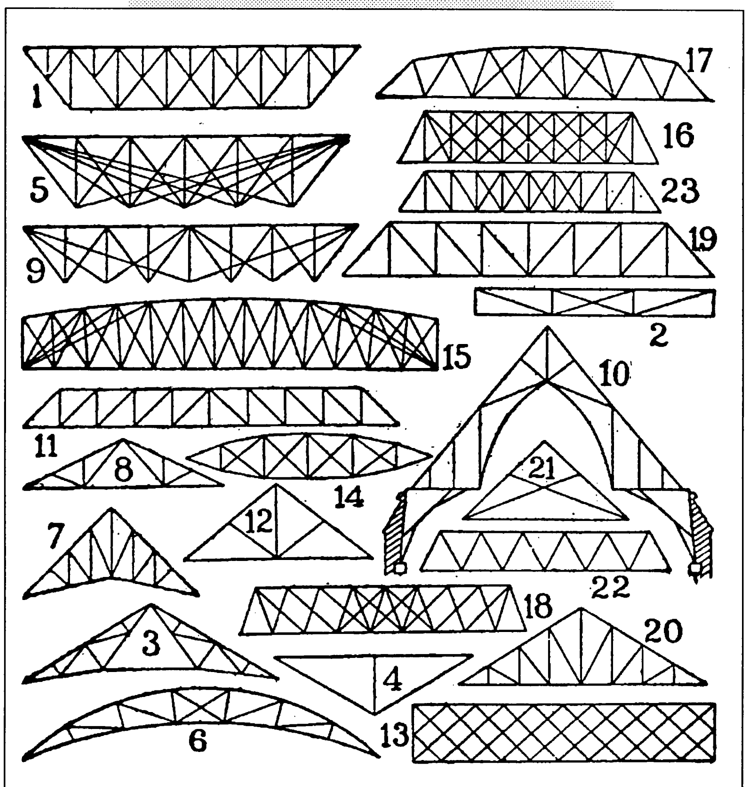
FINAL REPORT

STRUCTURAL STIFFNESS IDENTIFICATION OF BRIDGE SUPERSTRUCTURES



PETER W. HOADLEY, Ph.D.
Associate Professor
Department of Civil Engineering
Virginia Military Institute

JOSE P. GOMEZ, Ph.D.
Research Scientist

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Structural Stiffness Identification of Bridge Superstructures

Authors

Peter W. Hoadley, Ph.D., and Jose P. Gomez, Ph.D.

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530 Edgemont Road
Charlottesville, VA 22903

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Virginia Department of Transportation
1401 E. Broad Street
Richmond, VA 23219

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Abstract

Accurate measures of bridge stiffness are important when determining structural integrity. This information should be an integral part of any comprehensive bridge maintenance program, especially considering the nation's aging infrastructure. Informed decisions regarding the placement or repair of an existing bridge require knowledge of the in-situ structural state. Although static and dynamic field tests can provide accurate measures of in-situ stiffness, the instrumentation necessary to conduct such tests is time-consuming and labor intensive. A need exists for an accurate, cost-effective, and time-efficient method of measuring aggregate bridge stiffness. The falling weight deflectometer (FWD) is an instrument that may provide accurate measures of aggregate bridge stiffness in a timely fashion.

The purpose of this study was to determine the feasibility of using the FWD to measure bridge stiffness. Aggregate bridge stiffness in two bridges was measured using the FWD. These stiffness values were compared to values obtained from finite element models. A static field test was conducted on one of the bridges, and the stiffness values obtained were compared with the FWD results.

The FWD has the potential to be an effective tool for measuring structural stiffness in certain circumstances and may be capable of providing bridge engineers with crucial information in a timely, cost-efficient fashion. Further calibration of the FWD is necessary before it can be used in a comprehensive bridge maintenance program.

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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ABSTRACT

Accurate measures of bridge stiffness are important when determining structural integrity. This information should be an integral part of any comprehensive bridge maintenance program, especially considering the nation's aging infrastructure. Informed decisions regarding the placement or repair of an existing bridge require knowledge of the in-situ structural state. Although static and dynamic field tests can provide accurate measures of in-situ stiffness, the instrumentation necessary to conduct such tests is time-consuming and labor intensive. A need exists for an accurate, cost-effective, and time-efficient method of measuring aggregate bridge stiffness. The falling weight deflectometer (FWD) is an instrument that may provide accurate measures of aggregate bridge stiffness in a timely fashion.

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INTRODUCTION

Accurate measures of stiffness are important when determining the structural integrity of a bridge. This information should be an integral part of any comprehensive maintenance program, especially considering the nation's aging infrastructure. Information regarding the in-situ structural state of a bridge must be known by the engineer before decisions regarding repair or replacement can be made. Accurate measures of structural stiffness are also necessary for the calibration of computer models.

Bridge stiffness is derived from a combination of the stiffness of each structural component. Bending stiffness is the primary stiffness component of horizontal members (e.g., slabs and beams), whereas axial stiffness is the primary stiffness component of vertical members (e.g., piers). In-situ stiffness measures are necessarily a summation of the stiffness of each component.

