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16. Abstract

As a result of increasing highway construction and expansion, a corresponding need to increase traffic capacity in heavily populated areas, and ever-increasing constraints on available land for transportation use, there has been an increasing demand for alignment geometries and bridge configurations that result in more efficient use of available space. As a result of this demand, there has been a steady increase in the use of curved girder bridges over the past 30 years. Despites extensive research relating to the behavior of these types of structures, a thorough understanding of curved girder bridge response, especially relating to dynamic behavior, is still incomplete.

To develop an improved, rational set of design guidelines, the Federal Highway Administration (FHWA) initiated the Curved Steel Bridge Research Project in 1992. As part of this project, FHWA constructed a full-scale model of a curved steel girder bridge at its Turner-Fairbank Structures Laboratory. This full-scale model made it possible to conduct numerous tests and collect a significant amount of data relating to the static behavior of a curved girder bridge. However, relatively little information has been available on the dynamic response of curved girder bridges and this type of information is needed before a complete design specification can be developed.

The objective of this study was to develop a finite element model using SAP2000 that could be used for predicting and evaluating the dynamic response of a curved girder bridge. Models of the FHWA curved girder bridge were developed using both beam and shell elements and response information compared with experimental data and with analytical data from other finite element codes. The experimental data were obtained during dynamic testing of the full-scale bridge in the Turner-Fairbank Structures Laboratory and analytical response information was provided from finite element models of the bridge using ANSYS and ABAQUS. The primary focus of the study was the prediction of frequencies and mode shapes of the full-scale curved girder both with and without a deck.

Both experimental and analytical frequencies and mode shapes were calculated and compared. Although the more refined ANSYS and ABAQUS models provided response data that compared more favorably with the experimental data, the SAP2000 models were found to be more than adequate for predicting the lower modes and frequencies of the bridge.

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**FINAL REPORT**

# DYNAMIC ANALYSIS AND TESTING OF A CURVED GIRDER BRIDGE

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Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
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## ABSTRACT

As a result of increasing highway construction and expansion, a corresponding need to increase traffic capacity in heavily populated areas, and ever-increasing constraints on available land for transportation use, there has been an increasing demand for alignment geometries and bridge configurations that result in more efficient use of available space. As a result of this demand, there has been a steady increase in the use of curved girder bridges over the past 30 years. Despites extensive research relating to the behavior of these types of structures, a thorough understanding of curved girder bridge response, especially relating to dynamic behavior, is still incomplete.

To develop an improved, rational set of design guidelines, the Federal Highway Administration (FHWA) initiated the Curved Steel Bridge Research Project in 1992. As part of this project, FHWA constructed a full-scale model of a curved steel girder bridge at its Turner-Fairbank Structures Laboratory. This full-scale model made it possible to conduct numerous tests and collect a significant amount of data relating to the static behavior of a curved girder bridge. However, relatively little information has been available on the dynamic response of curved girder bridges and this type of information is needed before a complete design specification can be developed.

The objective of this study was to develop a finite element model using SAP2000 that could be used for predicting and evaluating the dynamic response of a curved girder bridge. Models of the FHWA curved girder bridge were developed using both beam and shell elements and response information compared with experimental data and with analytical data from other finite element codes. The experimental data were obtained during dynamic testing of the full-scale bridge in the Turner-Fairbank Structures Laboratory and analytical response information was provided from finite element models of the bridge using ANSYS and ABAQUS. The primary focus of the study was the prediction of frequencies and mode shapes of the full-scale curved girder both with and without a deck.

Both experimental and analytical frequencies and mode shapes were calculated and compared. Although the more refined ANSYS and ABAQUS models provided response data that compared more favorably with the experimental data, the SAP2000 models were found to be more than adequate for predicting the lower modes and frequencies of the bridge.

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## INTRODUCTION

As a result of ever-increasing constraints on available land for transportation use, there has been an increasing demand for alignment geometries and bridge configurations that result in more efficient use of available space. As a result of this demand, there has been a steady increase in the use of curved girder bridges over the past 30 years. These types of bridges offer the advantages of requiring less land use, fewer substructures and shallower sections while allowing vehicle speeds to be maintained. These bridges are found in on- and off-ramps to major highways and are much preferred in such locations because of simplicity of fabrication, speed of erection, and serviceability performance (Zureick et al., 2000).

Although there has been extensive research devoted to the behavior of curved girder bridges, a full understanding of their response, especially with regard to dynamic loadings, is still incomplete. To improve the state of knowledge relating to the behavior of curved girder bridges, the Federal Highway Administration (FHWA) in 1969 formed the Consortium of University Research Teams. Research from this effort contributed to the American Association of State Highway and Transportation Officials (AASHTO) *Guide Specifications for Horizontally Curved Highway Bridges* (AASHTO 1980). These specifications were never fully adopted into the AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996) because particular design issues were not completely addressed (Davidson, 2000).

To develop an improved, rational set of design guidelines, FHWA initiated the Curved Steel Bridge Research Project in 1992. To provide a major testing capability for the project, FHWA constructed a full-scale model of a curved steel bridge at its Turner-Fairbank Highway Research Center (TFHRC). This test structure made it possible to conduct numerous tests and collect a significant amount of data relating to several areas of curved girder bridge behavior. During construction, the bridge was instrumented, yielding information about the forces and stresses within a curved steel bridge during construction conditions. Since construction, the bridge has been used for tests involving bending, shear, bending-shear interaction, computer modeling, and cross-frame spacing among other things. This research is expected to benefit the AASHTO *LRFD Bridge Design Specification* (AASHTO 1998).

Among the areas of curved girder behavior not yet fully understood is the dynamic response, necessary for a complete understanding of curved girder behavior, and for reliable seismic design. Previous versions of the AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996) have treated dynamic loading as only a factor applied to a static load. Currently, the most general and practicable method for modeling of curved girders is the finite element method. Unfortunately, there has been little experimental work conducted on the dynamic behavior of curved girder bridges and, consequently, relatively little experimental data available. Such experimental data could provide a reliable basis for comparison with predicted response from various analytical or computer models and, thus, assist in evaluating the advantages of the various computer models.

## PROBLEM STATEMENT

As noted previously, limited data relating to the response of curved girder bridges are available, and this is particularly true for the dynamic response of these structures. Improved understanding of the dynamic behavior of a curved steel girder bridge could enhance the engineer's ability to design these bridges. In addition, the ability to model curved steel bridge dynamic behavior accurately would lend more credibility to static results obtained from computer models. The existence of the full-scale, three-girder curved bridge at TFHRC presented a unique opportunity to study the behavior of a curved girder bridge in a controlled environment. This is especially true for dynamic testing that had been neglected in previous curved girder bridge studies.

A major advantage of testing the curved girder bridge at TFHRC was that the bridge was available for testing in two stages of construction. FHWA allowed time for the bridge to be tested before the concrete deck was poured. This decision made it possible to test the structure as a bare steel frame. Later, the bridge was tested after placement of the composite concrete deck. This also yielded certain benefits when developing finite element models of the bridge. Using the same bridge for both bare girder and composite steel-concrete bridge models allowed for simpler, less time-consuming model development.

## PURPOSE AND SCOPE

The primary objective of this project was to develop a finite element model that could be used to predict the dynamic response of a curved girder bridge using SAP 2000 software. The specific bridge under consideration was the full-scale curved girder bridge in the Turner-Fairbank Structures Laboratory. To reach the goal of an accurate SAP2000 model for dynamic behavior, it was necessary to accomplish a number of secondary objectives. These included the following:

- basic model development using SAP2000, ANSYS and ABAQUS

- experimental testing of the curved girder bridge and collection of dynamic response data

- validation of SAP2000 models by comparing response information with corresponding data obtained from other computer programs and with experimental data.

## METHODOLOGY

The following tasks were conducted to achieve the study objectives:

1. Identify and become familiar with the details and characteristics of the FHWA curved girder bridge to be modeled and tested.

2. Become familiar with the capabilities and limitations of the SAP2000 software and develop a series of finite element models, ranging from simple to complex, which would represent the FHWA curved girder.

3. Develop more detailed computer models of the curved girder bridge using the commercially available finite element programs, ANSYS and ABAQUS.

4. Conduct tests on the full-scale curved girder bridge in the Turner-Fairbank Structures Laboratory with and without the deck to obtain experimental dynamic response data.

5. Compare the dynamic response predicted by the SAP2000 models with that predicted by the more sophisticated ANSYS and ABAQUS models and with the dynamic response measured experimentally.

### Description of Bridge

The initial full-scale model of the curved girder bridge in the Turner-Fairbank Structures Laboratory consisted of three steel plate girders connected by cross-bracing. Each girder had a depth of 48 in with a girder spacing of 8.75 ft and different cross-sectional properties as detailed in Table 1. The inner girder, designated Girder 1, has a radius of 191.25 ft; the middle girder, designated Girder 2, has a radius of 200 ft; and the outer girder, Girder 3, has a radius of 208.75 ft. The span lengths of the three girders are 86.06 ft, 90.0 ft, and 93.94 ft, respectively. In addition, web stiffeners were placed at the bearings and along the span of all three girders. The curved girder bridge was essentially a simple span with the support conditions at the ends providing no resistance to rotation.

The cross-bracing sections consisted of pipe sections having a 5-in outer diameter and 1/4-in wall thickness and were located at the end supports, quarter-span, and mid-span locations. The cross-bracing plan is shown in Figure 1. Figure 2 is a photograph of the cross bracing between two of the girders at one support.

**Table 1. Plate Girder Properties**

| Property | Girder 1 | Girder 2 | Girder 3 |
|---|---|---|---|
| Top Flange Thickness (in) | 7/8 | 7/8 | 1 |
| Top Flange Width (in) | 12 | 14 | 24 |
| Web Thickness (in) | 5/16 | 5/16 | 5/16 |
| Web Height (in) | 48 | 48 | 48 |
| Bottom Flange Thickness (in) | 7/8 | 1 | 1 3/8 |
| Bottom Flange Width (in) | 17 | 22 | 24 |
| Cross-sectional Area (in$^2$) | 40.375 | 49.25 | 72 |
| Moment of Inertia About 3 Axis (in$^4$) | 17752 | 22238 | 36692 |
| Moment of Inertia About 2 Axis (in$^4$) | 484 | 1089 | 2736 |
| Torsional Constant (in$^4$) | 6.72 | 10.61 | 47.5 |

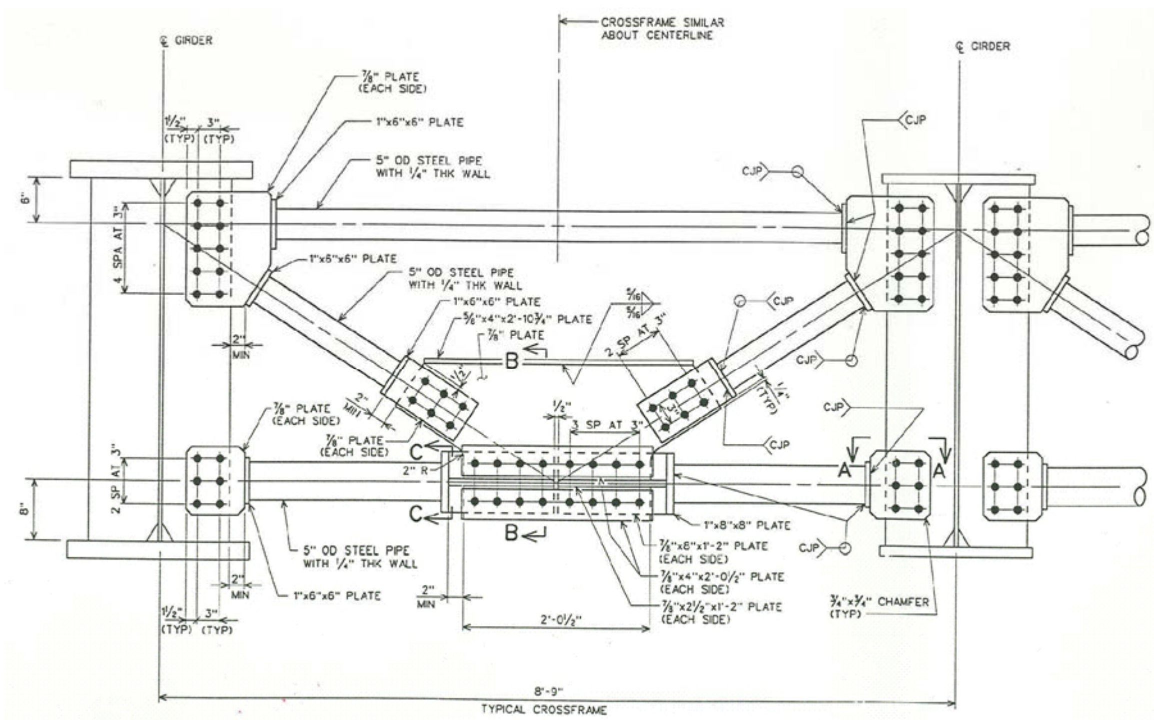


**Figure 1. Cross Bracing Plan**

In addition to the cross bracing, diagonal bracing, consisting of two WT6X29 rolled shapes, was installed at each end of the span. This bracing was connected from the bottom flange of Girders 1 and 3 at each support to the bottom flange of Girder 2 approximately 8 ft from the support. This can be seen in the plan for the original structure as shown in Figure 3 and in Figure 4.

This initial full-scale model of the curved girder bridge, without the deck, is shown in Figure 5. This particular model, but with a reduced number of cross bracings, was also the subject of the first dynamic test conducted as a part of this study.

