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# PRESTRESSED CONCRETE GIRDER

# BRIDGES IN FLORIDA

BY

**MOUSSA A. ISSA**

STRUCTURAL ANALYST

**MOHSEN A. SHAHAWY**

CHIEF STRUCTURAL ANALYST

STRUCTURAL RESEARCH CENTER, MS 80

FLORIDA DEPARTMENT OF TRANSPORTATION

TALLAHASSEE, FLORIDA 32310

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# DYNAMIC AND STATIC TESTS OF PRESTRESSED CONCRETE GIRDER BRIDGES IN FLORIDA

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Moussa A. Issa, Ph.D., P.E.
Structural Analyst

Mohsen A. Shahawy, Ph.D., P.E.
Chief Structural Analyst

Structural Research Center, MS 80
Florida Department of Transportation, Tallahassee

## SYNOPSIS

The paper presents the results of full scale static and dynamic tests on two prestressed concrete bridges. Both bridges contain a variety of AASHTO type girders and were designed to carry two lanes of HS20 loading. The critical spans were instrumented at quarter span (L/4) and midspan (L/2) with accelerometers, strain gages and deflection transducers. The bridge load testing apparatus consists of a mobile data acquisition system and two load testing vehicles, designed to deliver the ultimate live load specified by the AASHTO Code. For static testing, the bridge was incrementally loaded up to the full ultimate design live load. The test vehicles were loaded to be equivalent to HS-20 truck loads. At each load step the instruments were monitored and the results were compared to the analytical model before proceeding with the next load step. The dynamic load tests were performed with the two testing vehicles traveling at 55 MPH, 45 MPH, and 35 MPH. The results indicated an increase in the strain and deflection amplitudes, with an increase of vehicle speed. A linear relationship exists between the applied load and the measured strains and deflections. The AASHTO impact factor (I) appears to be conservative for short spans. The impact factor increased in a nonlinear mode with an increase in the speed parameter($a$). A comparison of the measured and analytical results for both dynamic and static tests is also discussed.

# INTRODUCTION

It is estimated that out of half a million existing bridges in the 50 states, nearly 105,000 are rated critically deficient.[1,2] The number of structurally deficient bridges reported in the last six years in the federal-aid system has risen from 28,070 to 37,300 - an increase of 33%.[3] In Florida thousands of existing highway bridges are older than 20 years. Throughout the state some bridges are posted for lower than original design loads.

In many cases the proper rating of a bridge cannot be achieved by the present methods. of analysis. In most cases the bridge is small and on an off-system road; such bridges do not seriously impact commercial users. However, other bridges are on major systems and the resulting detours do impact the public and the commercial users.

Bridges of questionable strength that are posted for lower loads or are scheduled to be replaced can be examined through a load test. The information collected from such a test can be analyzed to evaluate the true strength of the structure. This information can be used in making decisions on the future of such bridges. The Federal Highway Administration (FHWA) requires states to consider all possible alternatives, including rehabilitation; before approving bridge replacement. In spite of all attempts to police loads, overweight vehicles do use our roads and bridges everyday. Furthermore, higher loads are expected in the future. Therefore, one needs to know the actual safe loads that these bridges can carry.

Bridge load testing will allow a satisfactory overall strength evaluation of any bridge under question. The information provided will greatly increase the possibility of selective rehabilitation, rather than the current practice of replacing the entire structure.

In Florida a large number of new and old bridges have been field tested during the past five years. The experience gained from these tests indicates that load limitations imposed by theoretical analyses are not representative of the structures real capacities. Proof loading has consistently indicated that structures have greater residual strength than indicated by analysis or design.

## BRIDGE LOAD TESTING

In testing a bridge various structural elements need to be examined. The strength of these elements is generally determined by placing strain or deflection-transducer gages at critical locations along the elements. The bridge is then incrementally loaded to induce maximum effects. The collected data can then be analyzed and used to establish the strength of each component as well as the load distribution.

The FDOT's bridge load-testing apparatus consists of two testing vehicles, a mobile data acquisition system and a mobile machine shop. The two testing vehicles, shown in Figure 1, have been designed to deliver the ultimate live loads specified by the AASHTO Code. Each vehicle is a specially designed tractor-trailer combination, weighing, in excess of 200,000 pounds when fully loaded with concrete blocks. Detailed dimensions of the test vehicles are shown in Figure 2. Each vehicle can carry a maximum of 72 concrete blocks, each weighing approximately 2,150 pounds. Incremental loading is achieved by adding blocks with a self-contained hydraulic crane mounted on each truck. Each truck contains a remote control system allowing operation without a driver when a bridge's strength is in question.

The data acquisition system, consists of two distinct subsystems for static and dynamic testing. The data acquisition system was housed in a 23 foot motor home which serves a computer center, electronic workshop and general office for field crew. The data acquisition system is capable of high and low speed scanning for dynamic and static measurements.

Once a bridge is identified for load testing, a site survey and an analysis of existing plans and inspection reports provide further information on the feasibility of such a test. The plans and details of instrumentation and loading locations are then established. Next, the testing equipment and personnel travel to the bridge site. The instrumentation (strain or transducer gages, accelerometers, LVDT's, etc.) are mounted at critical locations. of the structure and tested for functional response.

**TESTING PROCEDURE AND INSTRUMENTATION**

The load testing procedure and the type of instrumentation used for the two bridges were essentially the same. The static and dynamic load testing procedures as well as the description of the instrumentation are presented below.

**STATIC LOAD TEST PROCEDURE**

Each testing vehicle was loaded to an initial weight of 100 kips (24 blocks). Initial readings of all instrumentation were recorded with no vehicles on the structure. The trucks were then driven and placed at predetermined critical load positions on the bridge. For the specified load position, strain and deflection readings were measured and recorded by the host computer. The trucks were then driven off the bridge.

The measured data was immediately analyzed, displayed and compared to the theoretical prediction; this process took approximately ten minutes. If the results of all strains and deflections were within acceptable limits, the loads could be safely increased. The loads on each truck were increased by 26 kips, the trucks were then driven back onto the same load position on the bridge and readings were again recorded. This procedure was repeated until the trucks weighed 204 kips each.

**DYNAMIC LOAD TEST PROCEDURE**

Each testing vehicle was loaded to a total weight of 100 Kips. This weight is equivalent to an HS20-44 truck. The initial readings of all the instruments were recorded with no vehicles on the bridge. The trucks were then driven over the bridge side by side at a constant speed of 55 MPH. The data for deflection, strain and acceleration was collected and stored in the data acquisition media and then transferred into the host computer system for data reduction and presentation. The data was collected at a sampling rate of 256 Hz for an approximate time of 30 seconds. The same test was repeated for vehicle speeds of 45 and 35 MPH. A real time display of deflection, strain and acceleration results can be monitored on the computer screen as the vehicles move over the bridge.

**BRIDGE INSTRUMENTATION**

Instrumentation for measuring strains, deflections and acceleration were installed at specified locations prior to testing. Based on the analytical model, these locations are chosen to provide the maximum response due to the loads.

**STRAIN GAGES**

The strain readings were obtained by using displacement transducers (DT's), designed to provide strain and displacement measurements. The DT's used for the static and dynamic tests were PI-5 TML gages manufactured by Tokyo Sokki Kenkyujo Co. of

Japan. The gages were mounted on the bottom flange of each girder using a common industrial epoxy and special mounting blocks developed by the FDOT research team. The lead wires from each gage were then connected to the data acquisition system. Figure 3 shows the details and location of instruments.

**ACCELERATION GAGES**

Acceleration transducers were used in the dynamic test to measure the acceleration of the bridge. The accelerometers were mounted at the bottom flange of the middle girders with a special mounting-block(see Figure 3). The full range of the accelerometer is 5g with natural and response frequencies of 85 and 50 Hz respectively.

**DEFLECTION GAGES**

Vertical deflections were measured, with linear variable displacement transducers (LVDTs). These particular LVDTs have a through-bore construction which allows a spring to be mounted at a fixed height above the core and coil. As the deflection occurs the spring will hold the core at a fixed elevation and allow the coil to move with the structure and along the core. As the core moves through the coil, the voltage output changes. This voltage change can then be read with the data acquisition system and converted to deflection.

**DATA ACQUISITION**

The data acquisition system and the strain readout box were both housed in a 23 ft. motor home which was parked adjacent to each bridge. Individual cables were run from each strain gage, accelerometer and LVDT to a control panel in the motor home. The control panel was connected to the data acquisition system. The data acquisition system is a MEGADAC system made by OPTIM ELECTRONICS INC.. The system is capable of collecting data at a low sampling rate (static test) and a high sampling rate (dynamic test). The maximum sampling rate of the system is about 250,000 samples per second, depending on the number of channels used.

## DESCRIPTIONS OF BRIDGES AND PRESENTATION OF RESULTS

The two test bridges will be referred to as Bridge A and Bridge B, in the following discussion. The test results of each bridge will be presented and discussed separately.

## BRIDGE A: 1-75 OVER CALOOSAHATCHE RIVER

Bridge A is located on F-75 across Caloosahatche River near Fort Myers, Florida. The bridge consists of six 45.5 ft continuous spans and was designed to carry two lanes of HS20 loading. Each span has five AASHTO type II girders spaced at 9'-3". The first three spans were instrumented.

### Static Test of Span A3

The static load test was performed on span 3. Figure 4 shows the load position of the testing vehicles during all static tests. Figure 5(a) shows the increase in the measured midspan strains at different stages of loading during the static test. Figure 5(b) shows the relationship between the applied load and the measured strain for girders is linear, and elastic. Neither cracking nor other signs of distress were observed at the maximum applied loads.

### Dynamic Test of Span A3

The maximum measured midspan dynamic and static strains for all girders with vehicles loaded to 100 kips; and traveling 55 MPH traveling speed was collected.

Figure 6(a) shows a typical time vs. midspan strain relationship at 55 MPH. The fundamental natural frequency of the bridge was calculated by the power spectrum of measured time-strain or time-acceleration data as shown in Figure 6(b). The dominant natural frequency of this test was about 10.5 Hz (cps).

**BRIDGE B: SR-55 OVER SUWANNEE RIVER**

Bridge B is located on SR-55 over the Suwannee River at Fanning Springs, Florida. The bridge consists of one-66 ft simple span and two-121 ft continuous spans. The bridge was designed for two lanes of HS20 loading. The first span (66 ft) consists of 5 AASHTO type III girders, spaced at 8'-8" while the remaining two spans consist of AASHTO type IV girders spaced at 5 ft. The instrumentation locations are at quarter span (L/4) and midspan(L/Z) for spans 131 (type III girders) and B2 ( type IV girders).

