Issues in Rating Steel-Stringer Bridges

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Nondestructive dynamic field testing and structural identification studies on three steel-stringer bridges (2, 20, and 43 years old) are presented. The bridges were rated by code procedures and by field-calibrated comprehensive three dimensional finite element models developed by structural identification. Experimentally measured and analytically simulated modal flexibilities were correlated with bridge deflections obtained under proof-load level truck load tests. Test results indicated that all three bridges, although constructed as noncomposite, exhibited composite action between the slab and girders. Although the composite action was nearly perfect in the 2-year-old bridge, the older bridges exhibited partially composite behavior caused by deterioration of the chemical bond and friction. The rating factors obtained by field-calibrated models exceeded the corresponding operating rating factors by about 2.5 to 4 times for the three test bridges. The rating process and the resulting factors helped to identify and conceptualize a number of unresolved important issues that influence bridge rating and management. Serviceability aspects that emerged as critical were studied through the relative contributions of various mechanisms to bridge deflections.

The AASHTO manual (*1,2*) guides most state Department of Transportation (DOT) operations related to the routine biannual inspection and rating of bridges. Consequently, condition indexes assigned by visual inspection influence critical bridge management decision about repairs, posting, rehabilitation, and replacement. In 1994, FHWA officials estimated a financing need of $90 billion to repair the bridges deemed deficient on the basis of visual inspection results.

Recent related NCHRP research projects (*3–8*) have led to the AASHTO guide (*9*) and the draft Load and Resistance Factor Design (LRFD) specifications (*10*). Others, such as Lichtenstein (*8*) and Galambos et al. (*11*), have recommended significant revisions and modifications to the AASHTO manual (*1,2*) and AASHTO guide (*9*) with respect to condition assessment and rating. A new Manual for Condition Evaluation of Bridges has been issued by AASHTO (*12*).

The AASHTO guide (*9*) encourages the use of facility-specific information in rating a bridge as well as experimentally calibrated analytical models. However, organizational and technical consensus methods need to be established for (*a*) generating bridge-specific objective information to adequately document a bridge's structural and loading conditions; and (*b*) modeling and simulation to reflect the actual loading environment, existing structural conditions, and all the critical structural response mechanisms of a bridge.

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CONDITION ASSESSMENT AND RATING OF STEEL-STRINGER BRIDGES

Appreciation of bridge behavior has evolved through allowable stress design (ASD), load factor design (LFD), and alternate load factor design (ALFD) (*11*). The corresponding rating provisions for ASD, LFD, and ALFD are respectively based on an evaluation of the ratio of the maximum stresses to their allowable counterparts, the cross-sectional force demands to the corresponding cross-sectional strength capacities, and the structural load demands to the capacity of a plastic mechanism. The recently issued draft LRFD specifications (*10*) require an explicit check of capacity and performance at all the critical limit states, including serviceability, fatigue, stability, deterioration, and collapse.

Hence, according to the spirit of the draft LRFD code provisions (*10*) and the AASHTO guide (*9*), it is desirable to use field-calibrated models and to evaluate performance at all of the critical limit states, including serviceability. The issue remains in the development of field-testing methodology and field-calibrated models that will allow rating a bridge and checking its serviceability. The main obstruction to objective condition assessment and rating has been lack of complete understanding of how bridges actually respond to many different external and intrinsic loading effects, actual contributions of different load-resisting mechanisms, their relative variation at different loading stages, and effects of aging and deterioration on both the load demands and capacities. The lack of a clear understanding of a bridge response also makes it difficult to formulate realistic and precise definitions for the structural limit states and the performances expected at these limit states.

OBJECTIVES AND SCOPE

Integrated analytical-experimental structural identification research has been conducted on a number of highway bridges in Ohio (*13–19*), on the basis of dynamic modal and truck load tests, development of field-calibrated finite-element models, and rating of bridges by a variety of linear and nonlinear analyses and limit-state definitions. Observations and findings from such research raise the need to review some concepts and applications proposed by the recent NCHRP reports (*8,11*).

The first objective of this paper is to discuss the characteristic problems in condition assessment of steel-stringer highway bridges with their specific resistance mechanisms. The second objective is to review the critical issues in rating steel-stringer bridges. The discussion focuses on steel-stringer bridges because this type composes the largest segment of the bridge population in the nation and more than one-third of nearly 40,000 bridges in Ohio.

Three steel-stringer bridge test specimens were evaluated and rated by field-calibrated models. The nondestructive tests conducted for structural identification included modal tests by impact as well as vertical and lateral forced excitation, followed by truck load tests for measuring global and local bridge response under different static loading patterns. 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.

STRUCTURAL IDENTIFICATION METHODOLOGY FOR INTEGRATING CONDITION ASSESSMENT AND RATING

The comprehensive bridge research project has been organized to solve the following issues before new inspection and rating tools are developed: (*a*) identification of the most important mechanisms that affect bridge behavior at different limit states and their proper incorporation into design, inspection, evaluation/rating, and maintenance management; (*b*) investigation of age and deterioration effects on these mechanisms; (*c*) verification of the possibilities to measure short-term and long-term bridge behavior in the field accurately and completely, and to develop experimental condition-assessment techniques that would help to reliably establish the global state of health of a bridge; (*d*) integration of rating with such an experimental condition-assessment procedure; and (*e*) development of analytical techniques that would reliably project the existing capacities of a bridge and its remaining service life from the results of experimental condition assessment.

