STATIC AND DYNAMIC FIELD PERFORMANCE OF VETERAN'S MEMORIAL CURVED STEEL BOX GIRDER BRIDGE

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Biography

Dr. Dongzhou Huang is currently a Sr. Structures Engineer PBS&J, Inc., Tampa. Engaged in bridge engineering for nearly 30 years, he has gained extensive professional and research experience in bridge design; bridge performance evaluation; dynamic, stability, and three-dimensional analysis of bridges; and has developed practical analytical and design methods for different types of bridge structures, especially box girder and curved girder bridges. He has published more than 60 technical papers, over 40 of which appear in peer-reviewed journals. He is an honorary Professor in the Department of Civil and Architectural Engineering at Fuzhou University, China.

Summary

Results of field tests and analysis conducted on the Veteran's Memorial Curved Steel-box Girder Bridge are presented. A total of 51 strain gages and 12 deflection gages were installed on the 614-foot radius bridge. Two FDOT test trucks with a total weight of up to 175.4 tons provided the static loading. One FDOT test truck with a total weight of 52.7 tons applied the dynamic loading. Three different theoretical models were developed to evaluate the test results. Test and analytical results show: (1) Current AASHTO Guide Specifications regarding the first transverse stiffener spacing at the simple end support of a curved girder may be too conservative for bridge load capacity ratings, (2) Current AASHTO Guide Specifications may greatly overestimate the dynamic loadings of curved box girder bridges with long span lengths, (3) A plan grid finite element model of about 20 elements per span in the longitudinal direction can be used to analyze the curved multi-girder bridges with external bracings located only over supports. The research results are instructive and applicable to bridge design and bridge load-rating activities.

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INTRODUCTION

Since the first edition of AASHTO's Guide Specifications for Horizontally Curved Box Girder Highway Bridges was published in 1980, there have been two more editions including two revisions to the Specifications. Some changes were based on valid research results and others were based on limited or uncertain research results and information. The current edition of the Specifications contains provisions that may result in unreasonably conservative load capacity ratings. There are many existing curved steel box girder bridges in Florida. Some of them are older than 20 years. Recently, in order to identify the actual static and dynamic behaviors of such bridges and provide more reasonable load rating capacities for existing bridges, The Florida Department of Transportation (FDOT) tested two curved steel box girder bridges. The purpose of this paper is to present the results of Veteran's Memorial Bridge field testing and impart better understanding of the static and dynamic behaviors of curved steel box girder bridges subjected to actual truck loading. First, brief descriptions of the bridge are given. Then, bridge instrumentation and test procedures are depicted. Finally, test and analytical results as well as design recommendations are discussed.

Figure 1: Curved Steel Box Girder Bridge

BRIDGE DESCRIPTION

Veteran's Memorial Bridge, built in 1997, is a curved steel box girder bridge that carries US 319 over Thomasville Road in Tallahassee, Florida. This bridge consists of a straight three-span continuous section and a curved three-span continuous section with an average radius of 614 ft. The curved section is the focus in this paper, unless otherwise noted. Measured along the bridge centerline, the end spans of the curved box girder bridge are 165.8 ft long and the middle span is 248.7 ft long. Overall and plan views of the bridge are shown in Figures 1 and 2. The cross-section of the bridge is comprised of two built-up steel box girders spaced 22.0 ft center-to-center and composite with the deck. All structural steel is Grade 50, except the transverse

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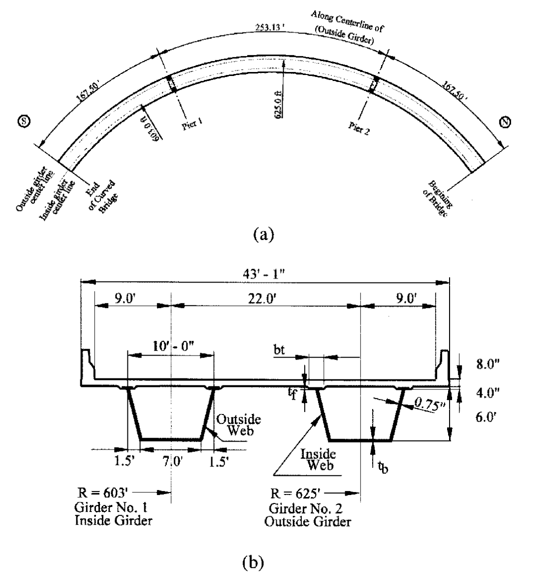