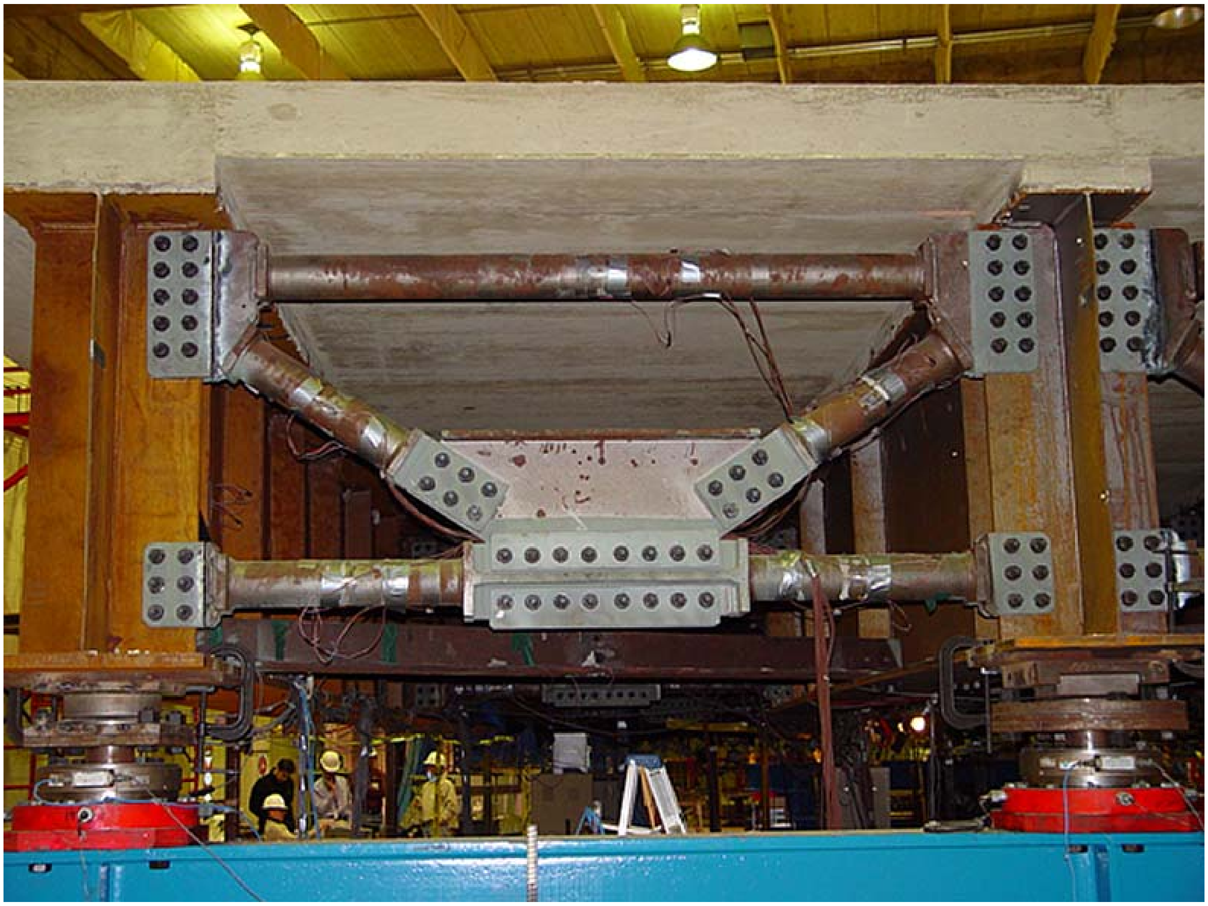

**Figure 2. Cross Bracing**

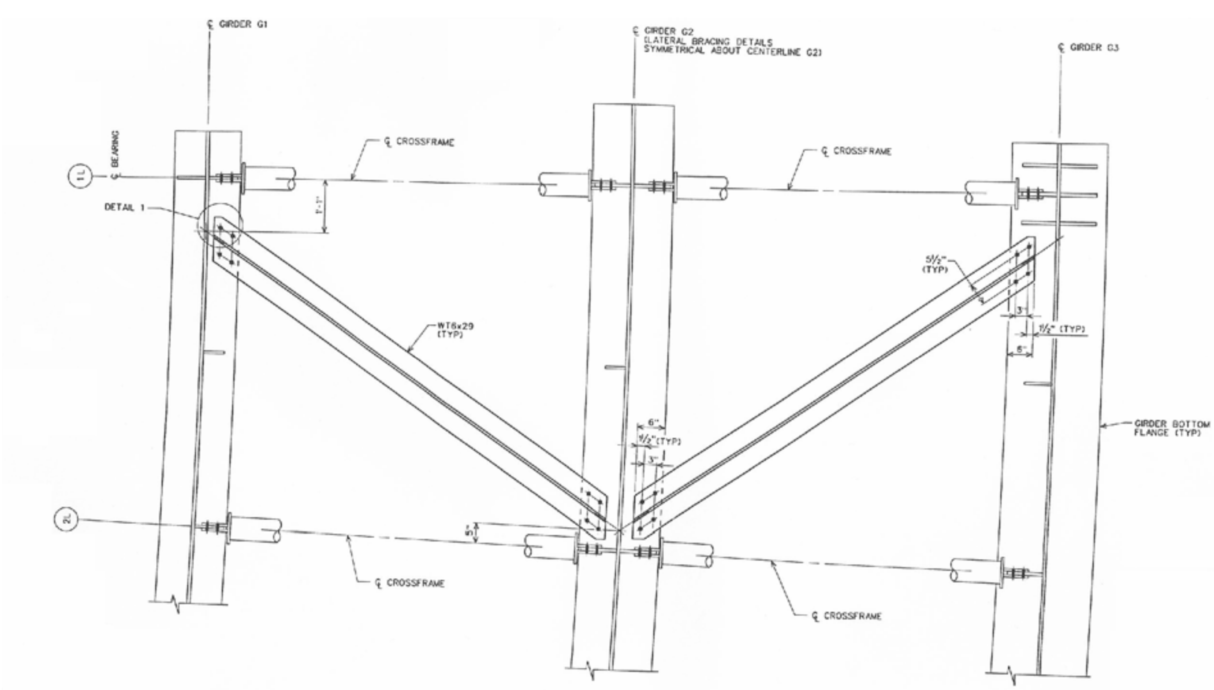


**Figure 3. Diagonal Bracing Plan**

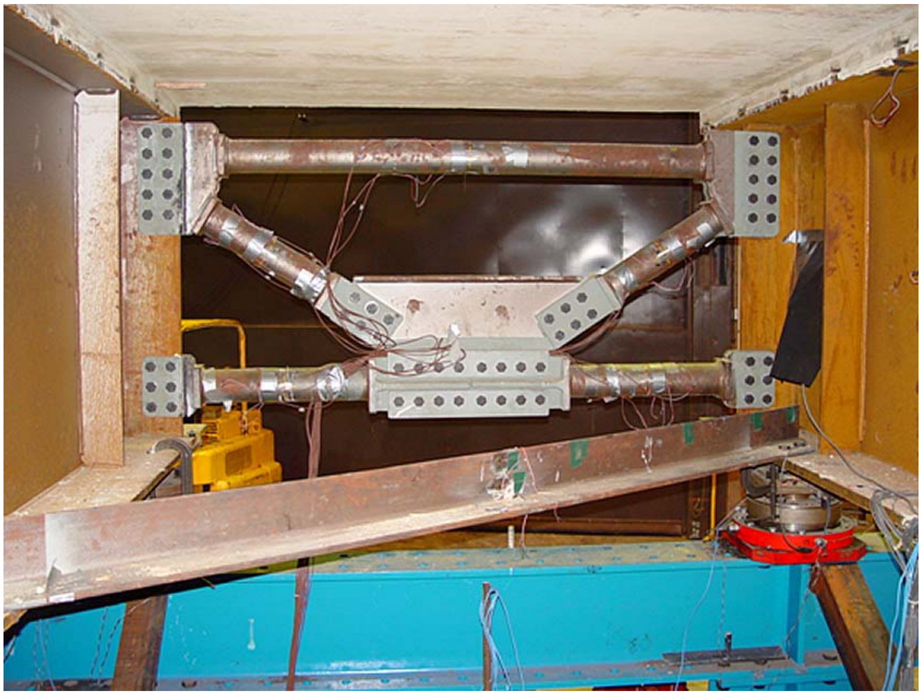

**Figure 4. Diagonal Bracing**

A subsequent modification to this model added a composite concrete deck that was cast in place. The design used an 8-in deck along with a 3-in bolster between the deck and the steel girders. Composite behavior was achieved by welding shear studs to the steel girders before the concrete was poured or formed. Figures 6 and 7 show the end of the bridge with the concrete deck and bolster fully composite with the steel girders. This final version of the bridge was the focus of the second dynamic test.

## Development of Bridge Models Using SAP2000

Initial models were relatively simple beam models and were used to become familiar with the software. At each stage of development, procedures for extracting dynamic response information in the form of natural frequencies and modes were implemented and evaluated. A number of issues, including boundary conditions, cross bracing, stiffeners, and curvature, were given careful consideration when the models were developed. The cross bracing for the bridge presented modeling challenges because of the complex connection between the pipes, and approximations employed for modeling this component are described later. The diagonal stiffener model was simpler because it was composed only of standard WT6X29 sections.

The first bridge model was composed of beams elements representing the girders; a subsequent, more sophisticated model consisted of shell elements representing the flanges and web of the girders. Each model was initially used as a bare-girder model and compared with tests of the bridge without a composite deck. Later, a concrete deck was added to each model, which in both cases consisted of shell elements rigidly linked to the girders. These later models were compared with the test results from the second experiment in which the composite bridge was tested.

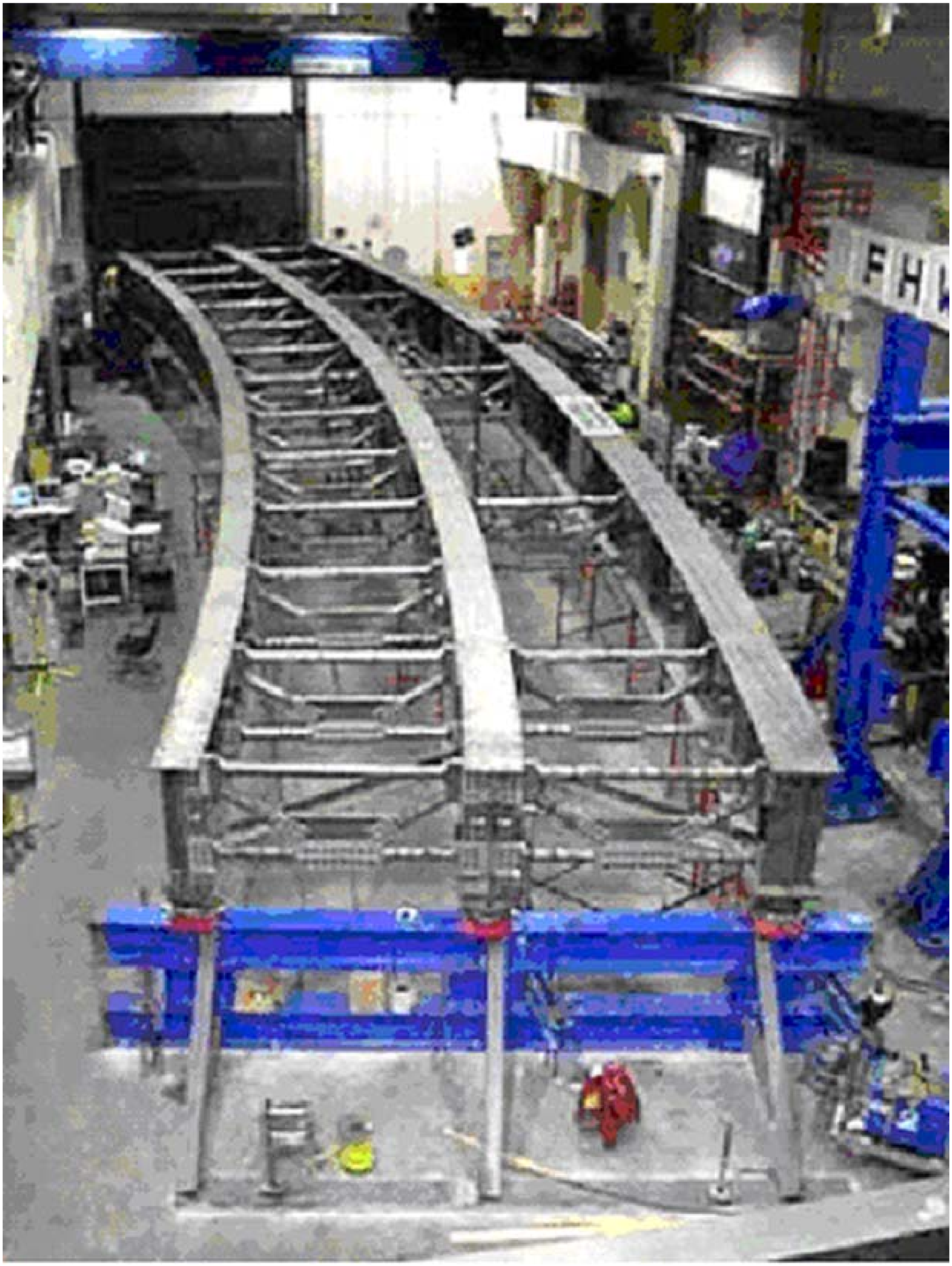

**Figure 5. Bridge Structure Without Deck**

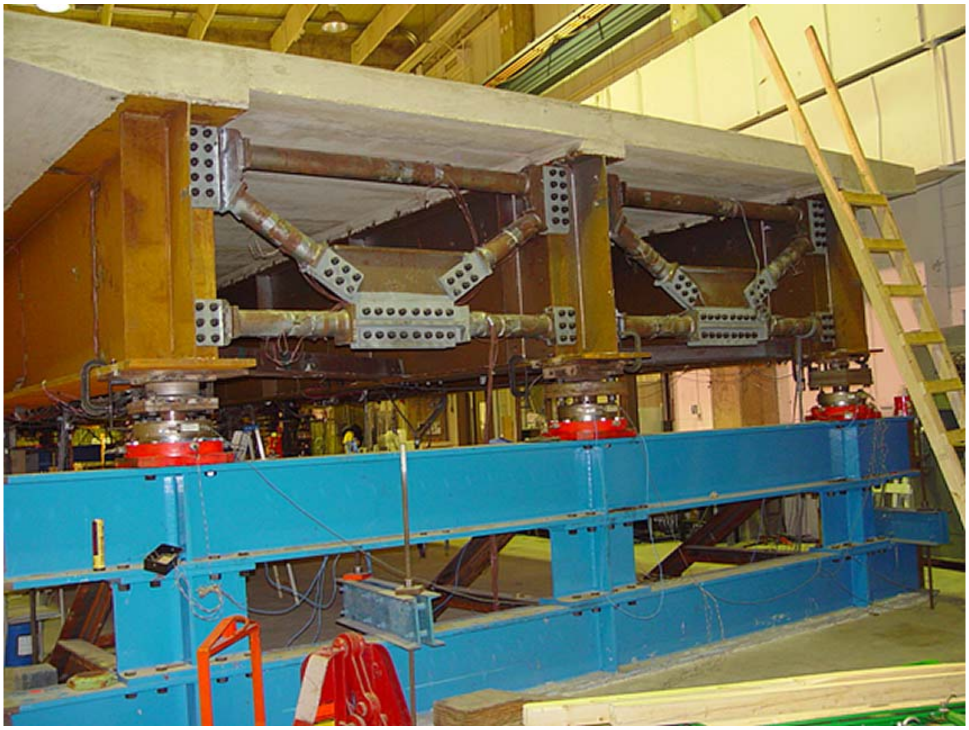

**Figure 6. Details of Deck, Girders, and Bracing**

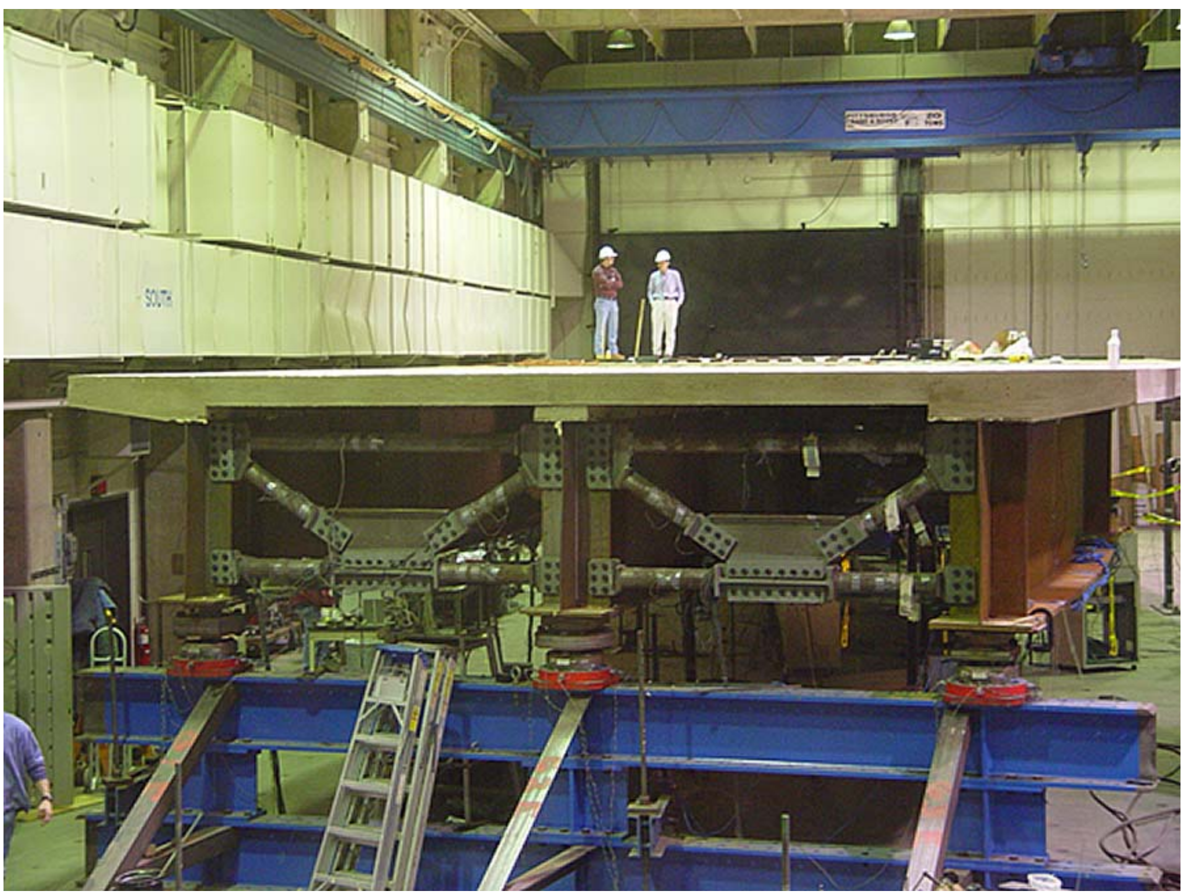


**Figure 7. Completed Bridge Prior to Testing**

From preliminary studies, the beam model was found to be too restrictive to provide meaningful response information. This was primarily a result of the beam model being unable to represent the rotational characteristics of the curved girders. Although a number of beam models were developed and evaluated, the data were not considered useful to this project and the results are not included.

**Shell Model**

To develop an improved girder model, the girders were modeled using SAP2000 shell elements. Plate bending behavior included two-way components, out-of-plane components, plate rotational stiffness components, and a translational stiffness component normal to the element plane. Cross bracing and diagonal stiffeners were modeled using beam elements whose stiffnesses were determined by detailed modeling of these components. Each girder was modeled using a series of quadrilateral shell elements to represent the bottom flange, web, and top flange of each girder. Flange elements were located at the mid thickness of the flange, and thus web elements extended to half the depth of the flange.

