

**RELIABILITY-BASED CONDITION ASSESSMENT OF
EXISTING HIGHWAY BRIDGES**

A Dissertation
Presented to
The Academic Faculty

By

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In Partial Fulfillment
Of the Requirements for the Degree
Doctor of Philosophy in
School of Civil and Environmental Engineering

Georgia Institute of Technology

August, 2010

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ACKNOWLEDGEMENTS

My years at Georgia Tech have been, and will always be, one of the most valuable experiences in my life. It is the people with whom I have the privilege to work that have made this journey so special and memorable.

I cannot express enough my appreciation to my co-advisor Dr. Bruce R. Ellingwood. Beyond his wealth knowledge, passion for research, and dedication to excellence, he has showed me what it means to be a professional engineer, an outstanding professor and an exceptional researcher. Working with him has shaped me in so many ways and his influence will no doubt propagate beyond my Ph.D. study and serve me well for many years to come. I also would like to extend my gratitude to my co-advisor, Dr. Abdul-Hamid Zureick, who has encouraged me with his knowledge, patience, enthusiastic support, and guidance throughout the study. I attribute my valuable experience in Tech and research contributions to them.

I am grateful to my committee members - Dr. Kenneth M. Will, Dr. Don White, and Dr. James I. Craig - for their insight, advice and critical assessment of my dissertation work. The research described in this dissertation was supported, in part, by the Georgia Department of Transportation under grant RP05-01. This support is gratefully acknowledged.

I also like to thank wonderful fellow graduate students, Quanwang, Chuang-Sheng, Guoqing, Murat, Ozan, Kursat, Eun, Soravit, Laura, Bo, Rulan and Alan, for

offering me technical knowledge, lively discussions and friendship along the road. I extend a special note of appreciation to Curtis for providing me with reliable experimental data for the calibration of my simulations and for his willingness to coordinate on many other tasks of this research project. Thanks also due to my best friends, sisterhood, Li and Ming, for continuing to be so thoughtful and encouraging.

My ultimate support is always from my families: Xuefang, Dingyi, Xiaoyu, Xiangge and Ranran, without whom I could not stand, and from my husband, Yong, an unexpected reward from this journey, who has brought immeasurable happiness to my life.

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SUMMARY

Condition assessment and safety verification of existing bridges and decisions as to whether bridge posting is required are addressed through analysis, load testing, or a combination of methods. Bridge rating through structural analysis is by far the most common procedure for rating existing bridges. The American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation (MBE), First Edition* permits bridge capacity ratings to be determined through allowable stress rating (ASR), load factor rating (LFR) or load and resistance factor rating (LRFR); the latter method is keyed to the *AASHTO LRFD Bridge Design Specifications*, which is reliability-based and has been required for the design of new bridges built with federal findings since October, 2007. A survey of current bridge rating practices in the United States has revealed that these three methods may lead to different ratings and posting limits for the same bridge, a situation that carries serious implications with regard to the safety of the public and the economic well-being of communities that may be affected by bridge postings or closures.

To address this issue, a research program has been conducted with the overall objective of providing recommendations for improving the process by which the condition of existing bridge structures is assessed. This research required a coordinated program of load testing and finite element analysis of selected bridges in the State of Georgia to gain perspectives on the behavior of older bridges under various load

conditions. Structural system reliability assessments of these bridges were conducted and bridge fragilities were developed for purposes of comparison with component reliability benchmarks for new bridges. A reliability-based bridge rating framework was developed, along with a series of recommended improvements to the current bridge rating methods, which facilitate the incorporation of various *in situ* conditions of existing bridges into the bridge rating process at both component and system levels. This framework permits bridge ratings to be conducted at three levels of increasing complexity to achieve the performance objectives, expressed in the terms of reliability, that are embedded in the LRFR option of the *AASHTO Manual of Bridge Evaluation*. This research was sponsored by the Georgia Department of Transportation, and has led to a set of *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia*.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Bridge structures in the State of Georgia are at risk from aging, leading to structural deterioration from service demands from increasing traffic and heavier loads, from aggressive environmental attack and other physical mechanisms, and from deferred maintenance. Condition assessment and evaluation of existing bridges may be prompted by changes in traffic patterns; concern about faulty building materials or construction methods; discovery of a design/construction error after the structure is in service; concern about deterioration discovered during routine inspection; and damage following extreme load events. A condition assessment may be conducted to develop a bridge load rating, confirm an existing load rating, increase a load rating for future traffic, or determine whether the bridge must be posted in the interest of public safety. The Bridge Inventory Management System in the State of Georgia lists 8,988¹ bridges, which are monitored by the Georgia Department of Transportation (GDOT). While rating calculations have yet to be performed on 1,587 of these bridges, it has been determined that approximately 1,982 (or 22%) of them require posting. Posting or other restrictive actions may have a severe economic impact on the state economy, which depends on the trucking industry for distribution of resources and manufactured goods. The economics of upgrading or posting a bridge makes it imperative that condition assessment criteria and methods

¹ As of August 1, 2009

(either by analysis or by testing) be tied in a rational and quantitative fashion to public safety, functional requirements and economics.

Condition assessment and safety verification of existing bridges, and decisions as to whether posting is required are addressed through analysis, load testing, or a combination of methods. Of these, bridge rating by structural analysis is by far the most common (and most economical) method. Load testing may be indicated when analysis produces an unsatisfactory result, when the analysis cannot be completed due to lack of design documentation or information, or when structural deterioration of the bridge renders the traditional analysis methods questionable or inapplicable. Until recently, the customary rating process used in most states has been described in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Condition Evaluation of Bridges, Second Edition*, which allows ratings to be determined through either allowable stress methods (ASR) or load factor methods (LFR). In recent years, the State of Georgia has utilized the LF method for the majority of those bridges in the state that have been rated. A third (and more recent) rating procedure found in the *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* is keyed to the AASHTO Load and Resistance Factor Design (LRFD) method, which is defined in the *AASHTO LRFD Bridge Design Specification, Fourth Edition*. The LRFR method is being introduced in bridge maintenance, and some states are beginning to use it to determine bridge ratings. The *AASHTO Manual for Bridge Evaluation (MBE)*, First Edition (2008) has included all three methods. These three competing rating methods may lead to different rated capacities and posting limits for the same bridge, a situation that cannot be justified from a professional engineering

viewpoint and has potentially serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures. Moreover, analytical methods with a fixed format are designed to be applicable to the entire highway bridge population. While they may be conservative for many bridges, they also may fail to properly consider all the risks facing a particular bridge, since the condition rating and capacity rating in the current rating practice is relatively weak. The cost and social impact of failing to meet a performance objective when evaluating an existing bridge, especially in terms of posting unnecessarily or failing to post when necessary, can be very large. Accordingly, the economics of upgrading or posting a bridge makes it imperative to determine condition assessment criteria and methods (either by analysis or by testing) that are tied in a rational and quantitative fashion to public safety and functional objectives.

The Georgia Department of Transportation has a need for condition assessment tools that can be used with confidence to determine whether or not to post certain existing bridge structures. To address this need, the Georgia Institute of Technology has conducted a multi-year research program, sponsored by the GDOT, aimed at making improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed. The research reported in this dissertation provides the technical basis for improving bridge rating practices in Georgia.

1.2 RESEARCH OBJECTIVES AND SCOPE

The research in this dissertation is aimed at developing a practical reliability-based analysis framework for assessing bridge load-carrying capacity and for developing

rational inspection/maintenance strategies and policies. To accomplish this objective, the following research tasks have been conducted:

- Critically appraise current bridge condition assessment procedures.
- Summarize and assemble applicable structural reliability analysis tools to support condition assessment of bridge systems.
- Select, test and perform in-depth analysis of sample bridges that are representative of bridges that are of most concern to the GDOT.
- Develop a reliability-based bridge capacity assessment framework which provides practical tools for incorporating available *in situ* data into reliability based rating analysis.

The scope of this dissertation is limited to bridges on the state primary and secondary system, including reinforced concrete tee, prestressed and steel girder bridges, which are subjected primarily to permanent gravity loads and vehicular loads. Interstate, wood or historical bridges and railway bridges are excluded.

1.3 OUTLINE OF THE THESIS

This thesis is organized into eight chapters.

Chapter 2 reviews the reliability bases for current bridge condition evaluation and AASHTO guidelines on bridge assessment, and summarizes the results of a survey of state DOTs conducted to investigate the engineering practices on bridge rating nationwide. In addition, documents used for bridge rating in the United Kingdom, Australia, and Canada were obtained to gain an international perspective on the subject.

Chapter 3 presents a general framework for bridge safety evaluation that directly addresses the deficiencies in current practice noted in Chapter 2. This framework has three levels of assessment of increasing complexity. In the first level, the deterministic member-based format of the AASHTO LRFR method is kept, and the correlation between visual condition rating and the capacity evaluation is established. The second level allows for the incorporation of site-specific data obtained from material tests, diagnostic load test and from in-depth structural analysis in rating calculations. In the third level, bridge system reliability is evaluated by incorporating proof load test results and routine inspection records regarding bridge performance history. This framework highlights the learning process in rating a given bridge and provides clear incentives to obtain quantitative *in situ* measurements in routine bridge inspections.

Four bridges typical of bridges of concern in rating and posting are summarized in Chapter 4 and the load testing and finite element analysis of these bridges is described. Finite element models of these bridges were developed to assist the design of the load tests and in the interpretation of the results. The bridge test results, in turn, were used to validate and improve the finite element modeling.

Chapter 5 presents the level-one assessment which is basically consistent with the current AASHTO LRFR method, but with one significant adjustment: a new method is introduced to correlate visually-based bridge condition ratings from routine periodic inspections with structural capacity. A revised set of values of ϕ_c tied to the AASHTO LRFR rating equations are developed to be consistent with the structural reliability-based philosophy embodied in the AASHTO LRFR and to incorporate recent developments in

bridge resistance degradation modeling and comprehensive databases of bridge condition rating history.

In the level-two assessment in Chapter 6, bridge resistance models at the component level can be “customized” for an individual bridge by incorporating available site-specific knowledge. This level of analysis reflects the fact that each bridge is unique in its as-built condition, and provides bridge engineers with an option to account for this uniqueness to achieve a better evaluation of the bridge performance when such effort is believed to be warranted. Tools are provided for incorporating the structural component knowledge gained from in-situ material tests, diagnostic load tests, and improved mechanical models for structural component analysis into the bridge safety evaluation process.

Chapter 7 presents the level-three assessment, which focuses on bridge safety at the system rather than component level and provides additional perspective on the (unknown) level of conservatism furnished by the current generation of reliability-based condition evaluation and rating procedures which are member-based. The possibilities of incorporating proof load test results and successful service performance history into the bridge rating framework are explored.

Finally, Chapter 8 summarizes the major research findings and conclusions, and outlines future research needs.

CHAPTER 2

REVIEW OF CONDITION EVALUATION PRACTICES FOR EXISTING BRIDGES

This chapter reviews current rating procedures for performing condition assessments of existing bridge structures. The review emphasizes current practices in the United States, but practices in several other industrialized countries are also summarized to provide additional context. This review is aimed at achieving a general perspective on technical issues associated with condition assessment methodologies used for bridges and other civil infrastructure applications.

2.1 RELIABILITY-BASED BRIDGE CONDITION ASSESSMENT FRAMEWORK

Uncertainties arise from variations in loads, material properties, dimensions, natural and manmade hazard, insufficient knowledge, and human errors in design and construction [Ellingwood, et al, 1980; 1982; Ellingwood and Galambos, 1982]. The uncertain nature of the data makes structural reliability theory a logical and powerful tool for performing quantitative performance assessments of existing structures. Moreover, probability-based limit states analysis provides a clear link between theoretical research and in-service experience, and also provides a theoretical basis for utilizing *in situ* data in the bridge evaluation process. Recent advances in bridge design and rating in the United States and elsewhere have a reliability basis.

The starting point for a quantitative evaluation of structural reliability is the description of the limit state of concern (flexural failure, instability, etc) by an expression

relating the resistance and load variables described above, derived from principles of structural mechanics. This expression, denoted the limit state function, is given by,

$$G(X) = G(X_1, X_2, \dots, X_m) = 0 \quad (2-1)$$

in which $X = (X_1, X_2, \dots, X_m)$ = vector of random resistance and load variables. The limit state is defined, by convention, as when $G(X) < 0$. Thus, the limit state probability is,

$$P_f = \int f_x(x_1, x_2, \dots, x_m) dx_1 dx_2 \dots dx_m \quad (2-2)$$

in which $f_x(x)$ = joint probability density function of \mathbf{X} and the domain of the multi-fold integration is that region of \mathbf{x} where $G(\mathbf{X}) < 0$. The limit state probability, P_f , is the quantitative metric of structural performance that is consistent with the uncertainties in structural resistance and loads.

Modern probability-based limit states design approaches, including the *AASHTO LRFD Bridge Design Specifications* [2007], have adopted the *reliability index*, β , as a measure of reliability instead of P_f . For typical structural engineering situations, the reliability index is in the range of 2 to 4.5. The reliability index is related, in a first-order sense, to the limit state probability by $P_f = \Phi(-\beta)$ for well-behaved limit state functions that are typical of those found in bridge design and condition assessment.

The target reliability index of 3.5 in the *AASHTO LRFD Specifications* for new bridge structures was determined by calibration to a spectrum of traditional bridge design situations (vintage 1985 and earlier) involving steel, reinforced and pre-stressed concrete construction. Gravity load situations were considered in this calibration exercise. A group of experts from the material specification committees participated in assessing the results of this calibration, and selecting target reliabilities. This target index of 3.5 was adopted directly in *LRFR* for capacity checking at the design load level. For

AASHTO legal load or State legal loads, the target index was chosen to be 2.5 by judgment [Moses, 2001; Minervino, et al, 2004]. In the latter case, the implied acceptable annual failure rate of an existing bridge would be at least an order of magnitude higher than a newly constructed bridge. The design load checking and legal load checking are comparable to inventory and operating level checking, which are discussed later in section 2.2.1.

When an existing bridge structure is evaluated, the knowledge gained from additional *in situ* data about the existing bridge or its components through field inspection, load testing, material tests, or traffic surveys, if available, could be applied to refine the probabilistic models of related random variables. Inspection, therefore, should lead to an improvement in the prior estimate of failure probability discussed above. The theoretical basis for incorporating additional information is provided by Bayes theorem [Ang and Tang, 2007]. The updated (posterior) failure probability of an existing bridge, P_f' , can be expressed as [Madsen, 1987; Ellingwood, 1996]:

$$P_f' = P[G(\underline{X}) < 0 | H] = \frac{P[G(\underline{X}) < 0 \cap H]}{P[H]} \quad (2-3)$$

in which, H is an event describing the available site-specific knowledge, such as the result of a bridge inspection or a proof load test. It is clear that strong stochastic dependence between the events $[G(\underline{X}) < 0]$ and $[H]$ will produce a tighter updated distribution, giving more confidence about the estimated random vector \underline{X} . If little is learned by the inspection or condition assessment, the correlation is weak; the prior and posterior estimates of the reliability will be nearly the same.

It should be emphasized that current condition assessment procedures for bridges do not utilize the valuable information regarding *in situ* condition that is reflected by the updating process summarized in Eq (2.3). The proposed methodology in Chapter 4 will remedy this deficiency in current practice.

The structural reliability theory framework described above provides a conceptual platform for the codified limit state bridge design and evaluation. The probabilistic models of the major random variables involved in the bridge reliability analysis will be discussed next.

2.2 CURRENT AASHTO GUIDELINES FOR BRIDGE EVALUATION

2.2.1 Bridge Rating by ASR, LFR and LRFR

Until 1970, the sole design philosophy embedded within *AASHTO Standard Specifications for Highway Bridges* was Allowable Stress Design (ASD). The allowable stress is established as a fraction of the load carrying capacity of a structural element (usually the yield or fracture strength in tension or point of instability in compression), and the structural action (stress in tension, bending or compression) from the applied loads may not exceed this allowable limit. Detailed procedures for rating existing bridges based on the ASD method first appeared in 1970 in the *AASHTO Manual for Maintenance Inspection of Bridges*.

Beginning in the early 1970's, as the design of reinforced concrete and steel structures was reformulated in terms of "ultimate strength" for concrete and "plastic" design for steel, the load analysis formerly used in ASD was modified as well, with adjustments to the load factors to reflect the relative uncertainty and predictability of different loads, such as vehicle loads, wind and earthquake effects. The new design

philosophy was referred to as Load Factor Design (LFD) and was incorporated in the *Manual for Condition Evaluation of Bridges (MCE)*, which was published by AASHTO in 1994 to replace the earlier *Manual for Maintenance Inspection of Bridges*. Although the 1994 manual contains some guidance for allowable stress rating (ASR), it clearly emphasized the load factor rating (LFR) method. Many State DOTs continue to use the 1994 *Manual*, with 1995, 1996, 1998 and 2000 interim revisions, in their bridge rating work¹.