The important issue is to design a comprehensive methodology that would integrate an objective condition assessment with bridge rating and life-cycle maintenance management. This research has led to an attempt to develop a bridge structural-identification methodology that integrates analytical modeling, experiment, damage diagnostics, and rating into a rational framework. A schematic of the methodology is given in Figure 1, entailing the following steps:

1. For a particular bridge, researchers first compile all of the information that exists on the original design, fabrication/shop and as-built drawings, and construction details, followed by maintenance records. The current conditions of the structure are documented and a preliminary finite element model is constructed.
2. The researchers use the a priori analytical model to design a modal test and conduct a pilot modal test labeled to calibrate the experiment. The pilot test typically does not require traffic control.
3. A rigorous modal test follows. This test is generally conducted by impact; however, in the case of flexible bridges, such as long span through truss or suspension bridges, forced excitation may be needed. Moreover, in case the lateral response characteristics of the

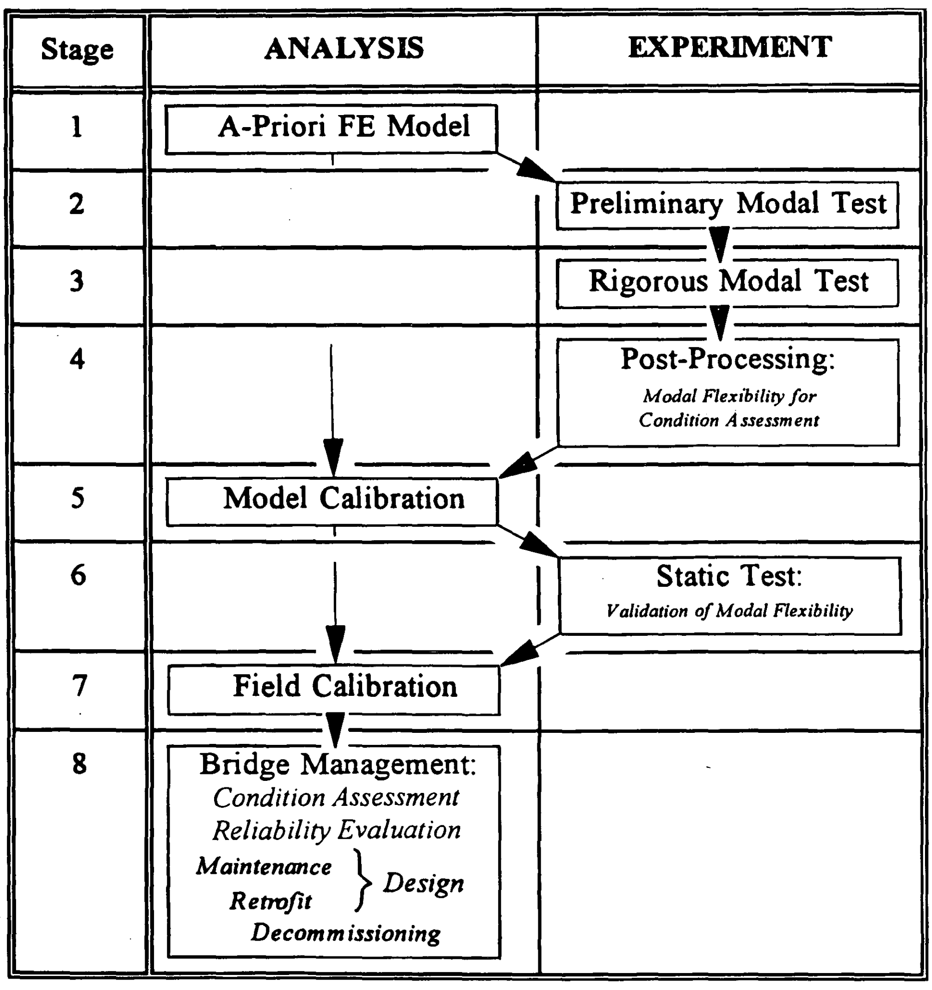


FIGURE 1 Structural identification method for bridge management applications.

bridge are included in the evaluation, forced excitation may have advantages over impact. The rigorous test requires traffic control; however, if impact is used the bridge may be tested in parts so that one or more lanes may be continuously kept open to traffic.

4. Post-processing the modal test data provides a wealth of information about the mechanical characteristics of the bridge: frequencies, damping coefficients, mode shapes, and most importantly, bridge flexibility. The flexibility from the modal test serves as a reliable objective bridge signature that is sensitive to damage and is typically more than 90 percent reliable if testing and post-processing are properly planned and carried out by experienced engineers.