For this model, each girder was modeled using 180 elements along the length of the girder, with 4 elements across the width of the top and bottom flange and 6 elements along the depth. Thus, each girder was represented by a total of 2,880 shell elements, each with dimensions of approximately 6 in by 6 in. The most severe aspect ratio in any of the elements used for this model was approximately 1.5 to 1, which is generally considered acceptable for reliable results.

Figure 8 shows the completed shell element bridge model in elevation, plan, and perspective view.

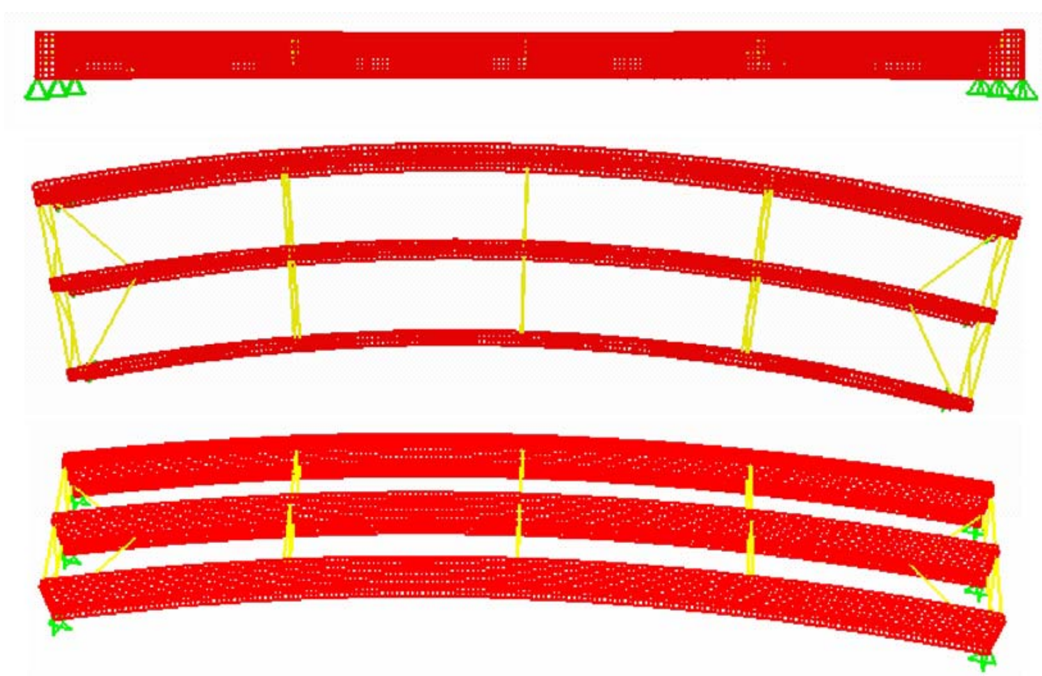

**Figure 8. SAP2000 Shell Element Model**

## Deck Model

A representation of the concrete deck was added to the SAP2000 shell model. The deck was modeled as a series of quadrilateral shell elements and connected to the girder models using rigid link elements. These rigid links coupled all corresponding degrees of freedom at the connected nodes and thus ensured that the deck and the girders would act in a composite manner. In the shell model, the rigid links connected the deck to the upper flange of the girders and extended from the midplane of the deck to the midplane of the upper flange, a distance of approximately 7.4 in. Figure 9 is a sketch of the shell model with deck.

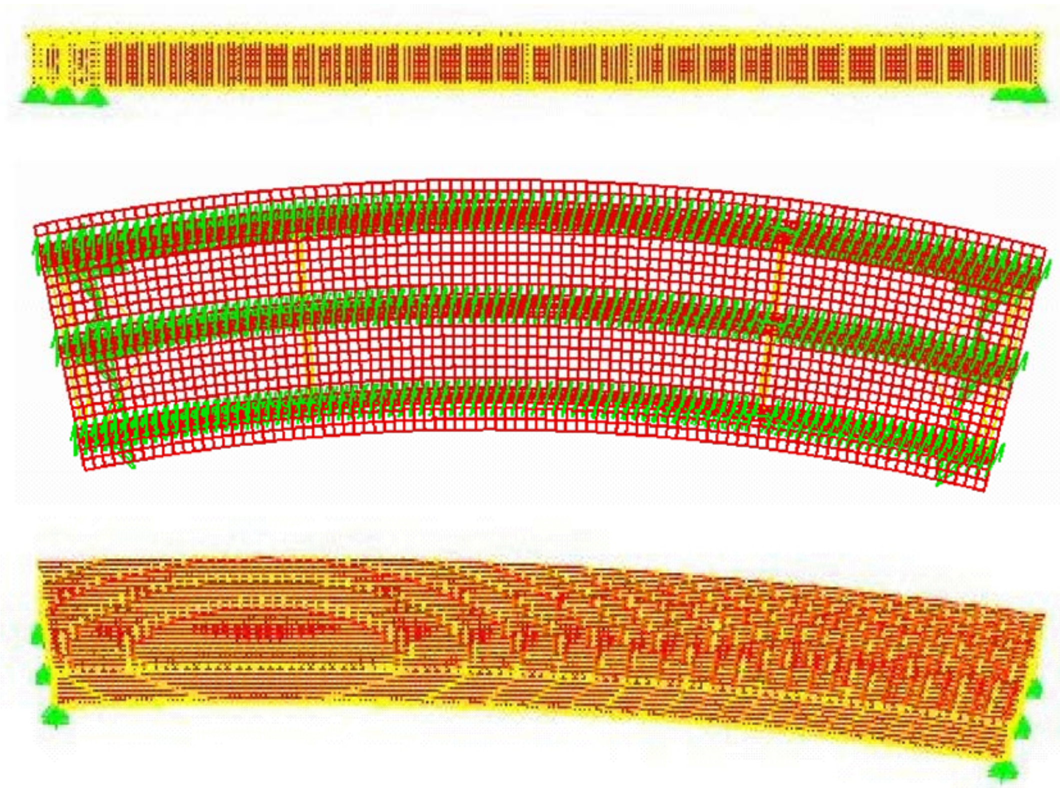

**Figure 9. Shell Element Model with Deck**

## Development of ANSYS and ABAQUS Models

ANSYS and ABAQUS are large-scale commercial finite element codes capable of linear and nonlinear, static and dynamic analysis with extensive modeling features. The primary reason for developing models of the curved girder bridge using these two codes was to validate the SAP models and to confirm dynamic response information obtained from the laboratory tests. Since these commercial codes employ more sophisticated representation of element deformations, results from these two codes should be more accurate than results obtained from SAP2000.

In the ANSYS models, the deck and the girders were modeled using SHELL63 elements. These elements included both membrane and bending stiffness, with 6 degrees of freedom at each node. For models incorporating the composite concrete deck, it was necessary to add a rigid link as described for the SAP models. In this case, the rigid link was represented by BEAM4 elements with zero mass and large stiffness. These link elements connected the edges of the girder top flange to the corresponding node of the concrete deck.

A limitation on problem size restricted the number of elements that could be used to model the bridge. Each longitudinal segment of the girder used 2 elements to model the top and bottom flange of the girders and 4 elements to represent the web. Again, to meet size restrictions, the ANSYS model had elements spaced to allow for a total of 40 elements in the longitudinal direction, resulting in a total of 320 elements for each girder.

The cross bracing was modeled using the ANSYS beam elements and represented the actual K-bracing geometry. Section properties were adjusted to account for the gusset plates. Since frequencies from the SAP models showed little dependence on the cross-sectional properties, these modified values should have little impact on the dynamic properties of the bridge model. Because of model characteristics, the cross bracing was connected at the web-flange intersection of the girders. The ANSYS model was restrained at the end points of the three girders, and all support points were treated as pinned. Sketches of the ANSYS model with and without the deck are shown in Figures 10 and 11, respectively.

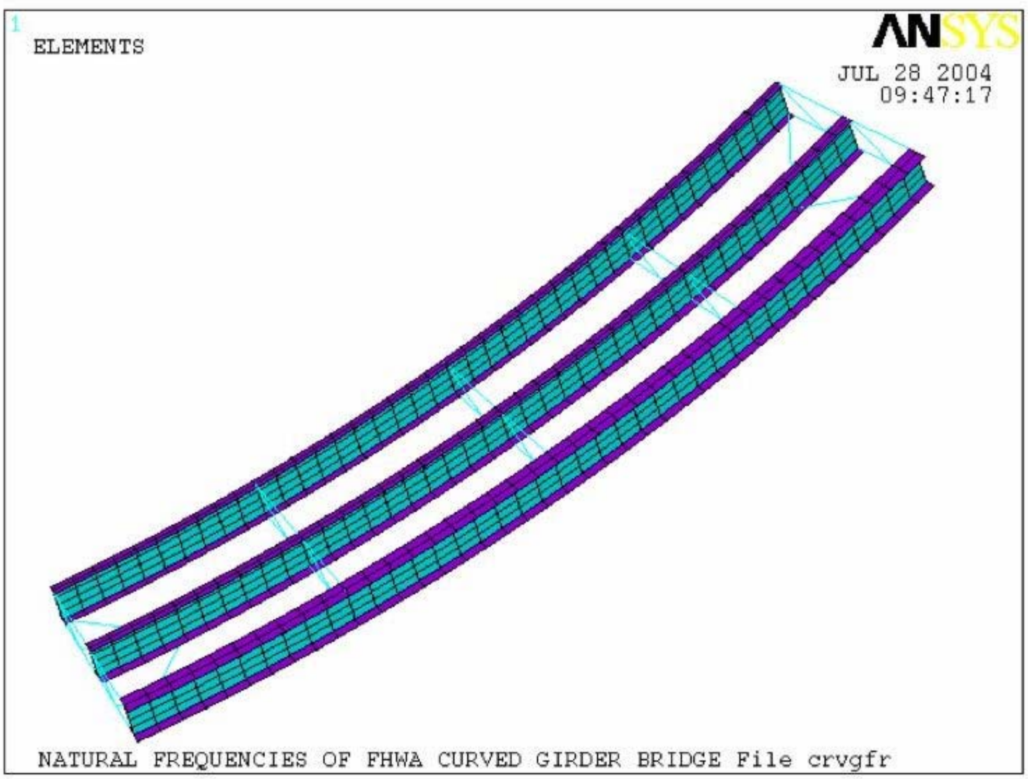


**Figure 10. ANSYS Model Without Deck**

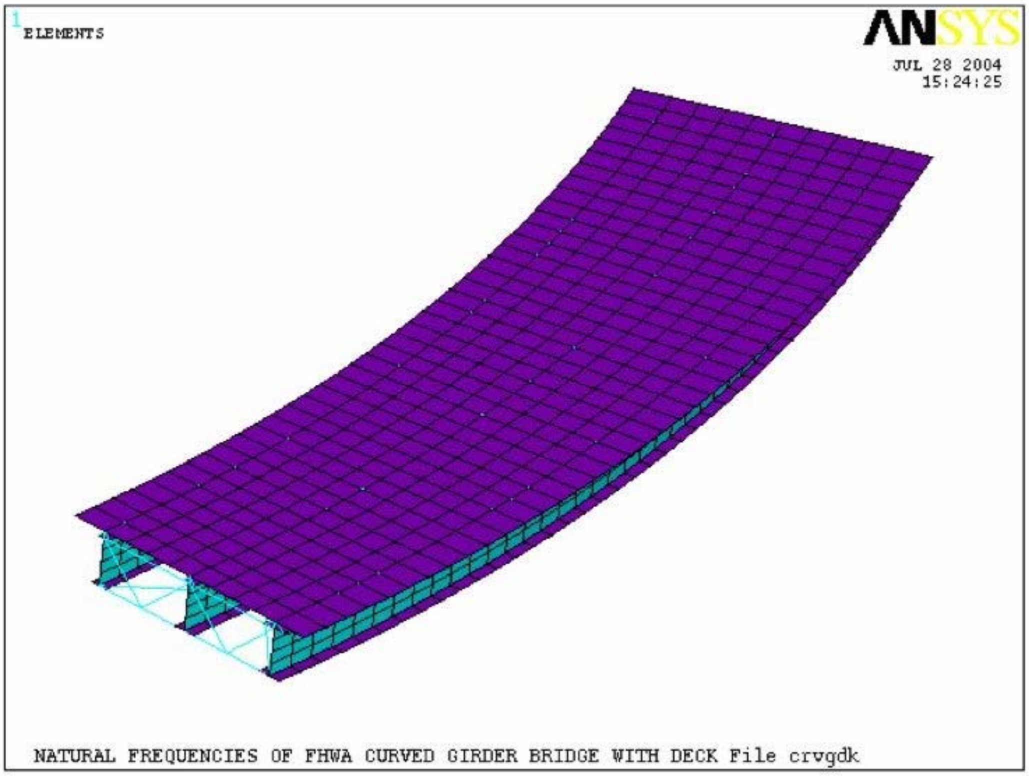


**Figure 11. ANSYS Model with Deck**

An ABAQUS model of the curved girder bridge was also employed to predict frequencies of the bridge. FHWA contractors had developed this model, and the input file was made available for use in this study. The model was far more detailed and complex than those developed as part of this study independently, and the input file was more than 1,200 pages in length. The only available version of this model was one that included the bare girders with all bracing and stiffeners but excluded the composite deck. It was possible to run the model and extract frequency data and plots of mode shapes for this case, but no other response information could be extracted. Figure 12 is a sketch of the ABAQUS model.

## Dynamic Testing Of Curved Girder Bridge

For this study, two tests were conducted on the Turner-Fairbank curved girder bridge. The initial test was completed on the structure with bare girders before the deck was placed. This test was conducted on August 19, 2003. The second test was conducted on January 22, 2004, after the deck had been placed and had cured to design strength. Both tests were completed with the objective of determining frequencies and mode shapes and involved the same test procedure except where noted.

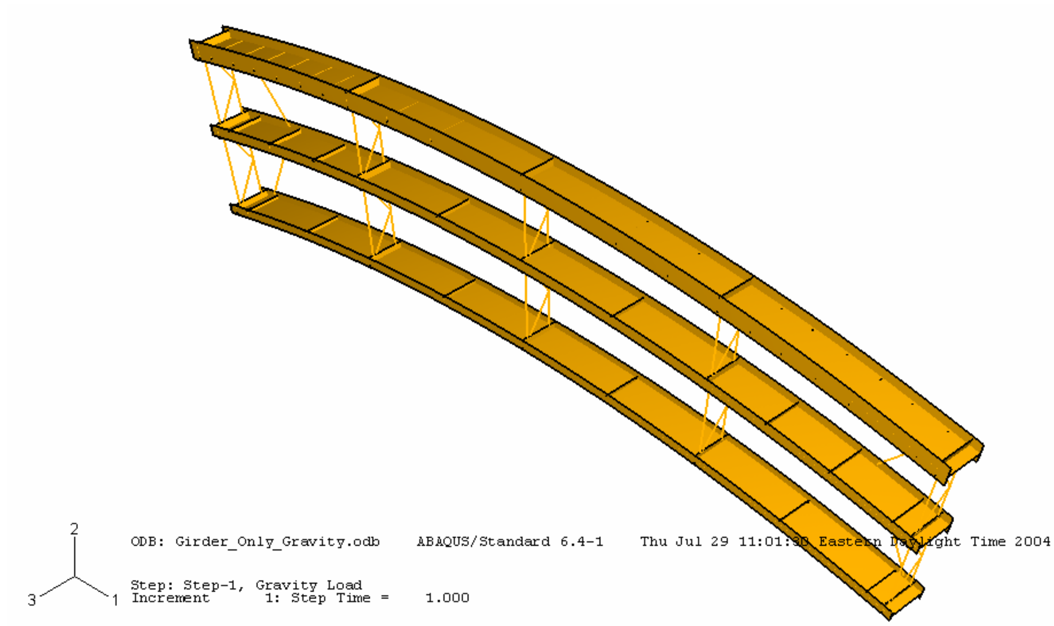


**Figure 12. ABAQUS Model**

The two tests used the same equipment including shaker, signal analyzer, accelerometers, and data acquisition system. The shaker used was an APS ElectroSeis, which uses a large electromagnet to move weights in response to a signal provided by the signal analyzer. For this project, it provided sinusoidal excitation to the shaker at specified frequencies.

The signal analyzer was an Onno Sokki dual-channel FFT analyzer. It was used to control the frequency of the excitation and provided output from one accelerometer that allowed confirmation of accelerometer output from the data acquisition system during the tests. Two types of accelerometers were used for the tests, both manufactured by PCB Piezotronics of Depew, New York. For vertical accelerations, PCB 393C seismic accelerometers were used, and for horizontal measurements, 302B03 accelerometers were used. The horizontal accelerometers had a lower sensitivity than the vertical ones by a factor of 10/3.