In 1994, the AASHTO Bridge Subcommittee voted to adopt the *AASHTO LRFD Bridge Design Specifications* and in 1998 designated LRFD as the primary design method for highway bridges. The *LRFD Bridge Design Specifications* (the latest edition is dated 2010) represented the first effort by AASHTO to integrate modern principles of structural reliability and the probabilistic and statistical models of loads and resistance into the design of highway bridges. LRFD introduced the reliability-based limit states design philosophy to achieve a more uniform and controllable safety levels for each applicable limit state. To extend this philosophy to the evaluation of existing bridges, AASHTO released the 2003 *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, which presents the first bridge load rating method in the United States to have a structural reliability basis.

At the present time, the ASR, LFR and LRFR methods of bridge rating are all included in *AASHTO Manual for Bridge Evaluation (MBE)*, First Edition, 2008 and are

¹ Wang, N., Ellingwood, B.R., Zureick, A. and O'Malley, C. (2009). "Condition assessment of existing bridge structures: Report of Task 1 – Appraisal of state-of-the-art of bridge condition assessment." Report of Project *GDOT No. RP05-01*, Georgia Department of Transportation, Atlanta, GA ([ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research Projects/05/2010](ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research%20Projects/05/2010)).

current use by State DOTs. A summary of these procedures and a critical appraisal of their relative merits are presented in this section.

Allowable Stress Rating (ASR) and Load Factor Rating (LFR)

The rating factors in both ASR and LFR are determined by [AASHTO MCE, 1994]:

$$RF = \frac{C - A_1 D}{A_2 L(1 + I)} \quad (2-4)$$

in which RF is the rating factor for the live load carrying capacity (expressed as a multiple of the design live load effect (from a rating vehicle) that can be carried by the bridge), C is the capacity of the structural member, D and L are, respectively, the dead and live load effect on the member, I is the impact factor to be used with the live load effect, A_1 is the factor on dead load, and A_2 is the factor on live load. The Rating Factor (RF) determined from Eq (2-4) is used to compute the rating of the bridge in tons as [AASHTO MCE, 1994]:

$$RT = (RF) \times W \quad (2-5)$$

where RT is the bridge member rating in tons, and W is the nominal weight (tons) of the rating truck used in determining the live load effect (L).

Both ASR and LFR methods rate bridges at two levels: Inventory and Operating. The Inventory rating level generally corresponds to the customary design level of allowable stress or strength, but reflects the existing bridge and material conditions with regard to structural deterioration. Load ratings based on the Inventory level allow a comparison of the estimated capacity of an existing bridge with the capacity for a new bridge, and therefore result in a live load which can safely carried by the existing bridge

structure for an indefinite period of time. In contrast, load ratings based on the Operating rating level generally describe the maximum permissible live load to which a structure may be subjected during a limited period of time. Allowing an unlimited number of vehicles to use the bridge at the Operating level may shorten the life of the bridge [AASHTO MCE, 1994]. Rating at the Operating level generally is the basis for decisions regarding traffic restriction and load posting.

Although the rating factor format for ASR and LFR is the same, the load factors (A_1, A_2) and the calculation of the capacity (C) used in Equation (2.1) are different. In ASR, $A_1 = A_2 = 1.0$ for both Inventory and Operating level rating; C depends on the rating level desired, with the higher value of C used for Operating level. In the LFR procedure, $A_1 = 1.3$, while A_2 equals 2.17 for Inventory rating and equals 1.3 for Operating level rating; the nominal capacity C is the same regardless of the rating level desired.

Load and Resistance Factor Rating (LRFR) Procedure

The general LRFR rating equation is (AASHTO LRFR, 2003):

$$RF = \frac{C - \gamma_{CD}DC - \gamma_{DW} \pm \gamma_P P}{\gamma_L LL(1 + IM)}$$

$$C = \phi \phi_C \phi_S R_n$$

$$\phi_s \phi_c \geq 0.85$$
(2-6)

in which C is the structural capacity, R_n is the nominal member resistance, DC is the dead-load effect of structural components and attachments, DW is the dead-load effect of wearing surfaces and utilities, P is the permanent loading other than dead loads (post-tensioning for example), LL is the live-load effect, IM is the dynamic load allowance, γ_{DC} is the load factor applied to the weight of structural components and attachments, γ_{DW} is the load factor for wearing surfaces and utilities, γ_P is the load factor for

permanent loads other than dead loads, and γ_L is the live-load factor. The resistance factor (ϕ) accounts for the general uncertainties in the resistance of a bridge member in a satisfactory condition and is the same as that used in LRFD bridge design. The condition factor (ϕ_c) accounts for increasing uncertainties in bridge member resistance once its condition deteriorates, and takes a value of 0.85 for members in poor condition, 0.95 for members in fair condition, and 1.0 for members in good condition. The system factor (ϕ_s) accounts for the level of redundancy in the structure. Bridges that are less redundant or non-redundant are assigned a lower system factor and therefore have lower rated capacities.

The LRFR method supports bridge evaluation for three general limit states that were introduced in the *LRFD Bridge Specification*: the strength-limit state (flexural or shear capacity), the service-limit state (deflections and rotations) and the fatigue limit state. The strength limit state is fundamental for public safety and is the main determining factor for bridge posting, closure and repairing. Service and fatigue limit states are applied selectively to bridges.

In the LRFR method, bridges are evaluated in a three-step approach for each limit state, as shown in Figure 2.1: design load rating (HL93), legal load rating (AASHTO/state legal trucks), and permit load rating (overweight trucks). An initial check first is performed using the HL-93 design load (Figure 2.2) using the dimensions and properties corresponding to the present *in situ* condition of a bridge. The bridge is rated using the same live and dead load factors as those used in the *LRFD Bridge Design Specifications*, which were calibrated to ensure a safety index of 3.5 (discussed subsequently in section 2.1.2 of this chapter). This check measures the performance of

the existing bridge in comparison to the expected performance of a new bridge, and serves as an initial screening check; a bridge resulting in a RF at this level larger than 1.0 requires no further analysis for any legal loads that result in member forces lower than the HL-93 design load. For example, the HL-93 load is designed to represent the member forces caused by the AASHTO legal loads through a single load case. Therefore any State legal loads that are equal to or less than the AASHTO legal load are covered by a HL-93 design load analysis. On the other hand, if a state has legal loads that surpass the AASHTO legal loads, those states must verify that HL-93 load case incorporates those legal loads.

If the bridge fails to pass the HL-93 design load check, a follow-up evaluation is performed using the AASHTO/State legal trucks (illustrated for the State of Georgia in Figure 2.3). The live load factor used at this level is calibrated to a safety index of 2.5 and varies in accordance with local truck traffic conditions at the bridge site (ADTT). The safety criteria, in comparison with the 3.5 in the previous step, are less conservative and reflect the substantial cost impact of strengthening an existing bridge or restricting traffic, as well as the shorter future service period expected compared to the 75 years that is typical for the design of a new bridge [Nowak, 1999; Moses, 2001]. The ratings determined using the legal loads are generally used as the basis for determining whether to post or strengthen the bridge.

In certain cases, a permit load rating may be performed to check the safety (and serviceability) of the bridge for vehicles above the legally established weight limit. This procedure is only necessary when an overweight vehicle is to use a bridge, and it is only allowed for bridges that yield $RF \geq 1$ at the previous legal load rating levels. The permit

live load factors were derived to account for the possibility of the simultaneous presence of one or more non-permit heavy trucks on the bridge when the permit vehicle crosses the span, as well as the site-specific traffic conditions described by the ADTT.

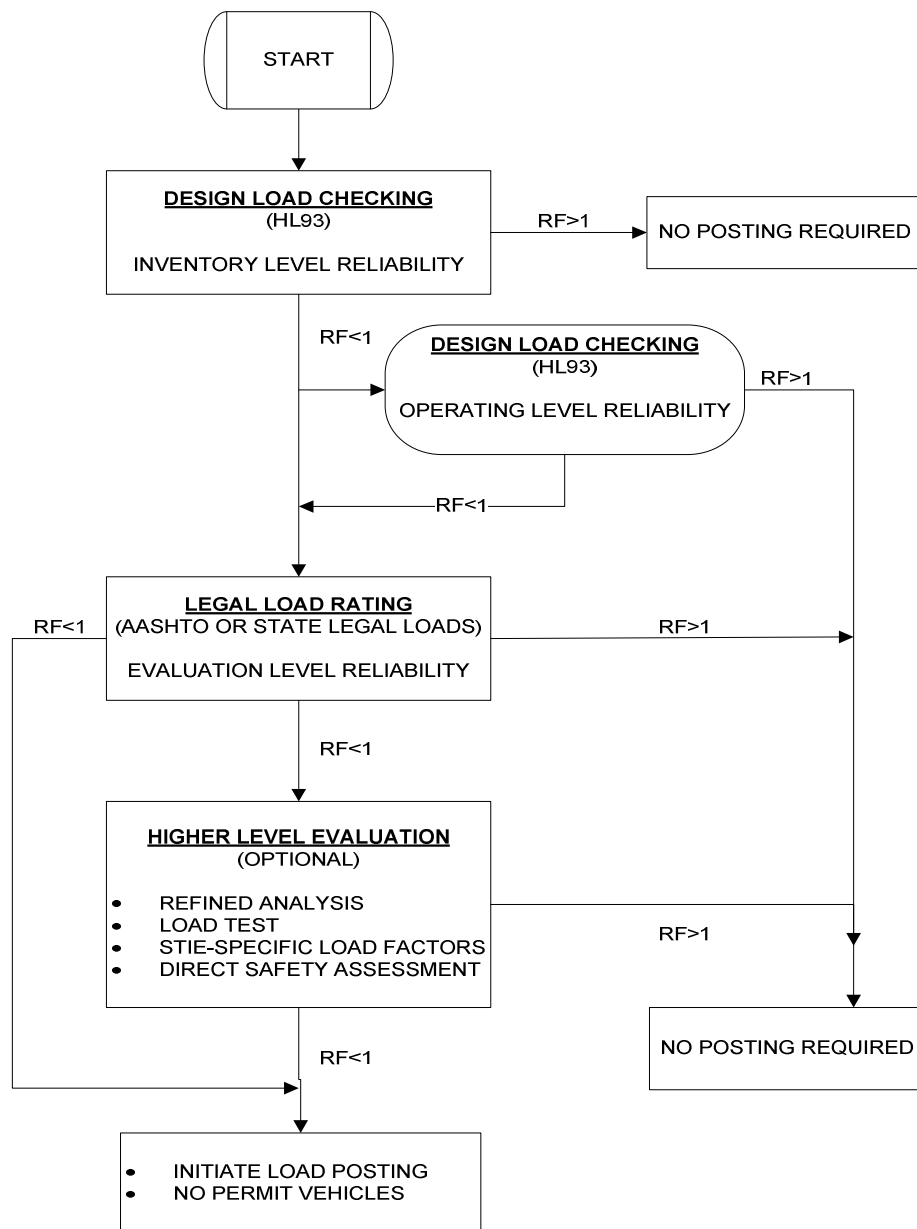


Figure 2.1 Loads and Resistance Factor Rating Procedure [AASHTO LRFR, 2005]

A comparison of the ratings used in the LRFR method (Eq.2.6) with those in the LFR/ASR method (Eq.2.4) shows three key improvements. First, LRFR attempts to reflect the *in situ* bridge resistance systematically and objectively through the use of the system factor (ϕ_s) and the condition factor (ϕ_c). In the LFR/ASR methods, the condition of the bridge, its redundancy, and any deterioration at the time of evaluation must be factored into the estimation of the capacity term (C) in a completely subjective manner. Second, the LRFR method considers dead load from factory-made members, cast-in-place members and wearing surfaces separately, with each assigned an independent dead load factor to account for the different degrees of variability in these components of dead load (discussed subsequently in section 2.1.2 of this chapter). In the LFR/ASR methods, all permanent loads are combined in calculating the dead load effect (D), to which an overall dead load factor is applied; adjustments that might be indicated by available *in situ* dead load measurements are difficult to handle in the rating process. Third, the LRFR method has provided a set of live load factors that ranges from 1.4 to 1.8, depending on the bridge's *in situ* traffic condition indicated by ADTT, for rating calculations at the legal load level. This improvement allows site-specific traffic data to be incorporated into the load rating process, which is a major advantage from applying probability-based structural reliability theory in existing bridge condition assessment and offers an important enhancement of the LRFR method over the traditional stress-based rating approaches.

The LRFR method further simplifies the bridge rating process by requiring the use of the HL-93 design load as the starting point in the rating and as a screening check for all other AASHTO/State legal loads. The HL-93 live load envelopes all types of legal

loads in the United States and provides a uniform reliability check for various span lengths with just this one load model. Otherwise, to achieve a uniform reliability for highway bridges using LRFR, rating calculations have to be applied to all three AASHTO legal loads individually, with each controlling short, medium, or long spans respectively [NCHRP 12-28, 2001; Minervino, et al, 2004]. In contrast, the HS-20 design load checking used in the ASR/LFR process does not envelope current trucks on the highway system and the ratings determined with this vehicle do not provide uniform reliability for bridges of varying span lengths. Finally, permit vehicles that are significantly heavier than the AASHTO/State legal loads may have very different configurations. While the LRFR method provides procedures and live load factors specific to permit vehicles ratings for bridges that have been demonstrated to have adequate capacity for AAASHTO/State legal loads, the LFR/ASR methods provide no guidance on permit checking.

Despite these improvements, the LRFR procedure has not been widely adopted for rating or posting bridges in the United States. A survey of State Departments of Transportation on bridge evaluation practices (presented in section 2.3) has revealed a number of issues and concerns with the LRFR method. Addressing these issues will facilitate the adoption of the LRFR, in a modified form, and provide an improved bridge rating methodology. Such improvements are the subject of the current research program, and are presented in Chapter 3, following the survey.

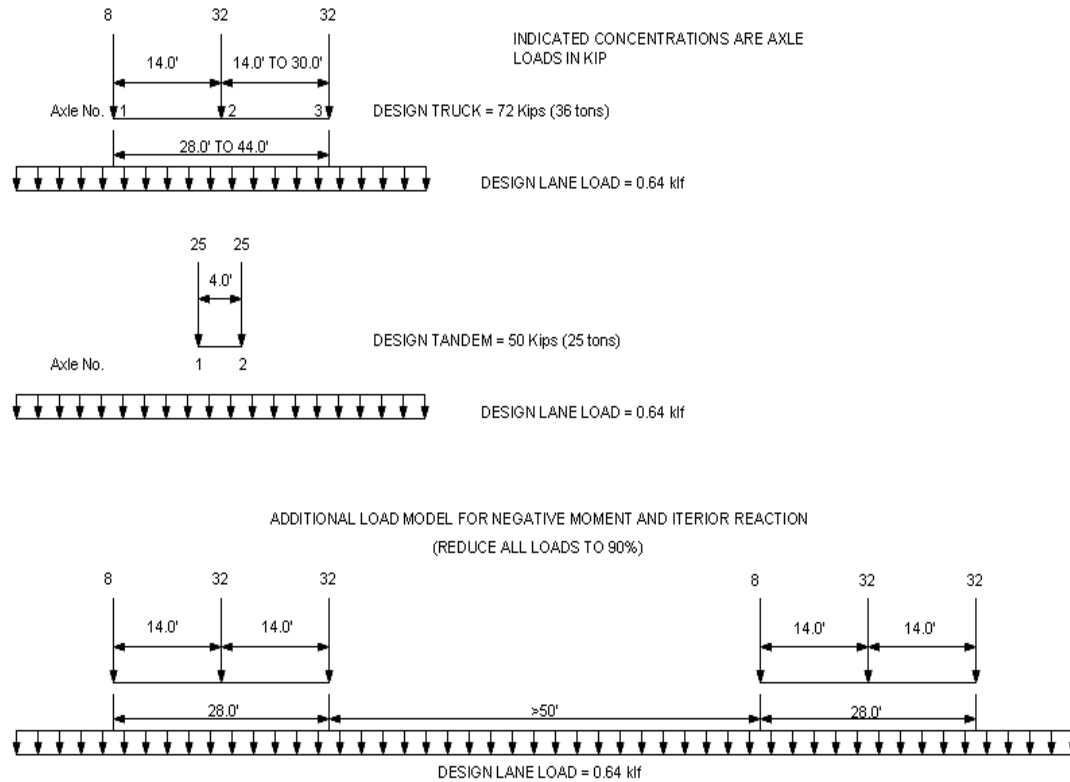


Figure 2.2 LRFD Design Live Loads (HL-93)

2.2.2 Probability Models and Supporting Data for Reliability-based Bridge Rating

As noted previously, the LRFD option in the *AASHTO Manual for Bridge Evaluation* [2003] is the first bridge load rating method in the United States to be based on modern principles of structural reliability and limit states design. The essential ingredients of a reliability-based design and evaluation include probabilistic models of the structural resistance and loads and a method for analyzing the reliabilities (or, conversely, the limit state probabilities) that are relevant to each bridge limit state. This section provides a brief summary of such methods and tools, as they have been applied to developing the *AASHTO LRFD Bridge Specifications* and the *MBE* and are expected to

be relevant to the current research program to develop improved bridge rating methods. Details are available in the archival literature [Nowak, 1999; Moses, 2001].