Aggregate bridge stiffness may be obtained by several nondestructive static or dynamic field test methodologies. One method computes stiffness from measured strains and displacements resulting from a static load. Lee, Ho, and Chung (1987) used static tests to verify the analytical model of a concrete bridge. Sanayei and Scampoli (1989) described how to obtain stiffness parameters from static tests. Another technique uses a bridge's natural frequency computed from measured accelerations created by a vibrating mass. An estimate of bridge stiffness can then be computed from the natural frequency. Yao published several reports describing dynamic techniques for stiffness identification (Hart & Yao, 1977), and Biswas, Pandey, and Samman (1989) described modal analysis techniques for highway bridges, which requires knowledge concerning natural frequency.

Although both static and dynamic field tests can provide accurate measures of stiffness, instrumentation for static and dynamic field tests is time-consuming and labor intensive. In addition, measurements are recorded only at specified locations. A need exists for an accurate method of measuring aggregate bridge stiffness that is more cost-effective and efficient than current static or dynamic field tests. The falling weight deflectometer (FWD), made by Dynatest, may provide accurate measures of aggregate bridge stiffness in a timely fashion. The FWD is a self-contained testing unit that is mounted on a trailer and towed by an instrumentation van. A test requires only a few minutes. The FWD is easily moved, so multiple tests at several locations on a bridge can be conducted in a short period of time.

Use of the FWD is not new to the stiffness identification process. Ullidtz and Stubstad (1985) used the FWD in the evaluation of pavements. Others have used it to measure the subgrade modulus under pavements. The Virginia Department of Transportation (VDOT) owns an FWD and uses it in the evaluation of pavements and subgrade moduli. The Naval Civil Engineering Laboratory has used the FWD to measure stiffness parameters on several wharfs owned by the U.S. Navy and to evaluate the structural integrity of a small bridge on a navy base. The information from FWD field tests was used to calibrate computer models of these structures, which were used to evaluate the need for repairs and the possible effect of making the repairs.

The FWD has proved to be an effective tool for measuring structural stiffness in certain circumstances and may be capable of providing the bridge engineer with crucial information in a timely, cost-efficient fashion. The FWD could become an integral part of a comprehensive evaluation and maintenance program for Virginia's bridges. It may be possible to use data from the FWD to develop a stiffness profile for a bridge. The history of a bridge's structural integrity could be developed over time and used in planning repairs and maintenance. VDOT's nondestructive testing personnel have shown great interest in expanding the applicability of the FWD to the evaluation of bridges.

PURPOSE AND SCOPE

The objectives of this study were:

1. Determine the feasibility of using the FWD to measure bridge stiffness.
2. Use data from the FWD to calibrate computer models.
3. Develop an evaluation program to rate the structural integrity of Virginia's bridges using information collected by the FWD.

MATERIALS AND METHODS

Description of the FWD

The FWD is a deflection testing device operating on the impulse loading principle. A weight is dropped from a preselected height onto a footplate that is connected to a baseplate by a set of springs. The weight is approximately 1.47 kN (330 lb), and the drop height can vary from 177.8 to 381.0 mm (7 to 15 in). The 304.8 mm (12 in) diameter baseplate is placed in contact with the bridge surface and helps to distribute the load uniformly over the loading area. The impulse load ranges from 22 to 66 kN (5 to 15 kips) and is varied by changing the mass and drop height. The duration of the impulse load ranges from 30 to 40 ms.

Velocities are measured with a series of velocity transducers (geophones). One geophone is located at the center of the loading plate. Six additional geophones can be arranged along a line radiating out from the loading plate to a maximum distance of 3.048 m (10 ft). Displacements are obtained by numerically integrating the velocities. The peak impulse load and corresponding displacement are recorded. Figure 1 shows a schematic of the FWD. The load versus time is represented by the function *P(t).*

The FWD is a self-contained unit that is mounted on a trailer and pulled by a van (see Figure 2). The data are recorded by a unit in the van, and the test is run by a laptop computer. One test takes only a few minutes and requires only one operator. A tremendous amount of useful information can be gathered in a short period of time.

The data from the FWD can be used to estimate the stiffness of a bridge at a particular point. The load measured by the FWD divided by the corresponding displacement represents the vertical stiffness at that point in the bridge superstructure. A stiffness profile can be developed by measuring stiffness at several locations. These data can be used to monitor structural degradation and calibrate computer models.