The data acquisition system consisted of a laptop computer with an add-on card allowing the computer to accept eight accelerometer signals. An advanced version of the LabVIEW Software package was installed that allowed for the recording of both time and frequency domain data during the test. For later analysis on computers without LabVIEW, test data were saved to a Microsoft Excel spreadsheet file.

The testing procedure was the same for both tests. Accelerometers were placed on the bridge at locations selected during an earlier test. The accelerometer locations and numbering codes are presented in Figure 13. A four-character alphanumeric code was used to identify each accelerometer on the bridge. The first value is an H, L, or V and describes the orientation of the accelerometer: H for horizontal-transverse, L for horizontal-longitudinal, and V for vertical orientation. The second digit of the accelerometer code is the number of the girder on which the accelerometer is placed numbered from 1 being the inside girder (smallest radius) and 3 being the outside girder (largest radius). The third value in the code is a letter referring to the

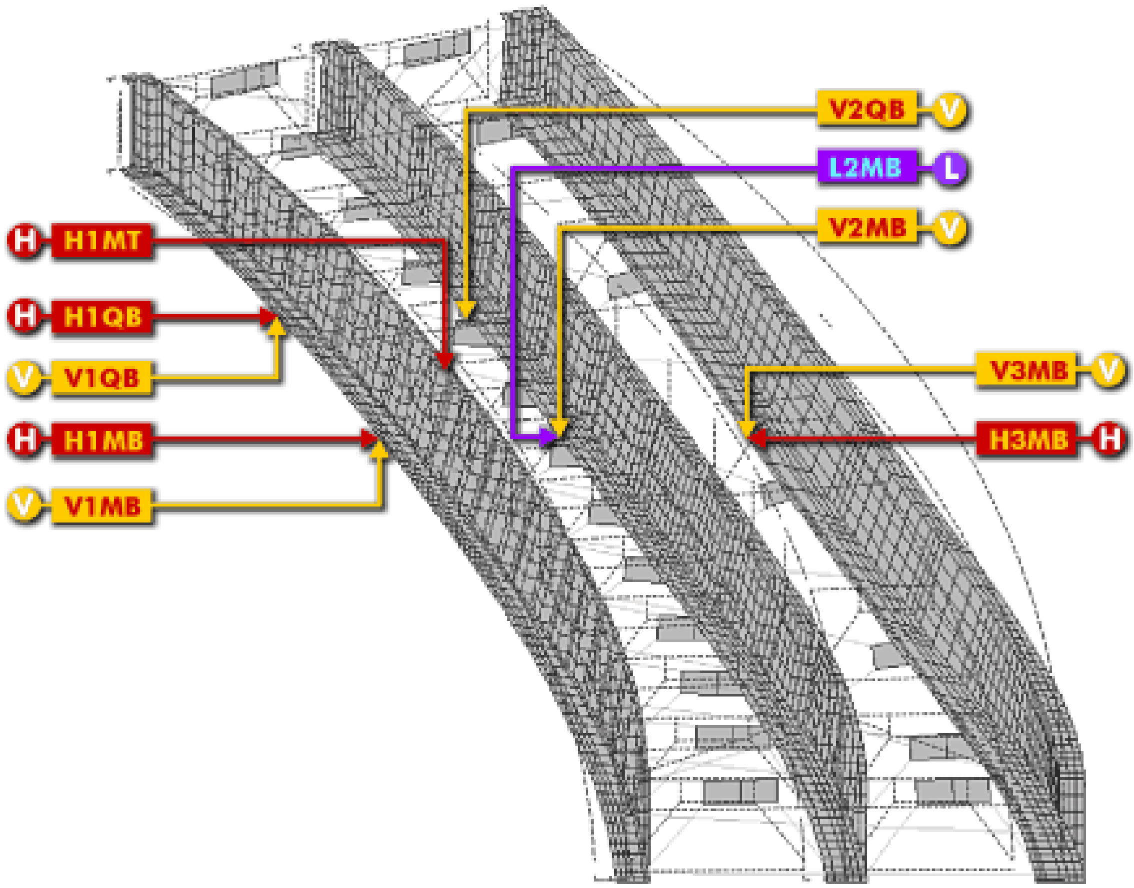


**Figure 13. Accelerometer Locations and Labels**

longitudinal position, with M representing mid-span location and Q referring to the quarter-span. The fourth letter, a T or B, indicates whether the accelerometer location is on the top or bottom flange of the girder. For the composite bridge structure test, the top flange was not exposed and the accelerometer H1MT was placed on the top of the deck above the top flange of Girder 1.

Once the accelerometers were located, the shaker was placed on the structure and oriented to provide first vertical and then horizontal transverse excitation. The excitation frequency was varied over the desired frequency range, and the resonant frequencies were noted. Once the frequency sweep had been completed, the excitation frequency was set to one of the resonant frequencies, and the frequency varied slightly until the maximum amplitude was encountered in a frequency domain plot. With the shaker continuing to excite the bridge at this frequency, data were recorded and saved to an Excel file for later analysis.

Recorded output from the horizontal accelerometers was multiplied by 10/3 to compensate for specified differences in sensitivity between the two types of accelerometers. To gain displacement data, and decrease the effect of any signal noise, a Fast Fourier Transform (FFT) was performed on acceleration data using Excel. These data were then plotted to obtain information in the frequency domain. This removed the noise and decreased the likelihood that spurious data were being used to obtain mode shapes. The Fourier analysis was completed on all data from all tests. Mode shapes derived from this could then be compared to those calculated by computer methods.

### Comparison of Response Data

Both analytical and experimental values of natural frequencies of the curved girder bridge were obtained during this study. Analytical values of frequencies were predicted from the computer models developed using the three computer programs: SAP2000, ANSYS, and ABAQUS. Experimental values of natural frequencies were measured during the two field tests at the Turner-Fairbank Structures Laboratory. Comparisons were made between the various values of natural frequencies obtained from computational and experimental efforts.

## RESULTS AND DISCUSSION

The primary results for this project were the frequencies and mode shapes from both computer models and experimental tests. In an attempt to develop and evaluate a viable SAP2000 model for dynamic analysis, results from the SAP shell models were compared to both the experimental results and the results of the other computational models. Similarly, other computational models were compared and validated against the results of the experimental dynamic tests. The determination of frequencies from the computational models and the laboratory tests was straightforward and easy to interpret. However, a determination of the mode shapes posed a variety of problems, which are discussed later.

### Computational Results

Results from each finite element model included mode shapes and frequencies. Numerical information from the ABAQUS mode shapes was not obtained for reasons discussed previously. However, pictorial data for mode shapes from ANSYS, ABAQUS, and SAP, as well as numerical data from SAP and ANSYS, were available.

#### SAP2000 Models

Results from the SAP2000 models included those for the beam models and the shell models. The models that used shell elements for the girders were evaluated with and without the composite concrete deck.

Frequencies and mode shapes for the shell element model without the composite deck are provided in Table 2. In this case, displacements were tabulated for locations corresponding to the locations of the accelerometers in the subsequent experimental tests. Displacements were normalized with respect to the largest value for each mode. Sketches of the first four mode shapes are shown in Figures 14a-d.

**Table 2. Mode Shapes and Frequencies for Shell Model Without Deck**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 |
|---|---|---|---|---|---|
| H1MT | -0.62 | -0.45 | 0.00 | 0.00 | 0.00 |
| H1MB | -0.13 | 0.12 | 0.00 | 0.00 | 0.00 |
| H3MB | -0.03 | -0.15 | 0.00 | 0.00 | 0.00 |
| H1QB | -0.05 | -0.03 | 1.00 | 1.00 | 1.00 |
| V2MB | 0.77 | 0.27 | 0.00 | 0.00 | 0.00 |
| V1QB | 0.40 | 0.68 | -0.08 | 0.30 | 0.35 |
| V3MB | 1.00 | -0.50 | 0.00 | 0.00 | 0.00 |
| V2QB | 0.55 | 0.19 | -0.15 | -0.23 | 0.22 |
| V1MB | -0.51 | 1.00 | 0.00 | 0.00 | 0.00 |
| Frequency (Hz) | 2.02 | 4.91 | 5.09 | 6.99 | 7.68 |

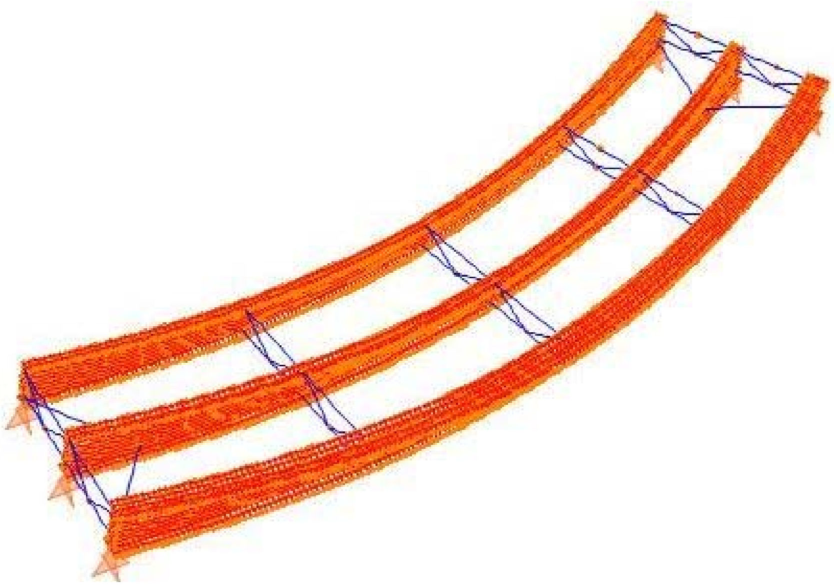

**Figure 14a. Shell Element Model Mode 1**

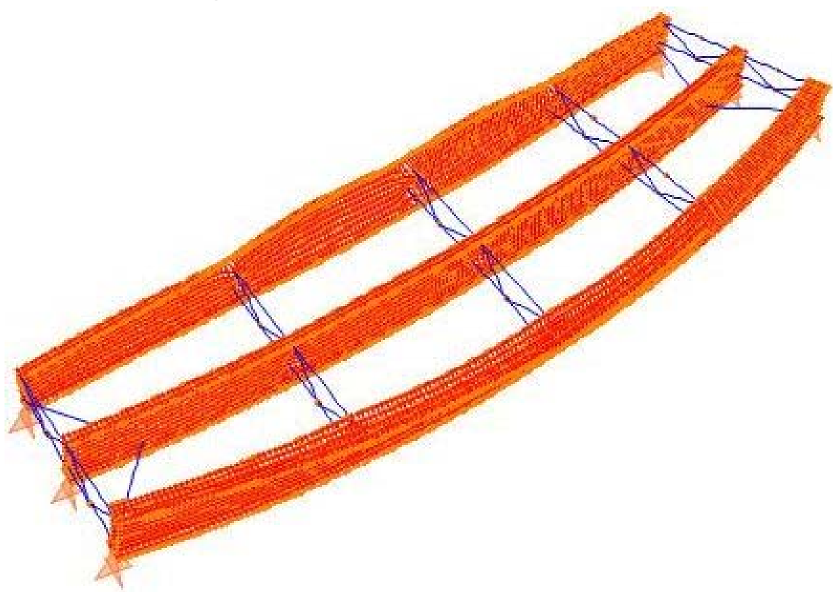

**Figure 14b. Shell Element Model Mode 2**

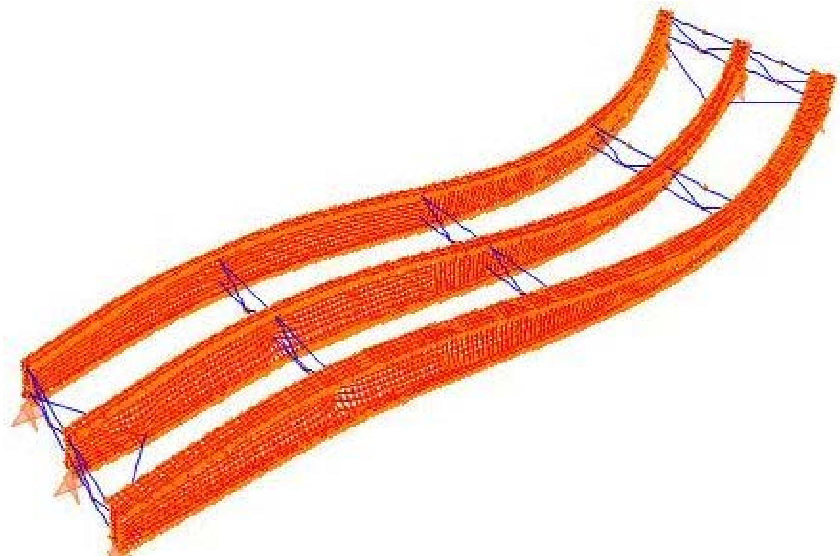

**Figure 14c. Shell Element Model Mode 3**

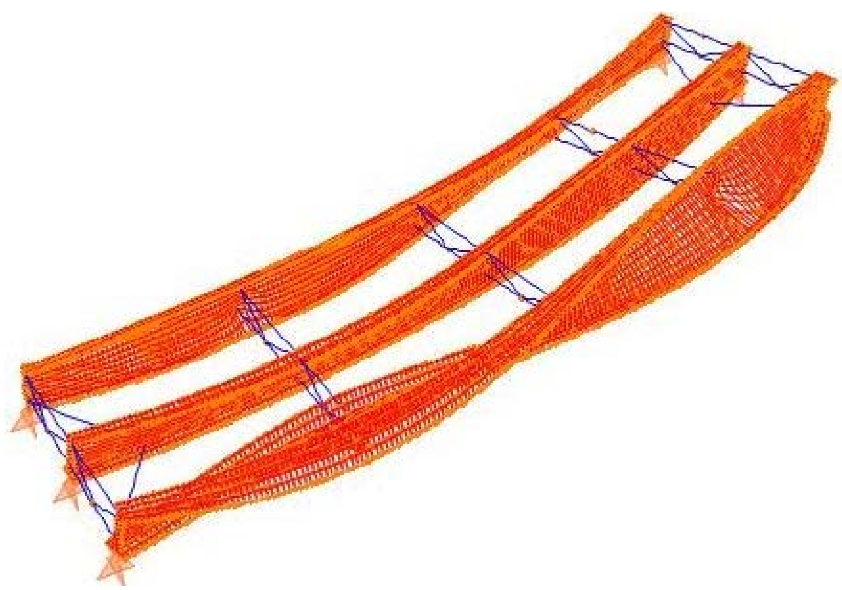

**Figure 14d. Shell Element Model Mode 4**

Frequencies and normalized mode shapes for the shell element model with composite concrete deck are provided in Table 3, with mode shapes shown in Figures 15a-d.