Structural Resistance Models

The capacity of a bridge depends on the strength of its components and connections. The strength, R , is a random variable having uncertainties that fall into three categories [Ravindra and Galambos 1978; Moses, et al, 1987; Tabsh et al, 1992]: material properties, M , including material strength, modulus of elasticity, cracking stress and chemical composition; fabrication, F , including geometry, dimensions and section modulus; and structural modeling, P , reflecting assumptions and approximate analysis methods. The mean and coefficient of variation for M , F and P are usually determined by material tests, simulations, observations of existing structures and engineering judgment.

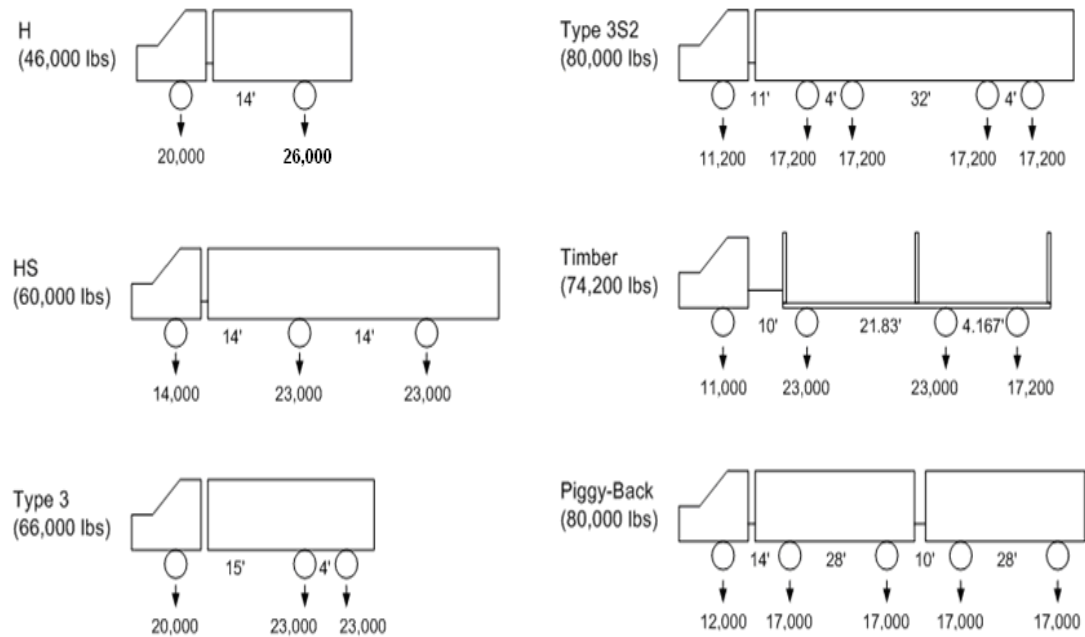


Figure 2.3 State of Georgia Legal Loads

In the development of the *ASHTO LRFD Bridge Design Specifications* [Nowak, 1999], R was determined as the product of the nominal resistance R_n and the three above-mentioned parameters, M , F and P :

$$R = MFPR_n \quad (2-7)$$

As a product of random variables that are assumed to be statistically independent, the resistance is modeled by a lognormal distribution with mean, μ_R , and coefficient of variation (COV), V_R , computed as follows:

$$\begin{aligned} \mu_R &= R_n \mu_M \mu_F \mu_P \\ V_R &= (V_M^2 + V_F^2 + V_P^2)^{1/2} \end{aligned} \quad (2-8)$$

in which μ_M , μ_F and μ_P are the means of M , F and P and V_M , V_F and V_P are the COVs of M , F and P , respectively. The statistical parameters of R used in the development of the *LRFD Specifications* for different types of structural components (steel girders, composite and non-composite, reinforced concrete T beams and pre-stressed concrete AASHTO-Type girders) in different failure modes (bending and shear) are presented in Table 2.1.

Dead Load Model

Dead load is the weight of structural members, nonstructural components and attachments, and traffic wearing surfaces. Because of the different degrees of variability, one must consider the components of bridge dead load from factory-made members (steel and pre-cast concrete), cast-in-place members (T-beams, slabs), and wearing surfaces (asphalt) separately. Generally speaking, dead loads can be predicted more accurately than live loads, as long as accurate records have been kept and the as-built condition agrees with the available drawings. In the study by Moses and Verma [1987], the bias

(defined as the ratio of the mean to nominal load) and COV of bridge dead loads were taken to be 1.0 and 0.10 respectively. Later in the *AASHTO LRFD* calibration [Nowak 1999], the dead load was divided into four components and each component was modeled with a normal distribution. Finally, Ghosn [2000] used 1.0 and 0.09 for the dead load bias and COV respectively in his study. These components of dead load are listed in Table 2.2 along with their statistical parameters; the “miscellaneous” category is the dead load portion from railings and luminaries.

Live Load Model

Bridge live load is produced by vehicles moving on the bridge. Variability in live load arises from uncertainties in vehicle weight, vehicle position, average daily truck traffic (ADTT), calculations of live load effect (including distribution of live load to supporting girders), and the likelihood of several heavy vehicles being on the bridge at the same time [Moses and Verma, 1987]. Traditionally, the static and the dynamic effects of the live load are considered separately and assumed to be statistically independent [Nowak, 1993; 1999].

Based on weigh-in-motion (WIM) data, Moses and Verma [1987] identified several variables to provide a simplified model for determining the maximum expected single truck load effect:

$$M = aW_{.95}mHIg \quad (2-9)$$

in which M is the predicted maximum dynamic live load effect; a is a constant which relates M to a reference loading model (taken as an AASHTO/legal rating vehicle); $W_{.95}$ is the 95th percentile characteristic value of 75-year maximum truck weight, assumed to be a random variable to reflect the possible errors (epistemic uncertainty) in load estimation

and site-to-site differences; the variable m reflects the influence of the dominant vehicle type and configuration at a site; the variable H reflects the overload events due to the multiple vehicle presence, such as side by side or following vehicles, and also reflects the probability that truck weight exceeds the 95th percentile in combination with closely spaced vehicles; variable I is the dynamic impact allowance and variable g is girder distribution factor. Except for the constant a , all of the variables in Eq. (2-9) are random variables with statistics based on studies and data collected on a number of sites.

Table 2.1 Statistical Parameters of Component Resistance [Nowak, 1999]

Type of Structure	<i>FM</i>		<i>P</i>		<i>R</i>	
	λ_{FM}	V_{FM}	λ_P	V_P	λ_R	V_R
Non-composite steel girders						
Moment (compact)	1.10	0.08	1.02	0.06	1.12	0.10
Moment (non-com.)	1.09	0.08	1.03	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.11
Composite steel girders						
Moment	1.07	0.08	1.05	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.11
Reinforced concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.13
Shear w/ steel	1.13	0.12	1.08	0.10	1.20	0.16
Shear w/o steel	1.17	0.14	1.20	0.10	1.40	0.17
Prestressed concrete						
Moment	1.04	0.05	1.01	0.06	1.05	0.08
Shear w/ steel	1.07	0.10	1.08	0.10	1.15	0.14

Table 2.2 Statistical Parameters of Dead Load [Nowak, 1999]

Component	Bias Factor	C.O.V
Factory-made members	1.03	0.08
Cast-in-place members	1.05	0.10
Asphalt	3.5 inch*	0.25
Miscellaneous	1.05	0.10

* mean thickness

The live load model used to calibrate the *AASHTO LRFD Bridge Design Specifications* is based on the weigh-in-motion data of 10,000 trucks taken at a site in Ontario in 1975, which included axle weights, gross weight and axle spacing for each vehicle [Nowak, 1999]. These 10,000 data points were assumed to define the upper 20% of the truck traffic at the site over a period of about two weeks. By finding the maximum bending moment and shear forces for each Ontario truck on different spans ranging from 10 ft (3 m) to 200 ft (60 m), the cumulative distribution functions (CDFs) of live load effect for various span lengths were obtained. Bridges with both simple spans and two continuous equal spans were considered. These CDFs were then extrapolated to a full lifetime (75 years) consisting of some 75 million truck load events and the 75-year maximum live load was fitted by a normal distribution.

Static and dynamic load effects were studied separately [Tabsh and Nowak, 1991]. On the basis of a finite element study of bridges with various span lengths, it was found that the ratio of the mean value of the 75-year maximum live load (without dynamic impact) to nominal (HL-93) live load is dependent on the bridge span and its COV is about 12%. The study also concluded that dynamic impact was dependent on three major factors: bridge dynamics, vehicle dynamics and road roughness; the mean value of the dynamic load factor does not exceed 0.15 for a single truck and 0.10 for two trucks side by side, and its COV is about 80%. For the static and dynamic combined load effect, the mean of this 75-yr maximum live load with respect to the design load model (HL93 in Figure 2.1) fell in the range 1.0-1.2, depending on span length, and the COV was found to be about 0.18.²

²Imai and Frangopol (2001) found that the maximum bridge live load was best modeled by a Type I distribution of extreme values. Bhattacharya et al. (2006) also found that the Type I distribution fits the

2.3 SURVEY OF BRIDGE RATING PRACTICES IN THE UNITED STATES

2.3.1 Development of Survey of Bridge Rating Practices

As part of NCHRP Project 12-46 that developed the *AASHTO LRFR Guide Manual* [2003], a survey questionnaire had been mailed to State Bridge Engineers in May, 1997, asking for current practices and views on technical issues pertaining to the inspection, evaluation and load rating of bridges. The responses to this questionnaire were valuable in developing the rating criteria in the *AASHTO LRFR Guide Manual*. However, in the intervening years, the state of bridge evaluation practices in the United States has continued to evolve. Accordingly, a follow-up questionnaire was prepared that requested additional information on a subset of topics covered in the older survey, with specific emphasis on bridge capacity evaluation practices that may have changed in the intervening years and would be of particular interest to the current research to develop improved bridge rating procedures and a set of *Recommended Guidelines*. The questionnaire was sent out to all states in November, 2005, and after four months, forty one responses (Table 2.3) were received and reviewed. A copy of this survey questionnaire can be found in Appendix A to this report.

The synthesis of the survey responses in Section 2.3.2 is presented without identifying the state or the respondent. These responses often were presented in sentence fragments; in that case, an attempt has been made to complete the view expressed in the comment with a minimum of editing. The survey questions fall into several general categories: when to load rate a bridge, when to update existing ratings, how to rate, when

experimental measurements of live load effect properly. Finally, Galambos, Ellingwood et al (1982) used the Type I distribution to model the 50-year maximum live load for building structures.

to post, and other performance issues (connections, fatigue, and scour). The following synthesis of the survey responses is organized around those categories.

Table 2.3 Responding States

Alabama	Iowa	Oklahoma
Alaska	Kansas	Oregon
Arizona	Maine	Rhode Island
Arkansas	Maryland	South Dakota
California	Minnesota	Tennessee
Colorado	Mississippi	Texas
Connecticut	Missouri	Utah
Delaware	Nevada	Vermont
Florida	New Hampshire	Virginia
Georgia	New York	Washington
Hawaii	New Mexico	West Virginia
Idaho	North Carolina	Wisconsin
Illinois	North Dakota	Wyoming
Indiana	Ohio	

2.3.2 Synthesis of Survey Response

When to rate?

In order to comply with FHWA regulations all states either perform a load rating analysis, or make a professional judgment as to the load capacity of their bridges. Most states are working toward 100% load rating, and most of those responding reported to have rated between 80% and 100% of their bridges. This intention is summarized by the response from a Western state: “Our goal is to rate all state owned bridges to determine the maintenance requirements and bridge load carrying capacities and to comply with the *National Bridge Inventory System* (NBIS). Also modeling all bridges will help in

overload permit evaluations.” Only five states have 60% or fewer of their bridges rated. One Western state is in the process of updating all of its ratings to include *both* Inventory and Operating and presently has over 90% rated at Inventory levels and approximately 5% rated at Operating levels. It is the policy of most states to rate all new bridges when they are designed or constructed. Existing unrated bridge structures are being evaluated and rated, as circumstances and resources permit. The rating of existing bridge structures in general begins with those for which design documents are available, and then continues to bridges without them. The rating of bridges without plans is typically performed in one of four ways: using plans from a similar bridge built at about the same time; by load testing the bridge; using results of load tests from a similar bridge structure; or by professional judgment.

When to update ratings?

As to when to update existing load ratings, the following is quoted from the response provided by a Midwestern state and is indicative of other responses:

1. There is a physical change in the condition of a bridge or a structural member, e.g., physical alteration in the structure; new beam or new deck, rusting or spalling or damage occurred to the structural member(s) resulting in section loss; change in the wearing surface; change in the super-imposed dead loads; excessive deflection or settlement observed; occurrence of an accident;
2. There is a request to re-evaluate the rating of a structure for a vehicle different than what was previously used such as for single trip permit load ;
3. There is a change from the method of analysis used for previous rating;
4. Special circumstances dictate re-analysis of the structure.

5. There is a change of the rating method (e.g. switch from ASR to LFR), rating software or the truck weight regulations.

A western state remarked that all their load ratings are being recalculated because: “the previous ratings were done by different individuals and are not consistent.” Some other states have specific policies on this issue such as revising load rating when “overlay changes more than 2 inches”, “steel section losses are more than 1/16 inches” or “primary member condition rating on the inspection report has changed by more than one point since last routine inspection if the initial rating was 5 or lower.”

What method to use in rating?

Thirty one (31) of the responding states reported that the LFR method is their primary rating method, but that they occasionally used the ASR method in cases where the LFR method does not appear to be applicable. The remaining ten (10) states reported to use a combination of ASR, LRF, and LRFR depending on what specifications governed the design of the bridge.

Nine (9) of the responding states reported that they were currently using LRFR on either all of their bridges or those bridges designed by *AASHTO LRFD Bridge Design Specifications*. Five (5) states were in the process of evaluating the suitability of LRFR, and were planning a transitioning from LFR to LRFR. Most of the remaining states cited either the lack of resources or readily available software as reasons for not making the transition, but noted that they would change to LRFR if mandated. Several specifically

said they were waiting for VIRTIS³ to offer a version that incorporated the LRFR method before they considered switching from LFR.

Several states which have considered transitioning to LRFR raised some significant questions and issues. One Midwestern state suggested that “the proposed updates to the guidelines do not inspire confidence in the manual.” A Western state responded “we are concerned with the high load factors; if we can not lower these factors through WIM (weight in motion) data, we may use older load rating methods on older bridges.” Two other Western states simply stated “it was too uncomfortable with the LRFR method to use it” and “not fully confident in this document.” The strongest opposition to transitioning to LRFR came from an Eastern state, which observed: “Too much work for no value. Ratings for concrete and timber do not correlate to real world. For timber, LRFR requires a “fudge” factor to get reasonable results for posting. For reinforced concrete bridges, the change from ASR to LFR resulted in a reduction of approximately 20% in posting values and changing from LFR to LRFR will result in another 15% to 20% reduction in the posting limits. On the other hand, with LFR and LRFR, “posting values for steel bridges increase.” This Eastern state also had serious questions as to the applicability of LRFR and its ability to perform its main function of providing a uniform reliability for all bridge structure types. A similar concern was expressed by a Midwestern state, which also doubted whether LRFR was suited for all bridge types. These apprehensions about the transition from the older methods to the LRFR method warrant further investigation. A summary of an investigation aimed at examining the differences in ratings using these methods through illustrative rating

³ VIRTIS is a bridge rating software package, developed by *Cambridge Systematics, Inc.*

calculations performed for four sample bridges selected from the Georgia bridge inventory will be presented in Section 2.3.3.

Most of the states using LFR employ the HS 20-44 vehicle for both Inventory and Operating ratings at the design load level. Some states use the full set of AASHTO Legal vehicles, HS-20, H-20, type 3, type 3-3, and type 3S2, for legal load ratings. In some other states, the AASHTO vehicles are modified and designated as “state legal loads.” These modifications typically consist of a scaled-up load and/or a redistribution of the load between the cab and trailer. There are also a few states with unique legal loads, such as logging trucks or other highly used regional vehicles.

When to post?

Answers to the survey question regarding the decision as to when to post a bridge had the widest variation of any of the answers. Twenty (20) of the responding states reported that they post a bridge when its Legal loads exceed the Operating level rating. Georgia and four other states use the Operating rating as the posting limit for bridges on the state system and the Inventory rating for bridges on the local system.

Some other states have more detailed policies regarding the posting limit, such as: “use Operating rating for bridges having a condition rating larger than 5, otherwise, use Inventory level rating”; “for fracture critical member use Inventory rating, for others use some value in between the Operating and Inventory levels based on engineer’s judgment”; “use Operating rating for concrete members and the average of the two for steel members”; or, “post when the Operating rating is exceeded, or when the Inventory rating is exceeded and posting will have minimal economic effects” One Eastern state specified that all structures need to be considered for posting if the structure’s Inventory

capacity rating is less than 30 tons for HS20 vehicle, 35 tons for 3S2 vehicle or 18 tons for the H20 vehicle, or when the gross tonnage of a “4 Axle” vehicle exceeds the structure’s Operating level capacity.” In another Eastern state, a bridge will not be posted if “the bridge can carry H15 at Inventory level and HS20 and all state Legal loads at the Operating level.” Several states don’t have specific criteria for posting, but will consider it if the structure has a rating factor less than 1.0 at the Inventory level for HS-15 vehicles or if the structure shows signs of major deterioration. There is no consensus among the states as to whether to post a bridge at Operating level or Inventory level ratings. Engineering judgment sometimes is used either to post a bridge whose rating would not normally entail posting, or to not post a bridge that is calculated to require posting.