**Static Test of Span B1**

The static load test was performed on span 1. Figure 7 shows the load position of the testing vehicles during the static test. Figure 8 presents the midspan deflections at different loading stages during the static test. A linear relationship exists between the applied load and the measured deflection and, also, between the applied load and the measured girder strain. These results indicated maximum measured deflection and strain of 0.385 inches and 195 microstrain respectively for girder 3. Neither cracking nor other signs of distress were observed at the maximum applied load.

**Dynamic Test of Span B1**

Typical dynamic test values presented in figures 9 and 10 resulted from vehicles loaded to 100 Kips each, and traveling at speeds of 55 MPH, 45 MPH and 35 MPH.

mode and also in static mode. These bridges were loaded to their ultimate design live load in the static test.

Span B1 recorded a maximum static deflection of 0.206 inch at a total vehicle weight of 200 kips, and a dynamic deflection of 0.261 inch with the two trucks traveling at 55 MPH. Table 1 presents the results of deflections and strains for typical interior bridge girders. The results indicated an increase in the strain and deflection amplitudes, with an increase of vehicle speed. A linear relationship exists between the applied load and the measured strains and deflections. The analytical amplification factors were about 20 percent higher than the experimental results. Neither cracking nor other signs of distress were observed during the static and dynamic tests of these bridges.

**BRIDGE FREQUENCY**

The Fast Fourier Transformation (FFT) technique was used to find the bridge fundamental frequencies. The measured strain-time, deflection-time and acceleration-time data were represented as the sum of contributions from all the vibrational modes and analyzing the data through time intervals. The output of this analysis is commonly presented in the form of a power spectrum, which is a diagram showing the dominant frequencies. Typical deflection-time, strain-time and acceleration-time curves were used to find the dominant vibrational frequency of the bridge after the passage of trucks, as shown in the previous figures(see Figures 6, 10 and 12).

The experimental results indicated dominant vibration frequencies of 10.5 Hz, 7.0 Hz and 2.55 Hz for AASHTO type 11 (span A3), type III (span B1), and type IV (span B2) girders, respectively. These results are summarized and plotted as a function of span length in Figure 14. In this figure a curve approximation of the data has been established. This curve expresses the relationship between bridge period, ($T = 1/f_b$), and span length and is approximated by the following equation:

$$T = (15 * L^2 + 1557 * L - 975) * 10^{-6} \quad (1)$$

Where L is the span length measured in feet and T is the bridge period in seconds. This expression fits the test data that has been collected at this stage and it will be modified as more data is collected in the future. A straight line approximation of T = 0.0024L was given by Walker and Veletsos (Reference 7).

The straight line approximation method matches the collected experimental results up to 75 ft spans It appears that the straight line approximation underestimates the bridge period by about 25%; this is 25% less than the curve approximation for the 121 ft span.

The analytical frequencies of these spans were 9.4Hz, 5.9Hz, and 2.5Hz for AASHTO type II (span A3), type III (span B1), and type IV (span B2) girders, respectively.

**AMPLIFICATION FACTOR**

The amplification factor for the static and dynamic results of deflections, strains, and bending moments was represented by the ratio of the maximum value to the absolute maximum response (see Figure 13).

Table 2 presents the impact, factor extracted from measured strain and deflection results at different vehicle speeds. The impact factor was defined as the ratio of dynamic to static response of strain or deflection.

**EFFECT OF SPEED**

Figure 15 shows the experimental and analytical results for a vehicle speed of 55 MPH, 45 MPH and 35 MPH. The figure shows a measured impact factor of 1.30 for the 121 ft span. The measured impact factors are higher than the values specified by the AASHTO code and less than the Ontario Highway Bridge Design Code (OHBDC) recommendations. Figure 16 shows the speed parameter ($a$ = VT/2L) versus the impact factor for AASHTO type III and type IV bridges. In general, the higher the speed parameter(a), the larger the impact factor:

**ACCELERATION**

The magnitude of force in a bridge component depends upon the vehicle weight speed and-suspension system; the bridge's coefficient of friction, weight surface condition and span length. There is a noticeable increase in the acceleration of the bridge results as the vehicle speed increases.

**SUMMARY AND CONCLUSIONS**

The following summary and conclusions can be drawn from the results of this field test:

1. The fundamental frequency of the bridge can be derived from strain-time, deflection-time and acceleration-time data.
2. The speed parameter(a) is the most significant variable to effect the dynamic behavior of the bridge.
3. The measured impact factors for these bridges were higher than the AASHTO code and less than the OHBD code.
4. It is difficult to evaluate a bridge behavior by strain and by deflection data collected from regular truck traffic.

5. A linear relationship exists between the bridge period and the span length for bridges less than 75 feet long. For bridges longer than 75 feet it is recommended to use the curve approximation method (Equation 1).
6. The AASHTO impact factor (I) appears to be conservative for short spans and low for the 66 and 121 ft spans. The impact factor increased in a nonlinear mode with an increase in the speed parameter (a).
7. The method recommended to find the bridge dynamic amplification factor (Impact) and vibration is the positioning of the test vehicles on the critical locations in the static test, and then comparing the results with the same test vehicles in the dynamic test.
8. The measured field test results indicated that these bridges have greater residual strength than predicted by analytical methods.

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**LIST OF TABLES**



**LIST OF FIGURES**

## TABLE 1

| DEFLECTING AND STRAIN RESULTS FOR A TYPICAL INTERIOR MID-SPAN GIRDER | | | | | | |
|---|---|---|---|---|---|---|
| Method | AASHTO Bm. Type | Interior Girdor No. | Static Results | Dynamic Results | | |
| | | | | 55 MPH | 45 MPH | 35 MPH |
| Strain (ue) | II | 3 | 110 | 117 | N/A | N/A |
| | III | 3 | 95 | 130 | 125 | 105 |
| | IV | 5 | 115 | 150 | 140 | 124 |
| Deflection (in.) | III | 3 | 0.2 | 0.261 | 0.224 | 0.207 |
| | IV | 5 | 0.338 | 0.438 | 0.428 | 0.385 |

## TABLE 2

| COMPUTATION OF IMPACT FACTOR FOR A TYPICAL INTERIOR MID-SPAN GIRDER | | | | | | |
|---|---|---|---|---|---|---|
| Method | AASHTO Bm. Type | Interior Girdor No. | Dynamic Results % | | | AASHTO EQUATION I=50/(L+125) |
| | | | 55 MPH | 45 MPH | 35 MPH | |
| Strain (ue) | II | 3 | 6.3 % | N/A | N/A | 29.3 % |
| | III | 3 | 36.8 | 31.6 | 10.5 | 26.2 |
| | IV | 5 | 30.4 | 21.7 | 7.8 | 20.3 |
| Deflection (in.) | III | 3 | 30.5 | 12.0 | 3.5 | 26.2 |
| | IV | 5 | 29.6 | 26.6 | 13.9 | 20.2 |

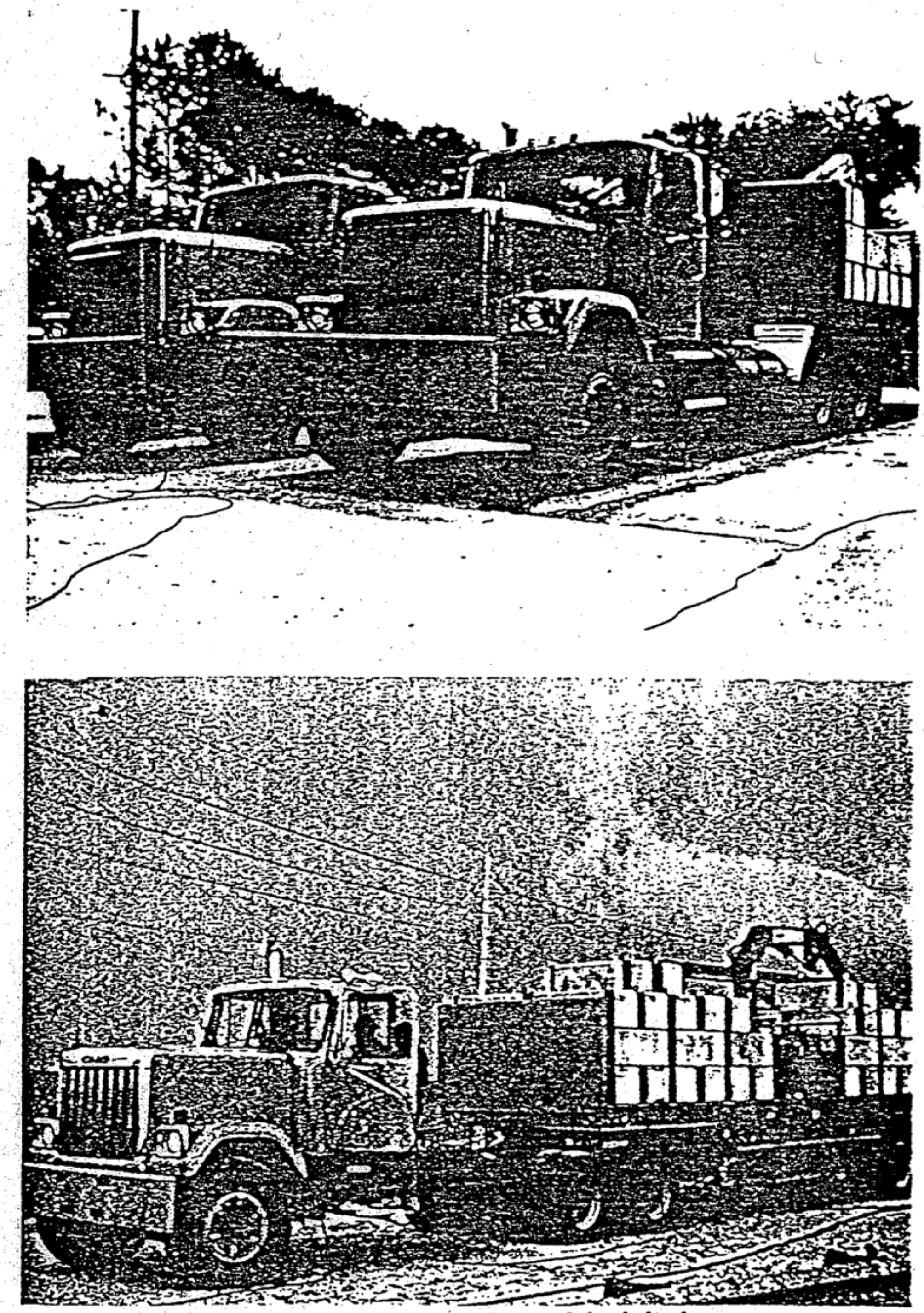
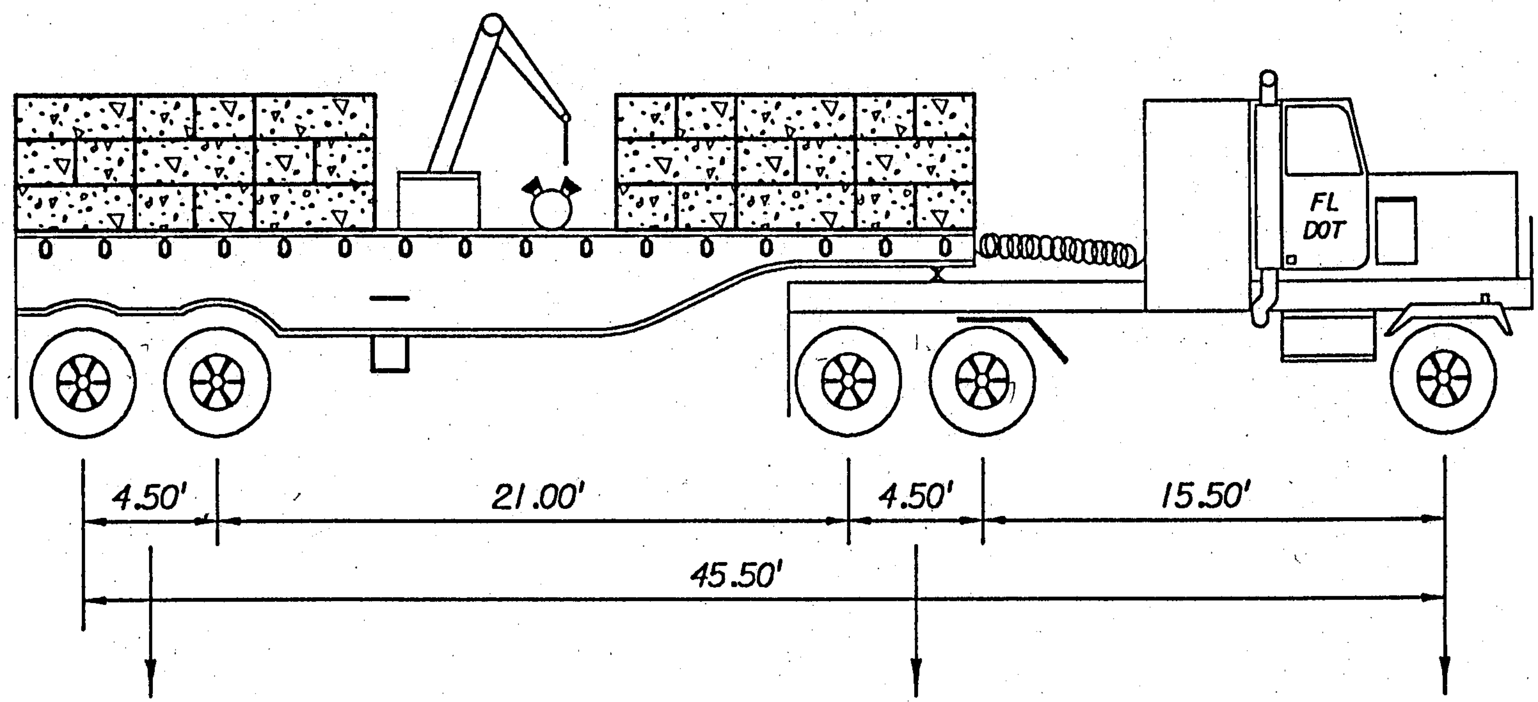


