Structural Identification of a Steel Stringer Bridge

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The concept of structural identification may help improve an understanding of the actual load-carrying mechanisms of bridges by integrating experimentation with analysis. A process of comprehensive structural identification incorporating dynamic and static tests for a 3-D finite element modeling of a three-span steel stringer bridge with continuous integral abutment is described. Researchers were able to conceptualize, then instrument and reliably measure, a number of critical local response mechanisms. These mechanisms were then incorporated in the analytical model, with resulting excellent correlation with the experiment. Bridge-rating factors, obtained by idealized models, increased by several times when the identified analytical model was used for rating. Field experimentation in the context of structural identification research greatly enhanced the reliability of the experiments and increased the benefit-to-cost ratio of the research.

A recent National Science Foundation study (*1*) defined three critical emerging research and application areas as "condition assessment technologies," "deterioration science," and "renewal engineering." The consensus is that "condition assessment" is the most important prerequisite for effective preservation (*2*).

The concept of structural identification (*3*) may hold the key for "carrying out meaningful large-scale assessments of the state of health of constructed facilities," which is identified by the National Science Foundation as a major problem in infrastructure preservation (*1*). In the last decade identification has been used as a component of structural control applications or as a tool to characterize buildings, bridges, and towers for conceptualizing their behavior; to test design assumptions; to establish effects of a loading environment; or to detect damage (*4,5*).

Inspired by the potential of the concept of structural identification, the authors have been conducting research in an effort to improve the state of the art for its implementation. Here they discuss steel stringer bridge behavior and demonstrate the potential of structural identification as a rational approach for condition assessment, damage diagnosis, and prediction of remaining capacities and service life.

OBJECTIVES AND SCOPE

The first objective is to identify local and global behavior mechanisms of continuous steel stringer bridges with integral abutments. These included modal tests by impact and vertical and lateral forced excitation, which were followed by truck-load tests for measuring global and local responses of the bridge under different static loading patterns by 60 transducers. The results of these experiments helped improve an understanding of some obscure local response mechanisms that significantly influenced bridge behavior at the service limit states.

The second objective is to present and discuss the rating coefficients of the test bridge on the basis of analyses of the experimentally identified 3-D finite element (FE) model. Significant discrepancies were observed when the corresponding rating coefficients were obtained based on analysis of idealized models recommended by the AASHTO *Guide* specifications. These discrepancies are discussed and the reasons for their existence are summarized.

STRUCTURAL IDENTIFICATION OF A STEEL STRINGER BRIDGE

The Westbound Cross-County Highway Bridge (Figure 1) in Cincinnati was selected as a test specimen because it represents a large population of bridges in Ohio. The noncomposite steel stringer bridge has two lanes, three spans (16.76 m, 23.77 m, and 16.76 m), and continuous, integral abutments. It was constructed in 1990 in accordance with the 1983 AASHTO specifications for two-lane HS 20-44 loading. It is skewed by 15 degrees 11 ft 16 in. The superstructure is composed of six 91.4-cm (36-in.) W-flange girders of ASTM A-36 steel, resting on elastomeric pads over the main piers and supporting a reinforced concrete (RC) slab 21.6 cm (8.5 in.) thick. At the abutments, the girders and deck slab are integrated together by a cast-in-place RC head-beam that rests on the abutment with a 2.54-cm (1-in.) preformed expansion joint filler. The abutment further functions as lateral brackets at each end.

3-D Analytical Modeling and Analytical Studies

An a priori 3-D FE model (Figure 2) of the bridge was constructed based on the nominal geometric and material properties presented in Table 1. Every effort was made to conceptualize and analytically simulate the 3-D geometry as well as the boundary, interelement, and span-continuity conditions of the bridge.

PC-based SAP90 (*6*) was selected as the software for structural analyses. The shell elements were used to model the

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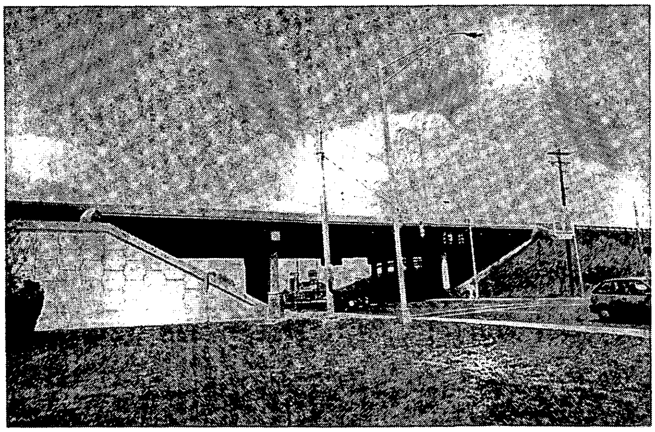


FIGURE 1 Test bridge.

deck; frame elements were used to model the girders, transverse diaphragms, pile caps, piles, and side barriers; and stiff frame elements were used to connect the elements together, preserving the local 3-D geometric attributes while ensuring interelement compatibility.

A sensitivity study of the a priori model was carried out for two reasons: to identify important response mechanisms with the associated model parameters and their possible ranges and to refine the a priori model. This sensitivity study revealed that the level of composite action between the girders and the deck, the manner of simulating the boundary conditions at the abutment, and the girder-pier continuity conditions at the piers were critical mechanisms.