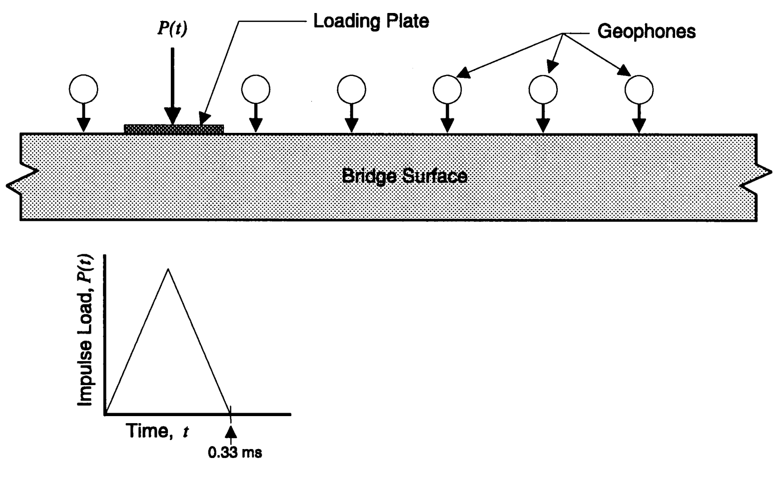

Figure 1. Schematic of the Falling Weight Deflectometer and Impulse Load Profile

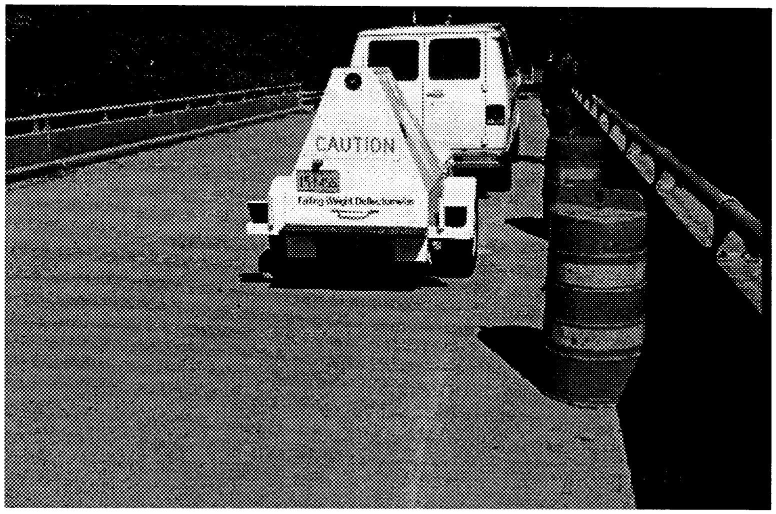

Figure 2. Falling Weight Deflectometer

Study Bridges

Two bridges were chosen to test the applicability of the FWD. The first was a two-lane prestressed concrete bridge over I-85 on Rt. 619 in Dinwiddie County, Virginia. The second was a steel bridge with a composite concrete deck that carries two lanes of a divided highway over the Dan River on Rt. 265 South near Danville, Virginia.

RESULTS

Rt. 619 Bridge

On July 8, 1994, an accident involving a recreational vehicle resulted in a fire on the northbound lane of I-85 directly under the bridge on Rt. 619 in Dinwiddie County. The bridge consists of six simple spans made up of precast, prestressed concrete beams and a cast-in-place concrete deck. The bridge was constructed in 1978 and is shown in Figure 3.

Following the accident, periodic visual inspections by VDOT's Richmond District Bridge Office indicated progressive spalling and deterioration of concrete on some of the prestressed beams. A series of nondestructive field tests were conducted to assess the amount of damage. Nondestructive measurements included ultrasonic pulse velocity, natural frequency determination, Schmidt impact hammer, and superstructure stiffness as measured by the FWD. A report describing the results of these tests was completed by research scientists at the Virginia Transportation Research Council (VTRC) in November 1994 (Lozev, Gomez, & Hoppe, 1994).

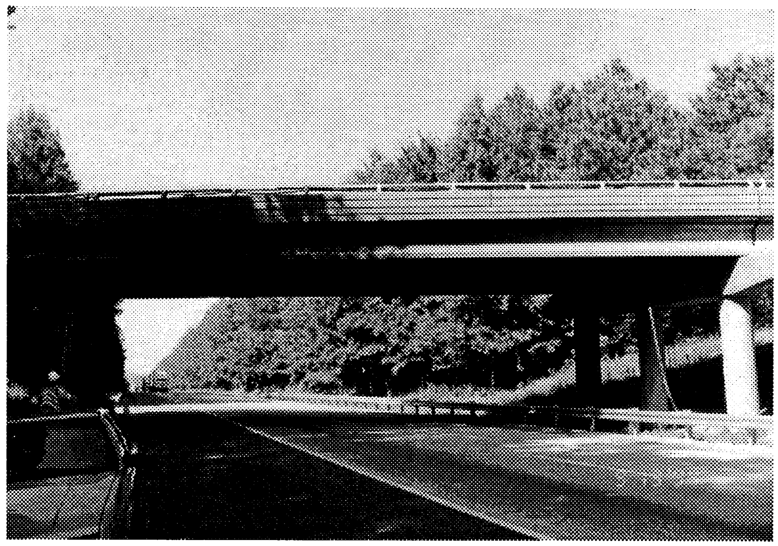

Figure 3. Burned Span of Rt. 619 Bridge

A schematic of the elevation and cross-section of the bridge is shown in Figure 4. Superstructure stiffness measurements were taken by the FWD at about 3.048 m (10 ft) intervals in the damaged span and in the identical span over the southbound lane of I-85. The location of the FWD loading plate in relation to the damaged girders is shown in Figure 4.

Stiffness measurements of the bridge were taken at 3.048 m (10 ft) intervals from the abutment at the west end. A maximum impulse load of 46.7 kN (10,500 lb) was applied at each location. The FWD mass was dropped 3 times at each location, and an average load and displacement were recorded. The load was then divided by the displacement to obtain a stiffness value for the bridge at a given location.

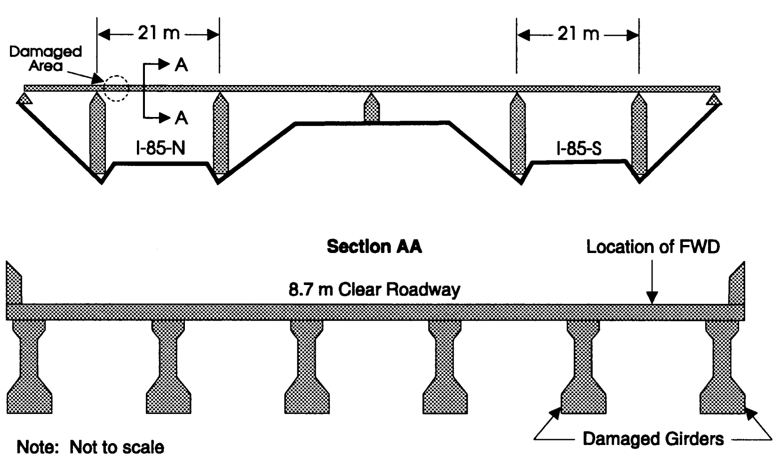

Figure 4. Schematic of Elevation and Cross Section of Rt. 619 Bridge

Figure 5 shows the stiffness versus distance from the east abutment for the entire bridge. Stiffness was the greatest at the supports and the least near the centerline.