Figure 1. Testing Vehicles




| | | | | |
|---|---|---|---|---|
| Weights: | 72 ballast blocks | 154,800 lb. | Load Transfer: | |
| | Equipment | 8,200 lb. | 5th wheel | 82,350 lb. |
| | Trailer | 24,000 lb. | Steering axle | 15,630 lb. |
| | Tractor | 17,000 lb. | Drive tandem | 83,720 lb. |
| | Total | 204,000 lb. | Trailer tandem | 104,650 lb. |

Note: All weights and dimensions are approximate and for information only.

**Bridge Testing Vehicle**
**Figure 2**

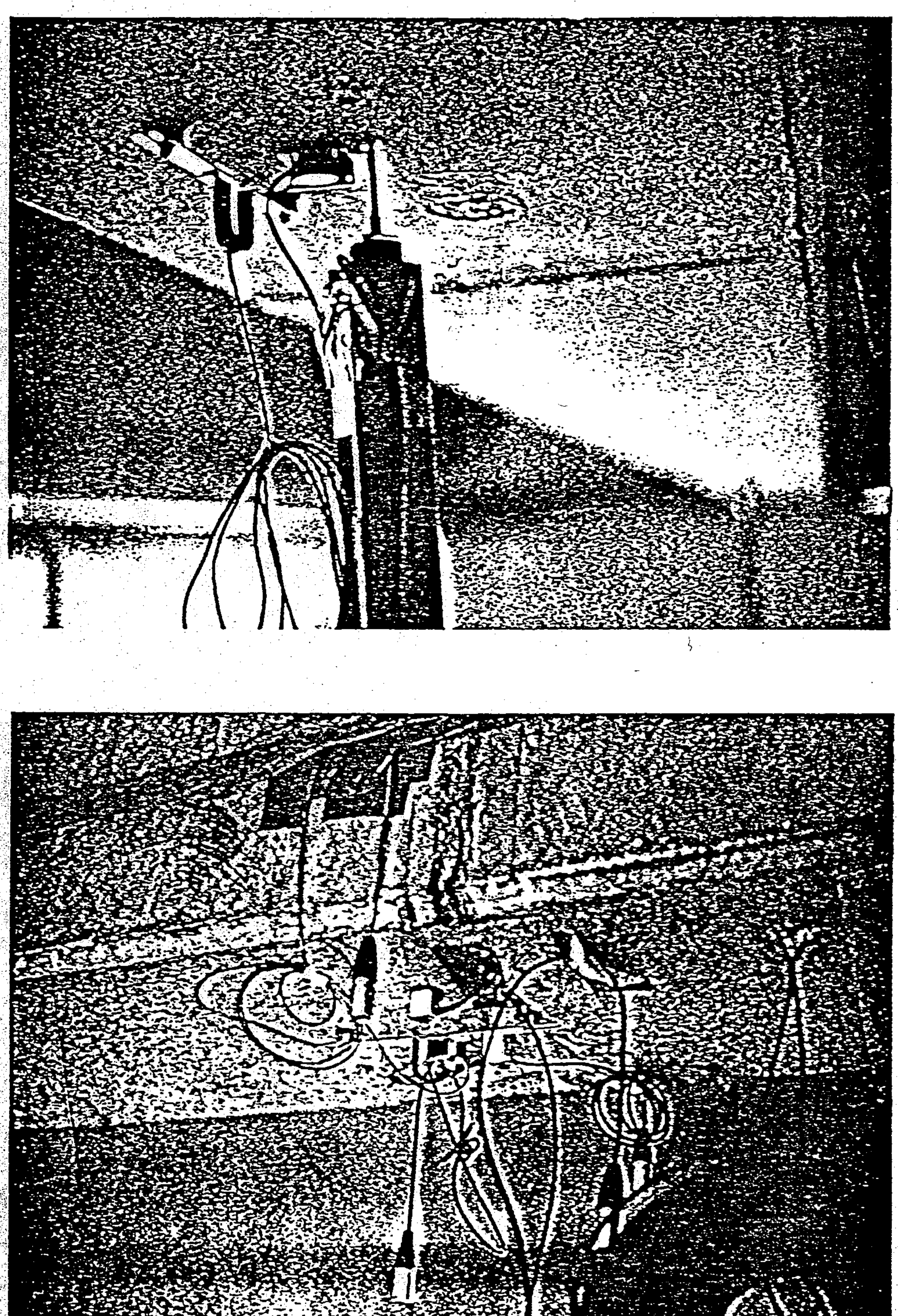
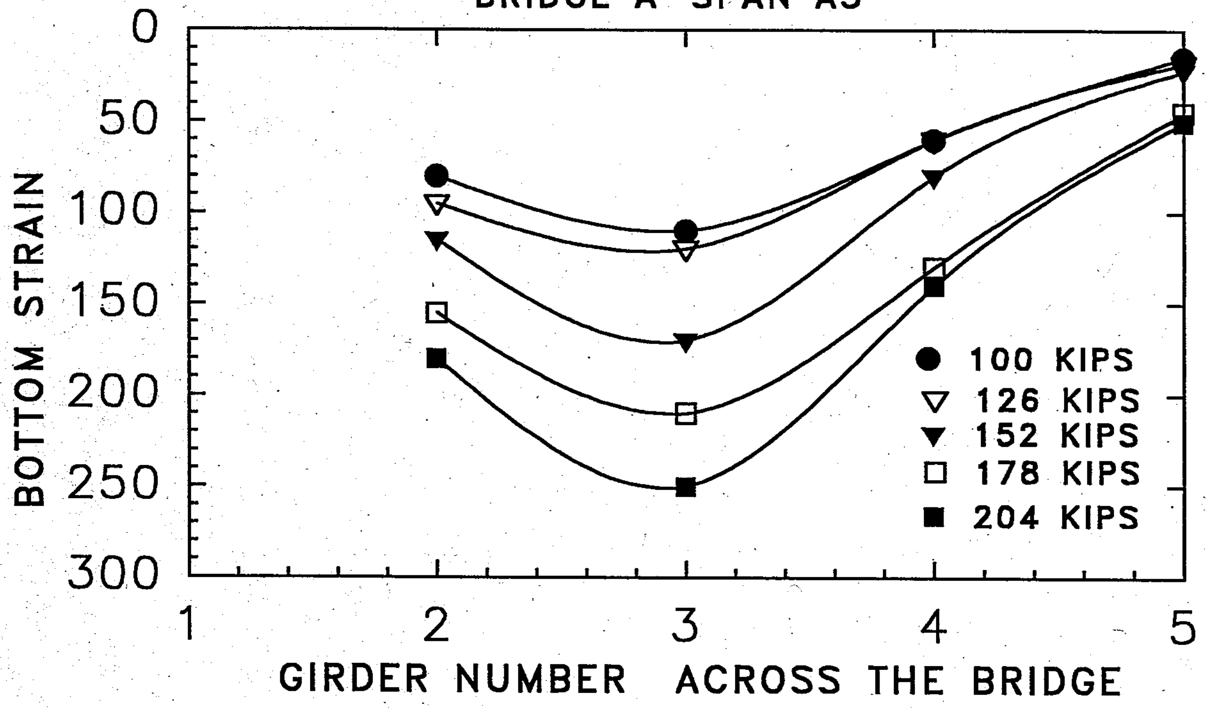