**Table 3. Mode Shapes and Frequencies for Shell Model With Deck**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 |
|---|---|---|---|---|---|
| H1MT | -0.66 | 0.39 | 0.00 | 0.00 | 0.00 |
| H1MB | 0.00 | 1.00 | 0.00 | 0.00 | -0.03 |
| H3MB | 0.26 | 0.89 | 0.00 | 0.00 | 1.00 |
| H1QB | -0.09 | 0.45 | 1.00 | 1.00 | -0.02 |
| V2MB | 0.64 | 0.35 | 0.01 | 0.14 | 0.00 |
| V1QB | 0.55 | 0.32 | 0.03 | 0.13 | -0.01 |
| V3MB | 1.00 | 0.57 | 0.00 | 0.00 | 0.00 |
| V2QB | 0.64 | 0.35 | 0.01 | 0.14 | 0.00 |
| V1MB | 0.78 | 0.48 | 0.00 | 0.00 | -0.01 |
| Frequency (Hz) | 3.82 | 5.39 | 7.68 | 11.21 | 11.70 |

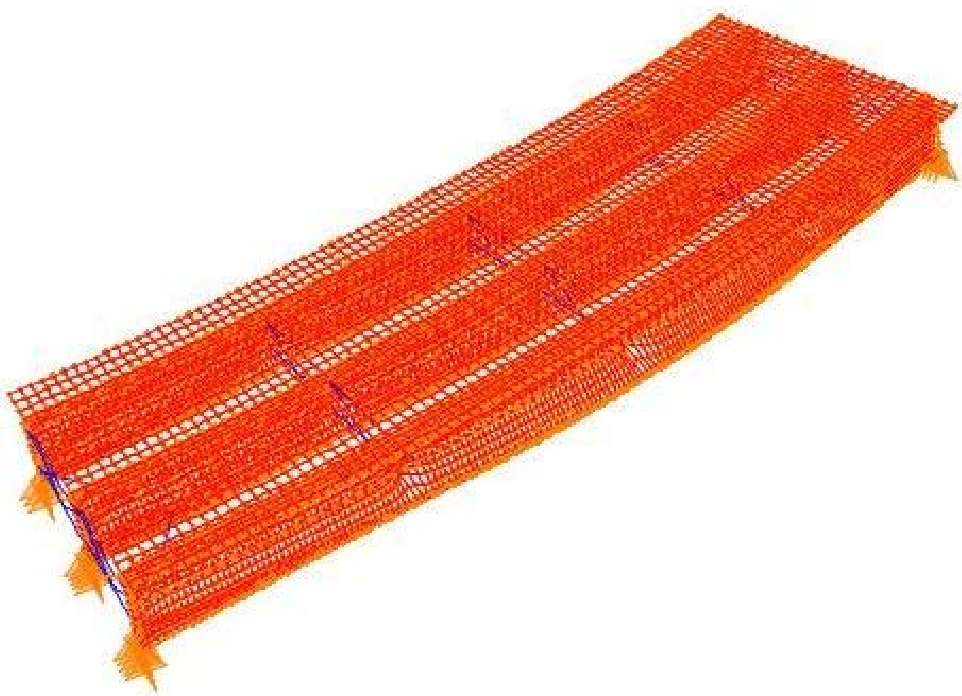

**Figure 15a. Shell Model With Deck: Mode 1**

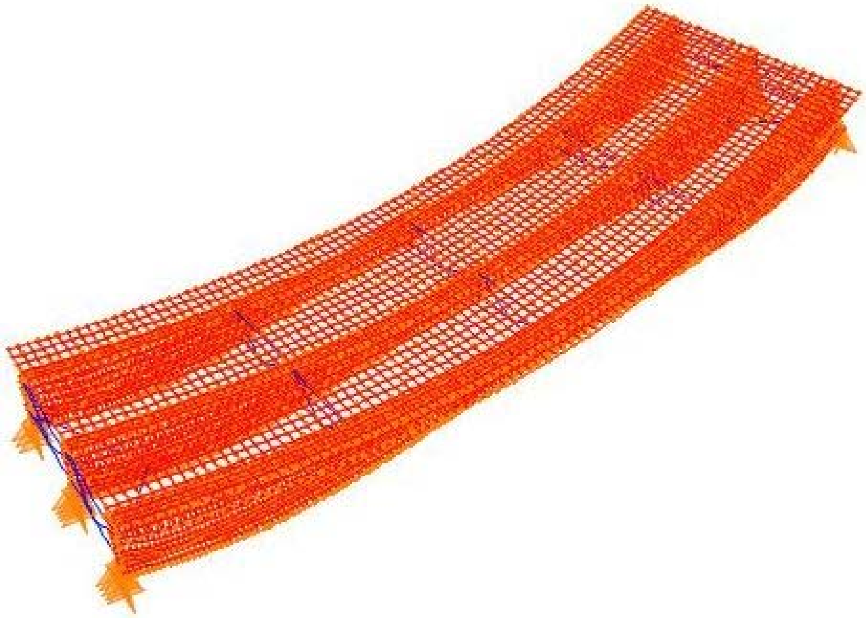

**Figure 15b. Shell Model With Deck: Mode 2**

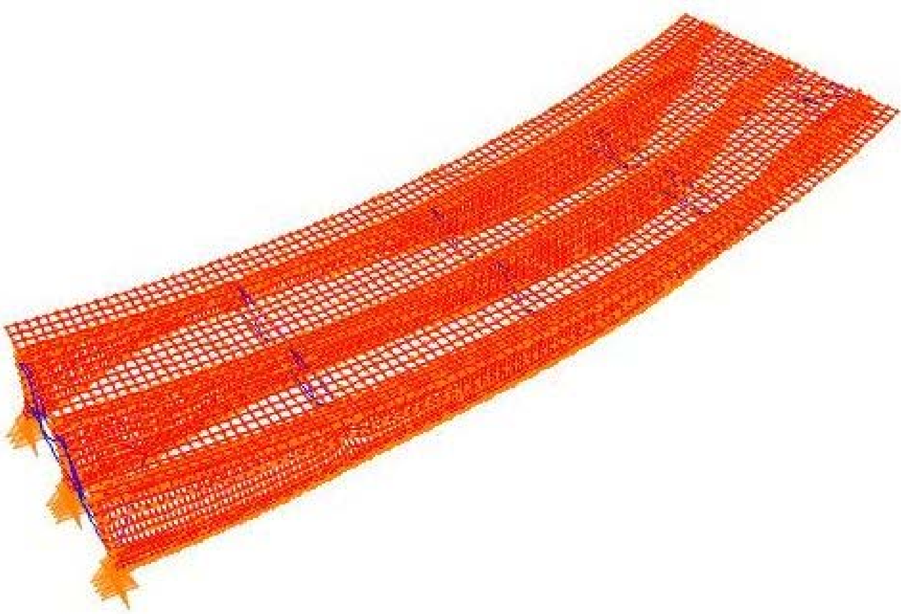

**Figure 15c. Shell Model With Deck: Mode 3**

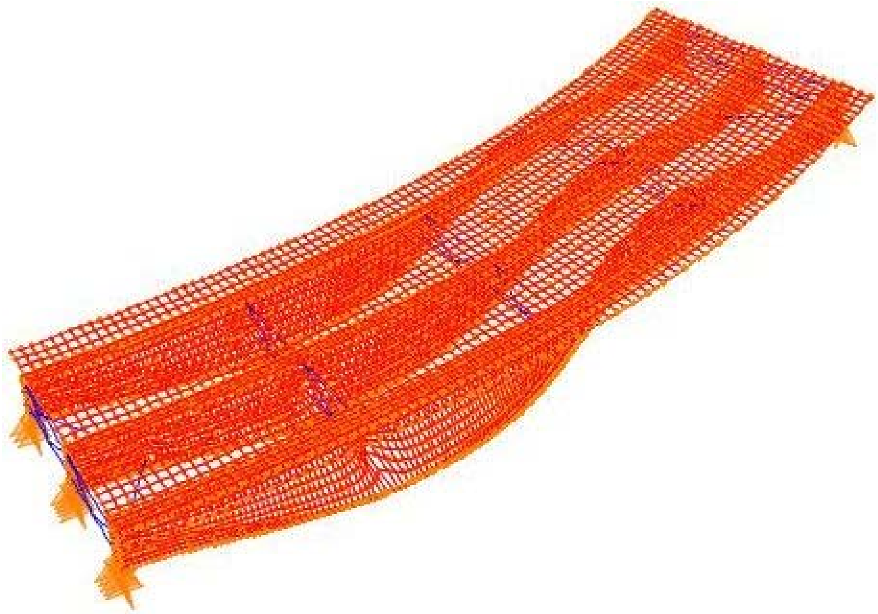

**Figure 15d. Shell Model With Deck: Mode 4**

## ANSYS Models

Two ANSYS models were used, one in which only the girders were modeled and one that included the composite deck. Frequencies and mode shapes were calculated for each model. Since the ANSYS models were able to represent the response of the curved girder bridge more accurately than the SAP models, it was anticipated that the ANSYS results would be more representative of the actual bridge.

For all of the modes considered, the mode shapes could be defined in terms of all of the nodal displacements. However, the only modal displacements tabulated and used for comparison purposes were those displacements at nodes corresponding to the location of the accelerometers in the collection of experimental data. Table 4 provides the frequencies and normalized mode shapes from the ANSYS model without the deck, and Figures 16a-d show the first four mode shapes.

**Table 4. Mode Shapes and Frequencies for ANSYS Model Without Deck**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 |
|---|---|---|---|---|---|
| H1MT | 1.00 | 0.01 | 0.04 | 0.23 | 1.00 |
| H1MB | 0.93 | 0.23 | -0.25 | -0.07 | 0.95 |
| H3MB | 0.93 | 0.23 | -0.25 | -0.07 | 0.95 |
| H1QB | 0.46 | 0.12 | -0.46 | 0.66 | -0.93 |
| V2MB | -0.07 | 0.52 | 0.38 | 0.35 | -0.01 |
| V1QB | 0.06 | 0.04 | 0.71 | 0.71 | 0.07 |
| V3MB | 0.22 | 1.00 | -0.24 | -0.29 | -0.11 |
| V2QB | -0.05 | 0.37 | 0.28 | 0.35 | -0.01 |
| V1MB | 0.08 | 0.05 | 1.00 | 1.00 | 0.10 |
| Frequency (Hz) | 2.05 | 3.32 | 5.61 | 5.81 | 10.37 |

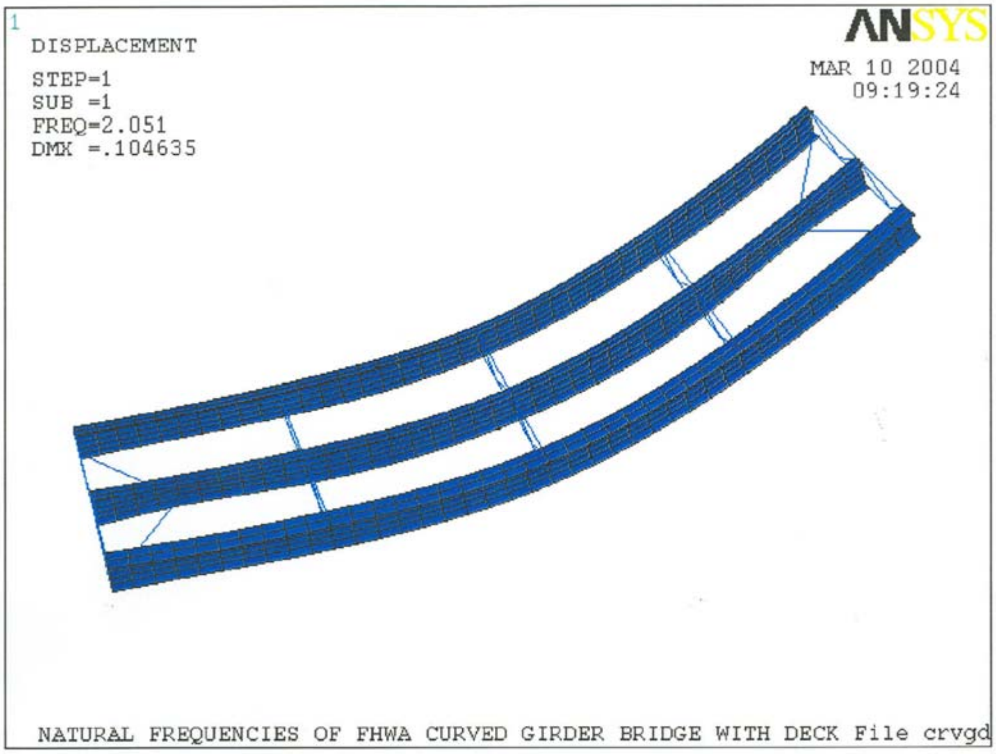


**Figure 16a. ANSYS Girder Model: Mode 1**

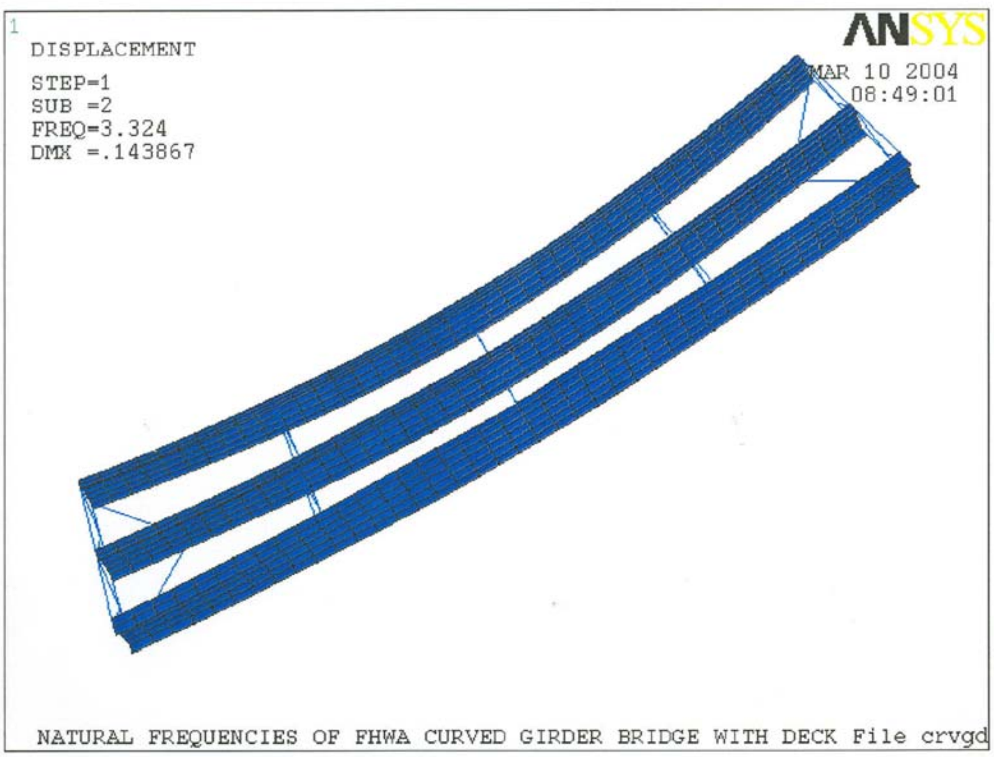


**Figure 16b. ANSYS Girder Model: Mode 2**

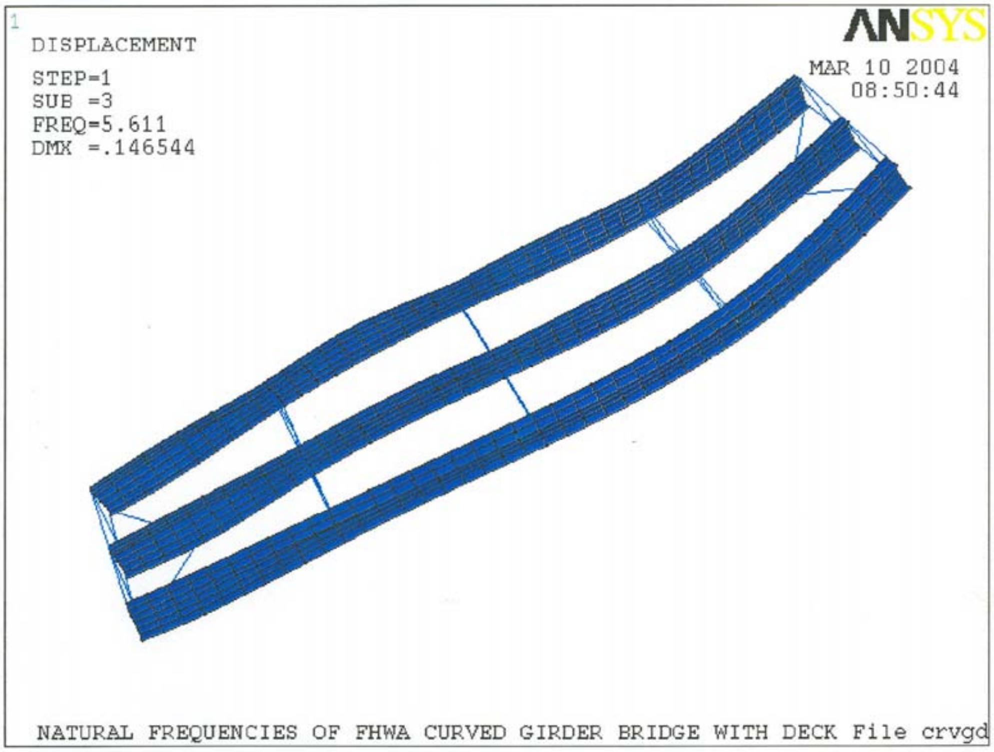


**Figure 16c. ANSYS Girder Model: Mode 3**

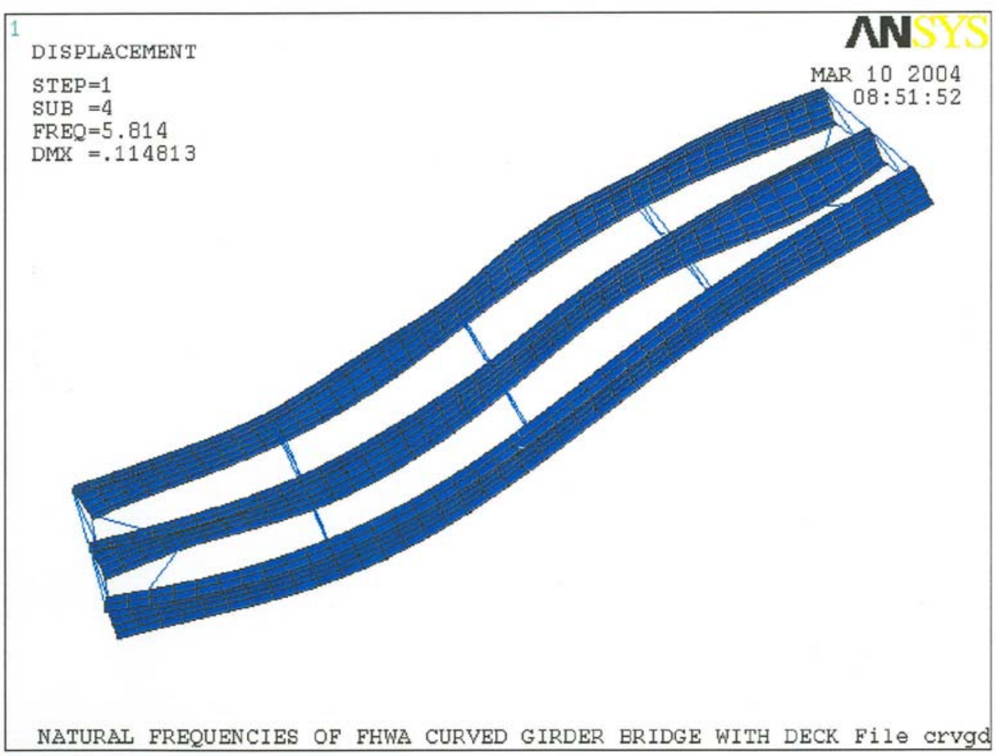


**Figure 16d. ANSYS Girder Model: Mode 4**

Normalized mode shapes and natural frequencies for the first four modes of the ANSYS model that included the composite deck are given in Table 5. Graphical representations of the first four mode shapes are shown in Figures 17a-d.