As to what percentage of the state bridge inventory has been posted, twenty (20) of the responding states reported posting fewer than 4% of their bridges, fourteen (14) reported that between 5 to 19% were posted, and the remaining seven (7) have posted over 20% of their bridges. This survey question was poorly phrased, however, leading some states to report the total percentage of posted bridges while others reported the percentages of state and local bridges separately. The percentage of posted bridges on local roads is typically anywhere from 10 to 100 times the percentage of posted bridges on state roads.

As to whether serviceability or fatigue limit states are considered when setting up the posting limits, twenty four (24) states, including Georgia, do not consider either; sixteen (16) consider serviceability, and the vast majority of states generally do not consider fatigue. Those that consider serviceability do so only for steel or pre-stressed concrete girder bridges.

When to load test a bridge?

Fourteen (14) out of 41 of the responding states, had performed some form of load testing for the purpose of load rating as a part of bridge evaluation practice. Five (5) other states reported that they had once performed very few load tests for the reason of academic research only. The remaining states have never used load testing as a tool for bridge condition assessment; one Eastern state remarked that the reason is “testing is too time consuming and expensive.”

Most of the load tests have been performed on structures that were in good condition but required posting according to standard rating analysis, on special construction such as FRP bridges, on those bridges without available plans or design documentations, or on those with serious deterioration that prevented an accurate theoretical strength calculation. One Western state noted that they performs test on bridges “deemed to be high risk, or fracture-critical.” The benefit of load testing results is best summarized by the response from one Western state:

- 1) To allow bridges to remain in service without traffic restriction
- 2) To avoid unnecessary repairs and needless replacement
- 3) To avoid repairs to bridges scheduled for replacement
- 4) To get more accurate load distribution factors, and
- 5) To compare calculated stresses with actual stresses

One other common use of load testing is in evaluation of overload permits. Two of the states that perform load tests do so extensively to prevent having to perform costly repairs, replacement or posting due to “unreliable *AASHTO* rating factors.”

Only one Southern state among the fourteen states that performed load tests used the provisions in Chapter 8 of the *AASHTO LRFR Guide Manual* (2003) to guide their load testing practices, although there is one other state that “follows NCHRP, Nov 1998-No.234, *Manual for Bridge Rating through Load Testing*, which is consistent with Chapter 8 of the *LRFR Guide Manual*.” One Western state reported that the reason for not using the *LRFR Guide Manual* is that “we are not yet sufficiently comfortable with it.” Two other western states, having performed load tests prior to the issuance of the *Guide Manual*, have also developed their own guidelines and testing procedures, which were reported to be in the process of being compared with the *LRFR Guide Manual*. Some states perform and analyze the load tests themselves, while states that do not have their own guidelines usually leave the testing and interpretation entirely to the Universities to which they contract the work. One Eastern State “uses the load test to determine live load distribution, which is then applied to LFR formula to update load rating factors.” Another Western state has a load testing protocol that involves taking “strain transducer measurements when the structure is under various loads. A model of the bridge is produced based on the strain transducer measurements. This model is then used to predict responses of the bridge to design loads and over-loads.”

Other performance issues - connections, fatigue and scour

Thirty seven (37) of the forty one (41) responding states do not assess the capacity of connections on a regular basis. Connections are routinely inspected in most states; however, they are checked for adequate capacity only if engineers suspect that the connections may govern the load rating of a bridge. For the four exceptions, one Eastern state stated that: “Our policy requires load rating of connections for all primary

components of a bridge unless the district Bridge Engineer concludes that the connections would not control the rating of the member.” A Western state does consider connections, but only those on continuous bridges with a splice at the piers; an Eastern state considers all types of connections, while another Western state examines “all areas of the structure.”

Most states normally do not compute the remaining fatigue life of a bridge unless fatigue cracking is found during inspections, with the typical reason being lack of sufficient truck volume data. Four states are exceptions. One Eastern state performs a 100% hands-on inspection of fatigue sensitive members; however, one can avoid this by calculating the fatigue life of bridges with low traffic counts, and then perform 100% hands-on inspection if the member has a remaining fatigue life of less than 10 years; One Western state “computes remaining fatigue life based on an arms length inspection” and performs such analysis on a 1 to 10 year cycle where the interval is usually 3 years for fatigue prone members as determined by fatigue life; Another Western state performs an in-depth inspection of all fracture critical members regardless of fatigue life, however, when the remaining fatigue life is finite or expired, the frequency of inspections increases. Finally, one Northern state performs fatigue analysis on selected bridges.

All states indicated that they perform some form of scour investigations on a regular basis. Most investigate scour for bridges that cross wade-able waterways during the FHWA-mandated 2 year inspections and all other bridges during a special underwater or scour investigation every 4 to 5 years. Two states report that they perform special scour investigations on any bridges identified as scour-susceptible following floods.

2.3.3 Comparison of Rating Methods through Sample Bridges

The survey of current bridge rating practices of State Departments of Transportation, summarized in Section 2.3.2, revealed considerable differences in current practices and concerns that the ASR, LFR and LRFR methods yielded substantially different ratings. The AASHTO *MBE* allows three rating methods but does not provide guidance as to which method should be used for specific circumstances. It is apparent that such discrepancies would be a barrier in routine bridge rating practices.

To determine the extent to which such discrepancies might exist and to quantify the magnitude of the rating differences that might result from the use of ASR, LFR and LRFR methodologies for typical Georgia bridges, a rating analysis with these three methods was performed for four typical bridges that had been identified for subsequent load testing and advanced analysis. A detailed description of the engineering characteristics of these sample bridges are provided in Chapter 4. Tables 2.4 and 2.5 present a summary of these rating results for flexure and for shear respectively [Wang, et al 2009].

In general, rating results by ASR and LFR are reasonably close in all cases. The LRFR legal load ratings for the HS20 vehicle fall between the Inventory and the Operating level ratings computed by either the LRF or ASR method for both moment and shear for all four bridges. In other words, the LRFR legal level ratings generally are more conservative than the LFR/ASR Operating level ratings and more liberal than the LFR/ASR Inventory level ratings. These results for typical Georgia bridges are consistent with what was found in the Survey,

Table 2.4 Summary of Sample Bridge Flexural Rating for Interior Girders

Bridge Type	Method	Rating Vehicle			
		<u>HS20</u>		<u>H93</u>	
		Inv	Opr	Inv	Opr
Conc. T (Straight)	ASR	0.70	1.25		
	LFR	0.75	1.25		
	LRFR	0.93		0.65	0.84
Conc. T (Skewed)	ASR	1.36	2.17		
	LFR	1.16	1.93		
	LRFR	1.77		1.27	1.65
Prestressed Girder	ASR				
	LFR	1.54	2.57		
	LRFR	1.95		1.34	1.73
Steel Girder	ASR	0.82	1.33		
	LFR	0.71	1.18		
	LRFR	1.08		0.72	0.93

Table 2.5 Summary of Sample Bridge Shear Rating for Interior Girders

Bridge Type	Method	Rating Vehicle			
		<u>HS20</u>		<u>H93</u>	
		Inv	Opr	Inv	Opr
Conc. T (Straight)	ASR	0.41	0.75		
	LFR	0.43	0.72		
	LRFR	0.61		0.45	0.58
Conc. T (Skewed)	ASR	0.94	1.44		
	LFR	0.84	1.40		
	LRFR	1.05		0.83	1.08
Prestressed Girder	ASR				
	LFR	1.43	2.39		
	LRFR	1.47		1.05	1.36

2.4 BRIDGE EVALUATION IN FOREIGN COUNTRIES

2.4.1 Bridge Rating in Canada

The provisions of Section 11 of the *Ontario Highway Bridge Design Code (OHBDC)*, 3rd edition pertain to the evaluation and posting of existing bridges other than soil-steel structures⁴ and pedestrian bridges. Provisions are given for the condition inspection, analytical load rating procedure, load testing and calculation of posting limit for bridges. In contrast to the requirements in the United States, evaluation is not mandated for every highway bridges and is not required on a periodic basis in Canada.

The *OHBDC* is based on the limit state design philosophy and a target reliability index, β , of 3.5 is used for both design and evaluation. There is no explicit reduction of β in evaluation, although a few adjustments can be applied to reduce those load factors used for design when evaluation is performed. Provisions are provided for ultimate, service and fatigue limit states checking, and only the ultimate limit state is specified to be used for determining the load carrying capacity, stability and load posting of bridges; the exceptions are masonry abutments, masonry piers and masonry retaining walls, for which serviceability is the governing limit state. Fatigue checks are performed only if the bridge owner wants to assess the remaining life of the bridge because of the observation of the physical evidence of fatigue-prone details or fatigue related defects. The method of fatigue life assessment is the same as in the *AASHTO LRFR Guide Manual*.

The rating process requires the use of three live load models, designated *OHBEL* levels 1, 2 and 3 respectively, with different gross magnitudes and configurations. These

⁴ Defined as a bridge comprised of bolted structural steel plates and engineered soil, designed and constructed so as to utilize structural interaction between the two materials. The *OHBDC* devotes an entire chapter to this type of bridge, as it does for concrete and steel bridges.

three live load models appear to be similar to the three AASHTO legal loads. The live load factors calibrated for bridge design are adopted in the capacity evaluation for most general cases, with some exceptions: the live load may be reduced by 10% for bridges with inspection intervals less than 5 years; the corresponding lane load equivalent used in evaluation is reduced as a function of *in situ* traffic volume or varies according to the road classes; live load factor may be reduced for multiple lane bridges with a certain level of redundancy, and the dead load factor can be reduced if the nominal dead load is carefully estimated. These reductions are not applied in controlled vehicle rating, which is a procedure that is comparable to the AASHTO permit load checking, and is conducted for specific vehicles for which permission must be granted prior to their using the bridge.

The *OHBDC* presents detailed provisions and curves for establishing posting limits according to the rating calculations performed for the three above-mentioned live load models. The provision regarding posting concrete bridges is similar to that in the *AASHTO LRFR Manual*, that is, a concrete bridge need not to be posted if it has been carrying normal traffic without signs of excessive cracking or deformation.

The *OHBDC* also states that a load test may be proposed as a part of the evaluation procedure when the analytical approach does not accurately reflect the actual behavior of the bridge. However, no detailed definitions and provisions are provided as to different types of load tests, loading patterns, instrumentation or interpretation of test results.

2.4.2 Bridge Rating in the United Kingdom

Document BD 21/01, *Assessment of Highway Bridges and Structures*, adopts a limit state format with appropriate partial safety factors for condition evaluation of most

highway bridges except for cast iron bridges and masonry arch bridges. It is stipulated that bridges built after 1965 should normally be evaluated for serviceability as well as for the ultimate limit states; bridges constructed before 1965 do not need to be assessed for service limit states. Requirements for fatigue endurance however are not included in the standard and the reason stated is that the past stress history of each structure, which could profoundly influence fatigue limit checking, cannot generally be determined to the accuracy level required for assessment.

No reduction in target reliability index from the corresponding values for design is explicitly stated in BD21/01; however, several adjustments are made to the live load model that have the effect of reducing the level of conservatism in the evaluation of existing bridge structures. In the UK, the bridge design live load model consists of a uniform distributed load (UDL) and a knife edge load (KEL) with the intensities of both components decreasing with bridge span lengths. The design load was derived by estimating the worst credible values of relevant loading parameters from available statistics. Adjustments are suggested in the evaluation to scale down this design load model for bridge situations that are less onerous than the above worst case scenario, while maintaining a consistent reliability level for the whole network; detailed scaling curves for the live load adjustment factors are provided in the document. Furthermore, in the absence of definite information about material characteristics in estimating the resistance of bridge component, the document assigns a set of values to materials which should be used in the initial assessment, mostly according to the construction period of the bridge. Structures which cannot sustain the assessment live loading after the checking calculation, and which are not scheduled for immediate replacement or

strengthening should be reevaluated for the other three lower live load models for posting; posting provisions can be found in the document.

Document BA 54/94, *Load testing for bridge assessment*, presents general instructions on load testing practices. The document states that the role of load testing primarily is to seek out the hidden reserves of strength, and the bridges most likely to be involved are those which contain features where such reserves may be found. Load tests are broadly divided into the two categories - Proving load tests and Supplementary load tests - which are analogous to the AASHTO Proof load test and Diagnostic load test, respectively. Because there is a risk of collapse during a proving test, or of damage to essential elements of the structure, such tests therefore are limited in the document only to those bridges which, on the basis of their analytical assessment, would have been closed to traffic or demolished. Bridges that previously have been subjected to proving tests need to be thoroughly inspected and reassessed at more frequent intervals. The document also emphasizes that extreme care has to be taken to extrapolate the results of tests carried out with fairly low levels of loading to those likely to occur at the ultimate limit state.

Instructions provided in BA 54/94 are rather general; detailed guidance on loading patterns and magnitude, testing procedures, and test results interpretations are not provided. Cautionary notes are provided concerning the effectiveness and the accuracy of load testing as a means of load capacity evaluation of existing bridges. Concerns expressed include: whether a static test load can adequately represent the ultimate limit state loading condition; whether a bridge deck should be fully loaded or partially loaded, in view of the fact that the collapse mode of a partially loaded deck may be different from

that when the whole deck is loaded as was intended in the design; and whether the benefit of a test is warranted, considering the risk to personnel.

2.4.3 Bridge Rating in Australia

Section 7 of the *Australian Bridge Design Standard* provides rating guidelines with a commentary. The concept of rating is based on the limit state design philosophy and both serviceability and ultimate limit states are considered. The ultimate action is defined as an action that has a 5% probability of being exceeded during the design life, which represents an average return interval of 2000 years; while the survivability action is defined as one having 5% probability being exceeded per year, corresponding to a return interval of 20 years.

The rating for strength is carried out for all strength limit states, e.g. moment, shear, compression, at all potential critical sections, with the lowest rating factor determined being the rating factor for the bridge. At the service limit state, a structure is checked for vibration and deflection. When a bridge is checked for the fatigue limit state, the cumulative fatigue damage at the critical details of the bridge must be carefully assessed, from which the nominal fatigue life of a bridge can be estimated. For the purpose of rating, the cumulative fatigue damage is defined as the sum of the damage in all previous years; the nominal fatigue life is considered having been reached when the cumulative damage sums to unity⁵, in which case, a program of inspection should be initiated to ensure that fatigue cracks are detected and suitably repaired before they endanger the bridge's ability to carry its applied loads.

⁵ Cumulative damage is assessed using the Palmgren-Miner linear damage accumulation model.

A bridge may be rated at each limit state, to a general rating capacity or to specific loading cases, using the same partial factor checking format as specified in the *Standard*. For the general rating case, which is comparable to the AASHTO design load level rating, the live load models and the corresponding load factors are the same as those used in the design of a new bridge. For specific loading cases, the live load can either be a legal load vehicle or an exceptional load, the former case being comparable to the AASHTO legal load level rating, and the latter being comparable to permit load checking. In all cases, the effects of the rating loads for the specific loading cases are determined using the gross weight and the configurations specific to the vehicles under consideration. Since the possibility of overloading at this step is unlikely, a reduced live load factor is permitted. Where the rating for a bridge is less than required for current general access vehicles, consideration shall be given to applying a posted limit on the bridge. Detailed regulation on establishing the limits for specific vehicles however is not presented in the document.

Two types of nondestructive test are defined in the *Australian Standard*: static proof load test and static performance load test, which are comparable to the two types of AASHTO load tests. The difference between the two types of test is in the magnitude of loading, and in the manner and the level of confidence in which the capacity of the bridge to carry the live load is determined from the test results. The *Standard* identifies the load test as an effective method of evaluating the performance and structural capacity of a bridge or bridge type. The document suggests that the proof test loading should be applied incrementally from a base load of 50% of the theoretical rated ultimate capacity in order to protect the bridge and the testing personnel, and the load response should be

continuously monitored to ensure that the bridge is behaving in an elastic manner. It also suggests that a numerical model of the structure should be developed prior to the test to assess the ultimate capacity, failure mode and the elastic limit under different loading configurations and to determine the maximum load needed for the test. The *Standard* provides some detailed formulas for updating ratings after a successful load test and also emphasizes that the adoption of the load testing results should only to apply to bridges of similar structural form, taking into consideration material properties and conditions.

2.5 CRITICAL APPRAISAL OF CURRENT BRIDGE RATING PRACTICE

Three different existing rating methods are currently utilized by state Departments of Transportation in the United States in their bridge rating work, as revealed in the survey responses. The ASR, LFR and LRFR methods are based on different design philosophies and therefore often produce different rating results and lead to different posting limits for the same structure; sometime the difference can be significant. One weak point of the current practice is that no clear policy is provided in the AASHTO rating guidelines as to which method should be used for specific circumstances. The LRFR method is relatively new; while having the most rational basis of the three, it still need to be tested and validated through research and practice for the bridge engineering community to develop confidence in its use. The large number of Inventory, Operating, and Legal loads clutters the analysis and rating process with many redundant calculations, especially in the ASR and LFR related procedures. Consequently, this situation causes differences in interpretations and practices from different DOTs regarding what triggers posting and whether to use an Inventory or Operating rating to

post and in which circumstances. These issues should be better stipulated in rating guidelines for safe practice and for consistent and unambiguous implementation.