Figure 3. Location of Instruments

EXPERIMENTAL RESULTS FOR AASHTO TYPE II GIRDER

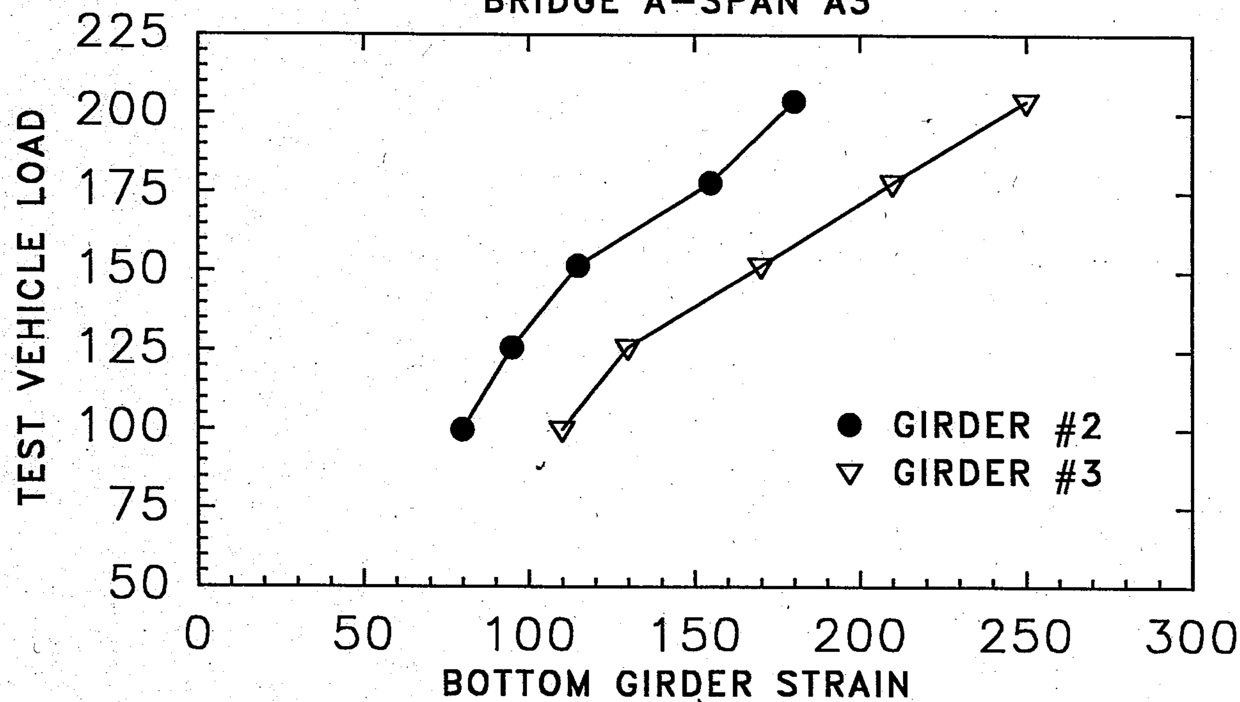
TRANSVERSE STRAIN OF MIDSPAN A3

BRIDGE A-SPAN A3




(a)

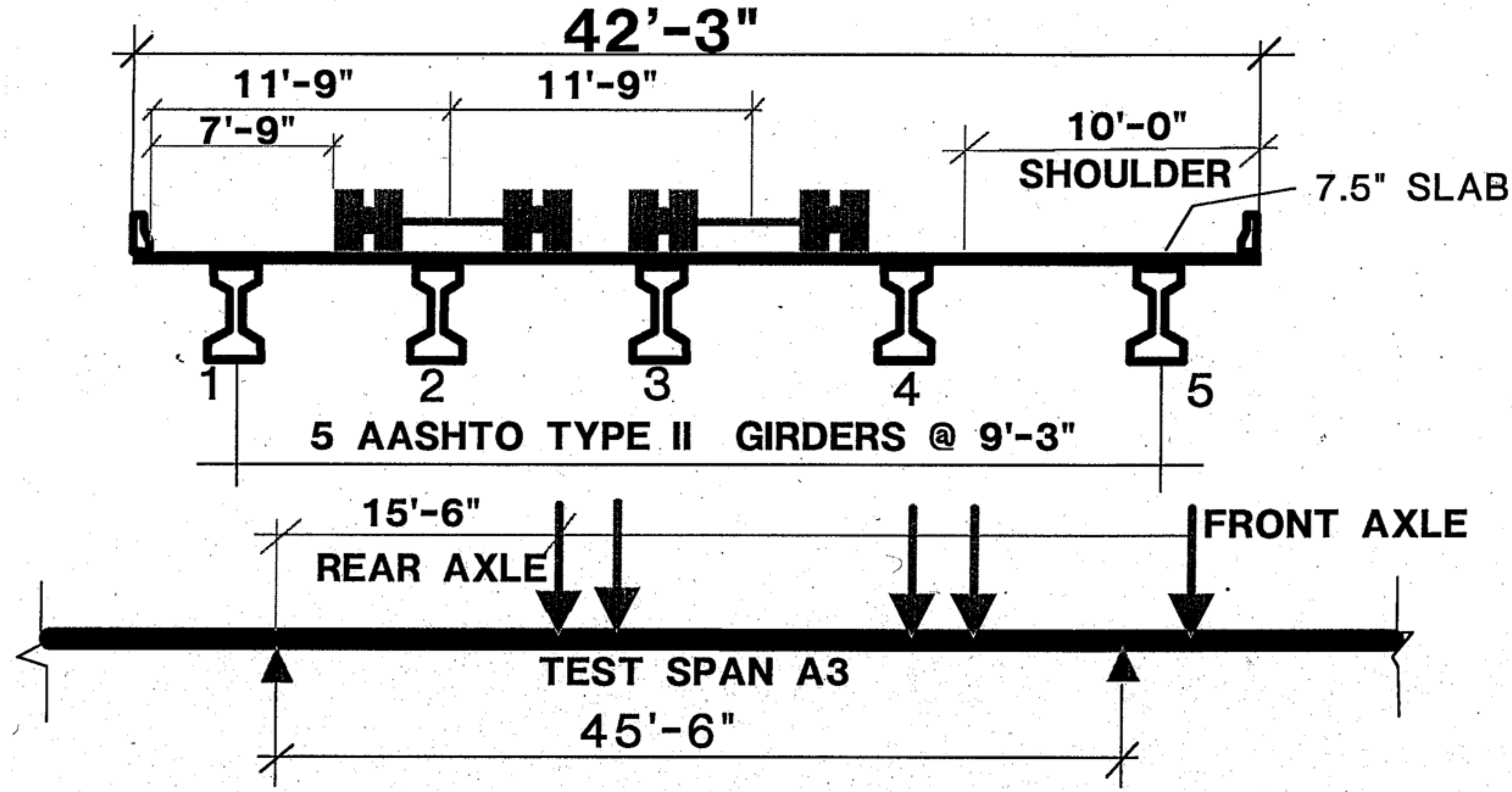
BRIDGE A-SPAN A3




(b)