**Table 5. Mode Shapes and Frequencies for ANSYS Composite Model**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 |
|---|---|---|---|---|
| H1MT | 0.07 | -0.07 | -0.24 | -0.17 |
| H1MB | 0.25 | -0.31 | -0.51 | 1.00 |
| H3MB | 0.26 | -0.28 | -0.51 | -0.43 |
| H1QB | 0.15 | 1.00 | 0.32 | -0.22 |
| V2MB | 0.60 | -0.30 | 0.41 | 0.38 |
| V1QB | 0.18 | -0.13 | 0.66 | 0.59 |
| V3MB | 1.00 | -0.31 | 0.18 | -0.20 |
| V2QB | 0.42 | 0.20 | 0.31 | -0.22 |
| V1MB | 0.20 | -0.07 | 1.00 | -0.42 |
| Frequency (Hz) | 3.28 | 6.09 | 12.09 | 13.10 |

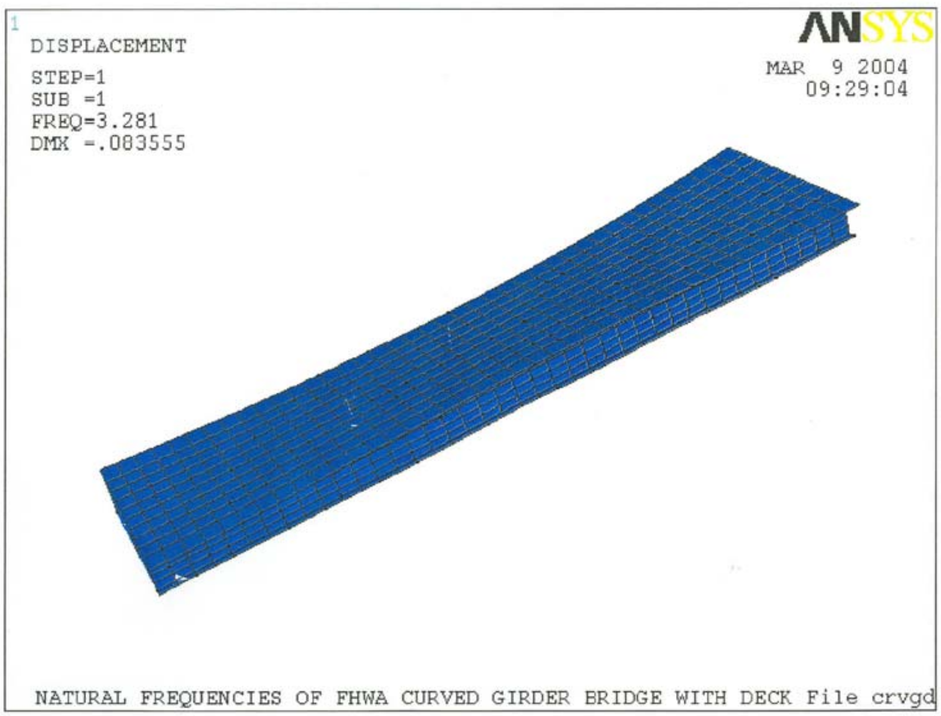


**Figure 17a. ANSYS Composite Model: Mode 1**

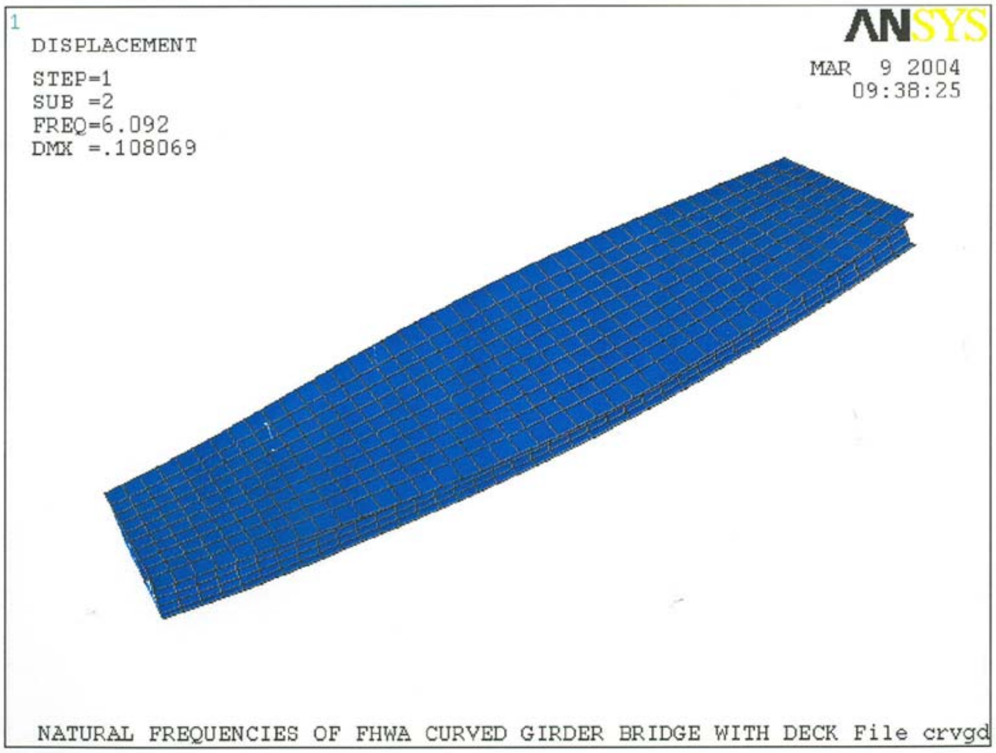


**Figure 17b. ANSYS Composite Model: Mode 2**

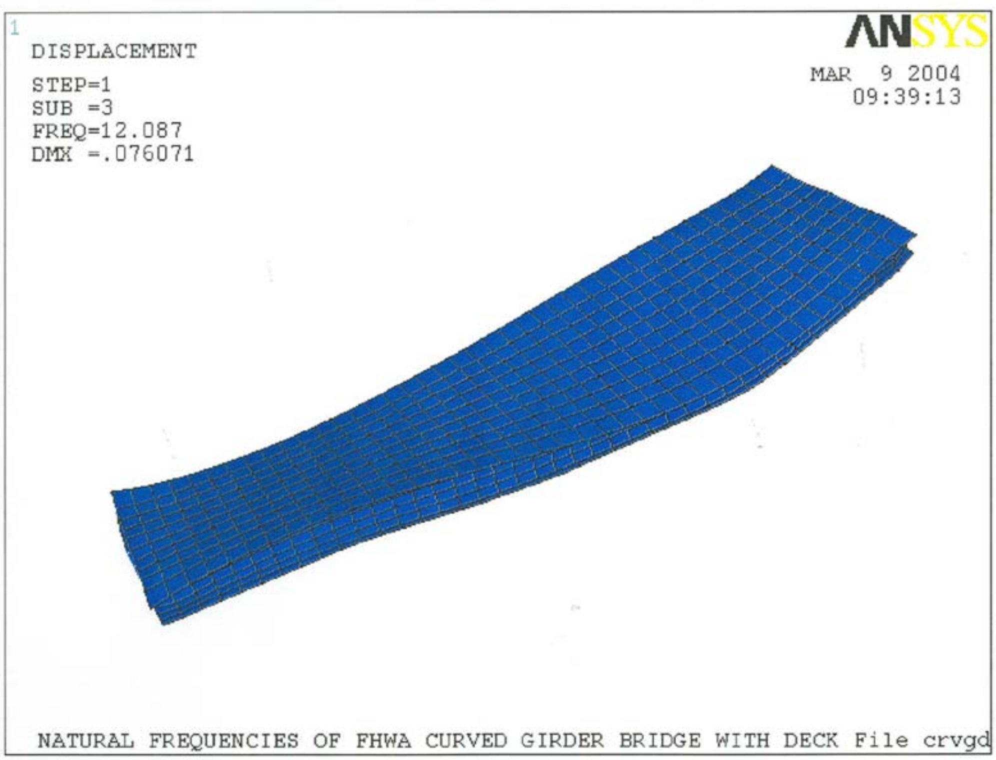


**Figure 17c. ANSYS Composite Model: Mode 3**

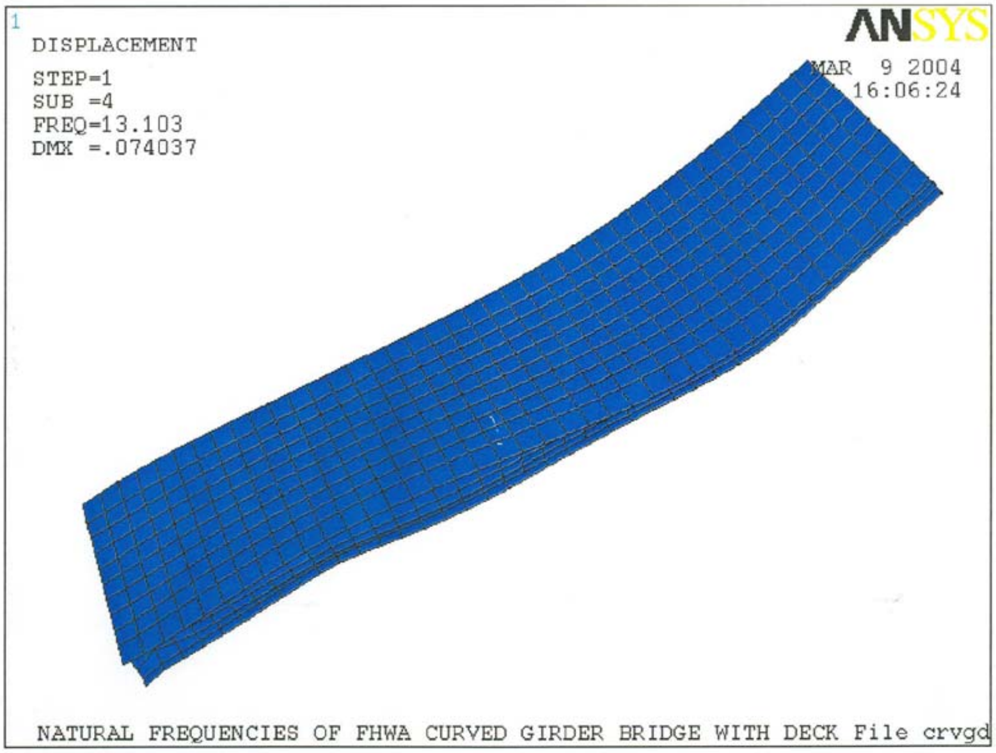


**Figure 17d. ANSYS Composite Model: Mode 4**

## ABAQUS Model

Although efforts were made to obtain an ABAQUS model including the concrete deck, and information available suggested that such a model existed, no composite ABAQUS model was available at the time of publication. Therefore, the only results from an ABAQUS model included in this report are those of the bare girder model provided by the FHWA contractors. The first five frequencies for this model are given in Table 6.

Although quantitative mode shapes were not available for this model, the first four mode shapes for this model are shown in Figures 18a-d. When considering the ABAQUS model mode shapes, it should be noted that the ABAQUS model was restrained at the supports against horizontal transverse and horizontal lateral deflection only on Girder 2, with Girders 1 and 3 restrained only against vertical deflection. As a result, the model displayed somewhat lower frequencies than those developed in other finite element codes.

**Table 6. Natural Frequencies from Abaqus Model**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 |
|---|---|---|---|---|---|
| Frequency (Hz) | 1.86 | 3.49 | 3.63 | 6.31 | 6.70 |