None of the current AASHTO manuals provides clear guidance as to when to revise existing load ratings. Therefore states DOTs normally make their decisions on revising a current rating based on judgment and on what has been observed during the field inspection. Most bi-annual inspections are visual and any insight that might be obtained from such an inspection on existing safety or load-carrying capacity necessarily will be qualitative rather than quantitative in nature. Either a way must be found to better quantify what a visual inspection reveals or a more sophisticated inspection strategy, including informative and non-invasive inspection technologies and optimal inspection intervals, should be encouraged, so that the decisions based on inspection data are well-substantiated.

The survey of the state Departments of Transportation also revealed that most states rely solely on analytical methods to evaluate the load-carrying capacity of existing bridges. Load testing as an effective alternative has been largely ignored. Due to the conservative nature of the analytical rating methods, this inevitably leads to some unnecessary bridge repairs, replacements or postings. The fact that most state DOTs do not perform any kind of load testing likely can be traced to a lack of guidance on load test to address practical issues including: under what circumstances a load test will be a good option for bridge rating, and under what circumstances one should choose a diagnostic vs a proof load test, and further, how to design practical load test procedures. Engineers should be provided general guidance as to whether a load test is worthwhile, considering the cost of a test, as well as specific instructions on field data acquisition and

interpretation. The current load test guidelines in Section 8 of the *AASHTO LRFR Guide Manual* (2003) do not provide engineers with enough details to bridge the gap between the concept and practice of load testing.

In the current bridge evaluation practice in the US, two major parameters used to describe a bridge's present condition are the condition rating (on a scale of 1 to 10) and capacity rating (on a scale of 0 to 1). The condition rating is based on visual inspection data and measures deterioration level of a bridge. The capacity rating, on the other hand, is performed in practice according to inventory data to check whether a given bridge meets the current design standards. One weak point of current practice is that the bridge's capacity rating is computed based on its design documentation and is not properly coupled with the bridge's deterioration state. Bridges are inspected periodically and the condition rating assigned by inspector on the basis of visual inspection identifies whether deterioration is occurring and, ideally, at what level. These inspection data clearly should be considered in computing the capacity rating and real time reliability of the bridge.

2.6 CLOSURE

In general, modern bridge rating procedures worldwide have adopted reliability principles as their basis. They have utilized the limit state philosophy to allow the safety checking to be performed in a deterministic manner without an explicit structural reliability assessment. The reliability indices for design are typically 3.5 or higher over the lifetime of the bridge. However, they permit lower reliability indices in the context of specific evaluations of individual existing bridges, either by explicitly reducing the target safety index in the calibration leading to the load factors for evaluation, which are lower

than those used in bridge design, or by directly scaling down the live loads used in the assessment to reflect the lesser requirements for evaluation compared to the design level.

The ultimate limit states are typically required as the governing limit states for safety checking for majority of the bridge types; serviceability and fatigue are not regularly mandated unless signs of distress or fatigue related defects are observed. Rating procedures and the assessment live load models vary the most from country to country, but for the most part, a check on design load is typically performed prior to the capacity estimation respect to actual vehicles; the latter, in general, is the basis for posting. The view towards load testing is different from country to country, which leads to different treatment of the provisions on this subject in different guidelines. Test protocols and details that are critical for a load test to be successful and informative may not be addressed. The AASHTO MBE (2008) has the most comprehensive provisions on load testing of the condition assessment guidelines reviewed.

CHAPTER 3

IMPROVED FRAMEWORK FOR BRIDGE RATING PRACTICE – A PROTOCOL WITH THREE LEVELS OF ASSESSMENT

The LRFR option in the *AASHTO Manual for Bridge Evaluation* extends the limit state design philosophy to the bridge evaluation process in an attempt to achieve a uniform target level of safety for existing highway bridge systems. However, the current capacity rating formulation in the *MBE* only supports capacity evaluation at one level of sophistication (considering individual members, with system effects addressed indirectly through a “girder distribution factor”) and treats existing bridges as “generic” structures when, in fact, they have individualized features that contribute to capacity. The uncertainty models of load and resistance embedded in the LRFR rating format represent typical values for a large population of bridges involving different materials, construction practices and site-specific traffic conditions. Although the LRFR live load model has been modified for some specific cases (average daily traffic, redundancy), the bridge resistance model can be better “customized” for an individual bridge by incorporating available site-specific knowledge to reflect the fact that each bridge is unique in its as-built condition. A rating procedure which does not incorporate *in situ* data properly may result in inaccurate ratings (and consequent unnecessary rehabilitation or posting costs) for otherwise well-maintained bridges, as indicated by many load tests (Nowak and Tharmabala, 1988; Bakht and Jaeger, 1990; Moses, et al, 1994; Fu and Tang, 1995; Faber, et al 2000; Barker, 2001; Bhattacharya, et al, 2005). Advancing the current bridge evaluation practice requires better understanding of bridge system behavior, better

utilization of available *in situ* data as well as better modeling of the live load process and other time-dependent factors such as fatigue, corrosion and concrete aging. This research develops an improved bridge evaluation framework that directly addresses the deficiencies in current practice noted in Chapter 2.

The improved practical rating framework illustrated in Figure 3.1 has three levels of assessment of increasing complexity. In the first level, the deterministic format of the AASHTO LRFR method is retained, but the correlation between visually determined condition rating and the capacity evaluation is established, so that the bridge deterioration can be taken into account quantitatively in the load rating process. If a bridge does not pass the first level checking, the second level assessment is optional. At this level, site-specific data about individual structural properties obtained from material tests, diagnostic load test and from in-depth structural analysis can be incorporated in rating calculations. Information needed for this second level assessment usually doesn't require significant cost investment and the checking format can still be presented in a deterministic manner. Finally, if a bridge exhibits unsatisfactory capacity ratings after the first two levels of assessment and if a further comprehensive evaluation is believed to be warranted, the third level of assessment rates the bridge at system level by proof load testing or by using information about its past performance. The proof load test can either be conducted on the real structure *in situ*, or it can be done "virtually" through a finite element analysis that is sufficient to describe the load-carrying mechanisms affecting the bridge's capacity and its load ratings. This third level of assessment involves a structural system reliability analysis which inevitably requires some level of probabilistic modeling and therefore is not given in a deterministic format.

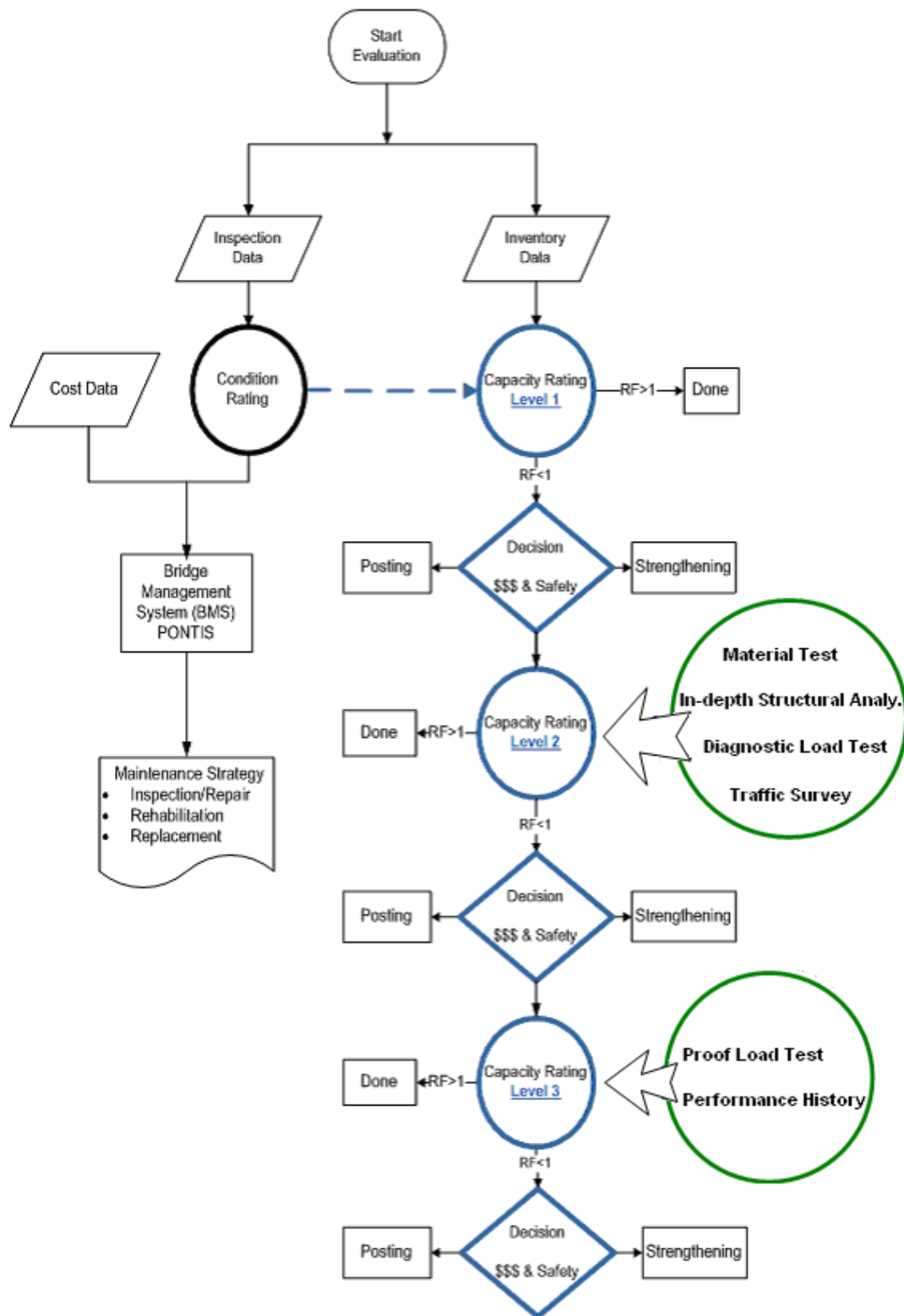


Figure 3.1 Improved Bridge Rating Framework

At the first level of the proposed rating framework, bridge condition rating as a measure of bridge's deterioration status is factored into bridge capacity rating metric. Statistical data on bridge condition rating history taken from the National Bridge Inventory (NBI) is analyzed, and quantitative models of material degradation and bridge degradation developed in other research studies are incorporated, leading to a quantitative correlation between the condition rating history of a large bridge population and the bridge degradation modeled stochastically. Using statistics presented by this correlation the bridge condition rating is incorporated in bridge capacity rating process through a simple condition factor with values that are consistent with the reliability requirement embedded in AASHTO LRFR.

The second level of assessment is designed to consider the uniqueness of each existing bridge and to incorporate its *in situ* information at the component level in the bridge rating framework. For example, material strengths *in situ* may be vastly different from the standardized or nominal values assumed in design and current rating practices due to strength gain of concrete on one hand and deterioration due to aggressive attack from physical or chemical mechanisms on the other. *In situ* load distributions in the bridge structure observed from diagnostic load test or a simplified FE analysis are often significantly different from the girder distribution factors specified in the code. Occasionally, certain bridge components might have a load carrying mechanism that can not be appropriately modeled by traditional code provisions. Proper consideration of these factors is likely to contribute to a more realistic capacity rating of existing bridges and practical guidance for doing so is required. The second level of rating analysis clearly highlights the learning process of a given bridge through field inspections and

provides incentives to obtain quantitative *in situ* measurements through modern non-destructive evaluation (NDE) techniques, such as Carpenter hammer sounding, Schmidt rebound hammer and ultrasonic pulse velocity.

In the third-level evaluation, a bridge is evaluated at its system level in situations where the first two level of analysis produce low ratings, or analysis is difficult to perform due to deterioration or lack of documentation. Bridge system reliability can be estimated by incorporating results from a properly conducted proof load test or information regarding performance collected in past routine periodic bridge inspections. The feasibility of using finite element modeling, validated through either systematic field inspection supported by NDE technologies or through diagnostic load tests, to conduct “virtual” proof load tests of bridge systems, is investigated in this research. An examination of the role and limitations of proof load testing is performed using FE models that have been validated by diagnostic load tests on several bridges believed to be representative of those bridges of the most concern of GDOT. Additionally, satisfactory bridge performance history provides information comparable to what is learned from a proof load test could also be favorable to the bridge capacity evaluation. This information is currently neglected in the load rating process and will be incorporated in the third-level assessment of the proposed evaluation framework.

CHAPTER 4

SELECTION, TESTING AND FINITE ELEMENT MODELING OF THE SAMPLE BRIDGES

In order to accomplish the study objectives stated in Chapters 1, four bridges that are representative of older bridges typically of concern with regard to rating or posting were identified from an examination of the Georgia bridge inventory. These four bridges subsequently were used in an integrated program of analysis and load testing to support the recommended improvements to the bridge rating process in the Chapter 3 framework.

4.1 SELECTED SAMPLE BRIDGES FOR TESTING AND ANALYSIS

The Georgia Bridge Management System database was examined carefully to determine general characteristics of bridges that would be candidates for posting. Of the approximately 2,000 Georgia bridges that require posting, 77% fall into one of three categories:

- Reinforced concrete T-beam bridges, representing 21%;
- Steel girder bridges, representing 53%; and
- Pre-stressed concrete I-girder bridges, representing 3%.

Figure 4.1 shows the primary structural types of bridges constructed over each decade from the 1940's to the present. Figure 4.2 identifies the number of bridges from each category that have been posted as unfit for some or all of the state legal load vehicles. Of those posted pre-stressed bridges, 57% were constructed after 1980; in contrast, only 2%

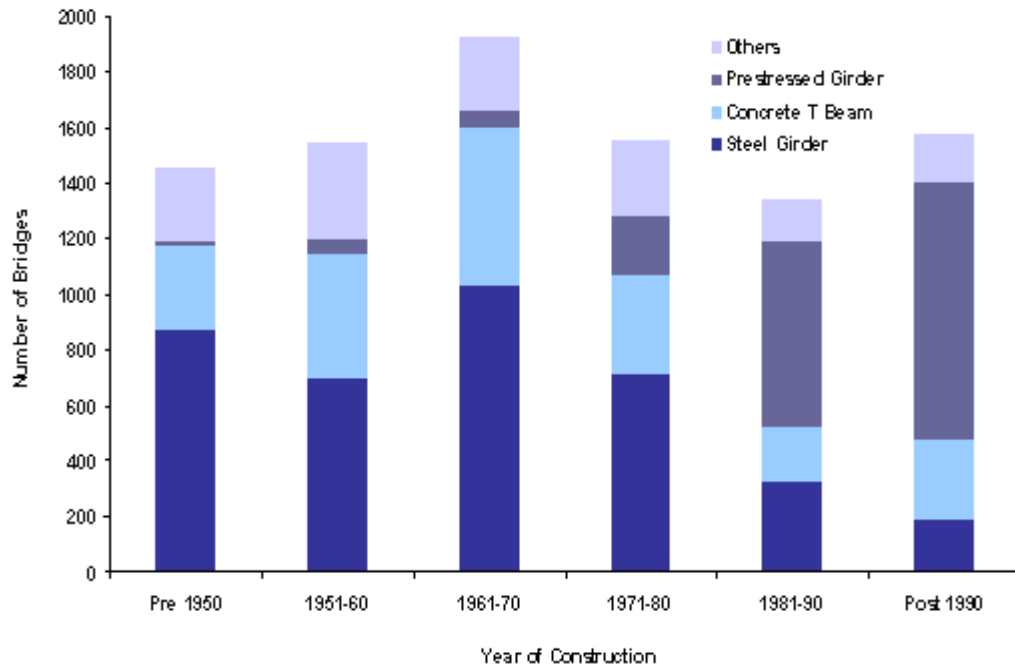


Figure 4.1 Bridge Categories Identified by Decade of Construction

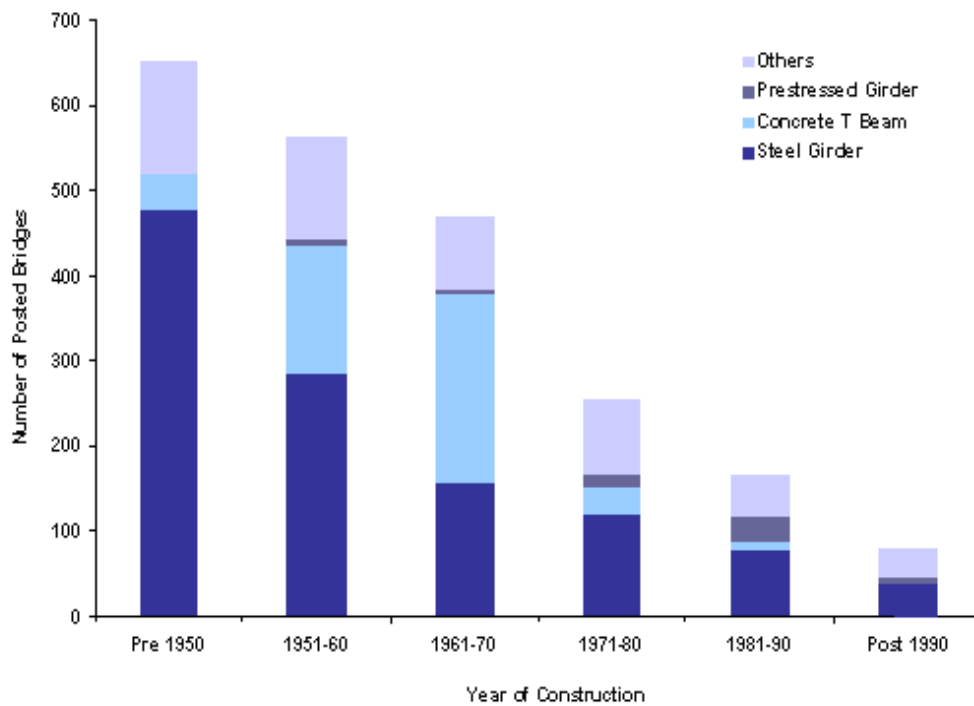


Figure 4.2 Posted Bridges Identified by Decade of Construction

of the posted reinforced concrete bridges and 10% of the posted steel girder bridges were constructed after 1980. Seventy-six percent of the bridges that are candidates for strengthening/posting in Georgia were constructed in the two decades following 1945, were designed for H-15 loading and had simple spans, with lengths of 40 to 70 ft.