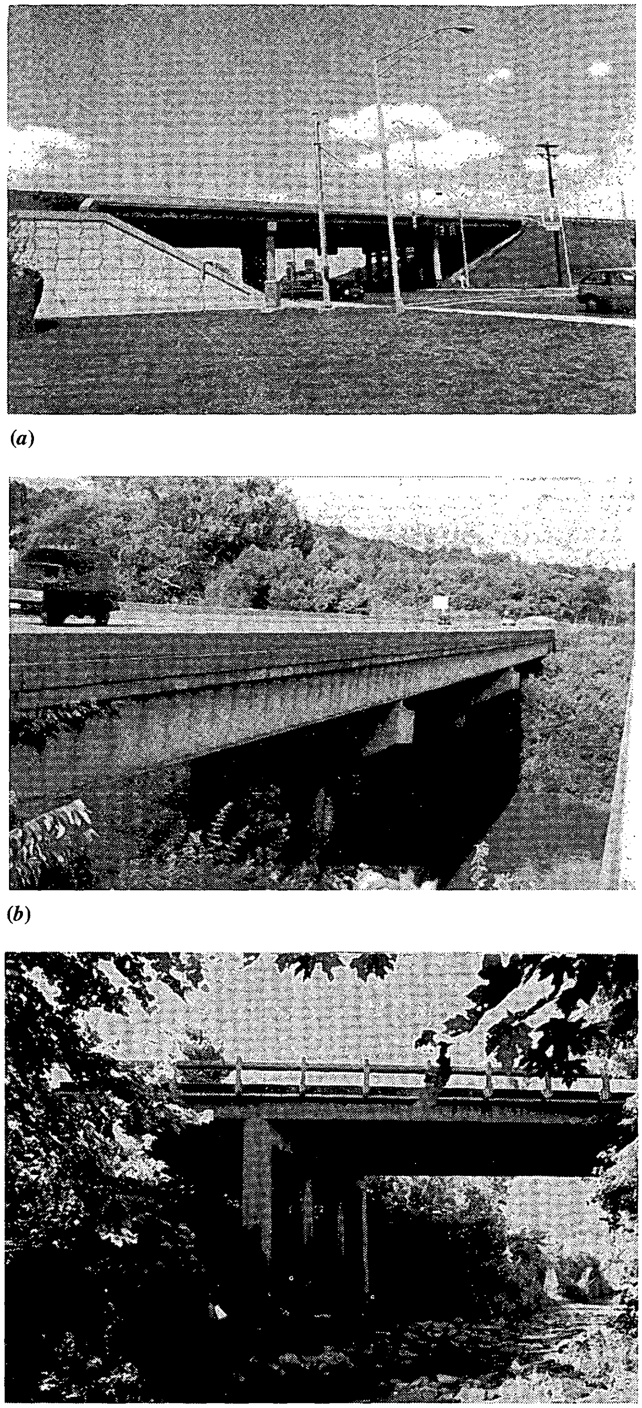
5. and 6. After a complete evaluation of the bridge conditions by studying the flexibility, if needed, an improved understanding of local bridge behavior is gained by additional instrumentation and measurement under truck loads. The researchers calibrate the finite element model such that the existing state and all the critical global and local behavior mechanisms are captured and simulated accurately.

7. The field-calibrated model then serves for reliable rating, projections of capacities, and design of effective maintenance, rehabilitation, or retrofit. The researchers have tested several bridges to damage and failure to demonstrate the reliability of the projected behavior and rating coefficients obtained by the methodology (*15–17,19*).

This methodology is intended as a research tool and is not for routine implementation on every bridge. Once the researchers test a sufficient number of samples from a recurring bridge type, a considerable amount of generic information is gained. This helps to design more practical experiments on other samples for routine applications. The generic bridge- and type-specific knowledge obtained from the applications of the methodology to a selected number of bridges provides an invaluable understanding of behavior fundamentals.

STEEL-STRINGER BRIDGE TEST SPECIMENS

The three steel-stringer bridge specimens, shown in Figure 2, were the HAM-42-0992, CLE-50J-0080L, and HAM-128-1006, which were, respectively, 2, 20, and 43 years old at the time of testing. Table 1 describes the main features of the three specimens, which represent a wide variety of continuous steel-stringer bridge design parameters related to girder spacing, spans, bearings, and abutments. All of the bridges were designed as noncomposite, that is, mechanical connectors were not provided between the steel girders and the concrete deck. None of the bridges had been rehabilitated for maintenance after construction or maintained with deck overlay.



(*a*)

(*b*)

(*c*)

FIGURE 2 Steel-stringer test bridges: (*a*) HAM-42-0992; (*b*) CLE-50-J-0080L; (*c*) HAM-128-1006.

STRUCTURAL IDENTIFICATION AND CONDITION ASSESSMENT OF TEST BRIDGES

The bridge structural identification methodology described earlier was applied to each of the three specimens, resulting in a field-calibrated three-dimensional (3-D) finite element model specific to each bridge. HAM-42-0992 was subjected to truck load testing after installing 64 channels of local strain and displacement instrumentation to verify the modal test results and to better understand the complex bearing pad and integral abutment response mechanisms. The structural identification results for HAM-42-0992 were reported earlier (*18*). An important finding from the structural identification of HAM-42-0992 and CLE-50J-0080L was related to composite behavior. Bridge displacements found by applying a uniform load to the measured flexibility were correlated with the resulting deflections simulated by the calibrated finite element model. Because the deflections from the finite element models, simulating rigid connections between the deck and the girders, closely correlated with those obtained from the experimentally measured flexi-