Figure 6 shows the stiffness versus distance from the support for the spans over the northbound and southbound lanes of I-85. The stiffness at the supports, about 0.75 GN/m (4,200 kips/in), was greater than the stiffness at the center, which was just over 0.18 GN/m (1,000 kips/in). The measured values for the support at 21.336 m (70 ft) were not consistent with the measurements at 0 m (0 ft). Since the span of the bridge is 21.0312 m (69 ft), the FWD may have missed the west support when the test was conducted.

Since the two spans yielded almost identical stiffness values, the damage to the pre-stressed girder did not significantly affect the aggregate stiffness of the span. More detailed conclusions regarding the extent of the damage can be found in the VTRC report by Lozev, Gomez, and Hoppe (1994).

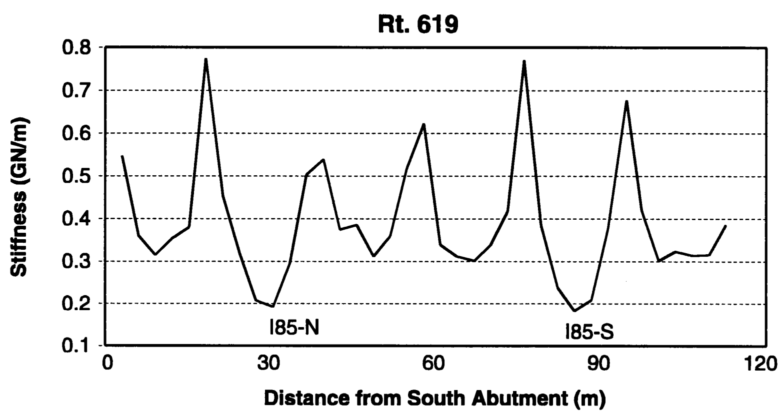

Figure 5. Stiffness Versus Distance as Measured from East to West

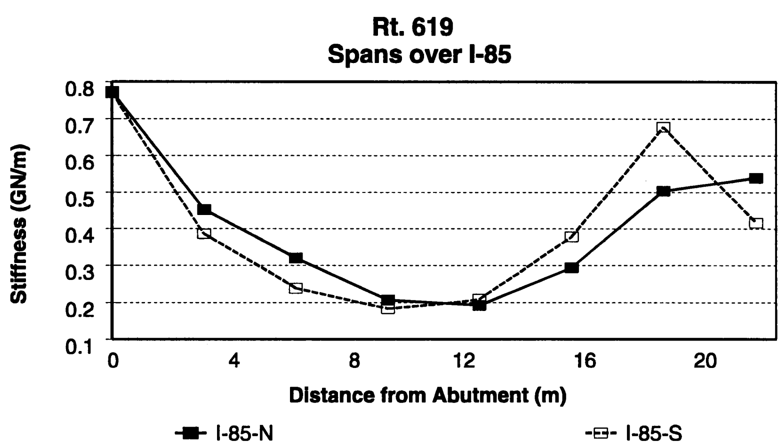

Figure 6. Stiffness Versus Distance for Spans Over Northbound and Southbound Lanes of I-85

The purpose of this study was to evaluate the information gathered by the FWD to determine its usefulness to the engineer. The FWD results shown in Figure 6 correlated well with the expected results. The stiffness near the center of the span was less than at the supports. The question remained as to how well the stiffness value obtained by the FWD correlated with the true value. A finite element model (FEM) of the bridge was developed to determine if the stiffness values as measured by the FWD could be duplicated analytically.

The bridge was modeled using the ALGOR finite element software. Six linear beam elements were used to model the prestressed girders. Plate elements connected to the beam elements with a rigid link were used to model the bridge slab. Table 1 lists the estimated cross-sectional properties of the girder and concrete. A modulus of 24.8 GPa (3,600 ksi) was used for the concrete.

Table 1
Cross-Sectional Properties of Rt. 619 Prestressed Concrete Girders

Depth (mm)	Web Thickness (mm)	Area (mm^2)	Moment of Inertia (mm^4)
1066.8	152.4	3.999×10^5	4.054×10^{10}

Unit loads were placed on the FEM to estimate the aggregate stiffness of the bridge at the same locations measured by the FWD. The results are shown in Figure 7. In the FEM, the stiffness at the support is infinite and is not shown in the graph. The FEM data exhibited the same general trend as the FWD data but yielded much lower stiffness values. There are many possible sources for the discrepancy. The material and cross-sectional properties of the bridge may not be represented accurately, and the FEM may not accurately represent the slab-beam interaction.