Following a review of the existing documentation on these bridges, site visits aimed at determining their suitability and testing feasibility, and discussions with State bridge maintenance engineering staff, four bridges were finally identified for diagnostic load testing, further in-depth finite element analysis and reliability assessment [O'Malley, et al, 2009].

- Reinforced concrete bridge – straight (Bridge ID: 129-0045)
- Reinforced concrete bridge – skewed (Bridge ID: 015-0108)
- Pre-stressed concrete girder bridge (Bridge ID: 223-0034), and
- Steel girder bridge (Bridge ID: 085-0018)

These bridges and their structural characteristics are summarized in the following paragraphs.

4.1.1 Reinforced Concrete Bridge – Straight (ID: 129-0045)

This bridge carries SR 156 over Oothkalooga Creek, was designed using the *AASHTO 1953 specification* for H-15 loading, and was built in 1957. It is located one mile west of Calhoun, GA in Gordon County. SR 156 is a two-lane road. The bridge has eight spans, seven of which are 40 ft (12.19 m) and one (over the channel) 45 ft (13.7 m). The girders are 18 ½ in x 24 ¾ in (46.99 cm x 62.87 cm), except for the long span which is 31 ¾ in, and are spaced 7.2 ft (2.19 m) apart. The bridge has a deck width of 32.3 ft (9.85 m) and a road way width of 25.7 ft (7.83 m). The bridge carries an ADTT of 458.

The concrete deck has a condition rating of 5, the supporting reinforced concrete T-beam superstructure is rated at 7, and the concrete bent and pier substructure are rated at 6. The latest inspection report indicates that all caps have minor hairline cracking, and that several areas of exposed cap reinforcement are present. All beams are reported to show signs of typical flexural cracking. The entire deck has moderate surface deterioration, scaling, and cracking. It has also been repaired in several notably bad sections. The bridge has not been posted, but was scheduled for replacement in 2008¹.



Figure 4.3 Straight T-Beam Bridge (ID: 129-0045, Gordon County)

4.1.2 Reinforced Concrete Bridge – Skewed (ID: 015-0108)

This 12-span structure over a long flood plain and a creek carries Old Alabama Rd. over Pumpkinvine Creek 3.7 miles south of Cartersville, GA in Bartow County. The two-lane bridge structure has a skew of 30 degrees and an ADTT of 709, was designed using the *AASHTO 1977 specifications* for HS-20 loading and dates to 1979. The eleven

¹ This bridge was demolished later on. Concrete samples were obtained; in situ concrete strength was measured. The in situ strength then is used to update the bridge rating in the second level evaluation presented in Chapter 6

spans over the flood plain are carried by 40-ft (12.19 m) reinforced concrete T-beams. The 70-ft (21.34 m) span over the channel is supported by AASHTO type II pre-stressed concrete girders. The current bridge ratings for substructure, superstructure, and deck are 6, 6, and 7 respectively, and the bridge is posted for three truck loads: H (21 tons), Tandem (19 tons), and Log (24 tons). There is minor cracking and spalling in a number of the bents and abutments, as well as in the T-beams, but none is in need of immediate repair.



Figure 4.4 Skew T-Beam Bridge (ID: 015-0108, Bartow County)

4.1.3 Pre-stressed Concrete Girder Bridge (ID: 223-0034)

This bridge carries State Route 120 over Little Pumpkinvine Creek approximately 5 miles south of Dallas in Paulding County GA. It was designed using the AASHTO 1989 for HS-20 loading specifications and was constructed in 1992. The main structural system consists of pre-stressed concrete I-Beams arranged in four simply supported spans. The bridge is 216 ft (65.8 m) long and is comprised of two 40-ft (12.2-m) Type II pre-stressed I-girder spans and two 68-ft (20.7-m) Type III prestressed I-girder spans. The

centerline of the bridge is essentially perpendicular to the girder supports. The bridge has a deck width of 43¼ ft (13.2 m) and a roadway width of 40 ft (12.2 m). The 68-ft (20.7-m) spans are comprised of five type III I girders that are composite with the 9⅞ in (232 mm) thick slab (Figure 4.5). The bridge is in good condition, with substructure, superstructure and deck condition numbers of 7, 8 and 7, respectively. It is not posted. The ADT is 6550.



Figure 4.5 Pre-Stress Bridges (ID: 223-0034, Paulding County)

4.1.4 Steel Girder Bridge (ID: 085-0018)

This bridge carries SR 136 over the Etowah River 5.7 miles east of Dawsonville, Georgia, in Dawson County. It was designed using the AASHTO 1961 specification, with interim revisions through 1963 for H-15 loading, and was constructed in 1965. The bridge is 196 ft (59.7 m) long and its four 49 ft (12.2 m) spans are supported by four steel girders spaced at 8 ft on centers; the two fascia girders are W33x118, while the two interior girders are W33x130, with a full-depth diaphragm located at mid-span. The

two-lane bridge has a (non-composite) concrete deck, with overall width of 32 ft (9.75 m) and a roadway width of 26 ft (7.92 m). The centerline of the bridge is perpendicular to the girder supports. The bridge was last inspected on June 30, 2005, and at that time the deck and substructure both were assigned a condition assessment rating of 6. That inspection report indicates that there is spalling, aggregate exposure, and transverse cracking in the deck in all spans. The bridge was determined to require posting, and has been posted for a 21-ton H load, 25-ton HS load, 23-ton Tandem load, 32-ton 3-S-2 load, and 27-ton Log load. The piles have minor pitting and the beams have minor deflections. The bridge carries an ADTT of 280.



Figure 4.6 Steel Girder Bridge (ID: 085-0018, Dawson County)

4.2 DEVELOPMENT OF FINITE ELEMENT MODEL OF SELECTED BRIDGES

Three-dimensional (3D) nonlinear finite element models (FEMs) of the superstructures of each of the four bridges were developed from design and construction documents obtained from the Georgia Department of Transportation. The purpose of these FE models was threefold. First, they provided a basis for comparison with the simplified analytical evaluation procedure currently used by GDOT for bridge rating and for identifying issues that might not be apparent with the existing component-based deterministic rating format presented in bridge evaluation manuals. Improving bridge rating guidelines requires an understanding of bridge system behavior subjected to extreme load events which may well exceed the load level applied in the diagnostic tests. Second, they were used to assist in designing the diagnostic load tests of the four bridges. Finally, once validated through the diagnostic load tests (described subsequently), they were used to conduct “virtual” proof load tests of other bridges of interest in the Georgia Bridge Inventory. These ‘virtual’ load tests, along with the system reliability analysis described in Chapter 7, are an essential ingredient of the technical support of the improved rating protocol introduced in Chapter 3.

All FE models of the sample bridges were developed using the ABAQUS commercial FE package [Simulia, 2006]. Prior to the conduct of the load tests, these FE analyses were performed using anticipated vehicle weights and arrangements to assist in designing test instrumentation, to identify test vehicle locations, and to anticipate and guard against potential bridge vulnerabilities that might become apparent during the diagnostic load tests. Following the load tests, FE analyses again were performed using the actual test vehicle weights and wheel locations measured during the tests, and

predicted responses were compared with test measurements to determine the accuracy with which FEA can predict bridge behavior.

In the FE analyses of all four bridges, the failure mechanism in the concrete was assumed to be either cracking in tension or crushing in compression. In the absence of information on the *in situ* strengths of steel and concrete, the strengths specified on the construction documents were assumed in bridge modeling and assessment. Of course, this assumption introduces uncertainty in the comparison of model predicted structural behavior and test measurements, as described subsequently. The stress-strain curve proposed by Todeschini [1964] was utilized to model concrete behavior under compression; in tension, the stiffness and strength reductions caused by cracking were taken into account by the smeared crack technique [Kupfer, 1973; Hillerborg, 1976], in which crack initiation is based on strength criteria and crack propagation is based on fracture mechanics-based energy criteria. Deformed bar reinforcement and prestressing strands both were assumed to have uniaxial elastic-plastic stress-strain behavior. All four sample bridges were simply supported by the pier caps or abutments; these supports were modeled by pin-roller boundary conditions.

For the reinforced and prestressed concrete bridges, the concrete deck, girders and transverse diaphragms were modeled using 3D continuum solid elements. Steel reinforcement was modeled using a distributed approach, in which the reinforcing bars were smeared into membrane layers and embedded in the concrete at appropriate locations. The pre-stressing strands in the girders were modeled individually using truss elements embedded in the solid concrete elements. The pre-stress in strands was replicated by applying an initial stress condition to the truss elements so that when the

bridge reaches self-equilibrium under such condition, the strands have the effective pre-stress indicated in the design documents. The compatibilities between rebars/pre-stressing strands and concrete were enforced. Shear reinforcement was ignored in the FE model of the bridge superstructure. For the steel girder bridge (Bridge ID 085-0018), each girder is modeled using beam elements for the flanges and shell elements for the web.

The FE model of the reinforced concrete T-beam span (bridge ID 129-0045) as shown in Figure 4.7 had 420,928 degrees of freedom. The FE modeling of other bridges was at a similar level of resolution. The research character of this investigation dictated this level of resolution; such a level would not be required for routine bridge condition assessment by analysis. More details of the FEMs can be found in O'Malley, et al [2009].

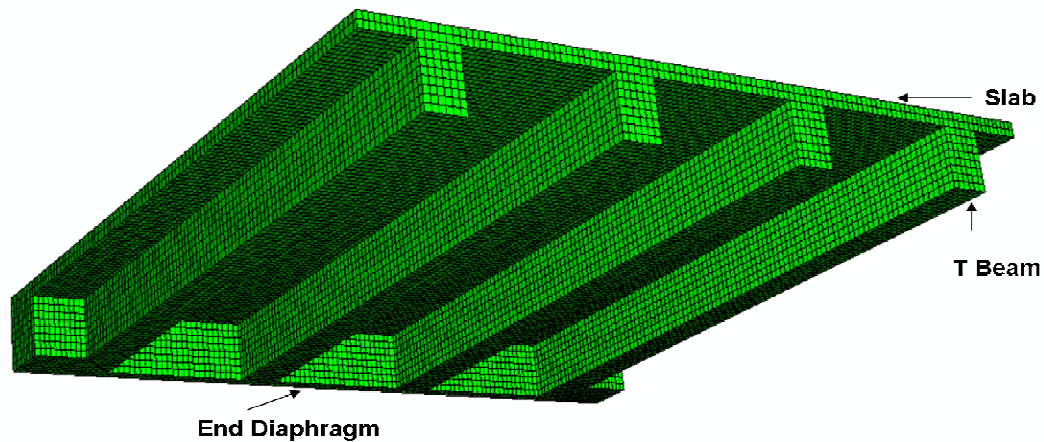


Figure 4.7: FEM of the Gordon County Bridge (ID: 129-0045)

4.3 VALIDATION OF FINITE ELEMENT MODELS OF THE SELECTED BRIDGES THROUGH DIAGNOSTIC LOAD TESTS

The load tests were performed by GDOT employees, following the test protocols instituted by the principal investigators and under their supervision. Each bridge was tested using up to four DOT trucks; the truck wheel loads were measured prior to each day's testing. Details of the testing program, including the bridge selection, instrumentation, testing process and the post-test assessment of the measurements, can be found elsewhere [O'Malley, et al, 2009]. A summary of the results of the analysis and load test conducted on the RC T-beam Bridge with straight approach (ID 129-0045) is presented in Figures 4.8 and 4.9. Figure 4.8 shows the test arrangement of the trucks on the bridge. The truck weight is summarized in Table 4.1. Figure 4.9 compares displacements at midspan predicted by the FE analysis and the displacements measured by potentiometers at the same locations.

When the RC T-beam Bridge with straight approach (ID 129-0045) was fully loaded by four DOT trucks, totaling 223 kips (1,068 kN) as in Table 4.1, the bending moment at mid-span of the bridge was 2.25 times the bending moment under the H-15 design load configuration. The maximum measured deflection of the beams at the mid-span under such loading was 0.28 inches (7 mm). The span/800 deflection limit for concrete T-beam bridge stipulated in AASHTO Standard Specifications [1992] (section 8.9.3.1) is 0.6 inches (15 mm). Clearly, the *1953 AASHTO specification* that was in effect at the time this bridge was designed incorporated a significant margin of safety. Notwithstanding its age and design load, there is no evidence from this assessment that this bridge was structurally deficient when evaluated according to modern bridge design and rating criteria. Similar observations were made for the other bridges analyzed in this

study. The implications of these observations will be examined in detail later in Chapter 7.

The comparisons between predicted and observed maximum girder deflections under load from four trucks for all four bridges tested are summarized in Table 4.2. The results indicate good agreement was achieved for all four bridges. The discrepancies were invariably within 20% and, in the majority of cases, were substantially less. Such differences can be attributed to various uncertainties associated with experimental data collection under field conditions and the many assumptions made in the FE analyses, including magnitude and homogeneity of *in situ* material properties and idealized boundary conditions. In view of these factors, results of the FE analyses of the four test bridges are considered sufficient to describe and quantify the load-carrying mechanisms that affect the bridge capacity and its load ratings.

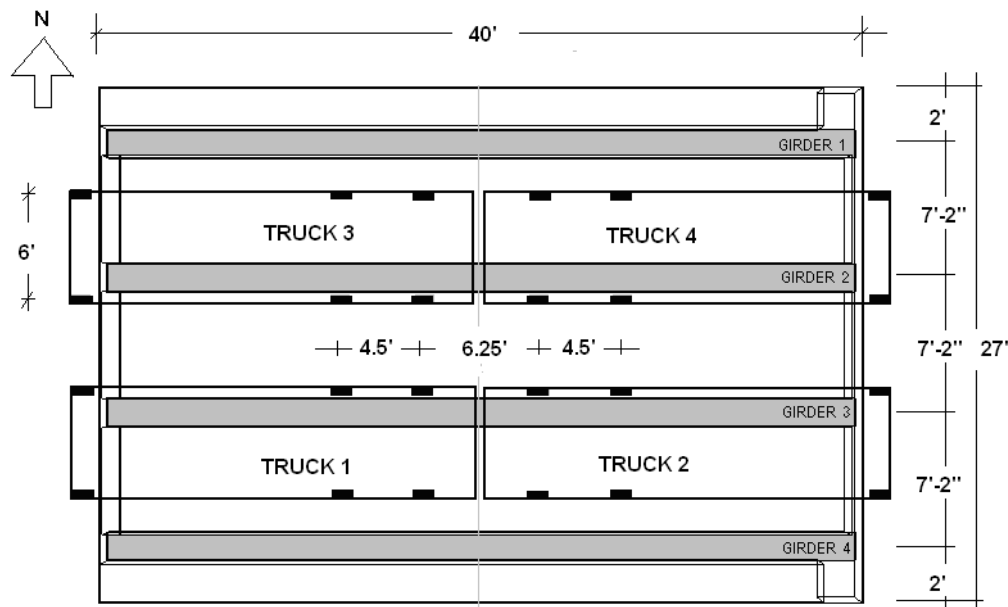


Figure 4.8: Schematic of Concrete Reinforced T-Beam Bridge

Table 4.1 Truck Weight (lb) Details for RC T-Beam Bridge (ID: 129-0045) Test

	Load on axle 1	Load on axle 2	Load on axle 3	Overall truck weight
Truck 1	18,400	19,100	19,000	56,500
Truck 2	19,100	17,400	17,100	53,600
Truck 3	19,500	19,300	19,000	57,800
Truck 4	17,800	18,700	18,600	55,100

Table 4.2 Comparison of the Maximum Deflections Measured in the Test and Predicted by FE Analysis

Bridge	ID	Measurement	FEM
RC bridge – straight approach	129-0045	0.28	0.26
RC bridge – skewed approach	015-0108	0.14	0.16
PC bridge	223-0034	0.20	0.22
Steel girder bridge	085-0018	0.36	0.43

4.4 CLOSURE

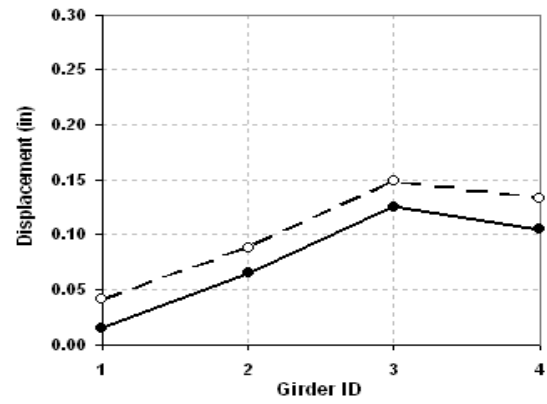
This Chapter has summarized the load testing and analysis phase of the study to examine current bridge rating procedures and to improve them using reliability –based

methods. Four bridges that are typical of bridges of concern in rating and posting were selected for load testing and analysis. Finite element models of these bridges were developed to assist the design of the load tests and in the interpretation of the results. The bridge test results, in turn, were used to validate and improve the finite element modeling. The measured deflections in all cases were in good agreement with those predicted by the FE model. All four bridges remained well within the elastic range when loaded to an intensity that is well above their design load. The maximum deflections measured during the load tests were on the order of 25% - 50% of the span/800 limit on deflection stipulated in the AASHTO design specifications. Experience with these load tests suggests that basing performance assessment of an existing bridge on global response measurements, such as displacement, as opposed to local responses, such as strain, minimizes the likelihood of errors in test interpretation and misjudgments of safety that may be prompted by spurious local non-homogeneous or material behavior. It was also observed that redundancy in measurements, through multiple gauges at a single location and gauges at multiple locations in a single element, is essential to achieve accurate conclusions from the condition assessment and should be utilized whenever practical.