Figure 2: Veteran's Memorial Curved Bridge, (a) Bridge Plan, (b) Typical Cross-Section

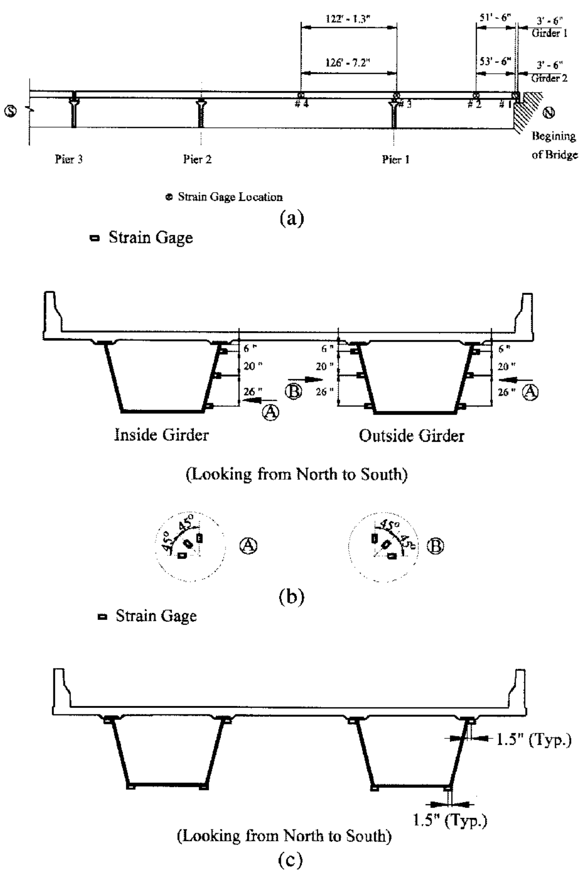


Figure 3: Strain Gage Locations, (a) Longitudinal Direction, (b) Transverse Direction at Section #1, (c) Transverse Direction at Sections # 2 to 4.

stiffeners are Grade 36. Top flange thickness varies from 0.875 to 2.75 inch and bottom flange thickness varies from 0.5 to 1.25 inch. The deck is 8.0 inches thick and 43.1 ft wide from outside to outside. Cross frames are spaced at 10.1 ft, except the end spacing is 8.9 ft. There is no external lateral bracing between box girders, except at sections over the supports. The principal dimensions are shown in Figure. 2.

INSTRUMENTATION

Strain Gages

Four critical sections were instrumented with strain gages as in Figure 3 to monitor the bridge response to test trucks. The 27 strain gages at Section #1 were used to monitor the shear response (Figure 3(b)). Sections #2 to #4 were each instrumented with eight strain gages located as shown in Figure 3(c), and oriented in the longitudinal direction of the bridge. Strain gages at Section #2 to #4 were used to monitor the section bending behavior.

Deflection Gage

A total of six Linear Variable Differential Transformer (LVDT) electrical transducer deflection gages were mounted at Section #1 of the outside girder, and three on each web. Section # 1 is located mid-way between the end diaphragm and the first transverse stiffener (Figure 4). The top and bottom gages were used to monitor torsion deformations of the girder, while the middle deflection gages were used to monitor local web deformation. Based on the current AASHTO Guide Specifications for Horizontally Curved Highway Bridges, the shear capacity at this location is comparatively low because the spacing of transverse stiffeners at the simple supports does not meet AASHTO Guide Specifications' requirement of one-half the web depth or less. The deflection gages were used to monitor the buckling behavior at Section #1. A total of six Displacement Transducer (DT) displacement gages (see Figure 5) were used to measure the bridge vertical displacements in order to monitor structural integrity and detect possible significant hidden defects. Deflection gages were located under the bridge deck to avoid interference with traffic and near mid-span to maximize the magnitude of the measured displacements.