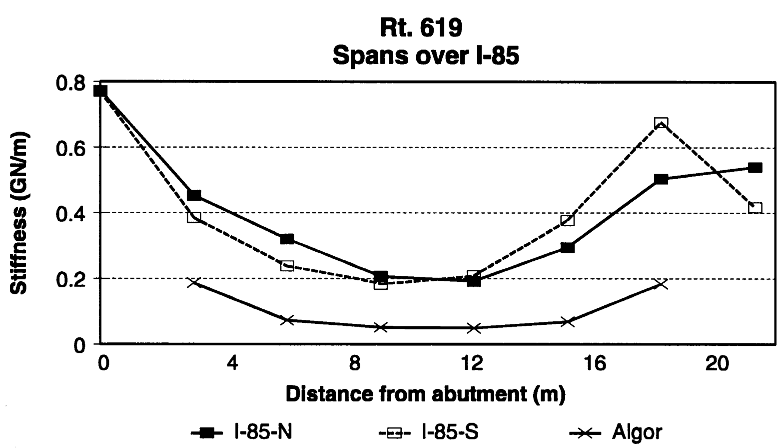

Figure 7. Stiffness Versus Distance for Spans Over I-85 Compared with Results from Finite Element Model

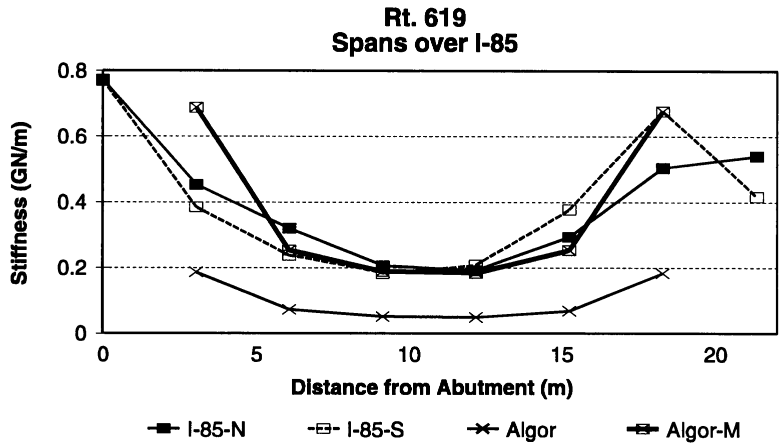

Figure 8. Comparison of Stiffness Versus Distance for Modified Finite Element Model

The FEM can be modified to represent the FWD data better by increasing the modulus of elasticity of the concrete. In an attempt to match the FEM data with the FWD data, the modulus was increased by a factor of 3.7 for the beam and plate elements. The data from the modified analysis (Algor-M) are presented in Figure 8. In the center of the span, the modified analysis and the FWD data correlated well. The points near the supports had stiffer values than those measured by the FWD because the stiffness at the supports in the FEM is defined as infinite.

Rt. 265 Bridge

Rt. 265 near Danville is a four-lane divided highway. The bridge carrying the south-bound lanes of Rt. 265 is 292.6 m (960 ft) long and divided into two four-span continuous sections. Each span is 36.58 m (120 ft) long and consists of five longitudinal steel girders supporting a composite concrete deck 2540 mm (10 in) thick. A schematic of the cross section is shown in Figure 9.

The flange thickness for each steel girder changes along the length of the girder. The different cross-sectional properties of the girder are shown in Table 2 as a function of the distance from the south abutment. The girders are symmetric about their centerline.

The FWD was used to test one 146.3 m (480 ft) section of the passing lane on the south-bound lane. The FWD was placed at three locations, as shown in Figure 9. Two tests were conducted at each location beginning at the center of the bridge and moving to the south abutment at 6.09 m (20 ft) intervals. Figure 10 shows the stiffness computations from two FWD tests and the average for girder 4 (G4, see Figure 9). The figure shows that the FWD data were replicable except for values near the supports. The stiffness at the supports (0, 36.58, 73.15, 109.73, and 146.30 m [0, 120, 240, 360, and 480 ft]) was greater than between the supports. The

Table 2
Cross-Sectional Properties of Rt. 265 Steel Girders

Distance from South Abutment (m)	Web Thickness (mm)	Web Depth (mm)	Flange Width (mm)	Top Flange Thickness (mm)	Bottom Flange Thickness (mm)	Area (sq. mm)	Moment of Inertia (mm^4)
0-23.7	11.11	1524	406.4	20.6375	31.75	38,221.92	1.561×10^{10}
23.7-31.5	11.11	1524	406.4	36.5125	31.75	44,677.3	2.064×10^{10}
31.5-41.5	11.11	1524	406.4	55.5625	55.56	62,096.6	3.145×10^{10}
41.5-48.3	11.11	1524	406.4	36.5125	36.5125	46,612.8	2.135×10^{10}
48.3-67.5	11.11	1524	406.4	20.6375	20.6375	33,709.6	1.328×10^{10}
67.5-72.0	11.11	1524	406.4	36.5125	36.5125	46,612.8	2.135×10^{10}

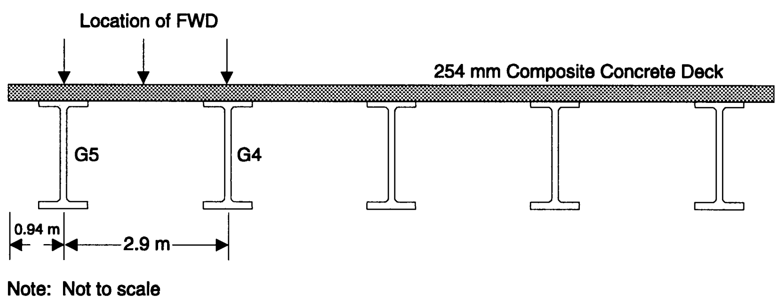

Figure 9. Schematic of Cross Section of Rt. 265 Bridge

stiffness should decrease steadily from the support to the centerline of the span, but the stiffness between the supports was surprisingly constant at about 0.18 GN/m (1,000 kips/in).