An eigenvalue analysis of the a priori model was carried out to predict frequencies, mode shapes, and modal density. The results served as a guide for discretizing the bridge for

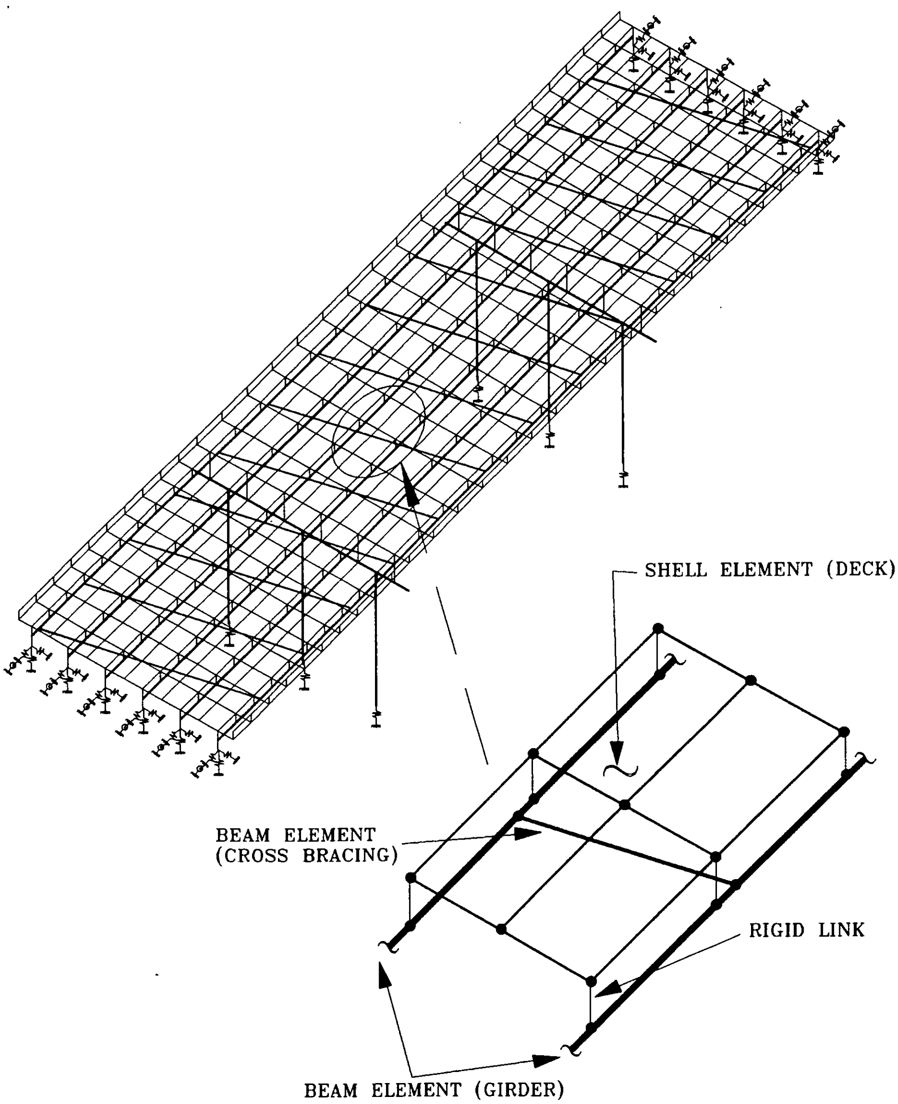


FIGURE 2 A priori finite element model of the test bridge: global attributes.

TABLE 1 Nominal Versus Calibrated Parameters of A Priori Model 1 and Model 2

Parameter	Nominal Value	Model 1 Calibrated Value	Model 2 Calibrated Value
MATERIAL PROPERTIES			
- CONCRETE: Modulus of Elasticity, E (MPa)	28042	Nominal	Nominal
: Shear Modulus, G (MPa)	10784	"	11700
- STEEL : Modulus of Elasticity, E (MPa)	200000	"	"
: Shear Modulus, G (MPa)	76907	"	"
SUPPORTING SPRINGS:			
- ABUTMENTS:			
: X-dir. translational spring (kN/cm)	Fixed	1.75E7	Fixed
: Y-dir. translational spring (kN/cm)	Fixed	1.75E7	Fixed
: Z-dir. translational spring (kN/cm)	Fixed	1.75E5	1.75E5
: Y-dir. rotational spring (kN-cm/rad)	Free	11.3E3	11.3E9
: Y-dir. rotational spring at the end of girder	Fixed	-	11.3E2
- PIER BASES:			
: Z-dir. translational spring (kN/cm)	Fixed	1.75E5	1.75E5
CROSS SECTIONAL PROPERTIES OF ELEMENTS			
- DECK SLAB (SHELL ELEMENT):			
: thickness (cm)	21.59	Nominal	Nominal
: mass density (kg/cu cm)	6.2183E-6	"	"
- STEEL GIRDER (BEAM ELEMENT):			
END SPANS (W36x150)			
: area (sq cm)	285	Nominal	Nominal
: moment of inertias (cm^4) (about major and minor axes)	376273 ; 11238	"	"
: torsional inertia (cm^4)	420	"	"
: mass / unit length (kg/cm)	5.7771E-5	"	"
MID SPAN (W36x170)			
: area (sq cm)	323	Nominal	Nominal
: moment of inertias (cm^4)	437043 ; 13319	"	"
: torsional inertia (cm^4)	629	"	"
: mass / unit length (kg/cm)	6.5473E-5	"	"
- BEARING PAD ELEMENT:			
: axial stiffness, AE/L, (kN/cm)	7215	11.68E4	5.83E4
- PILE CAP BEAM ELEMENT:			
: area (sq cm)	11239	Nominal	Nominal
: moment of inertias (cm^4)	14135760; 7828564	"	"
: torsional inertia (cm^4)	16632982	"	"
: mass / unit length (kg/cm)	6.9873E-4	"	"
- PILE COLUMN ELEMENT:			
: area (sq cm)	6568	Nominal	Nominal
: moment of inertias (cm^4)	3431745;3431745	"	"
: torsional inertia (cm^4)	6863490	"	"
: mass / unit length (kg/cm)	4.0836E-4	"	"

1 cm. = 0.3937 in.; 1 MPa. = 6.895 ksi.; 1 kN. = 4.448 kip force; 1 kg/cm = 1.7858 kip/in.

the modal tests, selecting the frequency band of interest, and locating the reference stations for optimum data acquisition during field testing.