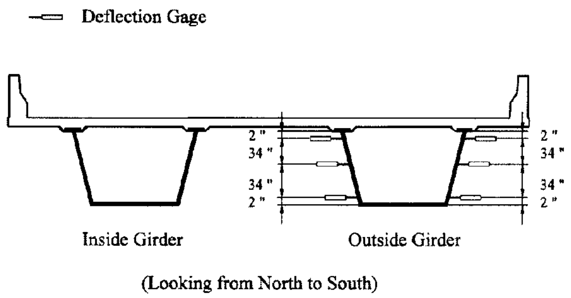


Figure 4: LVDT Deflection Gages at Section #1.

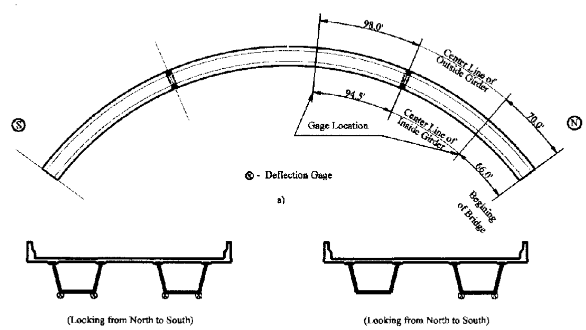


Figure 5: Locations of Vertical Deflection Gages, (a) plan, (b) Cross-section of Side Span, (c) Cross-section of Center spans.

TEST PROCEDURE

Static Test – Bending Capacity

Two Florida Department of Transportation Research Center test trucks (see Figure 6) were used to test the bridge under live load. The axle weights corresponding to the number of steel blocks loaded are presented in Table 1.

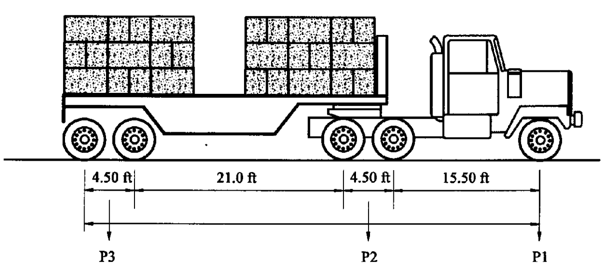


Figure 6: Test Vehicle

In order to detect bending behavior, a total of four static loading positions with two trucks side by side were determined to be the most unfavorable (stress-inducing) loading positions corresponding to each related span and girder. The detailed longitudinal and transverse loading positions are shown in Figures 7 and 8 (Case Nos. 3 to 6). Each truck carried 36 steel blocks for the static bending test. Initial readings of all gages were recorded with no loads on the bridge. The trucks were then driven and parked at the predefined critical load positions on the bridge. Strain and deflection readings were recorded for each loading case. After each loading case, the trucks were driven off the bridge and another zero-load reading was taken. The measured data was immediately displayed and compared with the theoretical predictions to reveal any anomalies.

Table 1. Tandem Weight of the Test Truck

Number of Blocks	Total Weight (kips)	Front Tandem P1 (kips)	Drive Tandem P2 (kips)	Trailer Tandem P3 (kips)
30	106.23	11.67	42.92	51.64
36	118.03	11.75	47.76	58.52
42	129.85	11.85	52.60	65.40
48	141.63	11.93	57.42	72.28
54	153.42	12.02	62.24	79.16
60	165.22	12.10	67.08	86.04
66	177.01	12.19	71.92	92.90