I-75 BRIDGE ACROSS CALOOSAHATCHE RIVER AT FORT MYERS

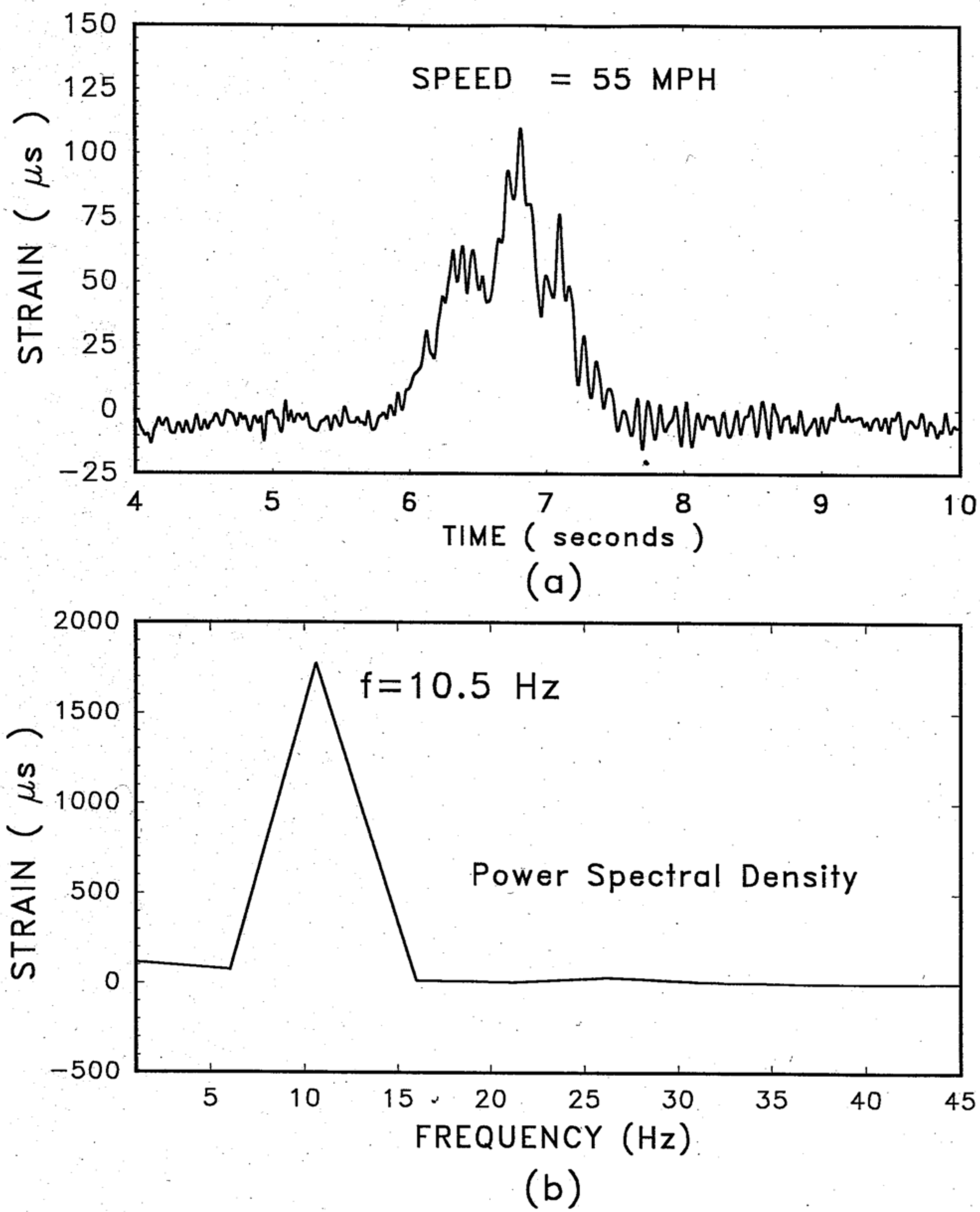
FIGURE 5



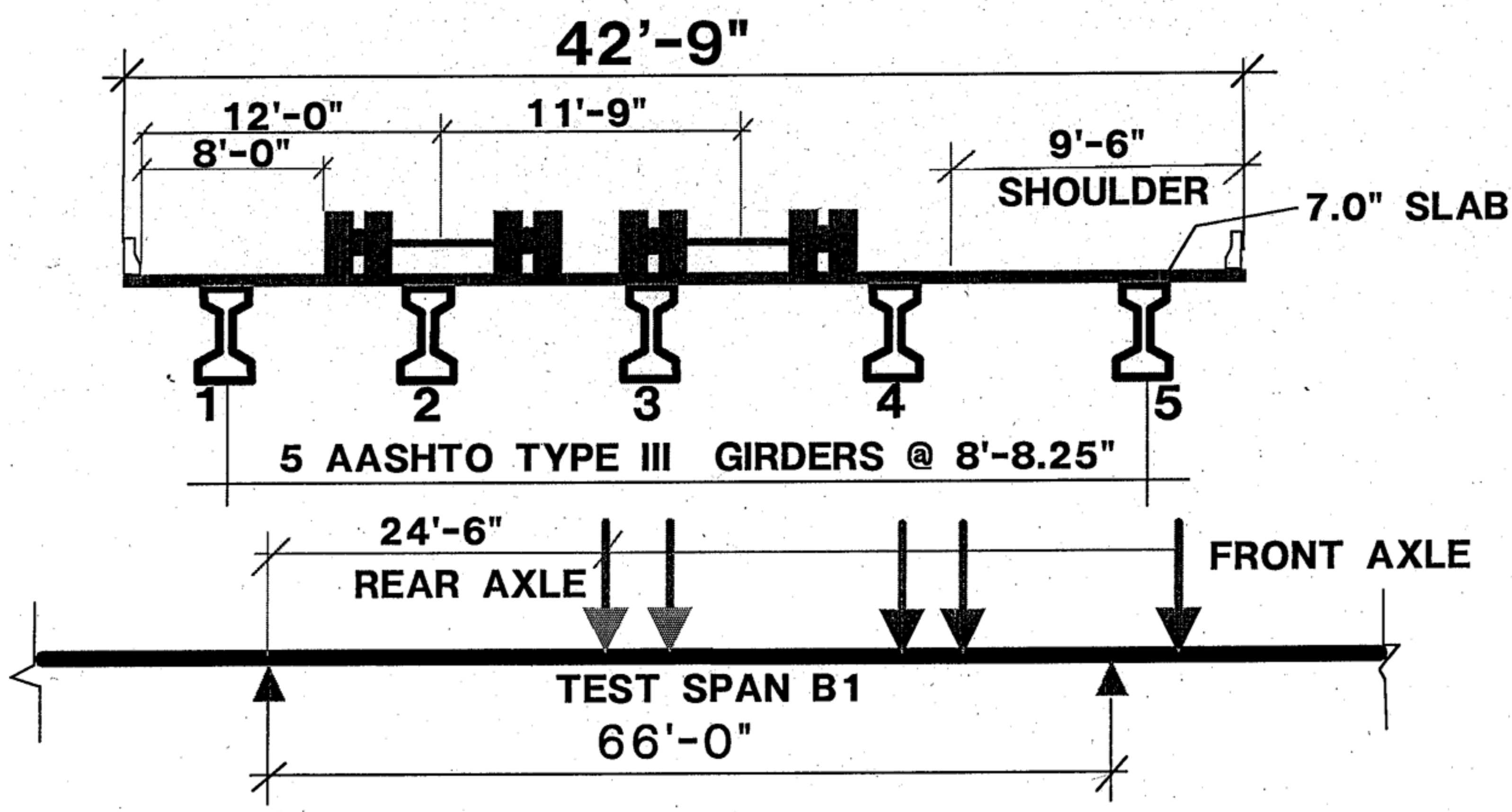

LATERAL & LONGITUDINAL POSITIONS OF TEST VEHICLES

I-75 BRIDGE ACROSS CALOOSAHATCHE RIVER AT FT MYERS

FIGURE 4




Typical Strain–Time and Frequency Results
Bridge A – Span A3
Figure 6

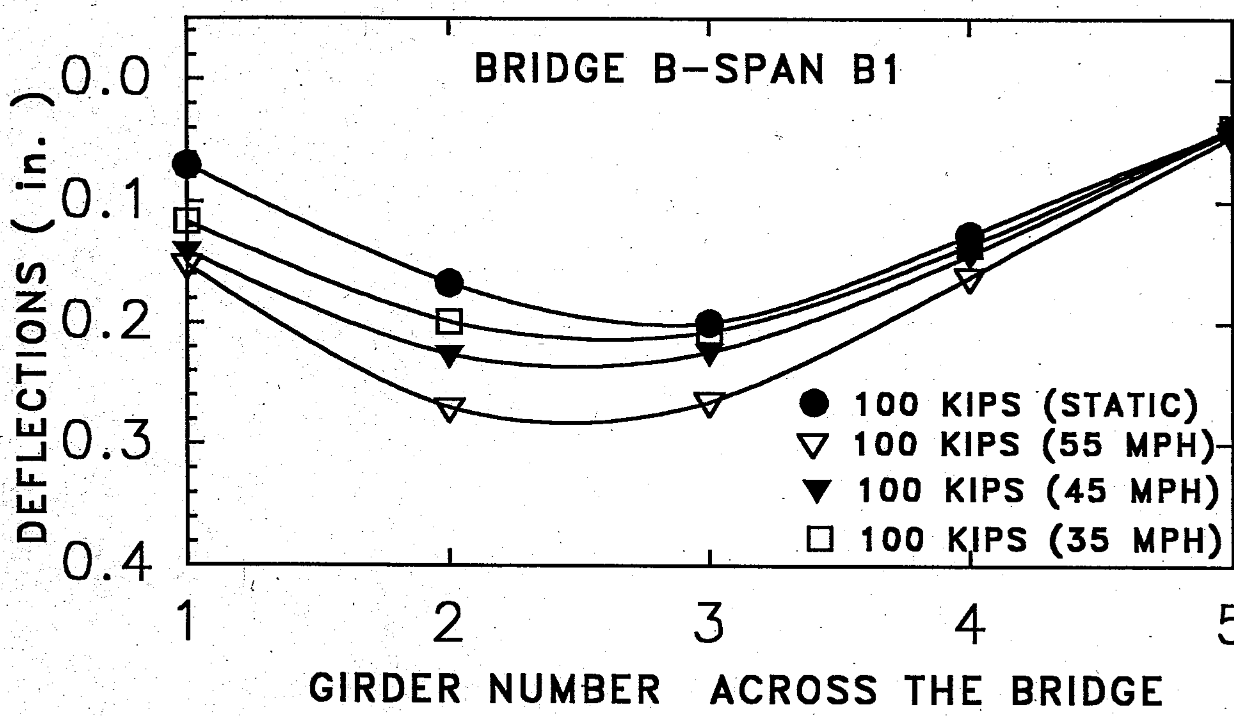



LATERAL & LONGITUDINAL POSITIONS OF TEST VEHICLES
SR-55 BRIDGE OVER SUWANNEE RIVER AT FANNING SPRIN

# FIGURE 7

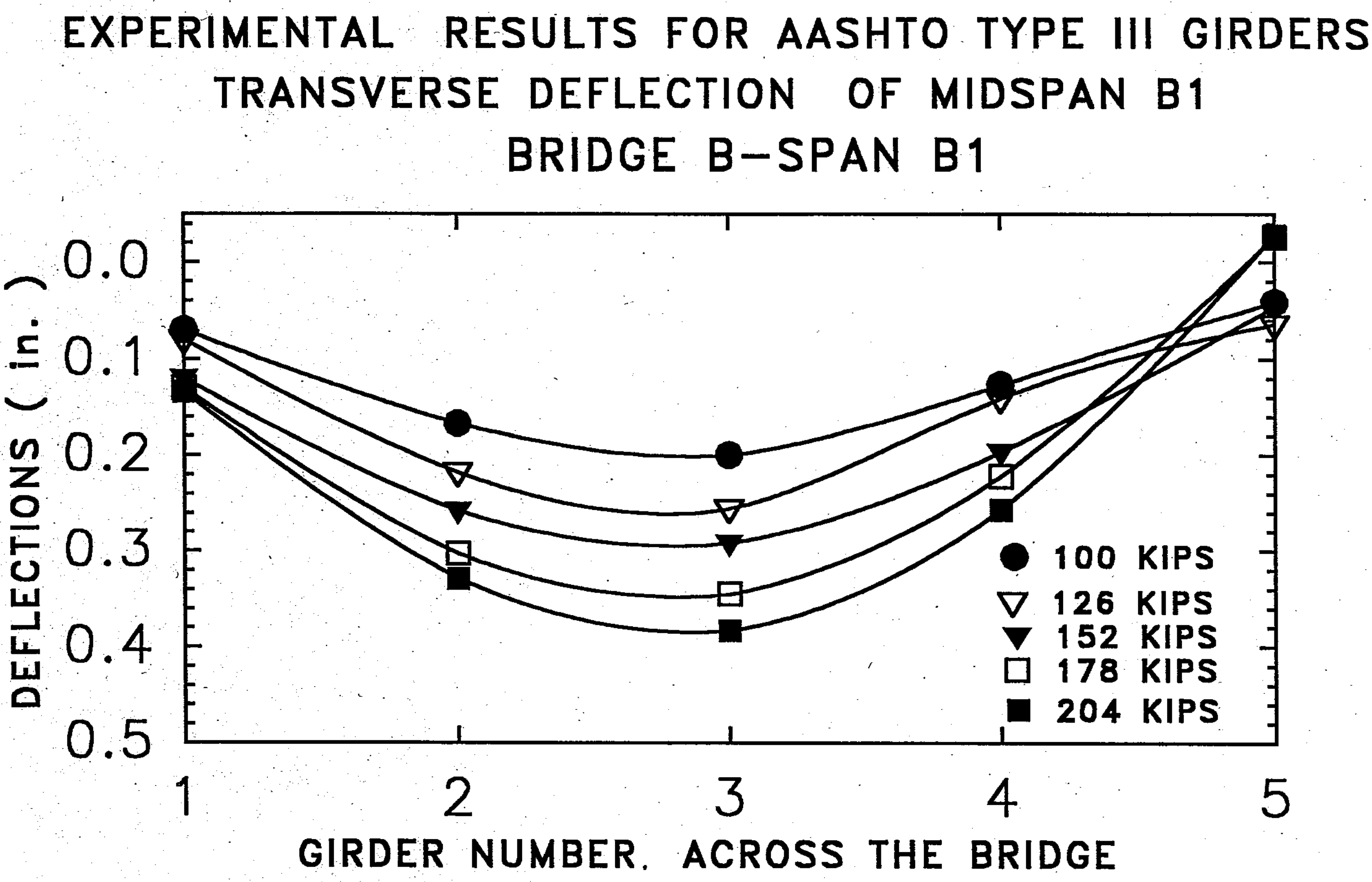
COMPARISON BETWEEN STATIC AND DYNAMIC RESULTS OF AASHTO TYPE III GIRDERS FOR DIFFERENT TRUCK SPEEDS