Modal Tests

The bridge was subjected to two separate modal tests to measure the dynamic characteristics in both the vertical and horizontal directions: the modal test by vertical impact and the modal test by horizontal forced vibration.

Procedures for reliable vertical impact testing of bridges have been reported by Raghavendrachar and Aktan (*7*). The study reported here is the first-time application of multireference impact testing to a steel stringer bridge.

The modal test by horizontal forced vibration was conducted to capture the lateral response characteristics of the bridge. The forced vibration testing was performed in the horizontal direction transverse to the traffic direction. The excitations were produced by a linear inertia-mass exciter that was integrated with multichannel signal-processing software and hardware as described by Somaprasad et al. (*8*).

Some of the bridge's natural frequencies, damping factors, and mode shapes obtained from the impact and forced-vibration tests are summarized in Figure 3.

Modal Flexibility As Bridge Signature

Many researchers have recognized that the frequencies, damping coefficients, and mode shapes of bridges do not serve as reliable condition indices. The authors have made similar observations. For example, the maximum change in the measured 20 frequencies of a slab bridge, after it yielded under

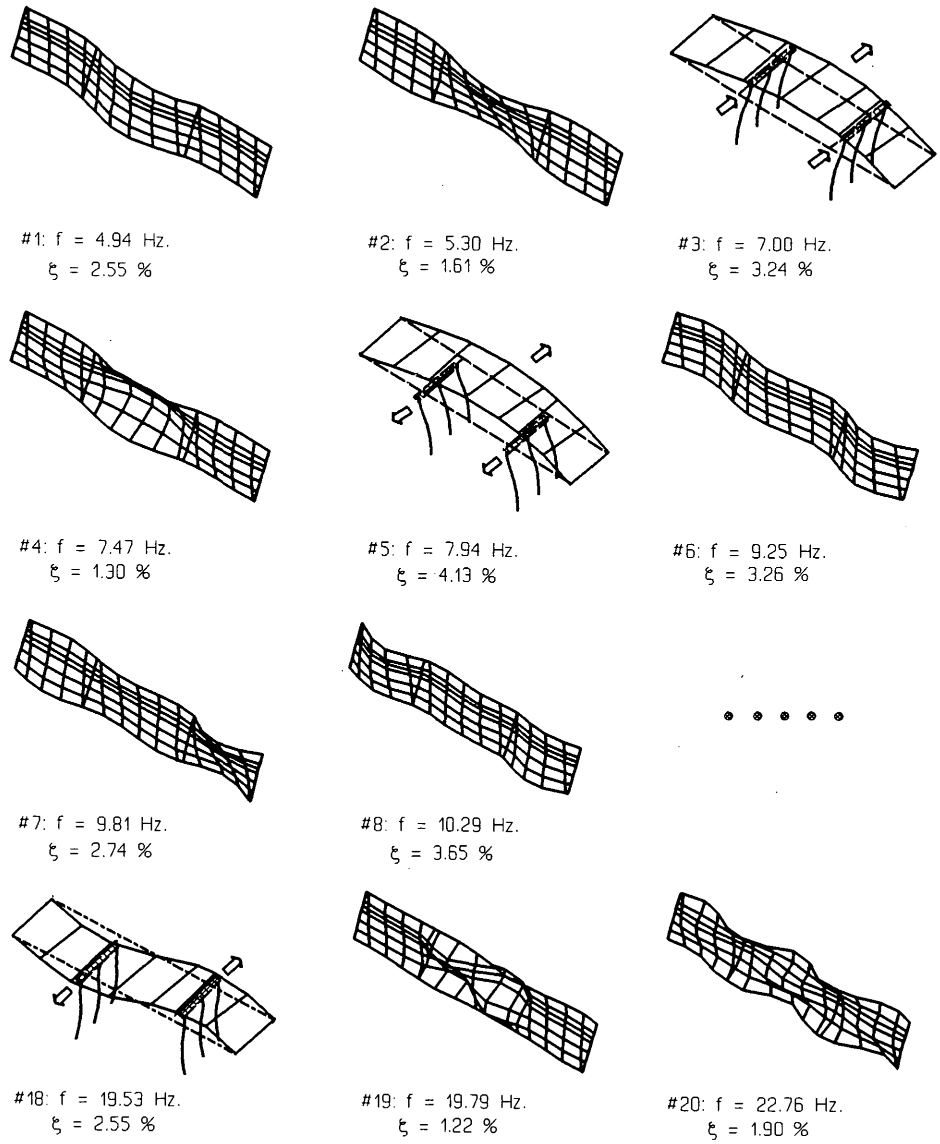


FIGURE 3 Mode shapes, frequencies, and damping ratios obtained from impact and horizontal forced-vibration tests.

a single-lane loading equivalent to 20 HS 20-44 trucks, was less than 5 percent (*9*). No appreciable changes were discerned in the mode shapes, whereas the changes measured in the damping coefficients were of the same order because of postprocessing errors. Moreover, frequency shifts in some modes on the order of 5 percent may also occur because of changes in bridge characteristics resulting from ambient effects as well as the inherent linearization, experimentation, and postprocessing errors in field modal testing of large bridges (*7*).

The authors therefore caution against using modal characteristics as signature. On the other hand, if a sufficiently large number of frequencies and mass-normalized mode shapes of a bridge may be accurately experimentally measured (generally about 20 modes would be needed), these may be transformed directly into a close measure of the flexibility matrix of the structure, termed "modal flexibility" (*7*). It should be clearly noted that multireference modal testing and postprocessing conducted with extremely stringent standards are required to accurately measure 20 mass-normalized modal vectors. Currently this may be accomplished only in the context of collaborative research between civil and mechanical structural engineers combining facility-specific experience with field modal testing of large constructed facilities.