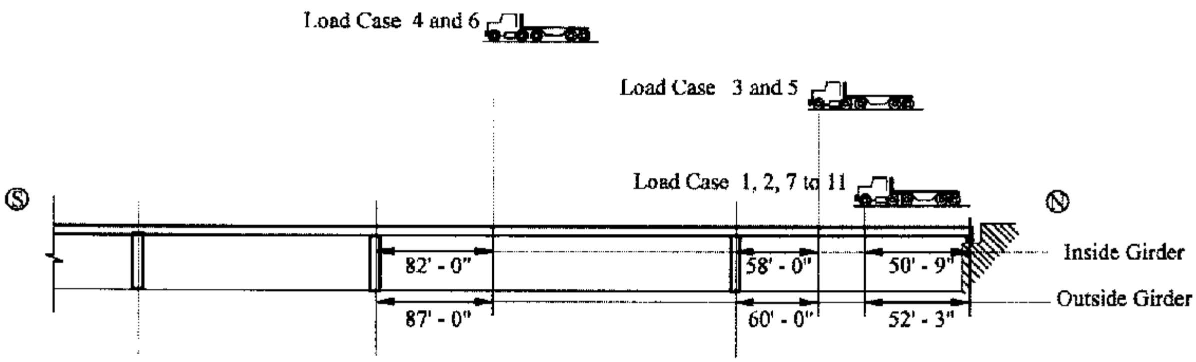


Figure 7: Truck Longitudinal Locations

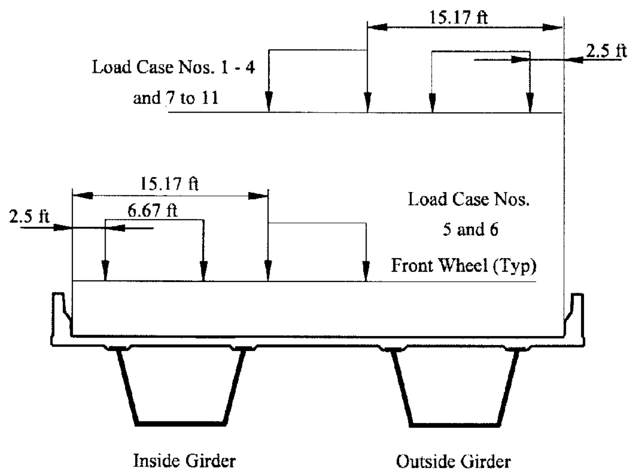


Figure 8: Truck Transverse Locations

Static Test – Web Shear

A total of seven static loading positions with two trucks side by side were chosen to identify the shear behavior at the end section of the bridge. Detailed longitudinal and transverse loading positions are shown in Figures 7 and 8 (Case Nos. 1, 2, 7 to 11). The test trucks were incrementally loaded from 30 blocks to 66 blocks. Strain and deflection readings were recorded and carefully observed for each loading case.

Dynamic Test

One FDOT test truck with 30 steel blocks, weighing slightly more than the HS20-44 design truck, was chosen for the dynamic testing. The test truck with 30 steel blocks has a total weight of 106.0 kips. The test truck was run along both inside and outside girders, individually, approximately 4.0 ft from the curb line. The truck speed was incrementally increased from a crawl to the design speeds of 35 mph.

TEST AND ANALYTICAL RESULTS

Static Test – Bending

Detailed test results can be found in Huang, et al (1). Only the maximum stresses at the critical sections are presented in this paper. Table 2 presents the maximum test and analytical normal stresses at two critical sections. The analytical normal stresses were determined based on three different finite element models (12) of plane grid (Figure 9(a)), curved grillage beam (Figure 9(b)), and shell-plate (Figure 9(c)).

In Plane Grid and Curved Grillage Models (2), the stiffness of the transverse beam element is determined based on the whole width of the element. The maximum warping stress is approximately evaluated by treating the box girder as an I-girder and using the warping equation for an I-girder (10). There are four curved lines in Figure 9(b). The inside curved lines represent the two curved box girders with equivalent section properties of

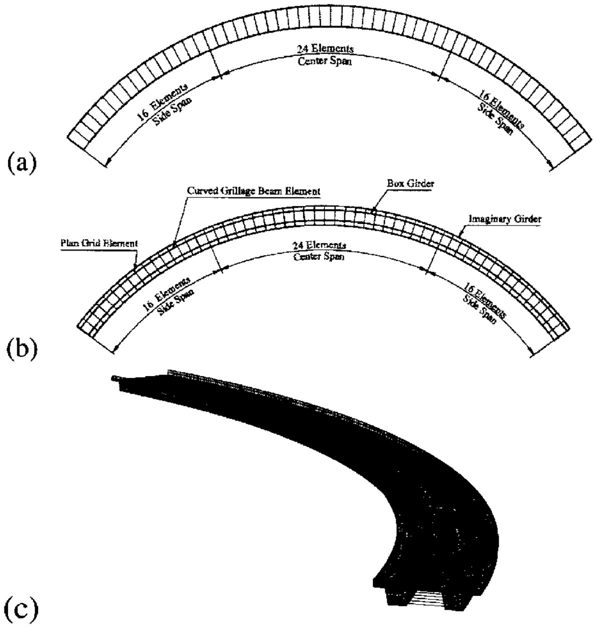


Figure 9: Analytical Model, (a) Plane Grid, (b) Curved Grillage Beam, (c) Shell-plate