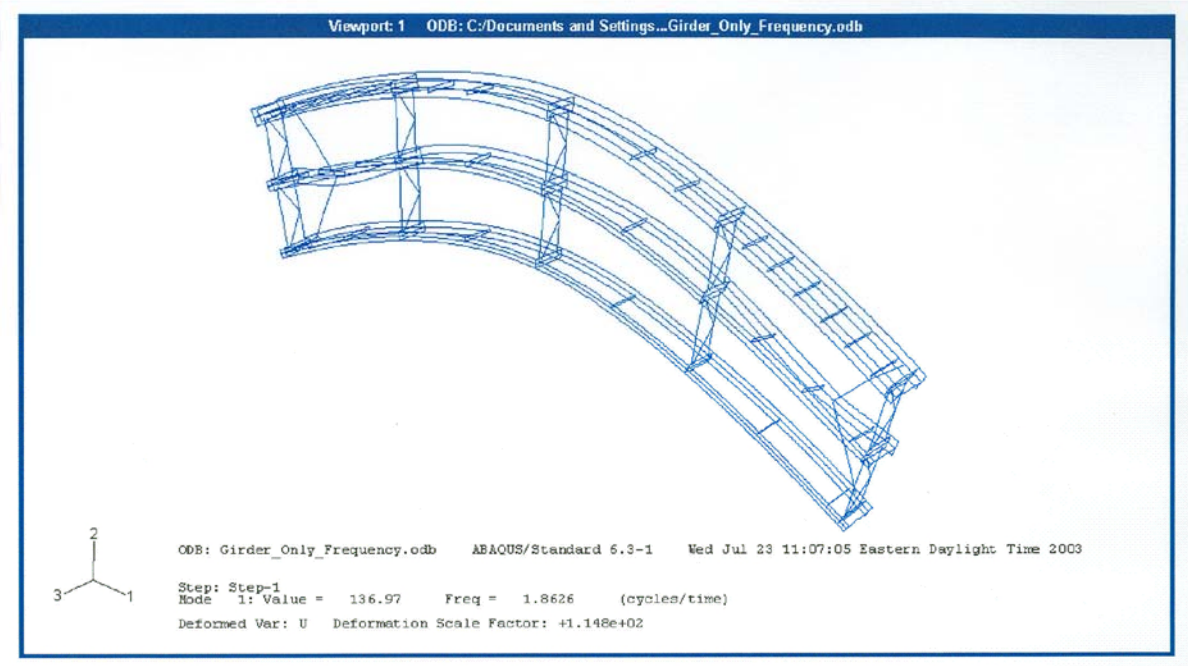


**Figure 18a. ABAQUS Model: Mode 1**

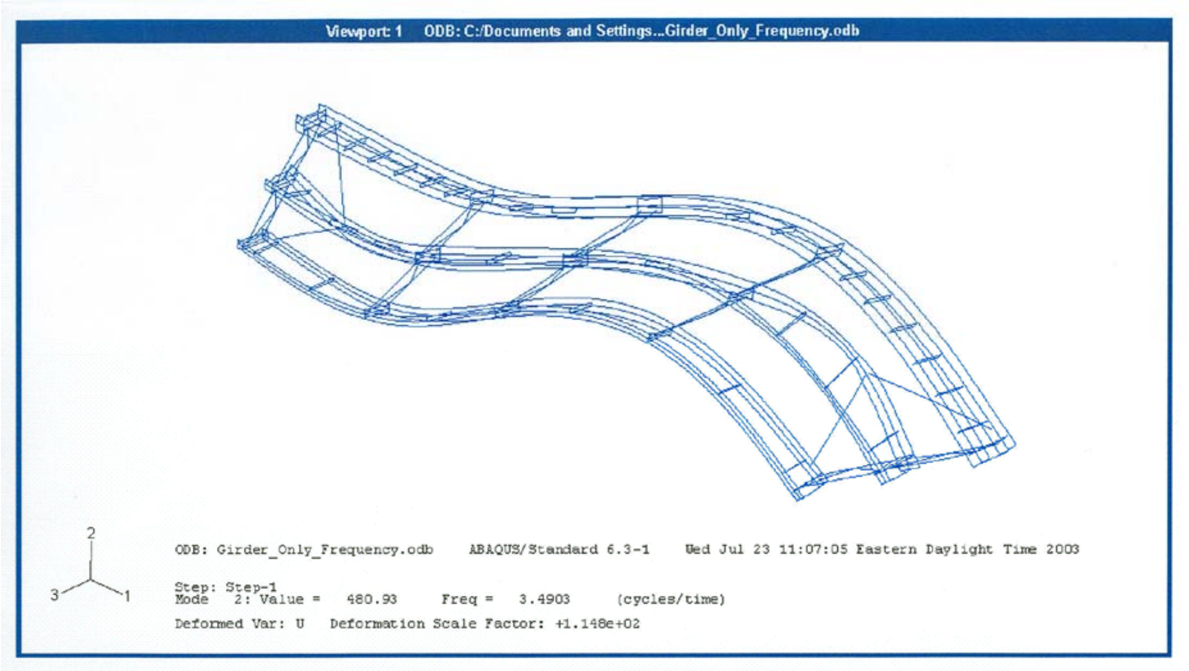


**Figure 18b. ABAQUS Model: Mode 2**

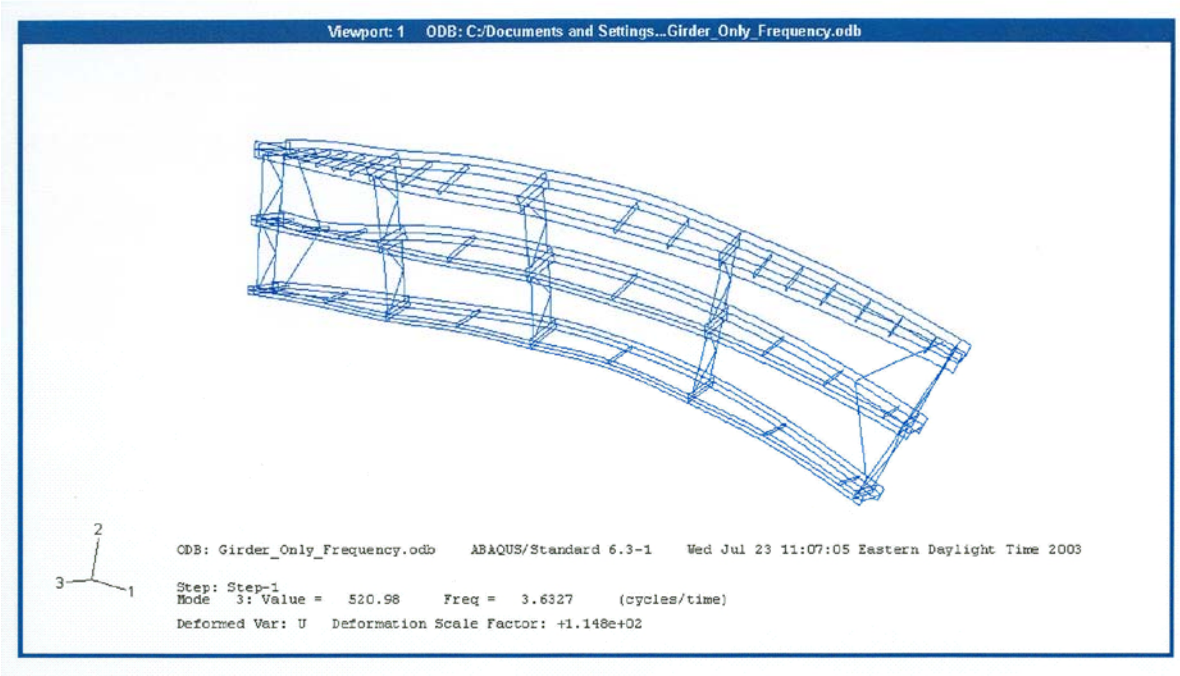


**Figure 18c. ABAQUS Model: Mode 3**

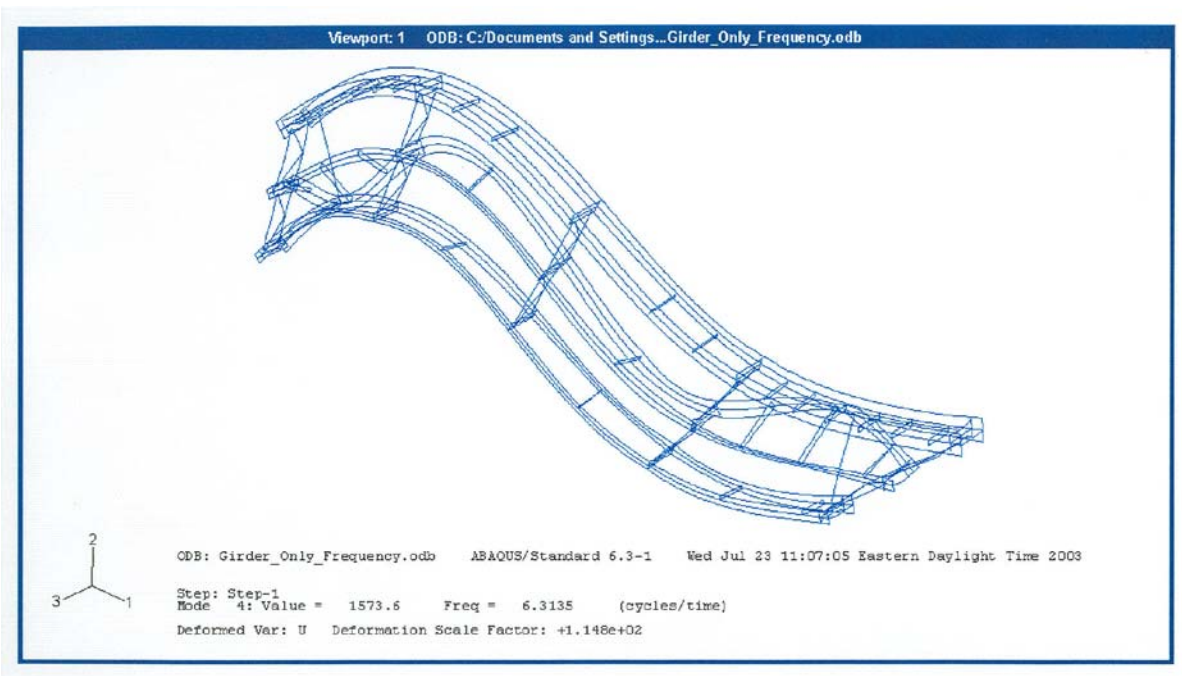


**Figure 18d. ABAQUS Model: Mode 4**

## Experimental Results

The data acquisition system used for recording frequencies and displacement amplitudes during the experimental phase of the study allowed for data to be recorded at only eight points along the structure. Thus, the mode shapes from any of the experimental tests were defined by only limited knowledge of the total deformation of the bridge. This made it extremely difficult to define the precise geometry of a particular mode and significantly limited comparison of mode shapes between computational and experimental results.

Results from the experiments included frequencies and modal amplitudes. Frequencies could be taken from the recorded test data or recorded during the test procedure. Mode shapes were found by performing an FFT on the time domain data to obtain frequency domain data as discussed previously. This resulted in time domain mode shapes with displacement known at all sensor locations.

### Test 1: Bridge Without Deck

Frequency and mode shape data from the bridge without a composite deck were recorded during the test on August 19, 2003. Frequencies from the first test were measured with the shaker oriented both horizontally and vertically. With the shaker oriented vertically, the first three frequencies were 2.46 Hz, 3.86 Hz, and 6.86 Hz. With the shaker oriented horizontally, the first three frequencies were 2.43 Hz, 5.32 Hz, and 11.40 Hz. The lowest frequencies in each case were essentially identical, and further data analysis showed that these frequencies corresponded to the same vibration mode.

Mode shapes were obtained for the first five frequencies of vibration. These included data from all eight sensors used for this test. The mode shapes were normalized with respect to the largest absolute measured displacement of any accelerometer. This normalized data are shown in Table 7.

Modes 1 and 2 correspond to the essentially identical frequencies mentioned previously. The similarity of the mode shapes further confirms that these frequencies correspond to the same mode of vibration.

**Table 7. Test 1 Frequencies and Mode Shapes**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 |
|---|---|---|---|---|---|
| H1MT | 0.75 | 0.72 | 0.04 | 0.21 | 0.07 |
| H1MB | 0.96 | N/A | N/A | 0.04 | N/A |
| H3MB | 1.00 | 1.00 | 0.19 | 0.04 | 0.08 |
| H1QB | 0.64 | 0.70 | 0.15 | 1.00 | 0.04 |
| V2MB | 0.00 | 0.01 | 0.02 | 0.00 | 0.02 |
| V1QB | 0.01 | 0.02 | 0.01 | 0.04 | 0.69 |
| V3MB | 0.47 | 0.47 | 1.00 | 0.01 | 0.16 |
| V2QB | 0.12 | 0.14 | 0.03 | 0.02 | 0.30 |
| V1MB | N/A | 0.06 | 0.34 | N/A | 1.00 |
| Frequency (Hz) | 2.43 | 2.46 | 3.86 | 5.32 | 6.86 |

### Test 2: Bridge with Deck

The bridge model with the deck was tested on January 22, 2004. As in the previous test, recorded data included frequencies and modal amplitudes. Frequencies and mode shapes were again measured using both horizontal and vertical orientations of the shaker. With the shaker in a vertical orientation, the first three frequencies were 3.21 Hz, 12.71 Hz, and 23.72 Hz. With the shaker in the horizontal orientation, the first three frequencies were 6.50 Hz, 12.43 Hz, and 21.31 Hz.

Only the first four frequencies and mode shapes were recorded for each shaker orientation. It was felt that response above the fourth mode would likely be of less importance than lower mode shapes. Thus, no mode shape data are considered here for the higher modes of the structure. Table 8 shows the normalized mode shape data. From the data in Table 8, it may be seen that the third and fourth frequencies in this test were quite similar and the difference in the mode shapes is clearly obvious. This would indicate that although the first two modes of the first test were the same, the third and fourth modes of the second test were clearly distinct modes of vibration.

**Table 8. Test 2 Frequencies and Mode Shapes**

| Accelerometer No. | Mode 1 | Mode 2 | Mode 3 | Mode 4 |
|---|---|---|---|---|
| H1MT | 0.02 | 1.00 | 0.07 | 0.04 |
| H1MB | 0.28 | 0.86 | 0.57 | 0.11 |
| H3MB | 0.03 | 0.05 | 0.08 | 0.05 |
| H1QB | N/A | 0.85 | 0.24 | N/A |
| V2MB | 0.68 | 0.10 | 0.46 | 0.05 |
| V1QB | 0.17 | N/A | N/A | 0.40 |
| V3MB | 1.00 | 0.02 | 0.07 | 0.15 |
| V2QB | 0.49 | 0.04 | 0.28 | 1.00 |
| V1MB | 0.24 | 0.42 | 1.00 | 0.02 |
| Frequency (Hz) | 3.21 | 6.50 | 12.43 | 12.71 |

## Comparison of Experimental and Computational Test Data

To meet the goals of this research project, it was necessary to compare responses from the various computational models and to compare computational responses to experimental test results. The primary emphasis is on comparison of natural frequencies because of the uncertainties in mode shapes and because frequency data are more definitive.

### Frequency Comparison

Each computational model and each experimental test generated at least five frequencies. These first five frequencies for all bare girder models and the bare girder test are summarized in Table 9. As discussed previously, the first two test frequencies were almost identical, and subsequent evaluation of the mode shapes indicated that these were, in fact, the same mode of vibration. Consequently, those two frequencies are shown in Table 9 as Frequency 1 and only four frequencies are shown for Test 1.

**Table 9. Girder Model Frequencies (Hz)**

| Model | Frequency 1 | Frequency 2 | Frequency 3 | Frequency 4 | Frequency 5 |
|---|---|---|---|---|---|
| SAP Shell | 2.02 | 4.91 | 5.09 | 6.99 | 7.68 |
| ANSYS | 2.05 | 3.32 | 5.61 | 5.81 | 10.37 |
| ABAQUS | 1.86 | 3.49 | 3.63 | 6.31 | 6.70 |
| Test 1 | 2.44 | 3.86 | 5.32 | 6.86 | |

Examination of the frequencies in Table 9 indicates that the fundamental frequencies predicted by all of the computational models were in reasonably good agreement, predicting a frequency of approximately 2 Hz, with the ANSYS and SAP Shell models predicting essentially the same frequency.

The similarity, however, did not carry over to the higher frequencies. For example, predictions of the second frequency from ANSYS and ABAQUS were closer to the test frequency than those predicted by other models, but for the third frequency, ANSYS and the SAP Shell model were better predictors. For the fourth frequency, the SAP Shell model and ABAQUS appeared to be better predictors. Above the fourth frequency, there was little agreement between any of the predicted or measured frequencies.

For comparisons with the experimental data, Table 10 includes the percentage difference between frequencies recorded during the experimental tests and frequencies predicted by the computational models. From Table 10, it appears that the ANSYS model is the best overall predictor and the SAP Shell model is more accurate for the third and fourth frequencies.

A similar comparison was made between measured and predicted frequencies for the bridge with the concrete deck. The three computational models included the SAP beam and shell models and the ANSYS model. Frequencies from these three models, together with the measured experimental frequencies, are included in Table 11. It can be seen that all three models predict similar values of the first frequency. It is also of interest to note that the models all remain fairly consistent in predicting the second frequency and are fairly close to the frequency results found in the second experimental test. A similarity between the computer model frequencies and the test frequencies becomes less readily apparent at higher frequencies.

Table 12 includes a percentage comparison between the three models and the Test 2 frequencies. In this case, the ANSYS model again seems to be more consistent in predicting frequencies over the entire frequency range.

**Table 10. Percentage Difference Between Test Frequencies and Predicted Frequencies for Bare Girder Model**

| Model | Frequency 1 | Frequency 2 | Frequency 3 | Frequency 4 |
|---|---|---|---|---|
| SAP Shell | -17.2 | 27.20 | -4.32 | -1.90 |
| ANSYS | -15.98 | -13.99 | 5.45 | -15.31 |
| ABAQUS | -23.77 | -9.59 | -31.77 | -8.02 |

**Table 11. Composite Model Frequencies (Hz)**

| Model | Frequency 1 | Frequency 2 | Frequency 3 | Frequency 4 |
|---|---|---|---|---|
| SAP Shell | 3.82 | 5.39 | 7.68 | 11.21 |
| ANSYS | 3.28 | 6.09 | 12.09 | 13.10 |
| Test 2 | 3.21 | 6.50 | 12.43 | 12.71 |

**Table 12. Frequency Comparison Between Composite Models**

| Model | Frequency 1 | Frequency 2 | Frequency 3 | Frequency 4 |
|---|---|---|---|---|
| % Difference from Test 2 Frequencies | | | | |
| SAP Shell | 19.0 | -17.1 | -38.2 | -11.8 |
| ANSYS | 2.2 | -6.3 | -2.7 | 3.1 |

## Comparison of Mode Shapes

### *Comparison of Frequencies*

Comparison of frequencies is relatively straightforward and can be useful in evaluating the effectiveness of computer models to predict dynamic response. Mode shapes generated by computer models may be even more useful in evaluating how well models can predict dynamic response since these shapes contain detailed information of displacements throughout the structure. However, in this study, the mode shape information obtained from the two laboratory tests was sketchy at best since displacement measurements defining the mode shapes were recorded at only eight locations on the structure. Although still of interest, the test data were insufficient to provide a reliable basis for evaluating the accuracy of mode shape predictions from the various computer models. Nevertheless, an attempt was made to compare mode shapes from the models to the measured mode shapes by comparing displacements recorded at the locations where accelerometers were placed in the two laboratory tests. In addition, mode shapes from the computer models could be evaluated subjectively by comparing the similarity of the plotted mode shapes available from the software used.

### *Numerical Comparison*

The numerical comparisons between the mode shapes found computationally with those found experimentally should be considered approximations at best. The displacements defining the mode shapes were normalized such that the largest value was 1.0. Thus, all shapes were artificially shifted to achieve this normalization. Nevertheless, it is possible to get a subjective sense of which models can more closely represent the shapes measured experimentally.

Comparison between the computational models and experimental tests for Mode 1 are presented in Table 13. To construct this table, the first and second modes of the test were averaged to gain an average value for the test displacements.

For this mode, there was some similarity between the ANSYS mode shape and the test mode shape. The SAP models do show some consistency, though neither seems to be close to the mode shape of the experimental data.