Figure 11 shows the stiffness computations from the FWD data for girder 5 (G5, see Figure 9). The computed stiffness at the supports was much more consistent for girder 5 than for girder 4. The stiffness values were the highest at the supports and decreased steadily to the centerline. The minimum stiffness at the centerline was about 0.14 GN/m (800 kips/in).

Figure 12 shows the stiffness data for the slab between girders 4 and 5. The stiffness at the support was somewhat less than for girders 4 and 5. The minimum stiffness at the centerline was about 0.14 GN/m (800 kips/in).

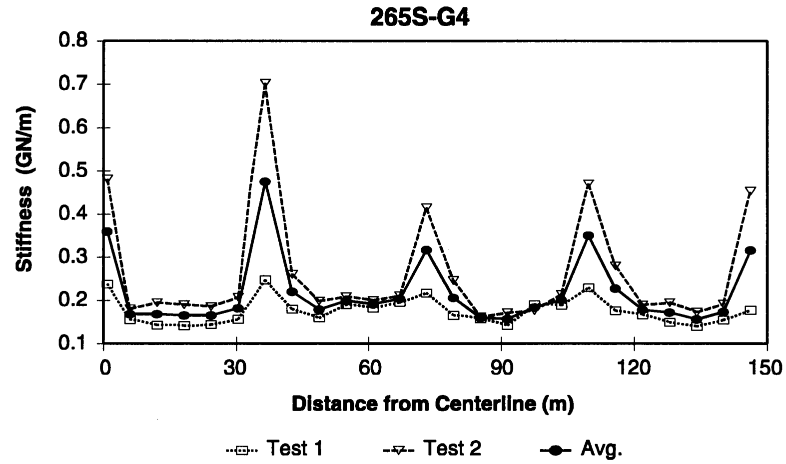

Figure 10. Stiffness Versus Distance Over Girder 4 for Two Tests Plus the Average

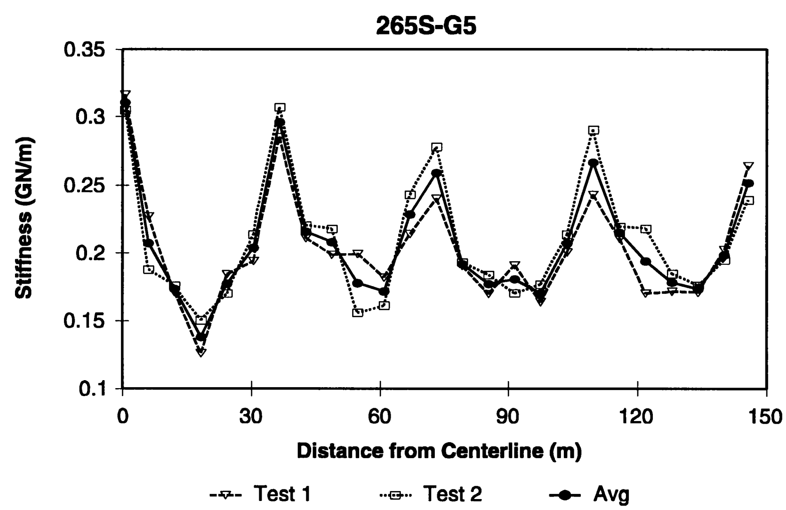

Figure 11. Stiffness Versus Distance Over Girder 5 for Two Tests Plus the Average

Figure 13 shows the average stiffness for girders 4 and 5 and the slab between the girders. The stiffness was almost identical at each location. This may indicate that the concrete slab had sufficient stiffness to effectively transfer vertical loads laterally to adjacent girders.

An FEM of the bridge was developed using ALGOR. The model consisted of linear beam elements representing the longitudinal girders and plate elements representing the slab. The beam and slab elements were connected by a rigid link from the center of the cross section of

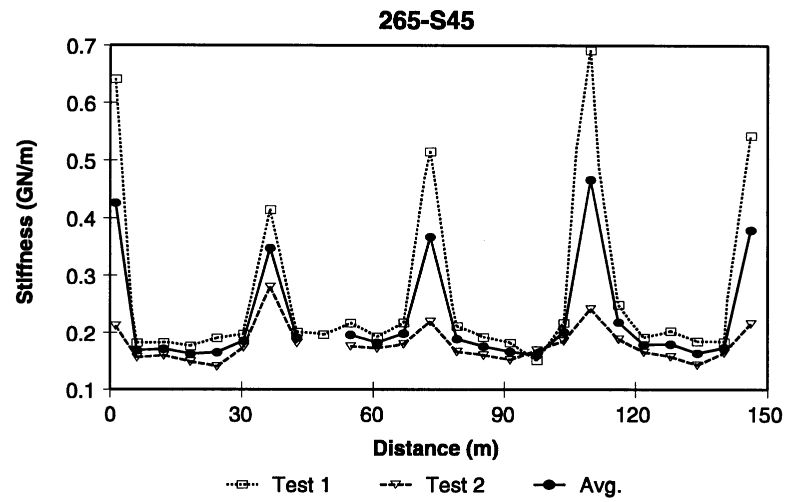

Figure 12. Stiffness Versus Distance Over Deck Between Girders 5 and 6 for Two Tests Plus the Average