a general box configuration, while the other two lines are imaginary girders whose stiffness is equivalent to that of the deck and are very small in comparison to the box girder. The imaginary girders mainly play a role in distributing the wheel loads to the related joints. Each curved girder is divided into 56 elements in the longitudinal direction. In the shell-plate finite element system, the bridge deck, barriers, and steel box girders are divided into a series of quadrilateral shell-plate elements, while the bracings and stiffeners are treated as three-dimensional frame elements. The barrier expansion joints were modeled. There are a total of 21,555 shell-plate elements and 5,417 three-dimensional frame elements.

In Table 2, Case 1 indicates the cross-sectional properties determined according to the original design assumptions with deck concrete modulus of elasticity of 3.824 x 10^6 psi, neglecting the barrier effect, while Case 2 represents the cross-sectional properties evaluated based on the test results with a deck concrete modulus of elasticity of 5.2 x 10^6 psi, considering the barrier section to be effective. In Table 2, the difference ratio is defined as:

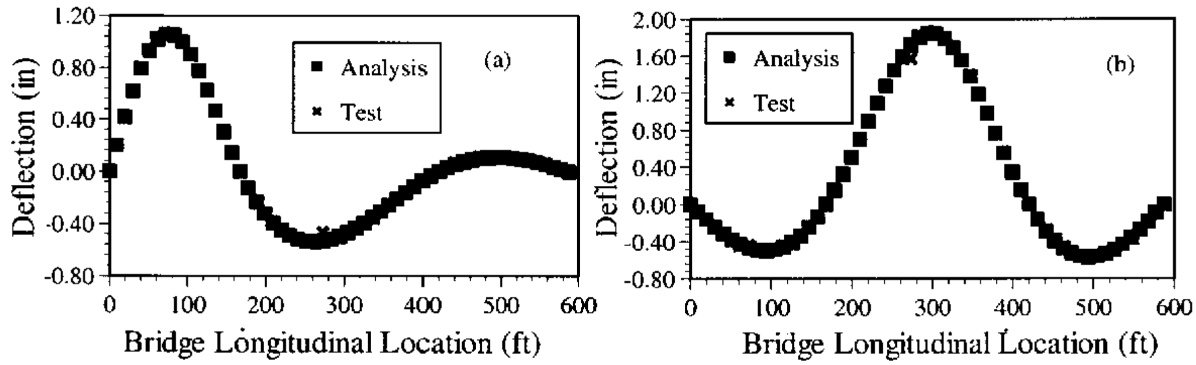
$$\text{Difference} = \frac{\text{Analytical} - \text{Test}}{\text{Test}} \qquad (1)$$

Table 2. Comparison of Maximum Normal Stresses at Bottom Flange

Section	Girder	Maximum Stress (ksi)						Different Ratio				
		Plane Grid		Curved Beam		Shell-Plate Case 2	Test	Plane Grid		Curved Beam		Shell-Plate Case 2
		Case 1	Case 2	Case 1	Case 2			Case 1	Case 2	Case 1	Case 2	
2	Outside	9.031	8.388	8.811	8.167	7.754	7.867	14.8%	6.6%	12.0%	3.8%	-1.4%
	Inside	7.741	7.125	7.963	7.062	6.985	6.731	15.0%	5.9%	14.0%	4.9%	3.8%
4	Outside	7.990	7.448	7.944	7.398	7.268	6.642	20.3%	12.1%	19.6%	11.4%	9.4%
	Inside	6.878	6.327	6.629	6.083	6.051	5.656	21.6%	11.9%	17.2%	7.5%	7.0%

From Table 2, it can be observed: (1) The difference ratios between the test and analytical results predicted by the plane grid model for Case 1 range from 15% to 22%, while for Case 2 they vary from 6% to 12%, (2) The difference ratios between the test and analytical results predicted by the curved grillage beam model for Case 1 range from 12% to 20%, while for Case 2, they vary from 4% to 11%, (3) The maximum difference ratio between the test and analytical results predicted by the shell-plate finite element model for Case 2 is less than 10%, (4) The actual deck concrete strength and barrier effect reduce the maximum normal stresses at some control sections by about 8% to 10%.