Relative changes in the local flexibility coefficients of adjacent nodes were shown to correlate strongly to damage in redundant offshore platform towers (*10*). More recently, the authors demonstrated that modal flexibility may serve as a reliable condition index for slab bridges (*7,11*). In this research, the modal flexibility of the test bridge was used for model calibration; this is discussed in the following section.

Analytical Model Calibration

The measured dynamic characteristics, as well as the modal flexibility obtained from a transformation of unit-mass-normal modal vectors, contain a wealth of information regarding the current state of the bridge. It is important to exploit rationally this ensemble of experimental results to quantify the analytical model parameters.

Because the number of independent parameters in the analytical model typically exceeds the number of experimentally measured characteristics even when a comprehensive modal test is conducted, no unique solution to the parameter identification and model calibration problem exists. On the other hand, a proper conceptualization of the bridge's behavior is possible by incorporating heuristic and rational procedures. The band of uncertainty in the numerical bounds of critical parameters may be considerably narrowed. Thus it may be possible to arrive at a sufficiently complete and reliable analytical model.

Calibration in the Modal Space

Analytical model parameters were adjusted until the correlation between analytical and measured frequencies and mode shapes improved while the sequencing of the measured modes was preserved. The calibrated a priori model is termed Model 1 from here on. Its analytical parameters are presented in Table 1. The frequency correlation between analysis and experiment are presented in Table 2. Improving the correlation further, particularly without violating the modal sequence, proved difficult. This difficulty indicated that certain fundamental response mechanisms may not have been properly simulated in the analytical model.

Testing "Completeness" of the Analytical Model in the Flexibility Space

Uncertainties prevailed in spite of efforts to generate a complete representation of the bridge. Although it is not possible to identify a unique analytical model for the bridge, it is important to ensure "completeness." The model should correctly and completely incorporate the 3-D geometry, boundary, and continuity conditions and the existing conditions of all the bridge components so that the global and local flexibility distributions and the 3-D displacement kinematics are correctly simulated. This would ensure that all the critical response mechanisms and the load paths are accurately simulated.

One possible test of model completeness is conducted by correlating the analytical flexibility of the model (after calibrating in the modal space) with the experimental "modal flexibility." The 3-D deflection profiles (Figure 4) permit such a correlation because these profiles correspond with loading the measured modal flexibility and the analytical model of

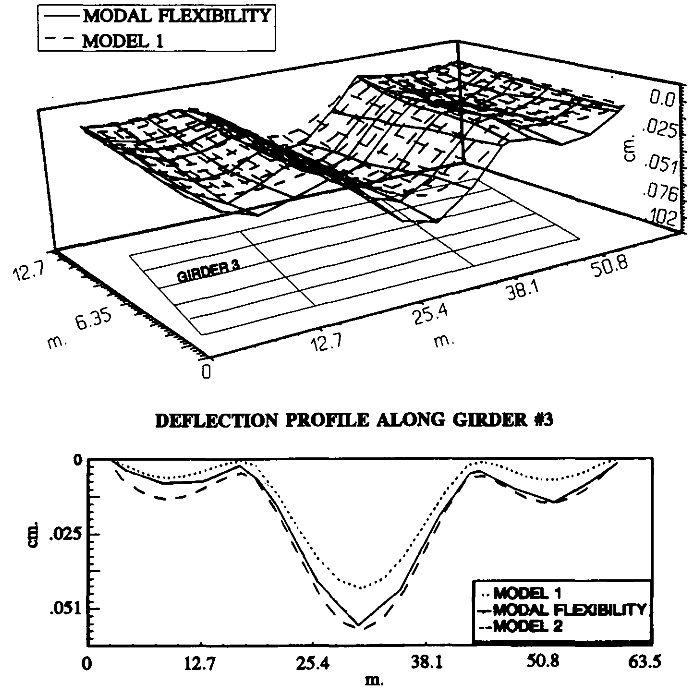
TABLE 2 Comparison of Experimental Frequencies with Analytical Counterparts

MODE NO.	A-PRIORI ANALYTICAL MODEL 1 (Hz)[a]	CALIBRATED ANALYTICAL MODEL 2 (Hz)[a]	EXPERIMENTALLY IDENTIFIED			
			IMPACT TEST		FORCED-VIBRATION	
			FREQ (Hz)	ζ (%)	FREQ (Hz)	ζ (%)
1	5.28	4.901	4.94	2.55	-	-
2	5.93	5.385	5.30	1.61	-	-
3	8.94[b]	7.609[b]	-	-	7.00	3.24
4	9.64	7.298	7.47	1.30	-	-
5	-	-	-	-	7.94	4.13
6	10.34	8.777	9.25	3.26	-	-
7	11.25	9.631	9.81	2.74	9.67[c]	2.45
8	11.30	9.831	10.29	3.65	-	-
9	11.99	10.374	10.58	3.10	10.84[c]	2.86
10	14.98	11.219	11.58	1.54	11.51[c]	3.40
11	15.47	11.429	12.01	1.53	-	-
12	16.03	12.597	13.34	1.05	-	-
13	15.89	14.211	14.56	2.16	-	-
14	16.18	14.268	14.90	2.21	-	-
15	18.24	15.341	15.71	1.47	-	-
16	20.50	15.612	16.49	1.36	16.42[c]	3.28
17	20.41	15.966	17.02	1.08	-	-
18	17.62[b]	16.64[b]	-	-	19.53	2.55
19	22.20	18.505	19.79	1.22	-	-
20	22.96	21.916	22.76	1.90	-	-
21	23.91	22.063	23.59	1.20	-	-