TABLE 1 Important Attributes of Tested Steel-Stringer Bridges

Bridge Features	HAM-42-0992 Cross County Highway	CLE-50J-0080L Little Miami River	HAM-128-1006 Paddy's Run
Construction year	1986	1970	1950
Design code	1983 AASHTO	1969 AASHTO 10th Ed.	N/A
Design loads	HS20-44	HS20-44	S-15-46
Number of Spans	3 (continuous)	3 (continuous)	3 (continuous)
Span lengths ft (m)	55, 78, 55 (16.8, 23.8, 16.8)	100.5, 125.5, 100.5 (30.6, 38.3, 30.6)	40, 50, 40 (12.2, 15.2, 12.2)
Roadway width ft (m)	42 (12.8)	50 - 55 (15.2 - 16.8)	33' 4" (10.2)
Skew	15° 11' 16"	32° 30' 00"	25°
Type of steel	ASTM A-36	ASTM A-36	N/A
No. of steel girders	6	7	5
Girder spacing (m)	7' 9" (2.36)	9' 6" - 10' 11.5" (2.90 - 3.34)	7' 4" (2.24)
Girder depth in. (m)	36 (0.91) W-flange beam	62 (1.57) plate girder	30 (0.76) W-flange beam
Capacity design	Noncomposite	Noncomposite	Noncomposite
Deck thickness in. (m) included surface layer	8.5 (0.216) 1.25 (0.032) latex modified	9 (0.229) 1 (0.025) monolithic	7.25 (0.184) 3/4 (0.019) monolithic
Design f_c' psi (MPa)	4500 (31)	4000 (28)	4000 (28)
Transverse reinforcement (%)	top = 0.71 bottom = 0.50	top & bottom = 0.74 to 0.83	top = 0.53 bottom = 0.71
Longitudinal reinforcement (%)	top = 0.24 bottom = 0.52	top = 0.28 bottom = 0.61	top = 0.24 bottom = 0.38
Pier support	elastomeric pads	rocker & bolster	sliding plate
Abutment support	full integral	rocker & bolster	sliding plate

bilities of HAM-42-0992 and CLE-50J-0080L, it was inferred that these two bridges were exhibiting perfect composite behavior. This inference was verified by measuring the strain profiles along the girders and deck-girder interfaces of HAM-42-0992 under truck loads that confirmed the composite behavior (*18*).

The finite element model for HAM-128-1006 and the simulated deflections along one of the girders under uniform loading of the measured and analytical model flexibilities are shown in Figure 3. The field-calibrated model is partially composite. It is observed that the measured and analytical flexibilities of this bridge correlate when a partial continuity between the deck and the girders is simulated in the analytical model, that is, a partial composite behavior. Moreover, at the northeast (right) end span, the measured flexibility is larger than the simulated one, whereas in the other spans the correlation is better. This reveals that the northeast end span has retained a lesser chemical bond and friction between the girder and slab relative to the other two spans. Figure 3 illustrates that one could discern regions that may have deteriorated more than others. It is important to note that the measured flexibility is about 10 percent more than in the case of simulated perfect composite action and 10 percent less than when no composite action is simulated. This characteristic is shown to have a significant effect on the serviceability and rating of the bridge.

NCHRP RECOMMENDATIONS FOR NONDESTRUCTIVE TESTING AND RATING

Lichtenstein, in the draft final report of an NCHRP paper (*8*), proposed a general scheme for diagnostic and proof testing of bridges together with expressions that would be used for arriving at a test-based rating factor. The recommendations in the report are applicable to truss bridges as well as others. Lichtenstein (*8*) defines diag-

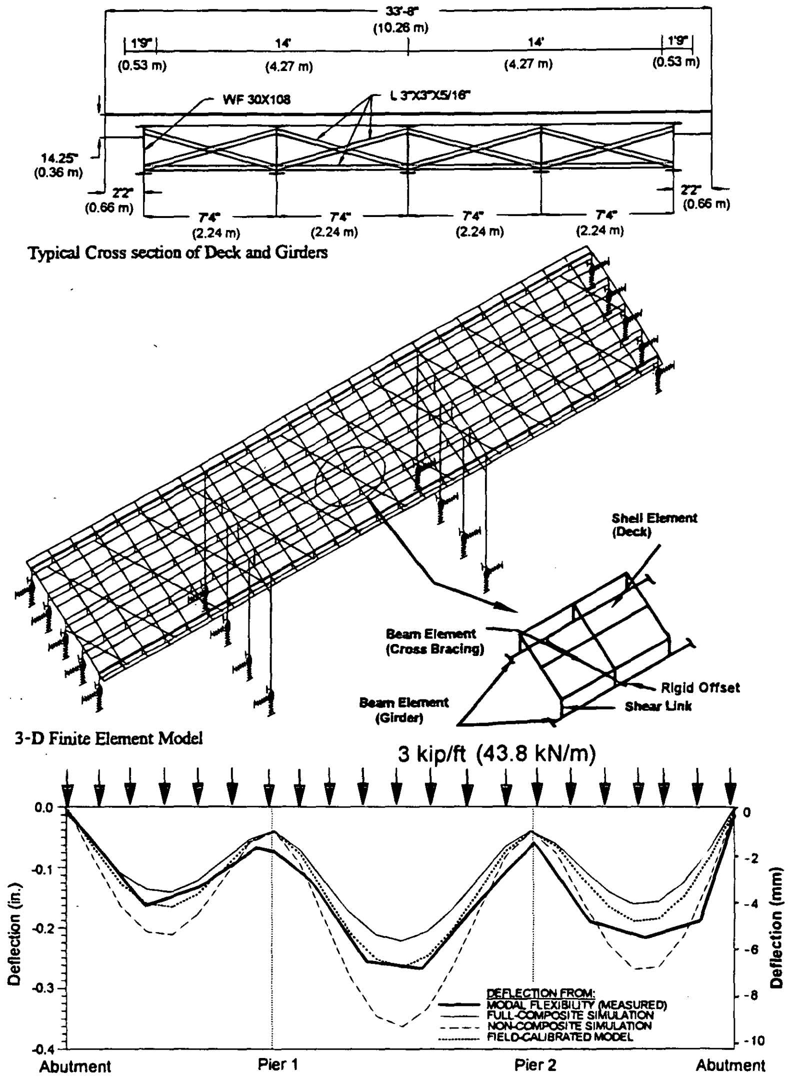


FIGURE 3 Finite element model and simulated deflections for HAM-128-1006.