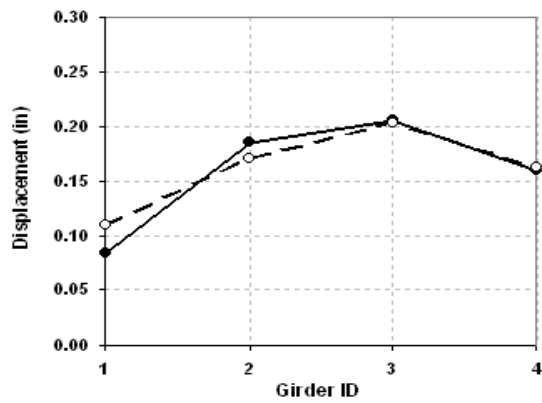
In Chapters 6 and 7, this combined analytical and experimental approach will be used to develop an improved bridge rating framework in which *in situ* material testing, condition data, and history of successful service life performance are properly integrated through system reliability analysis into an improved capacity rating metric.



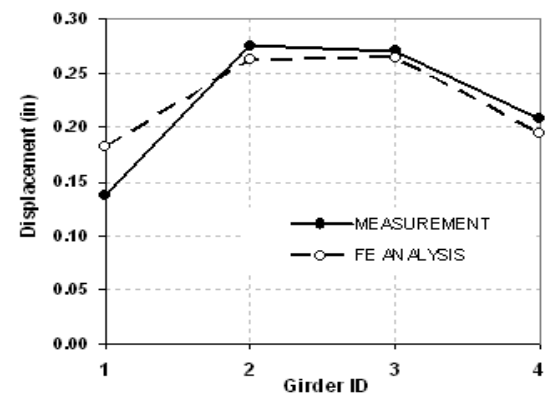
(a) Loaded with Truck 1



(b) Loaded with Truck 1 & 2



(c) Loaded with Truck 1, 2 & 3



(d) Loaded with Truck 1, 2, 3 & 4

Figure 4.9: RC T-Beam Bridge Girder Displacements Due to Truck Loadings

CHAPTER 5

LEVEL-ONE ASSESSMENT: CORRELATION BETWEEN CONDITION RATING AND CAPACITY RATING

5.1 INTRODUCTION

A bridge subjected to environmental attack and excessive traffic loads may experience changes in resistance, and its capacity rating should reflect these changes. Currently, in the *AASHTO Manual for Bridge Evaluation*, the physical condition of a bridge is reflected in the capacity rating equation (eq. 6-1; eq 6A.4.2.1-1 in the *MBE*) through a condition factor ϕ_c , where condition ratings from 1-9 that are assigned during routine inspections according to National Bridge Inspection Standard (1996) (as summarized in Table 5.1) are categorized into three qualitative descriptions of bridge condition – good, fair and poor - and these qualitative descriptions are then connected to the resistance calculation in the capacity rating through the condition factor, ϕ_c , as summarized in Table 5.2 (*AASHTO LRFR* Table 6-2).

The current categorization of bridge condition above is purely empirical and the development of the ϕ_c values is not theoretically supported by structural reliability analysis. This chapter introduces a new method to correlate bridge condition ratings with capacity ratings, taking into account both the underlying physics of bridge deterioration phenomena and bridge conditional rating history data from routine inspections. A revised set of values of ϕ_c that are tied to the rating equations have been developed to be consistent with the structural reliability-based evaluation philosophy embodied in the *AASHTO MBE* (LRFR option) and to incorporate recent developments in bridge

resistance degradation modeling and comprehensive databases of bridge condition rating history.

Four steps are taken. First, a state-of-the-art bridge degradation model that describes the physical process of bridge deterioration in a stochastic fashion is selected. Second, the average bridge condition rating history model that illustrates the condition rating as function of bridge service age is established. Third, the bridge condition rating history is linked to the statistical models of bridge resistance by mapping the condition rating history model in step two onto the bridge degradation model in step one. Finally, a reliability-based optimization technique is used to identify a set of ϕ_c values that express this correlation in a deterministic manner in the rating formula to satisfy the reliability requirement embodied in *AASHTO LRFR*. These steps are illustrated in the following sections, in which condition factors are developed for reinforced concrete bridges. The methodology introduced can be applied to other types of bridges as well.

5.2 STOCHASTIC BRIDGE DETERIORATION PROCESS

Quantitative models of deterioration of reinforced concrete structures have been developed in many research studies [Albrecht and Naeemi, 1984; Mori and Ellingwood, 1993; McCuen and Albrecht, 1995; Thoft-Christensen, 1998; Enright and Frangopol, 1998]. These models can be incorporated in a real-time bridge reliability assessment. The uncertainties in resistance of an existing bridge are at least equal to those of a newly designed bridge. Once the bridge begins to deteriorate, its mean resistance usually decreases and the uncertainty in resistance generally increases. Time-dependent structural resistance can be modeled as [Mori and Ellingwood, 1993]:

$$R(t) = R_0 g(t) \quad (5-1)$$

in which t is elapsed time, R_0 is the resistance variable of a newly-constructed bridge and $g(t)$ is the degradation rate. The mean and COV of random variable $g(t)$ can be expressed for many common deterioration mechanisms [Enright and Frangopol, 1998], in first approximation, as linear functions of time, as in eq (5-2):

$$\begin{aligned} E[g(t)] &= \begin{cases} 1, & t \leq T_0 \\ 1 - k_1(t - T_0), & t \geq T_0 \end{cases} \\ V[g(t)] &= k_2 t \end{aligned} \quad (5-2)$$

in which k_1 and k_2 are constants and T_0 is the mean time required to initiate corrosion. Bridges are exposed to many environmental stressors. The extent and the rate of strength loss, $g(t)$, depends on the aggressiveness of the environment and the properties of construction materials. Chemicals, moisture, and cycle of extreme temperature are the most common environmental factors that influence the strength of the structure.

For concrete bridges subjected to environmental attack, the strength degradation mechanism can be classified as affecting either the concrete or the steel reinforcement, or both. Concrete deteriorates because of internal pressures which are caused primarily by chemical reactions in the cement (sulfate attack), by chemical reactions between the cement and the aggregates (alkali-silica reaction), or by freeze-thaw cycles. Reinforcing steel deteriorates primarily because of corrosion. In addition, the corrosion products cause internal pressure that can lead to cracking and spalling of the concrete (Val and Melchers, 1997). Enright and Frangopol [2000] found that most damage of RC bridges is caused by water leakage through transverse joints in the deck and the corrosion is most frequent damage mode. Corrosion of reinforced concrete is a two-stage process consisting initiation (carbonation penetration or chloride ion ingress) and propagation (metal loss).

Table 5.1 NBIS Instruction for Superstructure Condition Rating

Condition Rating	Description
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION – No problems noted
7	GOOD CONDITION – Some minor problems
6	SATISFACTORY CONDITION – Structural elements show some minor deterioration
5	FAIR CONDITION – All primary structural elements are sound but may have minor Section loss, cracking, spalling or scour
4	POOR CONDITION – Advanced section loss, deterioration, spalling or scour
3	SERIOUS CONDITION – Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete.
2	CRITICAL CONDITION – Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	IMMINENT FAILURE CONDITION – Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affection structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION – out of service. Beyond repair

Table 5.2 Condition Factor, ϕ_c (AASHTO LRFR, 2005, Table 6-2)

Structural Condition of Member (Condition Rating)	ϕ_c
Good or satisfactory (≥ 6)	1.00
Fair (= 5)	0.95
Poor (≤ 4)	0.85

Corrosion initiates after an initiation time (T_0), at which time the steel reinforcement becomes depassivated due to carbonation or chloride ion ingress (Novokshchenov 1989; Whiting et al. 1993) and the chloride content in the concrete at the depth of the steel reinforcement reaches a critical concentration. The corrosion initiation time can be expressed as [Thoft-Christensen et al, 1977]

$$T_0 = \frac{X^2}{4D_c} [erf^{-1}(\frac{C_0 - C_{cr}}{C_0})]^{-2} \quad (5-3)$$

where X = concrete cover (cm); D_c = chloride diffusion coefficient (cm^2/year); C_0 = equilibrium chloride concentration at the concrete surface (in percent of weight of concrete); and C_{cr} = critical chloride concentration at which corrosion begins (in percent of weight of concrete). The corrosion initiation time is dependent on four random variables(X , D_c , C_0 , C_{cr}). The main descriptors of these random variables can vary considerably for different bridges.

Once corrosion has initiated, the cross-sectional area of reinforcement decreases with time at a rate that is dependent on the number of reinforcement bars actively corroding and the diameter of the individual bars. For the general case where the steel reinforcement is composed of bars of various diameters, which begin corrosion at different times, the time-variant area of steel, $A(t)$, is [Mori and Ellingwood, 1994; Thoft-Christensen et al. 1997; Enright and Frangopol 1998a]:

$$A_t = \frac{\pi}{4} \sum_{j=1}^n [D_j(t)]^2 \quad (5-4)$$

Where

$$D_j(t) = \begin{cases} D_{jo} & \text{for } t \leq T_{lj} \\ D_{jo} - r_{corr} (t - T_{lj}) & \text{for } T_{lj} \leq t \leq T_{lj} + D_{jo} / r_{corr} \\ 0 & \text{for } t \geq T_{lj} + D_{jo} / r_{corr} \end{cases} \quad (5-5)$$

and $D_j(t)$ = diameter of bar j at time t ; n = number of bars; D_{j0} = initial diameter of bar j ; r_{corr} = corrosion rate; t = elapsed time, and T_{ij} = corrosion initiation time for bar j . For rectangular nonprestressed members in which the strength of compression steel is neglected, the nominal flexure resistance M_n of a concrete beam is given by [LRFD 1994]:

$$M_n = A_t f_y \left(d - \frac{1}{2} \frac{A_s f_y}{0.85 f'_c b} \right) \quad (5-6)$$

where f_y = specified yield strength of reinforcing bars; d = distance from the extreme compression fiber to the centroid of the non-prestressed tensile reinforcement; f'_c = specified compressive strength of concrete at 28 days; and b = width of the compression face of the member.

To investigate the corrosion initiation time and strength degradation function for typical RC bridges and to define the coefficients in eq 5-2, Enright and Frangopol [2000] performed Monte Carlo simulations using eqs 5-3 through 5-6 and the statistics of key initial resistance and corrosion random variables listed in Table 5.3. The mean and coefficient of variation of the resistance random variables are based on the information presented in MacGregor et al [1983]. Values for the corrosion random variables are based on probabilistic corrosion studies summarized in Enright [1998]. Those studies found that the corrosion initiation time T_0 is lognormally distributed and is increasing with the depth of the concrete cover. The mean of the degradation function, $E[g(t)]$, decreased with time while its cov $V[g(t)]$ increased, as shown in Figure 5.1. For reinforced concrete bridges subjected to environmental attack with medium rate of corrosion, the mean value of T_0 is approximately 10 years and coefficients k_1 and k_2 in eq (5-2) equal to 0.0031 and 0.0027, respectively; substituting these parameters into eqs 5-2 yields the time-dependent

relations presented in Figure 5.2. In other words, the mean of the resistance $R(t)$ will reduce to 80% of its original value and the COV of $g(t)$ alone will increase to 13%, after a 75 year period of exposure. These statistics must be factored into the time-dependent reliability analysis of the reinforced concrete bridge.

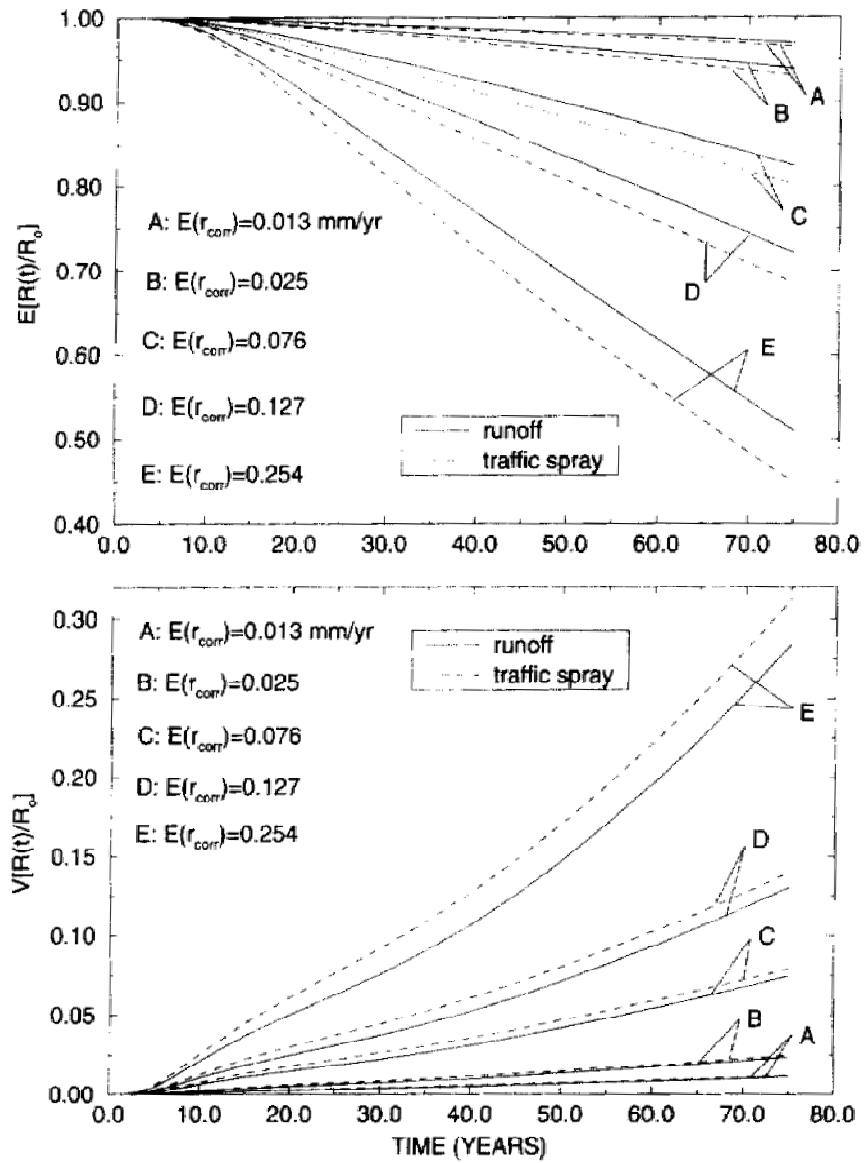
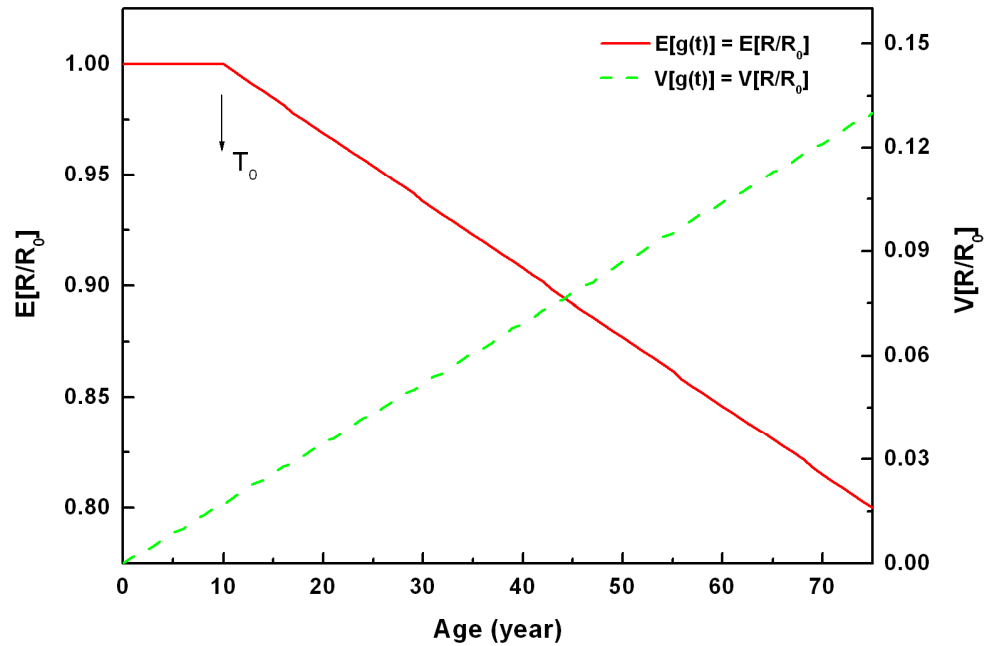