[a]Each frequency listed so that corresponding mode shape matches experimental mode shape for given mode number

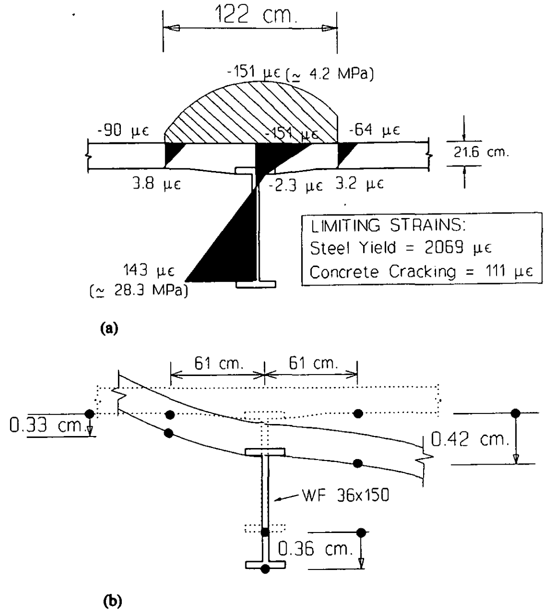
[b]Analytical transverse bending mode.

[c]Coupled modes.



1 m. = 3.28 ft.
1 cm. = 0.3937 in.

FIGURE 4 Correlation of bridge deflections under uniform load given by modal flexibility, Models 1 and 2.



1 cm. = 0.3937 in.
1 MPa = 6.895 ksi.

FIGURE 5 Strains and distortion of girder-slab under 4-222 kN (50 kip) trucks (Type 3 equivalent): (*a*) strain profile at midspan, Girder 3; (*b*) distortion of Girder 2 and deck at midspan.

the bridge by a uniformly distributed load. Normally deflections from the modal flexibility should be somewhat less because of modal truncation. However, the deflections given by the analytical flexibility are less than the deflections given by the modal flexibility (Figure 4), particularly with discrepancies at the piers and abutments. This indicated a lack of "completeness."

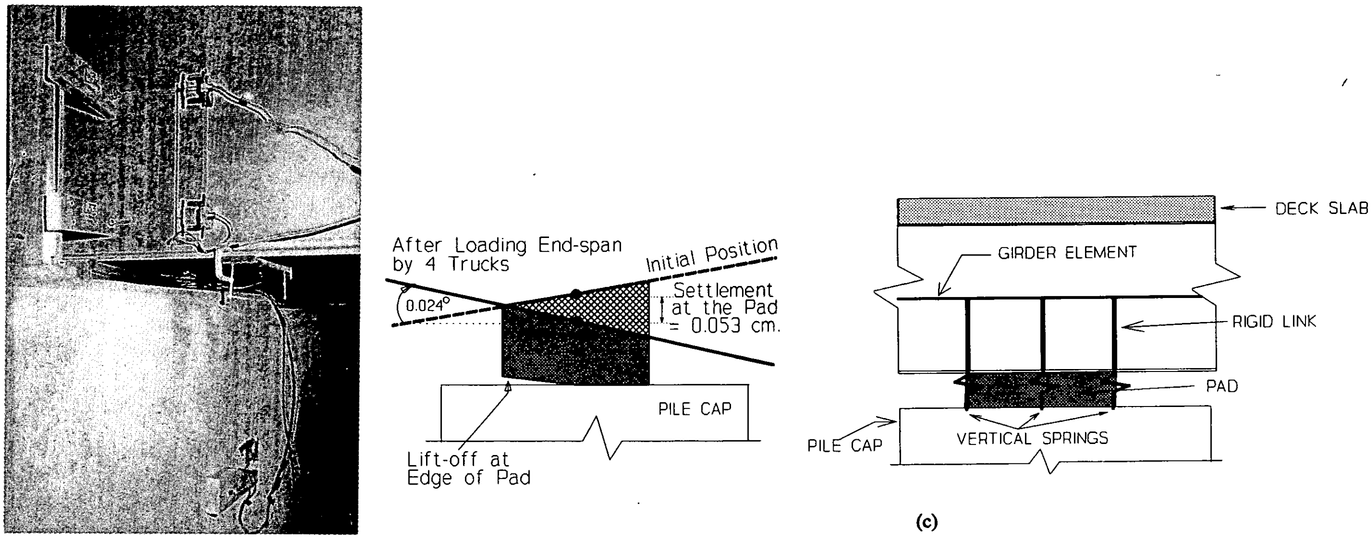
Additional Experiments for Completing the Model

Additional experiments were designed to better observe, conceptualize, instrument, and measure the critical response mechanisms of the bridge that are not adequately represented in the a priori model. The test bridge was loaded statically by positioning four trucks in various configurations. Each truck weighed approximately 222 kN (50 kips) and featured a tandem axle, making it equivalent to a Type 3 AASHTO vehicle. The bridge was extensively instrumented by 60 strain, distortion, and displacement transducers concentrated in one end span. Measured responses included closely spaced deflections along a girder and a lateral brace; the strain profile at a cross section at the midspan of a girder, including the RC deck; and displacements and rotations of a girder at the pier and at the abutment.

Figure 5 shows the strain profile at the middle of the end span when the bridge was loaded by all four trucks positioned back to back and side by side to maximize the positive moment demand of the girder at the instrumented cross section. The measured strains indicate a nearly fully composite action. The maximum girder flange strain indicates an incremental stress of only 27.58 MPa (4 ksi). Attenuation in the compressive strains in concrete at the top of the deck indicates an effective flange width of about six times the deck thickness. The displacement kinematics of the deck [Figure 5(*b*)] further confirms the composite action.