nostic tests as those conducted to verify predicted or nominal load effects. These tests are recommended for validating a model to rate a structure under design, legal, permit, or rating loads. Assumptions about material properties, boundary conditions, cross-section contributions, effectiveness of repair, impact of damage, and deterioration, among others, may be validated by diagnostic tests. Diagnostic testing is acknowledged to be much more elaborate than proof testing because both an analytical model and more stringent field measurements are required.

The load placement and load levels in a diagnostic test may be less than those at service load levels. However, the test should be conducted to rule out any possible nonlinearities at the service load level. After the test, the theoretical rating factor, which would be obtained prior to the diagnostic test, is multiplied by a factor K, which is larger than or equal to unity. This factor is based on the comparison of measured test behavior with the analytical model adjusted for site-specific considerations.

Lichtenstein's formulation (*8*) for making the best use of a diagnostic test for bridge rating is a significant contribution to the state of the practice. However, a large number of example applications and verifications are needed before the method is fully calibrated and the details of practical and meaningful applications can be streamlined. For example, suppose rating is based on an extremely idealized analytical model that ignores a number of relevant mechanisms that affect load distribution and the resistance of the critical element. Can this type of analytical deficiency be rectified by a diagnostic load test?

The following are some additional questions that come to mind: What if the critical element of the bridge is not properly identified or if the instrumentation is not properly designed and installed to capture the critical stress in the critical element? How should the load be placed to activate the most critical actions in the most critical element, particularly if the bridge has damage or deterioration or both? How would the bias and variance errors in the experimental results be evaluated? Obviously, the minimum qualifications and experience of a test team that can be entrusted with a diagnostic test should be clearly established and certified before a theoretical rating factor may be legally modified.

The experience of the authors based on the research reported here is that the strains measured during a diagnostic test have to be reliable within 20 $\mu\epsilon$ (the authors' truck load tests on reinforced concrete slab, steel-girder, and truss bridges have revealed that the critical strains in most bridges under legal trucks will correspond to 20 to 100 $\mu\epsilon$). Typically, a temperature change of several degrees may lead to comparable strains. Further specifications are needed for bridge instrumentation and diagnostic testing to benefit from NCHRP recommendations (*8*).

RATING OF TEST BRIDGES

The three test bridges were rated by the procedures followed by the Ohio Department of Transportation (ODOT) bridge bureau and based on the AASHTO Manual (*1,2*). The bridges were also rated by using the field-calibrated analytical models that were developed by structural identification. The field-calibrated finite element models simulated the 3-D geometry and incorporated all of the structural and nonstructural elements, as shown in Figure 3: reinforced concrete deck, cover plates, cross braces, parapets, abutments, piers, support and continuity including the composite action, flexibility at the interface with the integral abutment, flexibility at the pads, soil-pile interaction flexibility, and flexibility characteristics of each individual element of the bridges.

As the test bridges were rated by bridge-specific models, a number of important issues had to be resolved. Table 2 provides an overview of the issues in demand and capacity computation that are needed for rating. In computing demands, the main issues are analytical modeling, selection of linear or nonlinear analysis options and the corresponding software, simulation of live load, and interpretation of the results from analysis. In the case of capacity computation, the issues have to do with the approach, that is, whether ASD, LFD, or ALFD are to be adopted; incorporation of actual material properties and existing state of damage or deterioration or both; selection of linear or nonlinear analysis to compute capacity of materials, sections, or the complete structure; and interpretation of the results.

Clearly, there are many possible options and decisions to make in the rating process, and the analytical aspects of the process are complex in the case of bridge-specific rating on the basis of field-calibrated models. The field calibrated rating factors given in Table 3 were obtained by using linear analysis of the field-calibrated 3-D finite element models in conjunction with the load factor approach recommended by the AASHTO guide (*9*). Critical trucks and their positions were established on the basis of 3-D influence lines generated by using the 3-D finite element models. In computing capacity, nominal material properties were used. For calculating the flexural capacity of the composite or semicomposite girder-slab sections, a 3-D nonlinear section analysis software 3-DRCSA (*20*) was used.

