFNESS

A. PILE CAP STIFFNESS (NONCONNAH PARKWAY PIER 1)

Constants:

pile cap dimensions = 36 ft. x 36 ft.

number of piles = 156

stiffness per pile = 30 kips/inch

pile reduction factor = 0.356

effective dimensions:

$$36\text{ft.} - \frac{14\text{ft.}}{12\frac{\text{in.}}{\text{ft.}}}(13\text{ piles}) = 20.8\text{ ft.}$$

$$36\text{ft.} - \frac{14\text{ft.}}{12\frac{\text{in.}}{\text{ft.}}}(12\text{ piles}) = 22\text{ ft.}$$

equivalent circular radius (Equation 5.4):

$$r_o = \sqrt{\frac{(20.8\text{ ft.}) * (22\text{ ft.}) * (144\frac{\text{in.}^2}{\text{ft.}^2})}{\pi}}$$
$$= 144.8\text{ in.}$$

Pile cap stiffness (Equation 5.3)

$$\begin{aligned}K_{cf} &= \frac{8}{2 - 0.4} (14.65 \text{ kips/in.}^2) * (144.8 \text{ in.}) \\&= 10,617 \text{ kips/in.} * (12 \text{ in./ft.}) \\&= 127,280 \text{ kips/ft.}\end{aligned}$$

B. PILE GROUP STIFFNESS (Equation 5.7 and 5.8)

$$\begin{aligned}K_{gr} &= (30 \text{ kips/in.}) * (156 \text{ piles}) * (0.354) \\&= 1656.7 \text{ kips/in.} * (12 \text{ in./ft.}) \\&= 19,880 \text{ kips/ft.}\end{aligned}$$

C. TOTAL COLUMN FOUNDATION STIFFNESS

$$\begin{aligned}KF1F1 &= 127,280 \text{ kips/ft.} + 19,880 \text{ kips/ft.} \\&= 147,160 \text{ kips/ft.}\end{aligned}$$

VITA

Anthony Kevin Bowling was born in LaFollette, TN on June 26, 1970. Upon graduation from Campbell County Comprehensive High School in 1988, he entered the Civil Engineering program at the University of Tennessee, Knoxville. After three years in the program, he became a member of the Tennessee Air National Guard at McGhee Tyson Airbase in 1992 to financially assist in funding his education. Upon completion of the eye-opening experience of Basic Military Training School, Kevin re-entered the Civil Engineering program where he received his Bachelor of Science degree in May, 1994 and subsequently entered graduate school and received a Master of Science in December 1995.