Figure 6 indicates the vertical displacement and rotation of a girder at the pier as the end span was loaded by four trucks. Before the truck loading, the girder exhibited a counterclockwise rotation at the pad resulting from dead loads. Because of this rotation, one edge of the pad was observed to lift up while the opposite edge was firmly compressed against the pier-cap. Truck loading induced a rotation in the opposite sense while also resulting in a vertical distortion at the pad. The observed displacement kinematics resulting from the finite size of the pad were properly simulated by modifying the analytical model as shown in Figure 6.

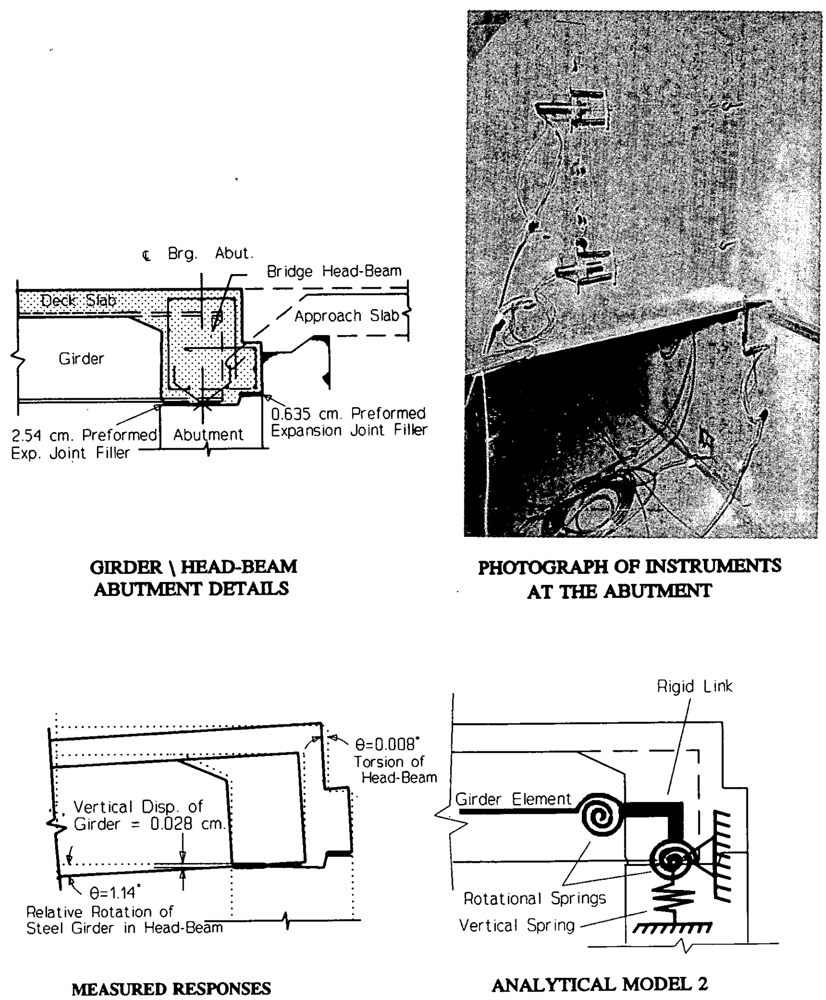
The construction details at the integral abutment are shown in Figure 7. This region was instrumented to measure the torsional rotation of the RC head-beam, vertical displacement of the steel girder relative to the abutment, and flexural rotation of the steel girder relative to the RC head-beam. The latter relative rotation occurred although the steel girder was encased within the RC head-beam. This type of relative rotation occurs as a result of the slippage of a steel element at an interface. The corresponding displacement kinematics were represented by modifying the analytical model to incorporate the girder-RC beam interface rotation, vertical displacement of the steel girder relative to the abutment, and the torsional twist of the RC head-beam relative to the abutment.



(a) (b) (c)

1 cm. = 0.3937 in.

FIGURE 6 Displacement kinematics at the pier under truck loading: (*a*) instruments at pier, (*b*) measured responses, and (*c*) analytical simulation (Model 2).



1 cm. = 0.3937 in.

FIGURE 7 Displacement kinematics at the integral abutment.

Calibrating and Verifying the Completed Model

Following the second round of experiments, the analytical model was modified for completeness, and some of the parameters were adjusted to simulate closely the measured local responses. The calibrated parameter values of the completed model are presented in Table 1 in the fourth column, labeled "Model 2." Correlation between the experimental and analytically simulated frequencies may be observed in Table 2 by comparing the third column with the fourth and sixth columns. The agreement between the measured frequencies versus those simulated by Model 2 is less than 2 percent for the lower modes, whereas errors of 5 percent are observed for the higher modes.

Correlation between the simulated and measured bridge characteristics is more definitively confirmed in the flexibility space in Figure 4, which indicates that the 3-D displacement profile under uniform vertical loading of the bridge given by Model 2 is close to the profile given by the modal flexibility. In fact, modal flexibility yields a slightly stiffer response as expected because of modal truncation.

Further verification of the fidelity of analytical Model 2 is shown in Figure 8, in which the simulated and measured vertical deflections of the bridge along a girder and a lateral brace at the end span under truck loading are compared. The responses labeled "truck" were measured directly when four loaded trucks were positioned on the bridge, as shown in Figure 8. The responses labeled "Model 1" and "Model 2" were simulated by analyzing the respective analytical models subjected to the same truck loads. The responses labeled "modal flexibility" are obtained by multiplying the experimental modal flexibility with the appropriate load vectors corresponding to the measured truck loads.