Figure 10 illustrates two typical comparisons of deflections between test and analytical results. The analytical results were calculated based on the actual bridge conditions identified from test results. From these figures, it can be observed that the test deflections are very close to the analytical results.



Static Test – Web Shear

Detailed test web shear stresses can be found in Huang, et al (1). Figure 11 shows web shear–shear stress curves of the outside web of the outside girder. The web shears were determined based on the plane grid model, including torsional shear that was determined by assuming that the closed thin-walled cross-section is non-deformable and that the uniform shear flow in the webs resist all external torsion. The shear stresses were determined based on the test strains. From this figure, it can be observed that all of the load-stress curves approximate a straight line. Figure 12 shows the load-deflection curves for the outside web of the outside girder. Note that all of the load-deflection curves appear to be linear and that the lateral deflections of the web are very small. From this figure, it can be observed that the maximum web shear is far less than the web buckling/shear strength.

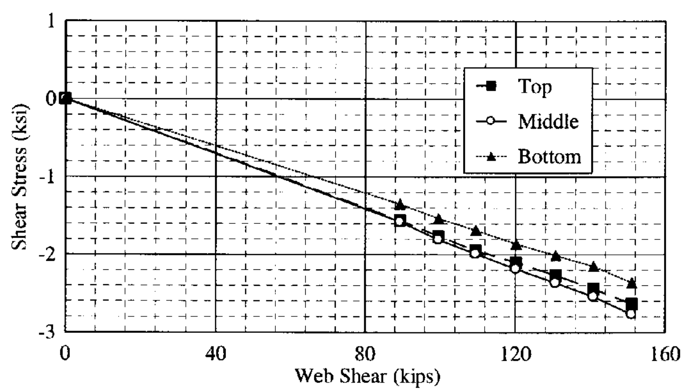


Figure 11: Web Shear-Shear Stress Curves of Outside Web of Outside Girder

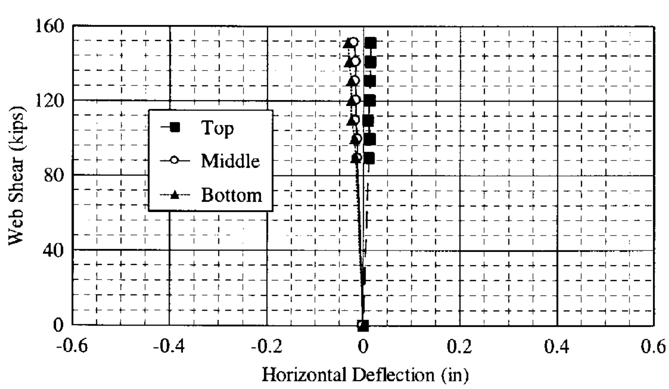


Figure 12: Web Shear – Lateral Deflection Curves of Outside Web of Outside Girder

In 1996, Davidson (9) conducted a series of nonlinear analyses of some curved steel webs subjected to pure shear force for different radii and depth-to-web thickness ratios by assuming initial deflection of one hundred-twentieth web depth. From his research, it can be concluded that: (1) The elastic buckling strength of curved web panels under pure shear is slightly greater than that of straight panels, (2) The ultimate strength of curved panels is not significantly different from the ultimate strength of straight panels, (3) For the end web of Veteran's Memorial Bridge, the load–deflection curve should be linear until the lateral displacement