IMPLICATIONS OF RATING FACTORS

The rating factors obtained for all the three test bridges (Table 3) reveal that those that are based on field-calibrated models exceed the corresponding rating factors obtained by ODOT procedures by

TABLE 2 Issues in Rating Steel-Stringer Bridges

Definition of Limit State and Analysis Approach (ASD,[a] LFD,[b] ALFD,[c] or LRFD[d])	
DEMAND COMPUTATION	**CAPACITY COMPUTATION**
(1) Analytical Modeling (a) Dimensional idealization (1-D, 2-D, or 3-D) (b) Member discretization (c) Boundary conditions (d) Continuity conditions (e) Analytical elements (f) Actual material properties (g) Existing damage and deterioration	**(1) Analytical Approach** (a) Local stress (b) Section capacity (c) Structural capacity (d) Actual vs. nominal material properties (e) Existing damage and deterioration
(2) Selection of Analysis Package (a) Linear or (b) Nonlinear	**(2) Selection of Analysis Package** (a) Nonlinear Cross-Sectional Analysis vs. (b) Finite-element analysis for different actions and failure modes
(3) Simulation of Live Load (a) Critical trucks (b) Truck configurations on the bridge (c) Truck loading and impact (d) Fatigue effect	**(3) Interpretation of Analytical Results** (a) Limit state considered for defining capacity (b) Localized stresses (c) Effective cross-section resultants (d) Structural strength capacity (e) Serviceability and fatigue considerations
(4) Interpretation of Analytical Results (a) Nodal forces/stresses (b) Localized stresses (c) Effective cross-section resultants (d) Structural demands and limit states considered	

Notes:
[a] Allowable Stress Design;
[b] Load Factor Design;
[c] Alternate Load Factor Design;
[d] Load and Resistance Factor Design.

TABLE 3 Rating Results for Test Bridges

Bridge	HAM-42-0992 Cross County Highway			CLE-50J-0080L Little Miami River			HAM-128-1006 Paddy's Run		
Procedure	M^+	M^-	M^-/M^+	M^+	M^-	M^-/M^+	M^+	M^-	M^-/M^+
AASHTO *Manual* (1983) BARS 1-D Model	1.64	2.14	1.30	1.27	1.62	1.28	1.57	1.63	1.04
AASHTO *Guide* (1989) Identified 3-D FEM	6.00	5.39	0.90	5.14	5.24	1.02	4.93	5.19	1.05
Ratio: *Guide/Manual*	**3.66**	**2.52**	0.69	**4.05**	**3.23**	0.80	**3.14**	**3.18**	1.01

Notes:

(1) All procedures consider Strength Limit State;

(2) Impact Factors specified according to AASHTO *Manual* are used in computing all of the rating factors.

about 2.5 to 4 times. It is important to note that if an inelastic rating approach such as one proposed by Galambos et al. (*11*) was adopted, the level of conservatism in the rating factors obtained from typical DOT practice would be even larger. From another perspective, whether inelastic rating would lead to more accurate rating factors, if it is not based on a field-calibrated model that incorporates all the critical elements of the bridge, is questionable.

The extreme conservatism in the rating factors based on typical DOT practices would have an important implication in permit load requests, which may be presently denied. Table 3 further reveals a lack of balance in positive and negative moment rating factors from the 1-D models other than for HAM-128-1006, whose negative and positive moment factors are closest to each other, as indicated by their ratios. In the rating factors based on field-calibrated models, all the structural and nonstructural elements were incorporated, including the cover plates provided for the splices over the bearing plates for the negative moment. The actual balance ratios of negative-to-positive moment capacities from the field-calibrated models are much closer than those from 1-D models.

Other observations from this bridge-specific rating research are (*a*) damage or deterioration, or both, as a result of aging affect demand and capacity and, more importantly, the failure mode; (*b*) the reinforced concrete deck affects both demand and capacity by its two-way flexural and shear capacities and, more importantly, by the compressive membrane forces that develop as a result of composite or partial composite action; (*c*) cross braces significantly affect demand, particularly in the negative moment regions; (*d*) composite action caused by chemical bond or friction, or both, between slab and girders, even without mechanical connectors, affects demand and capacity; (*e*) abutment fixity affects demand; and (*f*) deck parapets, beam cover plates, and size and flexibility of bearing elements are mechanisms that affect demand and capacity.

Present rating methods use analytical models that typically omit the reinforced concrete slab and the parapets as well as the lateral load distribution provided by the brace system. The composite action is also neglected if mechanical connectors are not provided. Bridge engineers generally justify omitting the composite action in noncomposite designs on the basis of the argument that the chemical bond and friction would be lost over time. On the other hand, this research has shown that a considerable level of composite action has been maintained even 40 years postconstruction. In fact, there is evidence that in the case of complete loss of composite action, stringer bridges lose a considerable amount of stiffness and the decks become unserviceable. More importantly, by omitting the slab, the interface of the slab with the girders, and the cross braces in the rating models, the DOTs are not evaluating the actual performance of these components, which are emerging as important as the girders for serviceability in conjunction with lifetime-cost management considerations. Clearly, the uncertainties in the rating factors and a lack of addressing serviceability in rating is affecting the reliability in bridge management.

Serviceability Versus Strength

Table 4 compares the critical deflections of the three test bridges (calculated by their field-calibrated finite element models, which represent the actual behavior of the bridges) normalized with the code-permitted deflection, showing that the actual deflections are far smaller than code-permitted values. The actual deflections under both lanes loaded by a T-3 truck at midspan were observed to be less than 50 percent of the deflection expected in design.

One may also observe an order of magnitude difference in the actual deflections of the three test bridges, although their rating factors, that is, expected strength capacities, are comparable (4.93 to 6.00).

The field-calibrated finite element models were used to further evaluate the contributions of two-way action of reinforced concrete deck, composite action, cross braces, and girders to the midspan deflection flexibilities of the test bridges. This was accomplished by simulating each one of these elements or mechanisms not to contribute to the stiffness of the bridge; that is, the stiffness provided by each mechanism was set to 0 in the field-calibrated finite element model, as indicated in Figure 4.