Whereas Model 1 is about 30 percent stiffer than is indicated by the measured deflections on the bridge, responses simulated by Model 2 show close correlation with their experimentally measured counterparts. This result illustrates the significance of correctly conceptualizing and simulating local

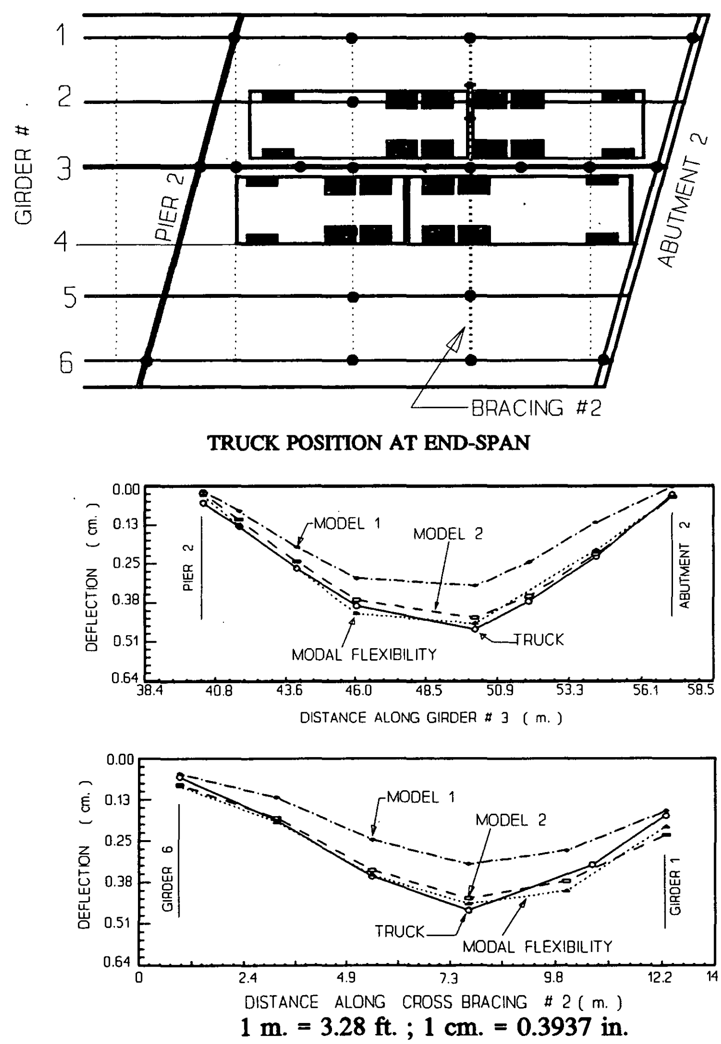


FIGURE 8 Correlation of truck-load deformations with simulations by analytical and modal flexibility.

response mechanisms for good analytical correlation. It is also significant that the deflection profiles given by the modal flexibility agree closely with the profiles measured directly under four loaded trucks clustered on one lane at an end span. The concentration of trucks in the experiment creates a considerably higher stress level under the loaded region than any legal two-lane loading configuration, even after allowing for the impact factors. The truck loading shown was permitted by highway officials during the experiments only because the critical bridge responses were being monitored in real time and the test could be stopped if the strains approached the limiting values shown in Figure 5. It follows, therefore, that this test may also be considered a proof-load test.

APPLICATIONS OF STRUCTURAL IDENTIFICATION

The agreement between the deflection profiles obtained from modal flexibility and those measured under truck loads indicates that modal flexibility, obtained by impacts applied at the dead load stress level of the bridge, may be considered a reliable condition index or signature capable of reflecting bridge conditions even at the upper levels of serviceability. This condition is attributed to the remarkable linearity of the new bridge even under proof-load stress levels. In the case of deteriorated bridges, such an accord between impact and proof-test results should not always be expected. For example, the writers observed considerable nonlinearity even at the service-load stages when a concrete slab bridge, which had extensive delamination at the shoulder regions, was loaded at these areas (*11*).

It is natural that if long-term deterioration is permitted or damage resulting from overloading occurs at the ultimate limit states, the structural condition of the new bridge (and therefore its flexibility) will change. For example, the chemical bond providing composite action between the deck and steel girders may deteriorate in aged or overloaded bridges, and the flexibility may increase significantly. In such cases, the analytical model identified cannot be relied on for estimating the strength capacity available. On the other hand, because the analytical model identified for the test specimen is shown to simulate closely bridge behavior even at the upper levels of the serviceability limit, it should serve reliably for rating the test bridge.

Once a completed analytical model is identified, more practical modal tests with only a sparse measurement grid, or truck load tests with only a few transducers measuring only some critical responses, may be carried out intermittently to update selected critical terms in bridge flexibility. For example, an impact modal test may be conceived for measuring only girder flexibility at midspan. Advances in sensor technologies should make it possible and practical to monitor continuously certain instantaneous and residual deflections of the bridge under truck loads. In this manner it may be possible to diagnose changes in structural condition. If future tests reveal a noticeable increase in flexibility relative to when the bridge was new, the identified 3-D FE model would have to be modified to incorporate mechanisms leading to increase in flexibility before it may be used for further predictions.

Rating the Test Bridge by Using the Identified 3-D FE Model

The test bridge was rated by the identified 3-D FE model. In the following discussion, some of the critical rating factors are compared against corresponding factors obtained by analysis of idealized models recommended by AASHTO (*12,13*). This phase of the study helped to evaluate the realism provided by different analytical modeling and analysis procedures for stringer bridges. Because the 3-D FE model captured the important global and local response characteristics of the bridge in its current state, rating the bridge with this model constituted an "academic" example of integrated field testing and bridge rating.