TRANSVERSE DEFLECTION AT MIDSPAN




SR-55 BRIDGE OVER SUWANNEE RIVER AT FANNING SPRING

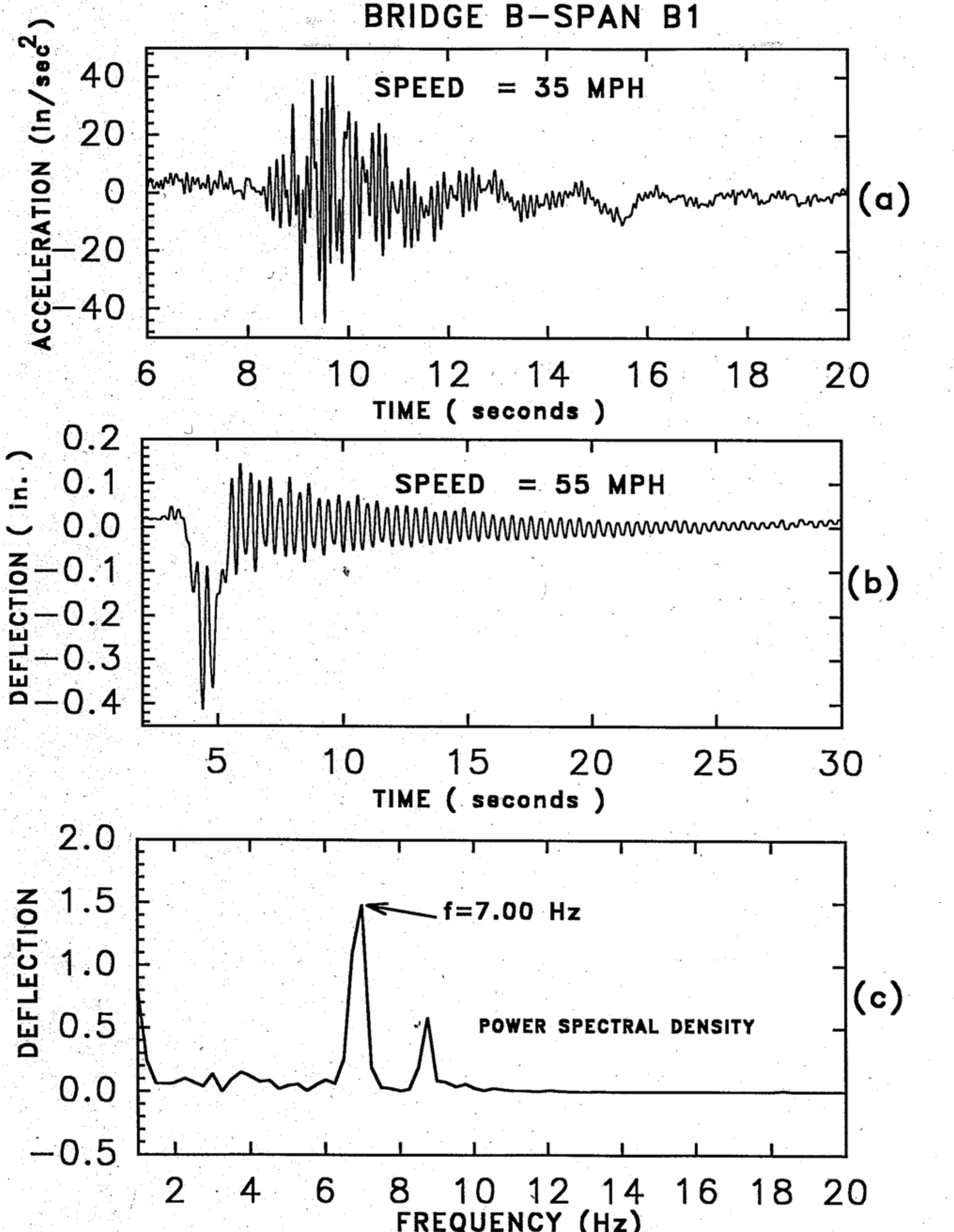
FIGURE 9




SR-55 BRIDGE OVER SUWANNEE RIVER AT FANNING SPRING

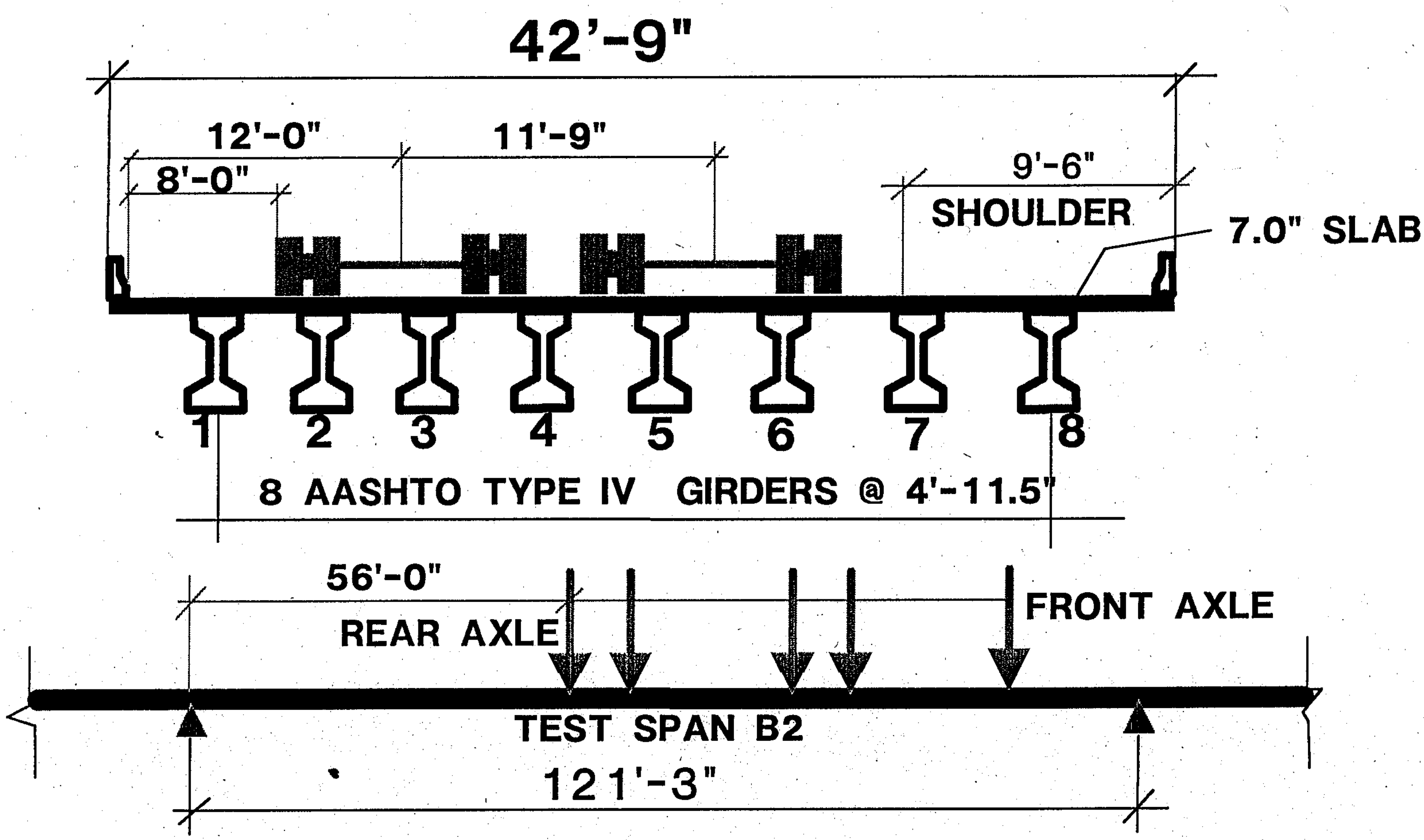
FIGURE 8

EXPERIMENTAL RESULTS FOR AASHTO TYPE III GIRDERS
BRIDGE B-SPAN B1




Typical Time–Deflection, Acceleration and Frequency Results

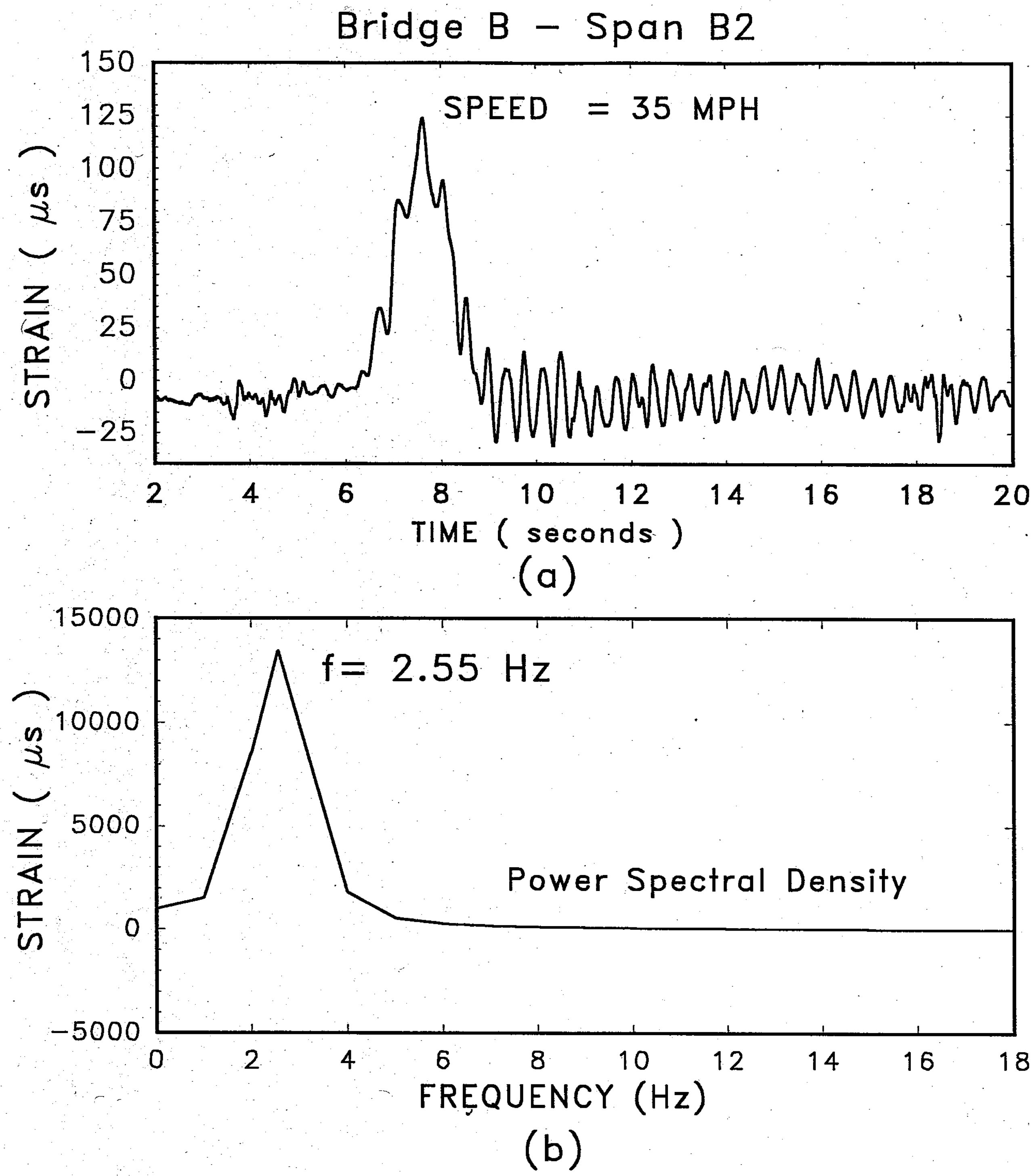
FIGURE 10




LATERAL & LONGITUDINAL POSITIONS OF TEST VEHICLES

SR-55 BRIDGE OVER SUWANNEE RIVER AT FANNING SPRIN

FIGURE 11




Typical Strain–Time and Frequency Results

Figure 12

For a speed of 55 MPH, the maximum measured midspan deflections for all girders is shown in figure 9. The equivalent values obtained from the static load test with vehicles of the same weight are shown in the same figure. Figure 10(a) shows a typical time-acceleration relationship for a 35 MPH vehicle speed. Figure 10(b) shows a typical time-deflection relationship for a 55 MPH vehicle speed.

The fundamental natural frequency of the bridge was calculated by the power spectrum of time-acceleration measured data as shown in Figure 10(c). The dominant natural frequency of this test was about 7.0 Hz(cps).

**Static Test of Span B2**

The static load test was performed on span 2. Figure 11 shows the load position of the testing vehicles during the static test. A linear relationship exists between the applied load and the measured deflections and, also, between the applied loads and measured strains for all the girders. These results indicate a maximum deflection and strain of 0.755 inches and 263 microstrain, respectively. Neither cracking nor other signs of distress were observed at the maximum applied load.

**Dynamic Test of Span B2**

Vehicles loaded to 100 Kips and traveling at speeds of 55 MPH, 45 MPH, and 35 MPH were used for the dynamic tests. For a speed of 55 MPH the maximum measured midspan strain was about 150 microstrain. Figure 12(a) shows a typical measured time-strain data for a speed of 35 MPH. The fundamental natural frequency of the bridge, was calculated by the power spectrum of time-strain measured data as shown in Figure 12(b). The dominant natural frequency of this test was about 2.55 Hz(cps).