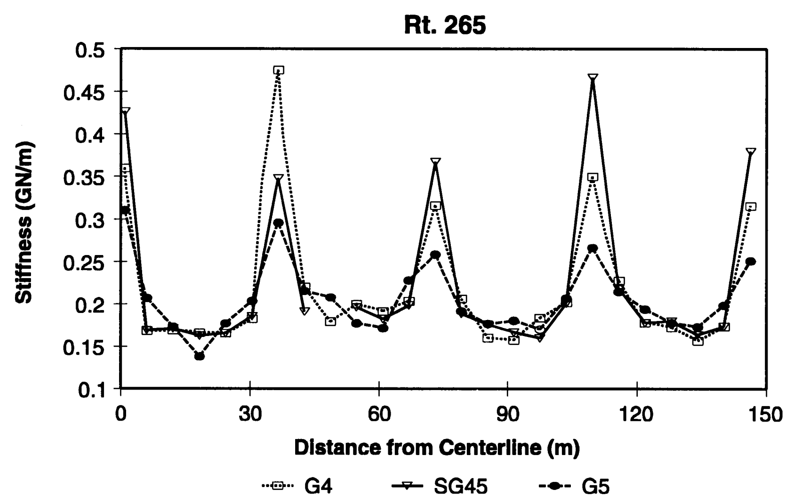

Figure 13. Stiffness Value Averages for Tests Over Girders 4 and 5 and the Deck Between Them

the beam to the center of the slab. The moduli for the steel and concrete were assumed to be 206.8 GPa (30,000 ksi) and 24.8 GPa (3,600 ksi), respectively.

Figure 14 shows the stiffness of girder 4 as computed by ALGOR and as computed from the FWD data. The general trend was similar, but the values as computed by ALGOR were much smaller than those computed from the FWD data. Figure 15 shows the same data for girder 5. Again, the stiffness as computed by ALGOR was less than that computed from the

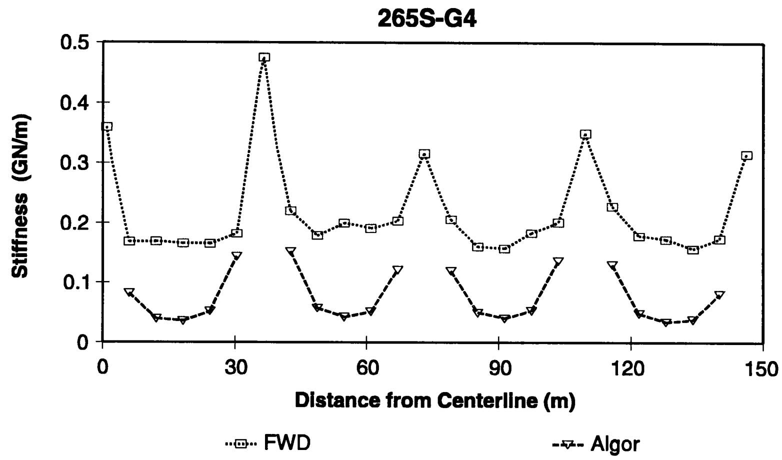

Figure 14. Comparison of Stiffness Values for Girder 4 as Determined From FWD and Finite Element Model

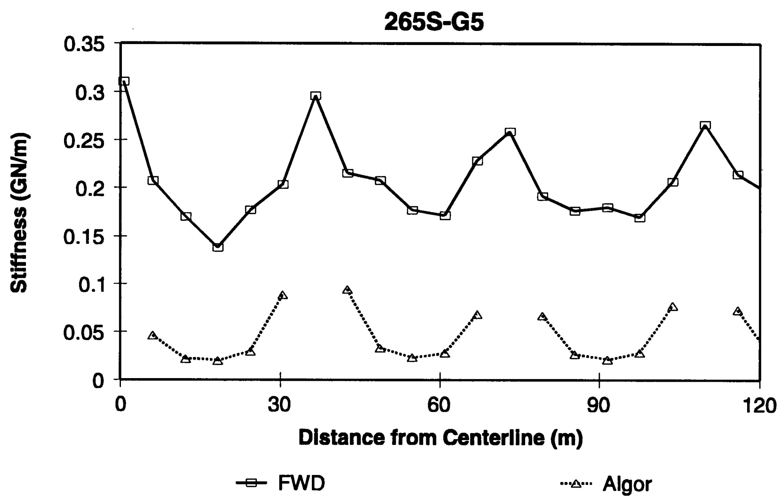

Figure 15. Comparison of Stiffness Values for Girder 5 as Determined From FWD and Finite Element Model

FWD data. The stiffness at the supports from the FEM is not included since it is infinite by definition.

The FEM was modified in an attempt to represent the measured data better. The moduli for the steel and concrete were increased. The results were plotted with the original FEM results and FWD data in Figures 16 and 17 for girders 4 and 5, respectively. Since the stiffness computations near the supports from the FEM reflected the perfectly rigid boundary conditions, the data near the ends should not be used for comparison. The correlation between the FWD and FEM data was better, but this was due to a modulus that was unreasonably high.