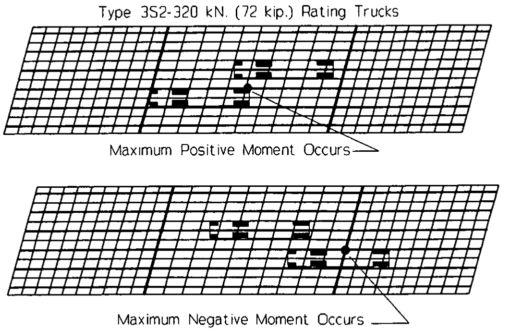
Issues To Consider in Rating the Bridge

The issues to consider in rating the bridge with the 3-D FE model were these:

1. *The limit-state that should govern rating*. Current practice is to consider yielding of the girder. However the truck-load test revealed that the bridge could maintain perfectly linear response at stress levels considerably exceeding any legal loading. Furthermore, evidence indicates that if the chemical bond in a noncomposite bridge is lost, significant serviceability and maintenance problems come soon after. Therefore it is rational to carry out strength evaluation of the bridge with the analytical model that reflects the linear service limit state. If maintaining the composite action resulting from chemical bond is a desired feature of the bridge, because of the evidence that the loss of composite action may render a bridge unserviceable and difficult to maintain, it should make even more sense to evaluate the bridge capacity for the limit state where this composite action is still available. Because the actual structural behavior is far more complex than the behavior of a simple steel beam, it does not make sense to evaluate the structural capacity based on beam steel yielding.
2. *Establishing critical elements, regions, and various capacities*. Slab and girders may be considered separate or compositely behaving elements on the basis of the measured strain distributions under truck load. Flexural and shear capacities of the girder would depend on cracking, separating, or yielding of the slab.
3. *Establishing the truck positions that would maximize demands at the critical regions*. In the case of a 3-D model incorporating the transverse load distributions between girders as facilitated by the slab and lateral braces, considerable analysis effort is required for locating trucks for rating.

Some of the resulting rating factors for the steel girder obtained from different recommendations and analysis approaches are shown in Figure 9. This figure shows the maximum positive and negative moments at the critical regions of the critical girder. Rating factors are also obtained based on the current Ohio Department of Transportation (DOT) practice, using the software BARS in conjunction with idealized 1-D beam models.

Significant differences are revealed in the rating factors based on the different approaches. Rating factors derived



Recommendations	Rating Truck	Rating Method	Rating Factors For	
			Positive Moment	Negative Moment
BARS (By: ODOT)	HS20	Allowable Stress Method	1.64	2.14
AASHTO 83 MANUAL (Operating Rating Factor)	T3S2	Allowable Stress Method	2.51	2.50
		Load Factor Method	2.71	2.77
AASHTO 89 GUIDE	T3S2	AASHTO Load Dist.	2.26	2.36
		3D FE Model 2	9.29	6.24

FIGURE 9 Rating factors for steel girder.

from the analyses of the 3-D FE model are several times larger than the corresponding factors obtained from planar idealizations of the bridge. The discrepancy caused by different approaches to computing demand is revealed when the two rating factors corresponding to the AASHTO *Guide* specifications are compared. Here, the same capacity computations and coefficients were used, and the only difference was in the demands computed from the AASHTO load distribution versus the prediction of the 3-D FE model.

In the case of positive moment the 3-D FE model incorporates composite action, reducing the girder demands. Therefore a rating factor of 9.29, which is more than four times the corresponding factor based on the AASHTO recommended load distribution, is obtained. In the case of negative moment the 3-D FE model yields a rating factor of 6.24, which is 2.6 times the corresponding factor obtained by the AASHTO load distribution. Such differences clearly demonstrate that in rating stringer bridges the uncertainty that may arise because of idealizations in analytical modeling may be of the same order of magnitude as the uncertainty that governs the loading envelopes.

It is also noted that if cracking of the slab is considered as the critical limit state, this may control the rating factor. What is important is that the idealized AASHTO load distribution does not permit correct evaluation of the demands of the concrete slab, whereas the 3-D FE model permits evaluation of all the possible critical components and regions of the bridge.

CONCLUSIONS

The concept of structural identification may hold the key for improving the state of the art in bridge field testing, condition evaluation, rating, health monitoring, and other applications. The authors have developed tools for demonstrating a comprehensive structural identification methodology and applied this to a newly commissioned continuous steel stringer bridge with integral abutments.

Modal flexibility is shown to serve as a conceptual, reliable, and comprehensive experimental signature, perhaps the best collection of numerical indices expressing bridge condition. In this application modal flexibility also served for testing the completeness of the 3-D FE model that is calibrated as a byproduct of the structural identification process.

The 3-D FE model of the bridge was completed by incorporating the observed local deformation kinematics at the critical regions and by calibrating the numerical model parameters so that the measured and simulated modal and flexibility characteristics of the bridge agreed to exacting goodness of fit. It is important that the test stress levels under truck loads considerably exceeded the stress levels that may be expected under legal loads even after allowing for impact and other safety factors. Because the 3-D FE model was shown to simulate the measured behavior under the test truck loading, it was considered a reliable tool for rating the bridge.

The rating factors obtained from analyses of the identified 3-D FE model exceeded those obtained by considering the AASHTO recommended load distributions by several times. This revealed that the uncertainties in rating resulting from failure to represent the load distribution mechanisms of steel stringer bridges correctly may exceed the uncertainties known to prevail in the load envelopes. Significantly, the 3-D FE model revealed the distribution of demands throughout the elements, including the slab and lateral braces, which are typically omitted in the rating process because of analytical limitations. These secondary elements may sometimes govern rating depending on the limit state considered. However unless a 3-D FE model is calibrated through structural identification, the potential for large errors cannot be overlooked. The confidence in the analysis results from a 3-D FE model that has not been verified and calibrated to exacting standards and cannot exceed the confidence in the estimates from the idealized analysis approaches currently used by experienced bridge engineers.

A global conclusion is that structural identification research of the type outlined here is essential for true appreciation of the limitations in current understanding of bridge behavior and in the manner of design, inspection, evaluation, and rating of bridges.

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