ANALYTICAL MODEL

The bridge was idealized as a simple beam, simply supported at the ends. The number of degrees of freedom of the beam depends of the number of equally-spaced concentrated masses along the span. A complete description of the mathematical model is presented in Reference 10.

The FDOT testing vehicle was used to analyze the bridge vibration based on the assumption that there was no eccentricity between the moving load and the centroid of the bridge girder. The test vehicle was idealized as a multiple-axle sprung load with special consideration of the interleaf friction in the suspension system. As shown previously, spans A3, B1 and B2 were modeled. Two percent (2%) of the critical damping was assumed in the analysis of these spans. The specified AASHTO lane load distribution factor was used in this analysis (Distribution Factor = S/5.5).

The vehicle started into motion at a distance of 180 ft. (four vehicle lengths) away from the entrance of the span. The movement continued until the entire vehicle left the bridge. An average road surface was assumed in the analytical study.

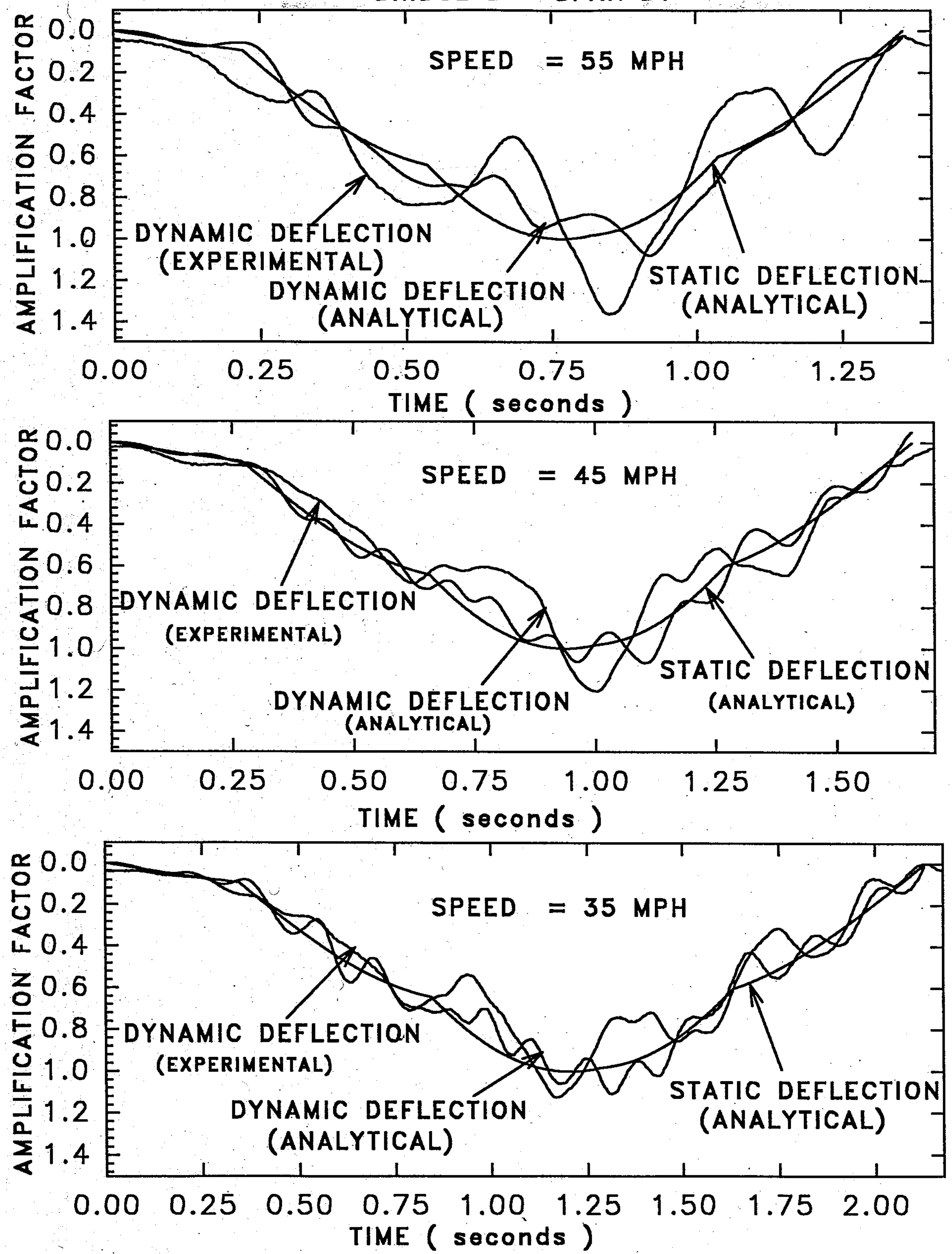
The computer program, solves for deflections, moments, shears and support reactions for static and dynamic responses. The amplification factors and the fundamental natural frequency of the bridges are also calculated.

## DISCUSSION AND ANALYSIS OF RESULTS

### STRAINS AND DEFLECTIONS

The measured deflections and strains of Bridge B were plotted and compared with the analytical results. Figure 13 shows the results at different vehicle speeds in dynamic

COMPARISON BETWEEN EXPERIMENTAL AND ANALYTICAL RESULTS FOR MIDSPAN DEFLECTION AT DIFFERENT TRUCK SPEEDS
BRIDGE B – SPAN B1