TABLE 4 Comparison of Deflections and Mechanisms Contributing to Stiffness of Stringer Bridges

Bridge	HAM-42-0992 Cross County Highway		CLE-50J-0080L Little Miami River		HAM-128-1006 Paddy's Run	
	Middle span L=78 ft (23.77 m)		Middle span L=126 ft (38.41 m)		Middle span L=50 ft (15.24 m)	
3-D FE Model	Deflection D in. (mm)	D / (L/800) (L/800=1.17" ≈29.7mm)	Deflection D in. (mm)	D / (L/800) (L/800=1.17" ≈29.7mm)	Deflection D in. (mm)	D / (L/800) (L/800=0.75" ≈19.1mm)
Calibrated	0.67 (17.0)	57%	0.88 (22.4)	46%	0.20 (5.1)	27%
Full Composite	0.67 (17.0)	57%	0.88 (22.4)	46%	0.13 (3.4)	18%
Noncomposite	1.19 (30.2)	102%	1.50 (38.1)	80%	0.27 (6.8)	36%
w/o RC deck	1.71 (43.4)	146%	1.67 (42.4)	88%	0.4 (10.1)	53%
w/o RC deck & X-braces	2.00 (50.8)	171%	1.69 (42.9)	90%	0.58 (14.6)	77%

Note: Deflections are for both lanes loaded by truck T-3 at midspan.

The calculated deflections (Table 4) of the different models reflect the importance of composite action and other bridge mechanisms under service loads. Figure 5 quantifies the significance and the relative contribution of different elements or resistance mechanisms participating in the global stiffness of the three test bridges. For each bridge, the contribution of each mechanism can be different, depending on span and other attributes. For example, in the case of CLE-50J-0080L, the girders are dominating the complete superstructure stiffness. For HAM-42-0992, which is on pads, the cross braces seem to have a much more significant contribution than in the case of CLE-50J-0080L. For HAM-128-1006, the effect of composite action dominates the stiffness, in spite of its 43-year-long service life at the time of testing, showing the contribution that saved the bridge from deck replacement and emphasizing the significance of this mechanism.

The implication of the observations from Table 4 and Figure 5 is significant because it enables one to infer, objectively, the actual contribution of each critical component to the total stiffness of the superstructure. Currently, the contribution of components is not controlled in an objective quantified manner because the deck and cross braces are empirically designed. Present codes and design traditions in bridge engineering have successfully ensured safety against inadequate strength. However, an evaluation of Table 4 indicates that relative flexibility and serviceability of typical highway bridges may not have been properly controlled in bridge design and evaluation practice.

It is now emerging that bridge evaluation should include an objective quantitative assessment of serviceability in addition to strength and safety. The related issues that should be taken into consideration are long-term performance, durability, toughness, or resistance to mechanisms that cause deterioration or aging, and damage caused by environmental attack. New codes, such as the Draft LRFD (*10*), have recognized that existing design processes may not be successful in providing long-term serviceability. This research showed the significance of quantifying serviceability in terms of critical deflection mechanisms in bridges possessing large reserves of strength. The research further demonstrated a structural-identification-developed integrated condition assessment and performance evaluation method to objectively quantify both the safety and serviceability.

CONCLUSIONS

It appears rational to emphasize, in the design, evaluation, and retrofit procedures, composite action over the longer term. In addition, evaluating a bridge by properly recognizing the contributions of the slab system as well as the cross braces would be important for understanding the state of force in these bridge components and their contribution to stiffness. This may help develop a uniform distribution of safety for all the critical components that transfer load. For example, although the evaluation of the steel girders of a stringer bridge may be conservative, the evaluation of the concrete deck or the braces may be unconservative. Moreover, ignoring some of the important mechanisms that govern the actual load distribution and structural stiffness does not permit controlling all of the attributes that lead to desirable safety and serviceability performance during a retrofit design.

All three steel-stringer test bridges had comparable strength rating factors, whereas their live-load deflection and vibration characteristics varied greatly. Current rating methods completely ignore serviceability performance as affected by deflection and vibration. The L/800 deflection limit does not reflect the serviceability performance of bridges.

In general, design, construction, inspection, and evaluation of steel-stringer bridges, because of their complexity, are particularly difficult to implement objectively, and these bridges make up the largest segment of the National and Ohio Bridge Inventories. The