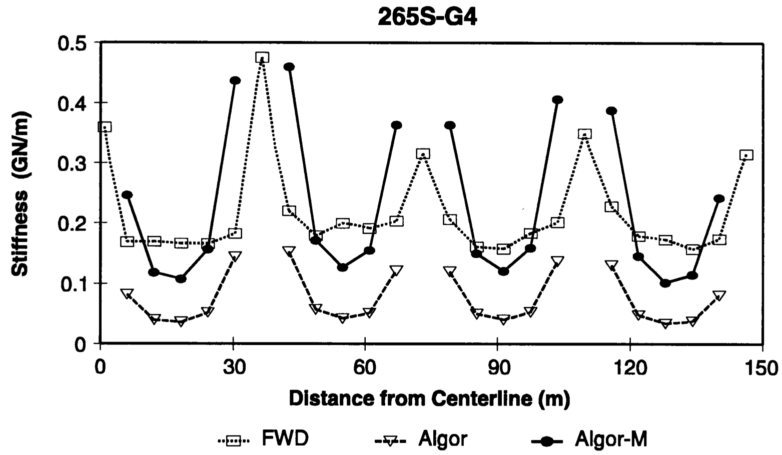

Figure 16. Comparison of Stiffness Values for Girder 4 as Determined From FWD and Original and Modified Finite Element Models

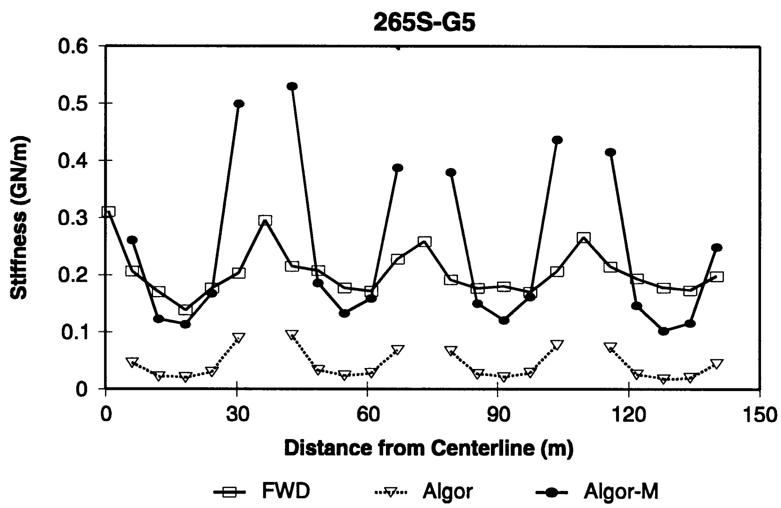

Figure 17. Comparison of Stiffness Values for Girder 5 as Determined From FWD and Original and Modified Finite Element Models

A static and dynamic load test was conducted in the fall of 1995 on this bridge. Linear variable differential transformers (LVDT) were placed at each girder along the midspan of the second span in from the south abutment of the southbound structure. A tandem truck loaded with aggregate and weighing 227.8 kN (51.2 kips) was placed at the quarter points along this span. Deflection measurements were determined with the LVDTs. These results are shown in Table 3 along with deflections determined from ALGOR at the identical locations in the computer model. The field measurements agreed well with the computer model, with the average difference being 16%.

Table 4 shows stiffness calculations based on the static load test, the FEM, and direct results from the FWD field test. Again, the FEM and static test results agreed quite well. However, upon comparing the static test results and the FWD results, the FWD results were shown to be consistently higher by a factor of 2.5.

Table 3
Static Data and Finite Element Results

Girder	**Result**	**9.14 m**	**18.29 m**	**27.43 m**
1	Static Test	3.5814 mm	5.2070 mm	3.3274 mm
	ALGOR	3.1243	6.1722	3.1243
2	Static Test	3.0480	4.9022	2.9464
	ALGOR	2.9718	5.3594	2.9210
3	Static Test	1.9558	2.1082	1.9050
	ALGOR	1.8796	3.5560	1.8796
4	Static Test	1.9022	1.5494	0.9906
	ALGOR	0.9144	1.8288	0.9144
5	Static Test	0.1270	0.2032	0.1524
	ALGOR	0.1016	0.2794	0.1270

Table 4
Calculated Stiffnesses (MN/m) from Static Data, FEM, and FWD

	9.14 m	**18.29 m**	**27.43 m**
Static	74.8	46.4	77.2
ALGOR	76.5	42.5	77.9
FWD	192.5	157.5	183.8

CONCLUSIONS

- Aggregate bridge stiffness values were obtained rapidly in all cases.
- Stiffness values were largest at the supports and smallest in the central portion of the spans, consistent with the expected structural response.
- The stiffness values obtained for the Rt. 619 bridge were not significantly different for the damaged and undamaged spans.
- FEM analyses yielded lower estimates of stiffness than the FWD results. The computer models were modified in an attempt to match the measured data, but the improved correlation was obtained through unreasonable assumptions.
- The stiffness values determined from FEM analysis and static test results for the Rt. 265 bridge were reasonably consistent, but results from the FWD were much higher.
- The FWD has the potential to provide a rapid estimate of the aggregate stiffness of bridge structures, although the stiffness values were consistently higher than those obtained by FEM analysis and field testing.

RECOMMENDATIONS

1. Although the FWD shows promise as a tool for determining the aggregate stiffness of bridge structures, additional tests should be conducted for calibration purposes. Since VDOT owns an FWD, further testing could be accomplished at very little additional cost.
2. The FWD should be used whenever a bridge test is conducted by VTRC staff. These static tests must be done before a final recommendation can be made regarding the use of the FWD as a component of a bridge maintenance program.

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