SR-55 BRIDGE OVER SUWANNEE RIVER AT FANNING SPRING

FIGURE 13

FIELD TEST DATA FOR BRIDGE PERIOD

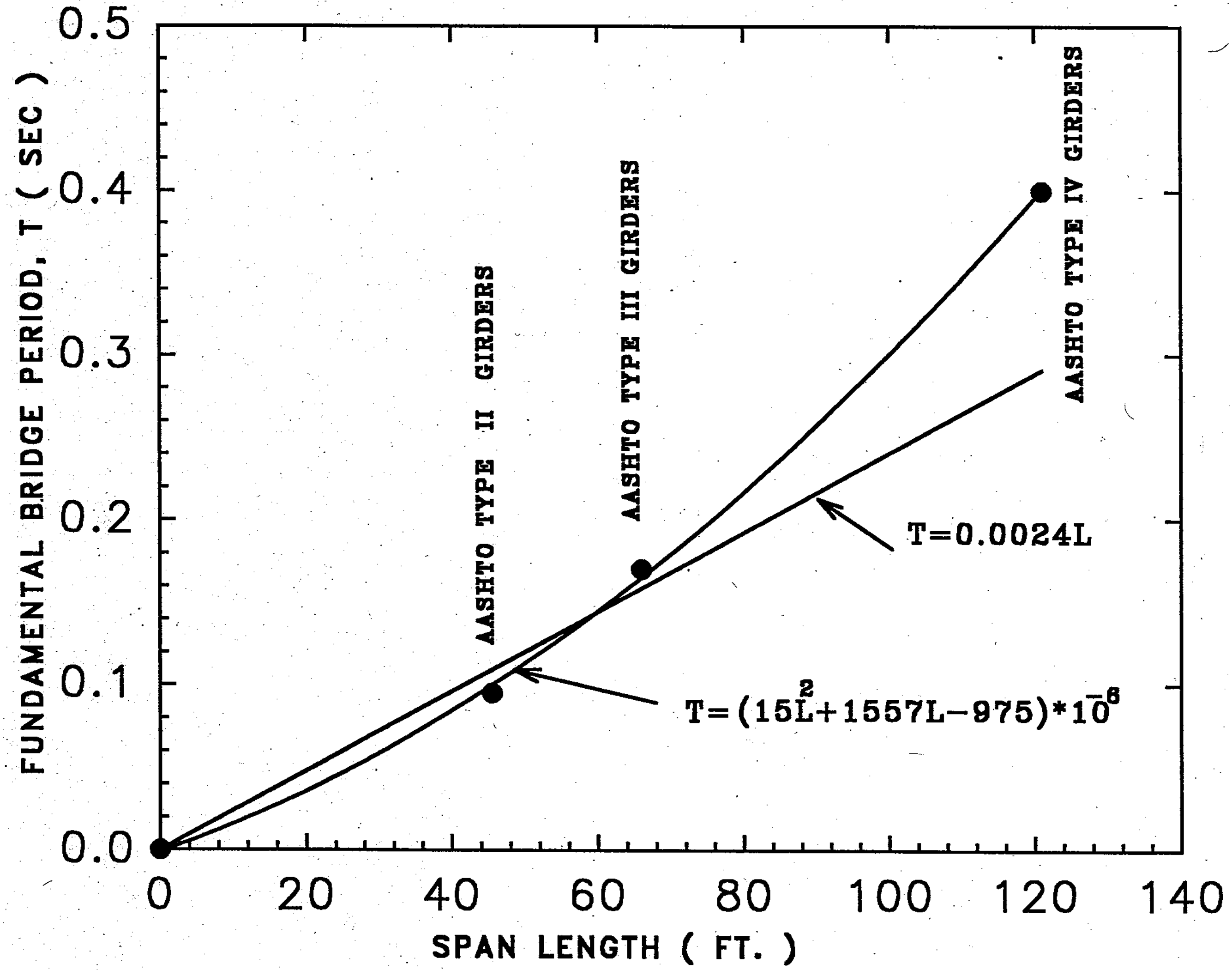



FIGURE 14

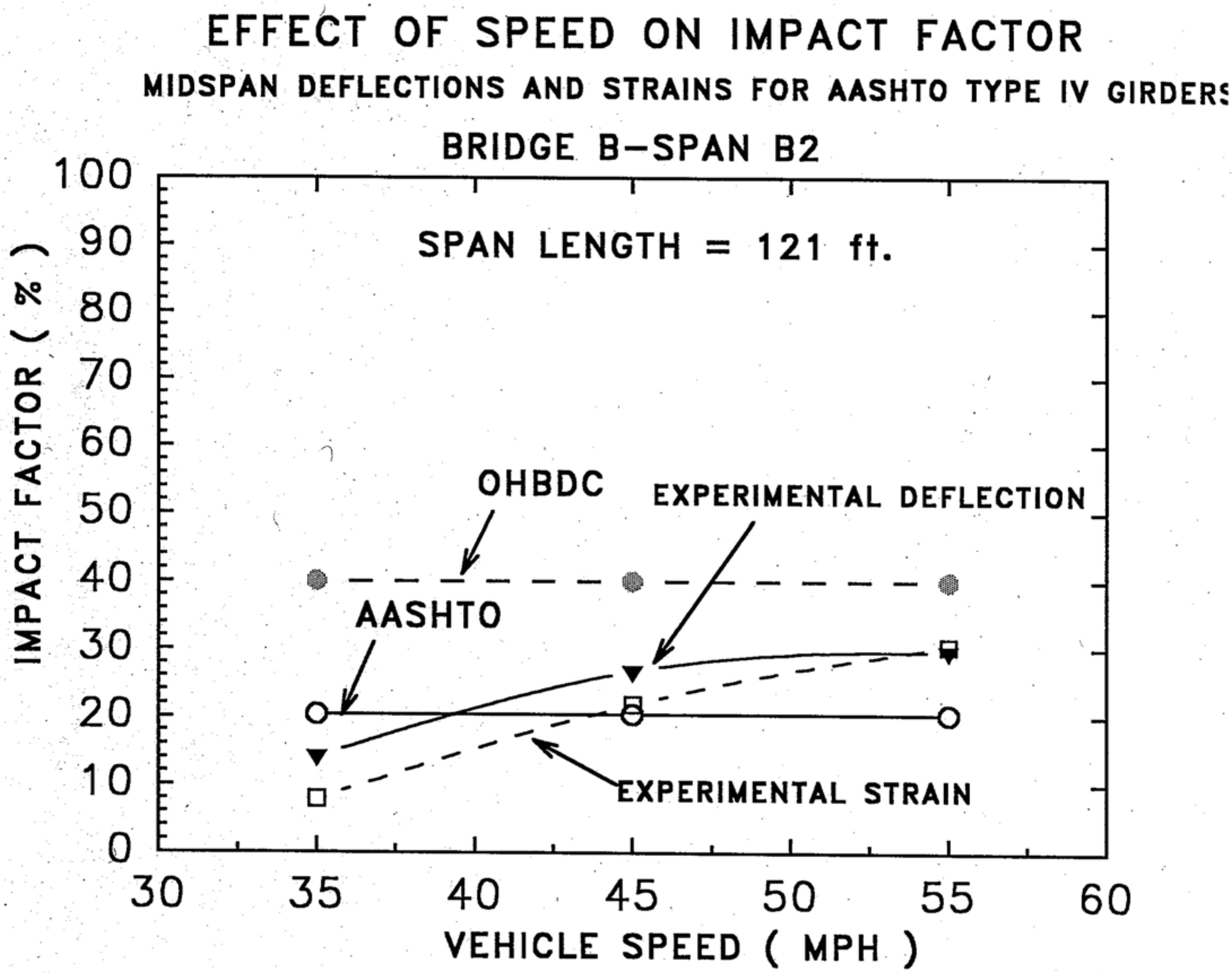
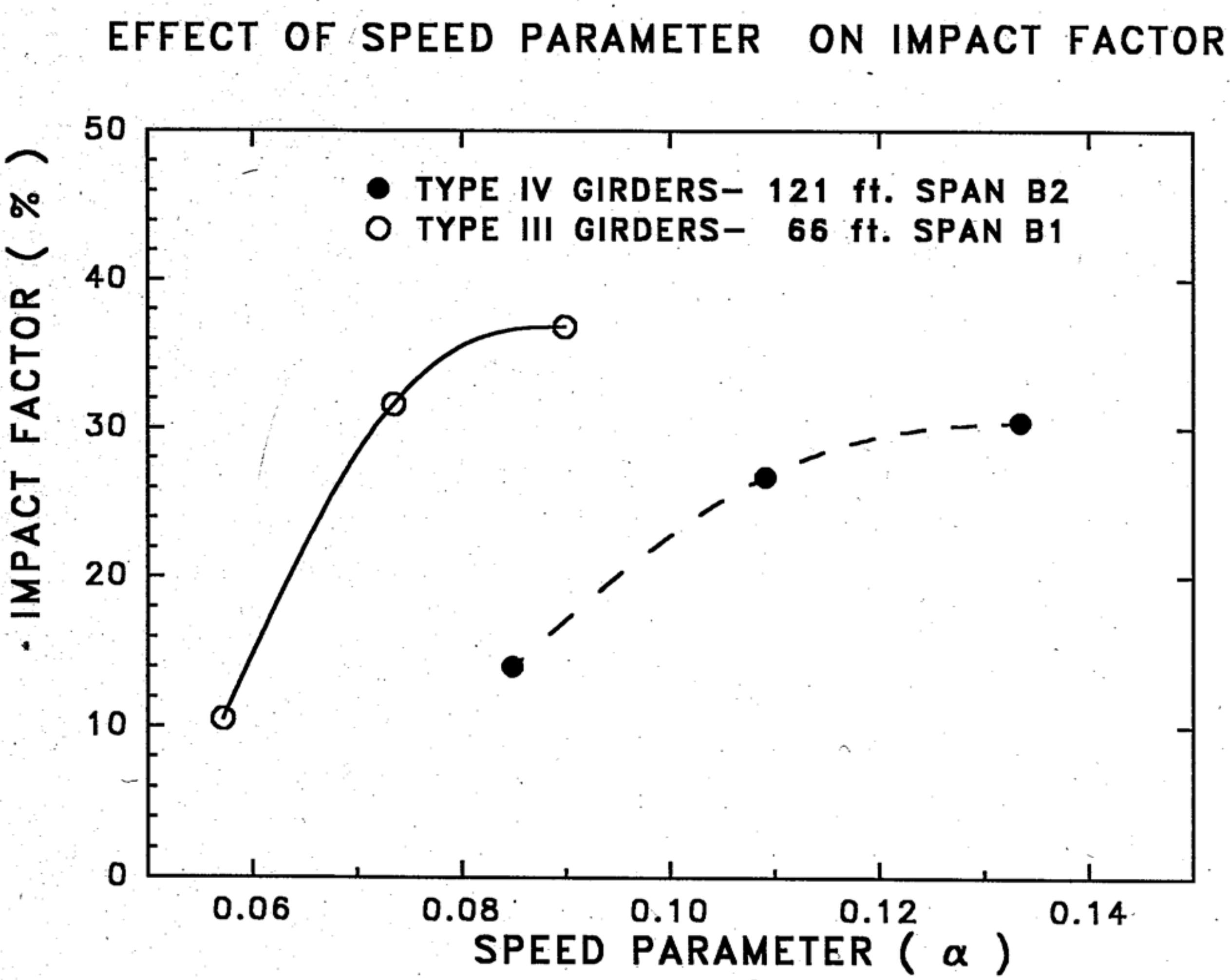

EFFECT OF SPEED ON IMPACT FACTOR
MIDSPAN DEFLECTIONS AND STRAINS FOR AASHTO TYPE IV GIRDERS
BRIDGE B-SPAN B2
SPAN LENGTH = 121 ft.
OHBDC
EXPERIMENTAL DEFLECTION
AASHTO
EXPERIMENTAL STRAIN
IMPACT FACTOR ( % )
VEHICLE SPEED ( MPH )
0
10
20
30
40
50
60
70
80
90
100
30
35
40
45
50
55
60

FIGURE 15




$\alpha = VT/2L$

FIGURE 16

KEY WORDS: Dynamic, Static, Prestressed Concrete Bridges, Frequencies, Strain, Deflection, Acceleration, Instrumentation, Amplification Factor, Speed, Impact, Bridge Load Test.