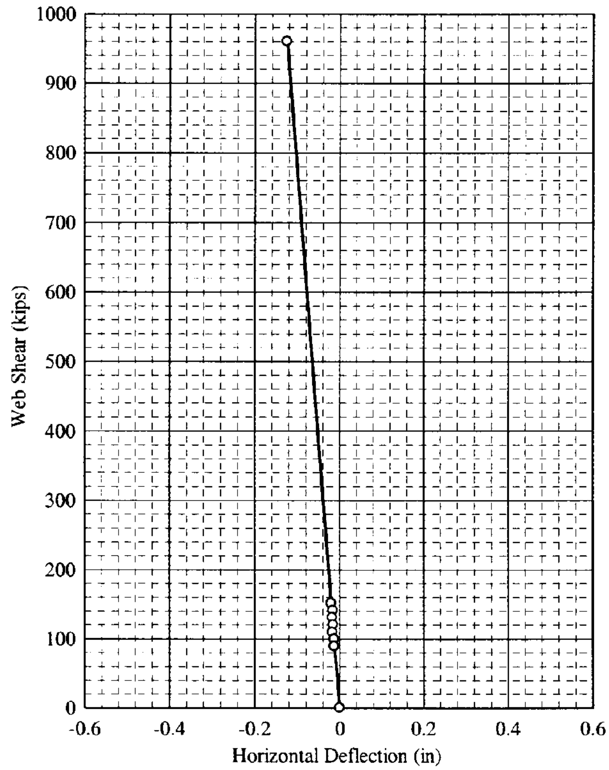


Figure 13: Prediction of Web Shear Capacity

exceeds 0.125 inch. Based on Davidson's nonlinear analytical results, we extend the field-tested shear–displacement curve to a lateral displacement of 0.125 inch and can obtain the lower bound of web shear capacity of 960.0 kips (see Figure 13). In other words, the web shear capacity can reach at least 960 kips. Due to the fact that the ratio of the stiffener spacing to web depth at the first transverse stiffener at the end supports of Veteran's Memorial Bridge does not meet the requirement specified in AASHTO Guide Specifications (3), most engineers have to treat this web as a non-stiffened (14). Then, the shear capacity of the web would be 306.6 kips, which makes the capacity rating of this bridge is very low. If the web is treated as a stiffened web, based on the research results obtained by Davidson (9), the web capacity evaluated by AASHTO codes is 932.2 kips, which is very close to the result obtained from the test results.

Dynamic Test

Some typical recorded time history curves are plotted in Figure 14 to illustrate the general concept of the dynamic effect of a moving vehicle on bridges. A moving vehicle will cause larger internal forces than its static weight will induce. In highway bridge design, impact factors or dynamic allowances are used to approximately account for the dynamic effect. To evaluate the dynamic allowance or impact factors specified in the current AASHTO guide Specifications, the test and analytical impact factors were investigated. The impact factor can be defined as

$$I_m = \frac{R_{dy}-R_{st}}{R_{st}} * 100\% \qquad (2)$$

R_{dy} and R_{st} represent the absolute maximum dynamic and static (crawl) responses, respectively. The maximum impact factors were obtained by changing vehicle speeds from crawl to the design speed and vehicle transverse positions from inside girder to outside girder. When the test results were sorted, it was found that the impact factors vary greatly from 4.8% to 101.7%. They relate to the magnitude of responses, truck locations, and bridge cross-sections. Normally, a smaller static response corresponds to a larger impact response. From an engineering point of view, the impact factors corresponding to the maximum static (crawl) responses at critical sections have practical meaning. Table 3 gives the maximum experimental and analytical impact factors corresponding to the maximum static responses. The description of the detailed theory for dynamic analysis is beyond the scope of this paper but can be found in Huang (4, 5, 12, 13). The impact factors obtained based on

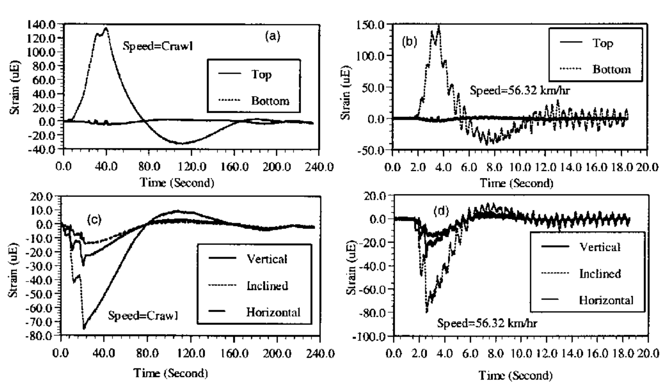


Figure 14: Typical Time Histories, (a) Strain in Outside Web of Outside Girder at Section 2 (crawl), (b) Strain in Outside Web of Outside Girder at Section 2 (35 mph), (c) Strain at Middle of Outside Web at Section 1(crawl), (d) Strain at Middle of Outside Web at Section 1 (35 mph)