A similar comparison for the second mode is presented in Table 14. Again, it may be observed that the second mode shape predicted by the ANSYS model appears to resemble the

**Table 13. Mode 1 Mode Shapes: Bare Girder Model**

| Accelerometer No. | Test 1 Mode 1 & 2 | SAP Beams Model | SAP Shells Model | ANSYS Model |
|---|---|---|---|---|
| H1MT | 0.73 | 0.00 | -0.01 | 1.00 |
| H1MB | 0.96 | 0.00 | -0.01 | 0.93 |
| H3MB | 1.00 | 0.00 | 0.01 | 0.93 |
| H1QB | 0.67 | 0.00 | 0.67 | 0.46 |
| V2MB | 0.01 | 0.41 | 0.89 | -0.07 |
| V1QB | 0.02 | -0.13 | 0.38 | 0.06 |
| V3MB | 0.47 | 1.00 | 1.00 | 0.22 |
| V2QB | 0.13 | 0.29 | 0.56 | -0.05 |
| V1MB | 0.06 | -0.18 | 0.55 | 0.08 |

**Table 14. Mode 2 Mode Shapes: Bare Girder Model**

| Accelerometer No. | Test 1 Mode 3 | SAP Beams Model | SAP Shells Model | ANSYS Model |
|---|---|---|---|---|
| H1MT | 0.04 | 0.00 | 0.00 | 0.01 |
| H1MB | N/A | 0.00 | 0.00 | 0.23 |
| H3MB | 0.19 | 0.00 | 0.00 | 0.23 |
| H1QB | 0.15 | 0.00 | 0.00 | 0.12 |
| V2MB | 0.00 | 0.48 | 0.05 | 0.52 |
| V1QB | 0.01 | 0.70 | 0.60 | 0.04 |
| V3MB | 1.00 | -0.05 | 0.00 | 1.00 |
| V2QB | 0.03 | 0.33 | 1.00 | 0.37 |
| V1MB | 0.34 | 1.00 | 0.00 | 0.05 |

mode shape from the test more closely than do the other computer models. Another way to illustrate the comparison of mode shapes is to examine the difference between the displacement magnitudes of the computational and experimental shapes. This was done for Mode 1 by plotting the difference in displacement magnitudes presented in Table 13; this plot is given in Figure 19. The plot may be somewhat misleading because the x-axis simply represents different accelerometers, not location distances.

Further information and discussion regarding the higher modes are not presented here since it was apparent that a similarity between experimental and computational mode shape data was questionable for lower mode shapes and any agreement at higher modes was unlikely.

Mode shapes were also compared for the structure with the deck in place. Once again, mode shapes for the three computer models under study were considered along with the mode shapes recorded from the second experimental test. The comparison for the first mode of vibration is provided in Table 15.

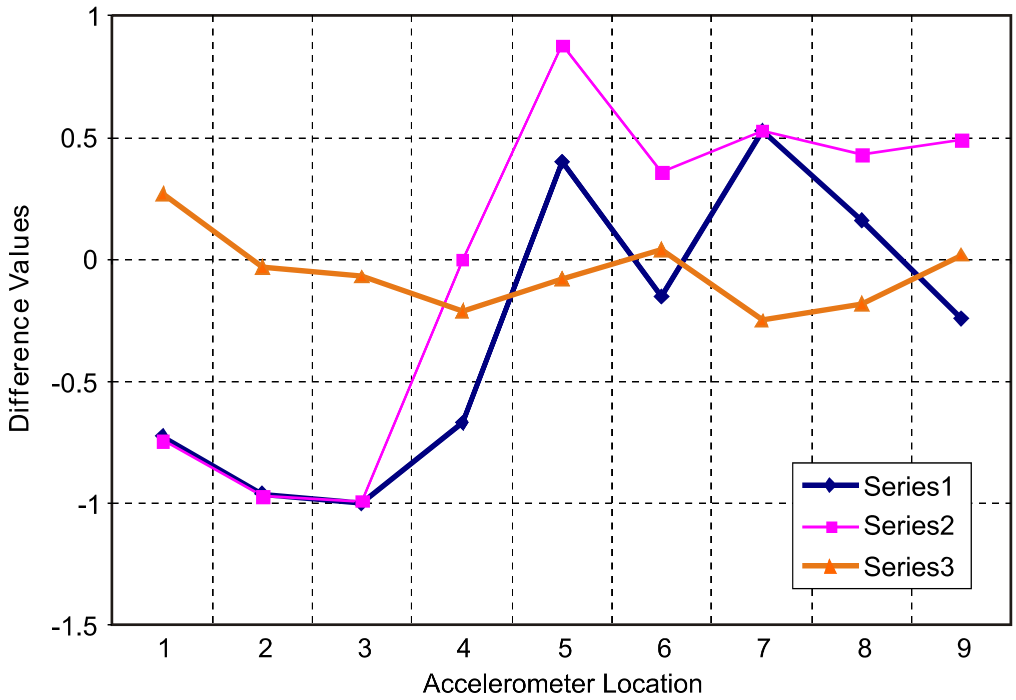


**Figure 19. Difference Between Calculated and Measured Displacements: Mode 1 Bare Girder Model**

**Table 15. Mode 1 Composite Mode Shapes**

| Accelerometer No. | TEST 2 Mode 1 | SAP Beams Model | SAP Shells Model | ANSYS Model |
|---|---|---|---|---|
| H1MT | 0.02 | -0.04 | -0.66 | 0.07 |
| H1MB | 0.28 | 0.00 | 0.00 | 0.25 |
| H3MB | 0.03 | 0.01 | 0.26 | 0.26 |
| H1QB | N/A | 0.00 | -0.09 | 0.15 |
| V2MB | 0.68 | -0.86 | 0.88 | 0.60 |
| V1QB | 0.17 | 0.51 | 0.55 | 0.18 |
| V3MB | 1.00 | 1.00 | 1.00 | 1.00 |
| V2QB | 0.49 | 0.59 | 0.64 | 0.42 |
| V1MB | 0.24 | 0.73 | 0.78 | 0.20 |

From the data in Table 15, it can be seen that all three models have mode shapes that seem to have the largest displacement value at the same location, namely the location of accelerometer V3MB. This implies that large bending is occurring in this mode between all three models and the experimental test. In addition, the mode shapes are quite similar at other points, with nearly all having very similar vertical displacements at multiple accelerometers. Horizontal accelerometers become less accurate between the models and tests, but these are much less important, as this mode shape for the experimental structure was found with the shaker in a vertical orientation. Based on this, it can be concluded that all these mode shapes are probably representing the same mode of vibration of the structure

As was done for the first mode of the bare girder model, the computed mode shapes can also be described in terms of the differences between the computed normalized displacements of the computed modes and those measured in the experimental tests. A plot of these differences for the Mode 2 mode shapes is presented in Figure 20.

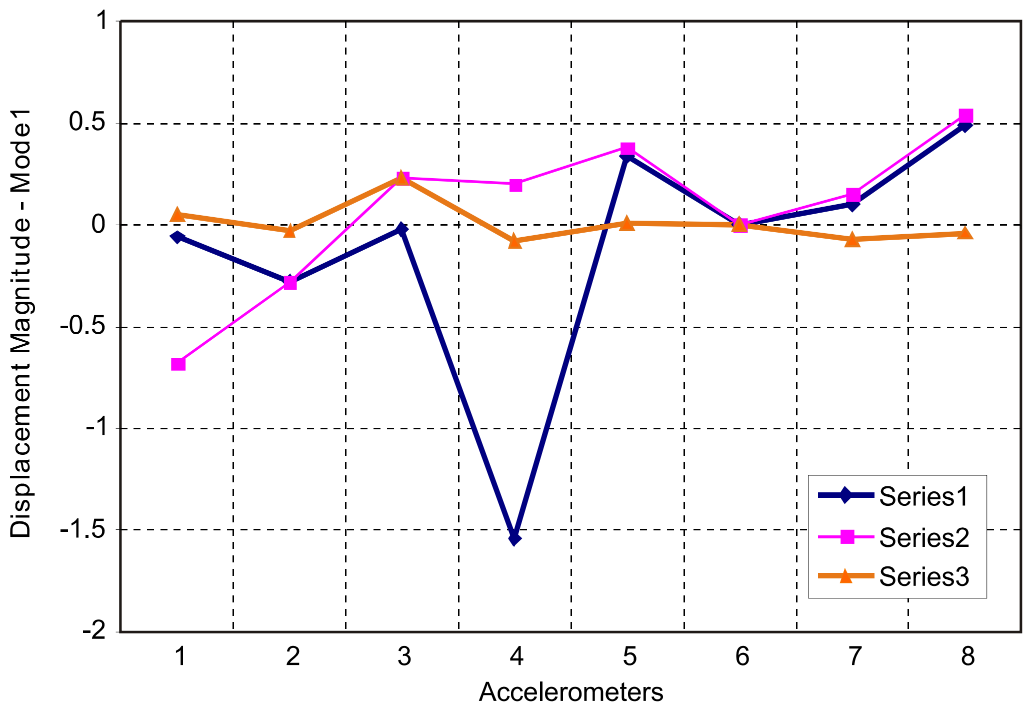


**Figure 20. Difference Between Calculated and Measured Displacements: Mode 1 Model with Deck**

From this plot, it is evident that, to some extent, the mode shapes predicted by all of the models closely approximate the measured shape. However, the mode shape predicted by the ANSYS model is clearly the best approximation of the mode shape measured in Test 2. It is likely that the ability of the various models to approximate the frequencies and mode shapes as measured from the experimental test is due to the presence of the deck, which dominates the manner in which the bridge responds. In the bare girder model, despite the presence of the cross bracing, the relative flexibility of the structure resulted in deformations of the girders, and consequently mode shapes, that were very difficult to represent from the relatively simple computer models.

The computed mode shapes for Mode 2 can be compared to the measured shapes in a similar manner. This comparison is provided in Table 16. Mode shape similarities are not as readily apparent as for Mode 1. In fact, for Mode 2, there is little similarity between any of the computed mode shapes and the measured response. A comparison between the models and test results for the higher modes can be performed in a similar manner but is not discussed further because of the lack of meaningful information provided.

**Table 16. Mode 2 Composite Mode Shapes**

| Accelerometer No. | TEST 2 Mode 2 | SAP Beams Model | SAP Shells Model | ANSYS Model |
|---|---|---|---|---|
| H1MT | 1.00 | -0.15 | 0.39 | -0.07 |
| H1MB | 0.86 | -0.41 | 1.00 | -0.30 |
| H3MB | 0.05 | -0.42 | 0.89 | -0.13 |
| H1QB | 0.85 | -0.27 | 0.45 | -0.31 |
| V2MB | 0.10 | 0.13 | 0.52 | -0.07 |
| V1QB | N/A | 0.69 | 0.32 | -0.28 |
| V3MB | 0.02 | -0.77 | 0.57 | 0.20 |
| V2QB | 0.04 | 0.10 | 0.35 | 1.00 |
| V1MB | 0.42 | 1.00 | 0.48 | -0.31 |

## DISCUSSION

The results of this study give an idea of how effective SAP2000 can be for the dynamic analysis of curved girder bridges. In both the composite and noncomposite structures, the SAP2000 beam and shell element models came within 20 to 30 percent of most frequencies of vibration. Major exceptions within the first three frequencies included the third frequency for the composite shell model, the first frequency for the composite beam model, and the third frequency for the noncomposite beam model. An average of the results of the two SAP models came within 20 percent of the test frequency in all cases except for the first frequency of the composite structure. The results of SAP models also mostly fell within about 20 percent of the results predicted by ANSYS models. In this particular case, the results of the shell model came much closer to those of the ANSYS model in the majority of cases. When mode shapes were considered, the SAP models were not quite as valuable. Although the data from both SAP models seemed to predict the first mode shape of the composite structure, they did not seem similar on most other shapes to either the ANSYS or test data.

Several sources of error may have adversely affected these results. Within the test itself, there was little knowledge about what types of testing were performed on the bridge structure by others before these tests were conducted. Further, any other information about changes to the structure was not available to the project team. A further testing issue was encountered with the 302B03 accelerometers used to measure the horizontal displacements of the structure. A significant amount of noise was encountered from these accelerometers during the testing of the composite structure. This noise was not as significant with the bare girder structure, in which the signal strength was great enough to overcome the noise. Increasing the gain in the composite test did allow some signal to get through but also amplified the noise.

Other problems may have occurred as a result of the methods used to model the bridge. The SAP beam model in and of itself accepts several limitations by the use of beam elements. Deformation of the girders themselves is impossible in this model because of the simplifications included. In addition, the use of rigid links totaling more than one-half the girder's depth plus more than one-half the web depth probably leads to a structure that is too stiff in the primary bending direction. This is probably to blame for the excessively high first frequency encountered within the composite SAP beam model. A final source of error in this model is that the cross bracing must be assumed to be a single beam rather than a multi-beam frame. This likely leads to compromises in the accuracy of this model. The SAP shell and ANSYS models also accept compromises. These models lack the web stiffeners that are present in the actual structure. Web stiffeners were excluded because of limited knowledge about their orientation and size and because of the complexity in modeling these stiffeners. This problem may have caused the odd mode shapes seen in the SAP composite shell model.

## SUMMARY

FHWA research into curved girder bridge behavior is ongoing via the Curved Steel Bridge Research Project. Previous studies evaluated static behavior and behavior during construction, which led to an exceptional volume of research in the various areas of curved girder bridge behavior. The current study successfully developed and evaluated computer models for dynamic analysis of a curved girder bridge by comparing the results of models and experimental results.

Experimental tests were conducted on the Turner-Fairbank curved girder bridge to determine the lower frequencies of vibration. The first test was completed using the bare girder structure without inclusion of the concrete deck. The second test was conducted after placement of the deck. Results from these tests were analyzed and extracted from data recorded on the test dates.

Basic and advanced SAP models and additional models using ANSYS were developed to predict the dynamic response of the tested bridge. An ABAQUS model developed by FHWA contractors was modified to obtain dynamic response data. All SAP and ANSYS models were modified to add a composite concrete deck to the models used to analyze the bare girder case.

Models used in all three finite element codes were compared and contrasted with each other and with the results of the experimental tests.

## CONCLUSIONS

- Comparisons of the SAP model results and the experimental results indicated that the SAP analysis is adequate for the lower frequencies of vibration but not acceptable for higher frequencies.

- Although the SAP beam and SAP shell models are acceptable for low frequencies of vibration, the SAP shell model is generally more accurate.

- Based on comparison with test results, the ANSYS models appear to be more accurate than SAP models for low frequencies of vibration.

- When compared with test results, none of the finite element models used in this study was accurate for the higher frequencies of vibration.

- Limited mode shape data drawn from the experimental results did not support absolute conclusions about the effectiveness of SAP2000 or the other finite element programs for obtaining mode shapes.

- Limited mode shape displacement data developed in this test suggest that future tests should include more thorough mode shape information, including instrumentation of both half-spans of the bridge.

- Testing using the 302B03 accelerometers introduce a significant amount of noise into the experimental results.

- The limited availability of information about the characteristics of the bridge under study added an unnecessary element of uncertainty that could have been removed with more complete information.

## RECOMMENDATIONS

1. *A much more complete series of tests should be conducted to obtain better experimental mode shapes for an actual bridge. Such a study should include both numerical and graphical mode shapes for the composite bridge structure.*

2. *Consideration should be given to whether 302B03 accelerometers should be used along with the 393C accelerometers or whether it may be necessary to use only the latter type of accelerometers for a given test.*

3. *Additional work to validate SAP models should include dynamic response from vehicles, impact, or other real-world bridge loading.*

4. *Further validation of SAP models should include the application of harmonic loads at the location of the shaker on the actual bridge.* This would allow a more accurate simulation of the test than did the eigenanalysis used for this project and would allow the use of static rather than dynamic analysis routines. This would also avoid use of the SAP eigensolver that is internal to the program but not particularly well documented in the SAP user's manual.

5. *Further studies should be conducted with respect to the effects of secondary bridge parameters on the dynamic response of a curved girder bridge. The effects of cross bracing, diagonal bracing, deck connections, parapets, and other elements found on an actual structure should be evaluated.*

## COSTS AND BENEFITS ASSESSMENT

Because the purpose of this study was to develop a finite element model that could be used to predict the dynamic response of a curved girder bridge using SAP2000 software, assessment of the cost and benefits must be qualitative. A major benefit of this study was the successful attempt to calibrate the finite element model with the limited experimental study. The results and recommendations will aid bridge engineers who use finite element modeling to assist in the design and analysis of curved bridges. The cost savings will include the time saved in developing accurate finite element models of actual bridge structures using SAP2000. The results and recommendations from this study will also assist those engineers who conduct dynamic field tests on actual bridges.

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