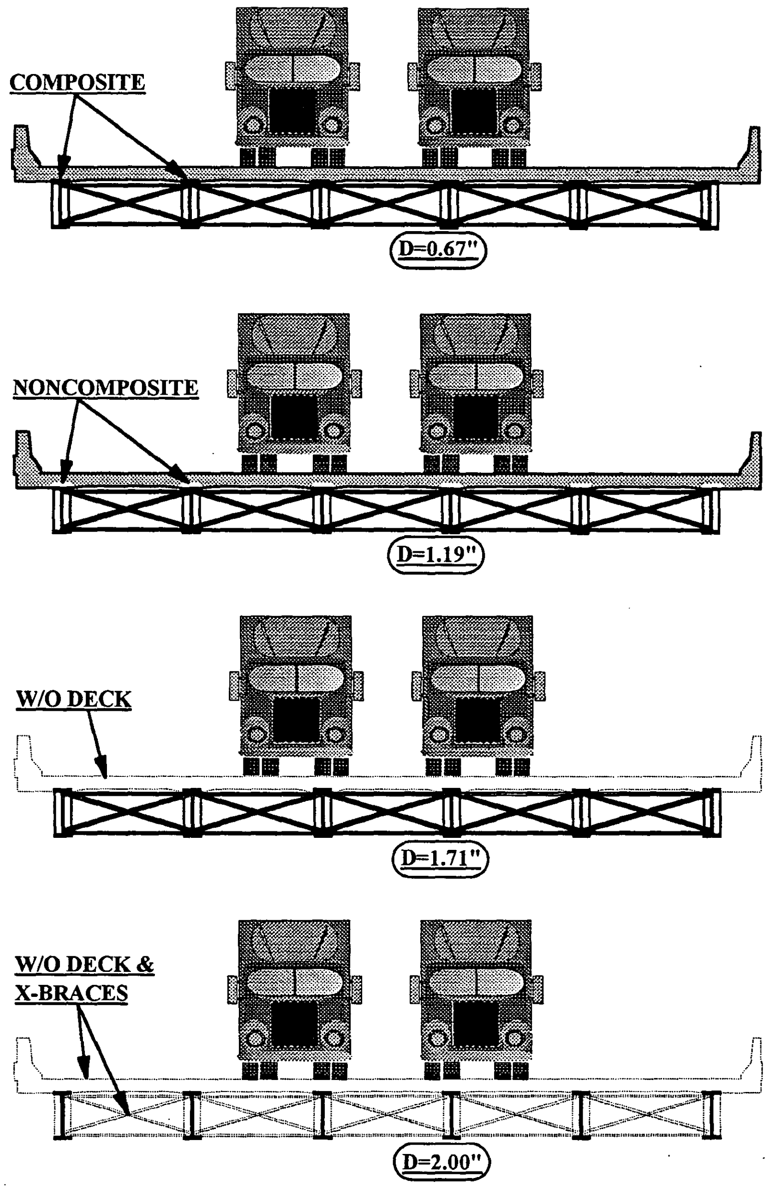


FIGURE 4 Comparison of deflections of HAM-42-0992.

replacement value of the 15,830 steel-stringer bridges in Ohio's inventory would exceed $2.5 billion, and more knowledge of their actual behavior is needed.

It is helpful to determine contribution by slab, girder, and cross braces to understand flexibility and safety and determine ways to improve rating.

RECOMMENDATIONS

As a result of the research reported here, the following recommendations are made:

1. For starting the design, selection of the dimensions and spacing of the steel stringers, assuming a stiffness based on noncomposite action at the interface with the concrete deck, provides excellent serviceability and redundancy in design. However, it is more desirable to use a complete 3-D model in design, incorporating the girders, cross braces, and the reinforced concrete deck, to arrive at a more reasonable estimate of resistance/capacity. It is also important to check for deflection and vibration problems with more rational procedures than just checking individual girders. Research demonstrated that L/800 is not an adequate measure of serviceability. Unfortunately, there are many reports of excessive deflection and vibration in the case of new bridges designed by incorporating composite action.

2. Mechanical girder-slab interface connectors provided during construction as well as other proven local details at the interface may facilitate long-term maintenance of composite action. This has been determined as a very desirable mechanism for performance at service and at ultimate limit states.

3. Cover plates that provide additional negative moment capacity at regions where negative moments are critical may lead to a

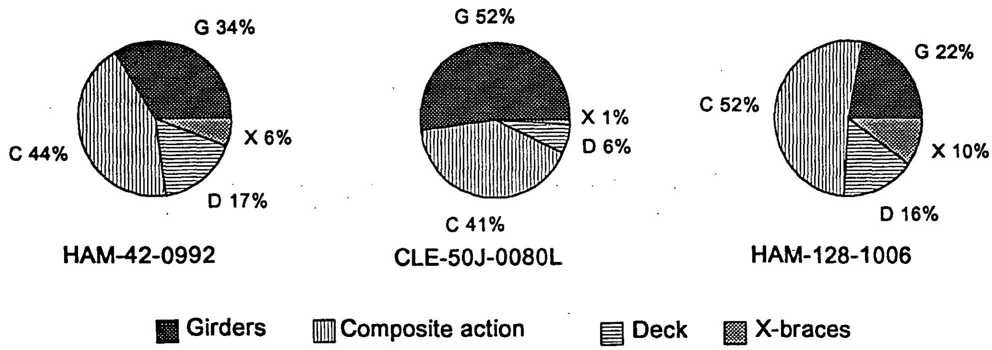


FIGURE 5 Contribution of mechanisms participating in stiffness.

more balanced design between negative and positive moments. Currently, designs based on AASHTO and conducted by analyzing 1-D models without incorporating the cover plates may appear not to be well balanced for different senses of moment.

4. This research revealed that integral abutments do significantly decrease positive moments in the end spans and increase the stiffness and serviceability by providing rotational stiffness. Some states permit integral abutment designs up to 800 ft. Their use in retrofit should be encouraged.

5. Provision of mechanical connections between the pads and the pier caps are recommended for lateral stability in the case of accidents. The flexibility provided by the pads at the supports regulated the negative moment demands.

6. Lateral cross-braces should be explicitly designed and incorporated in the global design. The stiffness provided by these braces was found to significantly enhance lateral redistribution of negative moment demands. Some of these recommendations on evaluation and rehabilitation are already incorporated into the Ontario Highway Bridge Design Code (*21*).

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