AASHTO Guide Specifications (3) are also given in Table 3. From this table, It can be seen that: (1) The impact factors of normal stress for this bridge with span length larger than 50.0 m are much smaller than those predicted by AASHTO Guide Specifications 1993 Edition, and smaller still than those of AASHTO Guide Specifications 2003 Edition; (2) The analytical impact factors reasonably agree with the test results; (3) the analytical impact factors of torsion reasonably agree with the results obtained by AASHTO Guide Specifications 1993 Edition. It should be mentioned that the analytical impact factors of normal stress do not include the effect of warping stress. To check that the test impact factors can be used for bridge capacity rating, the impact factors of the bridge subjected to two AASHTO HS20-44 Design Trucks were analyzed and it was found that the impact factors induced by two HS20-44 design trucks (6) are normally smaller than those induced by one FDOT test truck (12).

TABLE 3. Maximum Impact Factors (%)

Method	Type of Truck	Girder	Normal Stress		Deflection		Vertical Shear at North End	Torsion at North End
			Section 1 (Span= 167.5 ft)	Section 2 (Span= 253.1 ft)	Side Span (Span= 167.5 ft)	Center Span (Span= 253.1 ft)		
Test	FDOT Truck	Inside	14.0	12.5	13.3	-	-	-
		Outside	8.0	11.0	14.8	13.9	6.3	-
Analysis	FDOT Truck	Inside	10.6	9.4	12.6	12.9	10.2	19.8
		Outside	9.2	8.7	9.6	9.7	9.2	21.9
	HS20	Inside	7.0	5.9	8.7	10.3	9.4	10.1
		Outside	8.4	7.1	8.1	9.2	6.0	16.3
AASHTO Guide1993	HS20	Inside	27.9	23.8	23.9	22.2	18.6	25.8
		Outside	27.8	23.6	23.8	22.1	18.4	25.8
AASHTO Guide 2003	HS20	All	35.0	35.0	35.0	35.0	40.0	40.0

CONCLUSIONS AND RECOMMENDATIONS

In this paper, the static and dynamic test results of Veteran's Memorial Bridge are presented. Fifty-one strain gages and twelve deflection gages were installed on the bridge. Three different theoretical models were developed to evaluate the test results. Through investigation, the following conclusions can be obtained.

1. If the actual geometry and material properties are known, the bridge bending behavior due to live loads and its load capacity can be accurately predicted by a finite element model. For Veteran's Memorial Bridge, the difference between the maximum experimental and analytical results obtained by using a plane grid finite element model at critical sections is less than 13%.
2. Current AASHTO Guide Specifications regarding the first transverse stiffener spacing at the simple end support of a girder may be too conservative for bridge load capacity ratings. The test results support the findings by Davidson (1996) that the equations formulated for the buckling and ultimate strengths of straight plate girders under pure shear can be conservatively used for curved girders. It is suggested that the shear capacity of a curved steel box girder at the simple end support, with the first transverse stiffener spacing smaller or equal to the web depth and with sufficient buckling strength of the bearing support stiffeners, may be evaluated as a stiffened web. Research is continuing in this area.
3. Test impact factors vary from 0.05 to 1.02. Normally the larger the static response, the smaller the impact factor. Impact factors corresponding to the maximum static stresses at critical sections are comparatively small. The current AASHTO Guide Specifications (2003) may significantly

overestimate the effect of dynamic loadings on curved steel box girder bridges with long span lengths.

4. The test results show that nearly all concrete cross-sectional areas of the bridge deck and barriers are composite with the steel box girders. For the Veteran's Memorial Bridge, the actual concrete section and barriers contribute about 9% to the capacity increase. Test results show that the maximum warping stresses at critical sections amount to about 9.3% of their corresponding bending stresses.
5. Test and analytical results show that a curved grillage finite element model or a plan grid finite element model with about 20 elements in the longitudinal direction per span can be used to analyze composite curved multi-box girder bridges with external bracings located only over supports.
6. Experimental and analytical results show that under bridge design loading (such as two trucks side by side) the entire deck-width cross section including barriers can be treated as effective concrete area for bridge internal force analysis.

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