Figure 5.1 Mean and COV of $g(t)$ of Time-variant Bending Resistance with Different Corrosion Rate r_{corr} (Enright and Frangopol, 2000)

**Table 5.3 Random Variable for MC Simulation
(Enright and Frangopol, 2000)**

Variable (units)	Description	Mean	Coefficient of variation
f_y (Mpa)	Steel reinforcement yield strength	310.5	0.12
f'_c (Mpa)	Concrete compression strength	19.0	0.18
D_M (mm)	Initial diameter of flexure reinforcement	35.8	0.02
d_M (mm)	Initial depth of flexure reinforcement	68.73	0.03
X_{M1} (mm)	Cove depth of flexure steel, layer 1	5.08	0.05
X_{M2} (mm)	Cove depth of flexure steel, layer 2	12.70	0.05
D_c (cm ² /yr)	Diffusion coefficient	1.29	0.10
C_o (wt % conc.)	Surface chloride concentration	0.20	0.10
C_{cr} (wt % conc.)	Critical chloride concentration	0.025	0.10
r_{corr} (mm/yr)	Corrosion Rate	0.15	0.30



**Figure 5.2 Time-dependent Mean and Coefficient of Variation of
Bridge Degradation Function $g(t)$**

5.3 SYNTHESIS OF THE CONDITIONAL RATING DATA FROM NBI

Bridge condition ratings (see Table 5.1) for individual components are assigned following inspections which usually are conducted once every two years. If such condition ratings for a bridge are available for a relatively long period of time, the data can be used to establish a deterioration model for that bridge. National Bridge Inspection Standard (NBIS) coding guide identifies the specific bridge elements that must be inspected and provides instructions on how to conduct an inspection. Although state transportation agencies are not required to make their bridge inspection programs identical to that describe in the coding guide, they are required to have databases that can easily be converted to NBIS format for reporting to FHWA so that data from various states can be combined to form the National Bridge Inventory (NBI).

In a review of the NBI database, Bolukbasi, et al (2004) found that the average condition rating history of non-interstate RC bridges often can be modeled by a 3rd order polynomial, as shown in Figure 5.3:

$$C(T) = 8.662 - 0.146T + 0.003T^2 - 3.09E5T^3 \quad (5-7)$$

where $C(T)$ is the condition rating of the bridge at age, T , in years. This model yields a condition number 4 in 70 years. Jiang and Sinha (1989) developed a similar polynomial model with slightly different coefficients, which indicated 71 years to a condition state 4. Weyers et al (1989) computed an average condition rate deterioration rate which indicates 65 years to condition state 4 and 78 years to condition state 5. These models are consistent with one another, and the model raised in the study by Bolukbasi, et al (2004) is adopted in this study. The condition rating assigned during routine inspections varies from inspector to inspector, and thus is a random variable that might affect time-

dependent reliability analysis. A study on the accuracy of inspection documentation [Phares et al, 2004] has revealed that the distribution of condition ratings is normal.

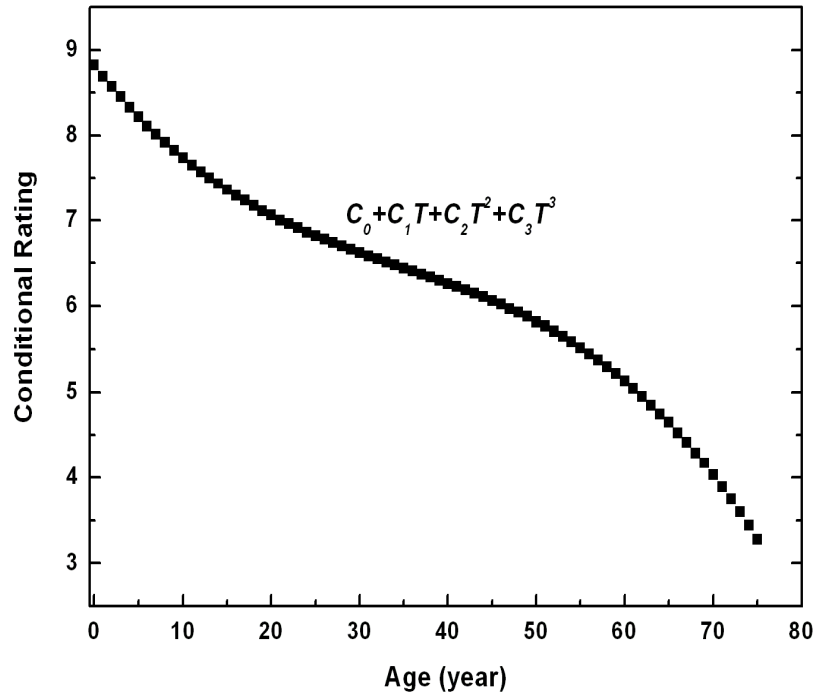


Figure 5.3 Average Condition Rating as Function of Time for Non-interstate Highway Bridges (Bolukbasi et al, 2004)

5.4 CORRELATION BETWEEN CONDITION RATING AND RESISTANCE MODEL

The correlation between condition rating $C(T)$ and the statistical descriptors of degradation $g(t)$ is developed by mapping the average condition rating history of non-interstate concrete bridges, as shown in Figure 5.3, onto the 75-year stochastic resistance degradation model with medium degradation rate in Figure 5.2, leading to the rating-dependent mean and COV of function $g(t)$ shown in Figure 5.4 by the solid lines. When

flexural resistance is considered, R_o is described by a lognormal distribution with mean of 1.14 times the nominal flexural strength R_n and COV of 13%, respectively (Nowak, 1999). The dashed lines in Figure 5.4 show the mean, $E[R/R_n]$, and COV, $V[R/R_n]$, of the resistance modal as a function of condition rating. By mapping the resistance with an average degradation rate on the mean condition rating history, the proposed statistical descriptions of resistance as a function of condition rating are finally independent of corrosion rate.

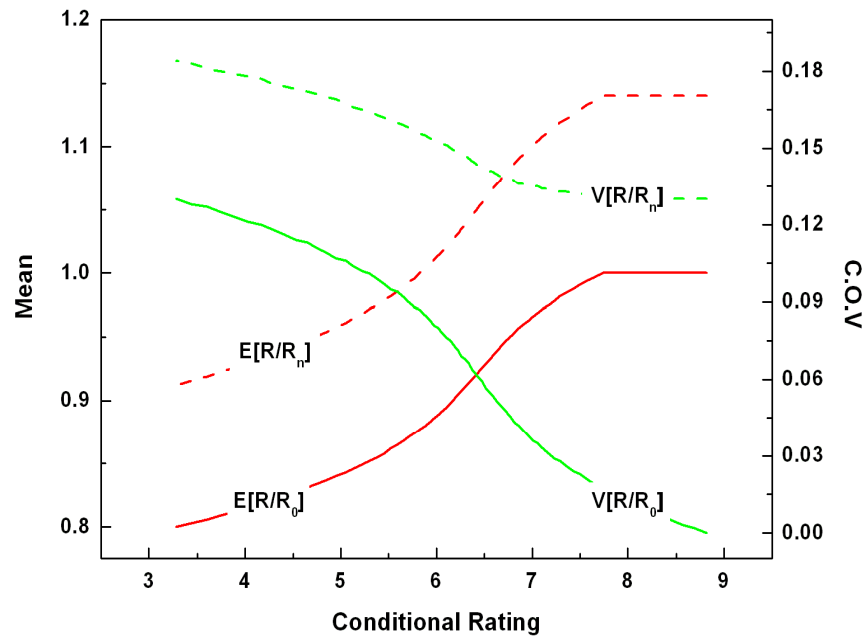


Figure 5.4 Time-dependent Mean and COV of Bridge Flexural Capacity

5.5 CALIBRATION FOR THE CONDITION FACTOR, ϕ_c

Using the statistics in Figure 5.4 along with the load models used in the AASHTO LRFD (Nowak, 1999), the bridge condition rating values can be included in the

estimation of time-dependent failure probability and reliability index of a given bridge. To further facilitate the bridge rating practices which utilize a deterministic format, a set of ϕ_c -values necessary to achieve the target reliability requirements consistent with the AASHTO LRFR method was obtained by minimizing the mean-square error between the target β_T and the reliability achieved by the use of specific values of ϕ_c , as illustrated in Figure 5.5. The difference in target reliability (β_T), 3.5 at the inventory level vs 2.5 at the operating level, is reflected in difference in the live load factor used for these levels, therefore does not affect the calibration of ϕ_c . The optimal condition factors determined from this analysis are presented in Table 5.4.

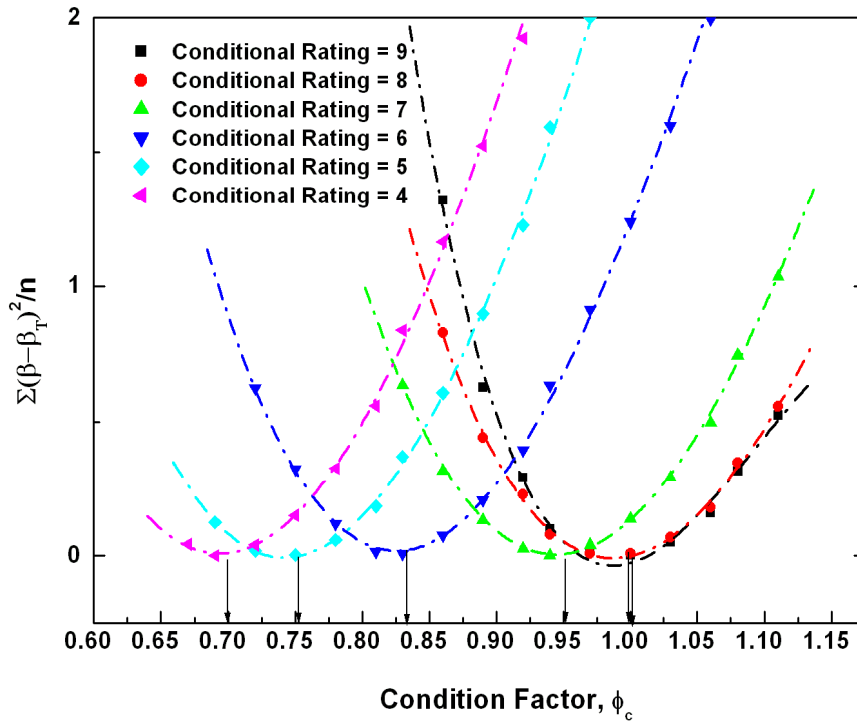


Figure 5.5 Optimal Condition Factors for Different Condition Ratings

5.6 CLOSURE

This chapter illustrated a methodology for incorporating qualitative measures of bridge condition into a quantitative reliability-based evaluation of bridge load rating. Correlation between condition rating and capacity rating was established through a set of condition factors with sound reliability basis. The development of ϕ_c was illustrated for reinforced concrete bridges, but the methodology could easily be applied to other types of bridges if supporting time-dependent deterioration statistics for those bridges similar to those presented in Figure 5.3 become available. The AASHTO LRFR deterministic rating format (eq 2-6), together with the revised values of condition factor ϕ_c developed in this chapter, should be incorporated in the first-level assessment of the proposed rating framework.

Table 5.4 Proposed Condition Factors

Structural Condition Rating (SI&A Item 59)	ϕ_c
≥ 8	1.00
7	0.95
6	0.85
5	0.75
≤ 4	0.70

CHAPTER 6

LEVEL-TWO ASSESSMENT: BRIDGE RATING USING COMPONENT-LEVEL INFORMATION

The level-one assessment introduced in Chapter 5 utilizing the deterministic format in AASHTO LRFR incorporates only limited quantitative *in situ* knowledge and therefore could produce overly conservative ratings for many well-maintained older bridges [Nowak and Tharmabala, 1988; Bakht and Jaeger, 1990; Moses, et al, 1994; Fu and Tang, 1995; Faber, et al 2000; Barker, 2001; Bhattacharya, et al, 2005]. The models of uncertainty in load and resistance (discussed in section 2.2.3) used in calibrating the LRFR rating equation (eq. 2.6) represent typical values for a large bridge population involving different materials, construction practices and site-specific environmental and traffic conditions. Although AASHTO has modified the live load model in LRFR rating to account for differences in site-specific average daily truck traffic, the resistance model also should be “customized” for an individual bridge by incorporating available site-specific knowledge to reflect the fact that each bridge is unique in its as-built condition.

The second level assessment in this Chapter provides bridge engineers with an option to include additional site-specific information in the bridge rating process to achieve a better evaluation of bridge capacity, when such effort is believed to be warranted. Knowledge gained from diagnostic load tests, from validated finite element analysis or from *in-situ* material tests can be used to revise the LRFR estimates (Chapter 5). An investment in the level-two assessment often can be justified, particularly for bridges that carry heavy traffic or have level-one rating factors in the range of 0.7-1.0.

6.1 RATING USING IN SITU MATERIAL STRENGTH

Section 4.1.1 summarized the load test of Bridge ID 129-0045, a reinforced concrete T-beam bridge that was designed according to the *AASHTO 1953 Design Specification* for H-15 loading, and was constructed in 1957. The specified 28-day compression strength of the concrete was 2,500 psi (17.2 MPa), while the yield strength of the reinforcement was 40 ksi (276 MPa). This bridge was load-tested in September, 2006. Subsequent to the conduct of the load test, the bridge was demolished in May, 2008, providing an opportunity to secure drilled cores to determine the statistical properties of the *in situ* strength of the 51-year old concrete in the bridge.

Four-inch diameter drilled cores were taken from the slab of the bridge prior to its demolition. Seven (7) cores were taken from the slab at seven different locations along both the length and width of the bridge. Cores also were taken from three of the girders which were in good condition after demolition; these were cut into 8-in (203 mm) lengths and the jagged ends were smoothed and capped, resulting in a total of fourteen (14) girder test cylinders. Tests of these 4 x 8 in (102 x 203 mm) cylinders conformed to *ASTM Standard C42*, and the results are presented in Table 6.1. An analysis of these data indicated no statistically significant difference in the concrete compression strength in the girders and slab, and the data were therefore combined for further analysis. The mean (average) compression strength of the concrete is 4,820 psi (33 MPa) and the coefficient of variation is 12%, which is representative of good-quality concrete [Bartlett and MacGregor, 1996]. The mean strength is 1.93 times the specified compression strength of the concrete. This increase in compression strength over a period of more than 50

years is typical of the increases found for good-quality concrete of this vintage by other investigators [Washa and Wendt, 1975].

If these results are typical of well-maintained older concrete bridges, the *in situ* concrete strength is likely to be substantially higher than the 28-day strength that is customarily specified for bridge design and also is used in condition evaluation. Accordingly, the rating criteria should provide the Bridge Engineer with incentives to use the best possible information from *in situ* material strength testing whenever feasible when performing a bridge rating [Ellingwood et al 2009]. It is customary to base the specified compression strength of concrete on the 10-percent exclusion limit of a normal distribution of cylinder strengths [ACI Standard 318-05]. Using Bridge ID 129-0045 as an example, the 75% lower confidence interval on the 10-percent exclusion limit of compression strength, f_c , for the sample of 21 tests can be expressed as,

$$f_c = (1 - kV)\bar{X} \quad (6-1)$$

in which \bar{X} = sample mean, V = sample coefficient of variation, and k equals 1.520 (Ellingwood, et al, 2009) to obtain the 75% lower confidence interval on the 10th percentile value of the distribution, based on 21 samples. Substituting the statistics in Table 6.1, one would obtain $f_c = (1 - 1.520 \times 0.12) \times 4,820 = 3,941$ psi (27.17 MPa), a value that is 58% higher than the 2,500 psi (17.2 MPa) that otherwise would be used in the rating calculations.

In the FE modeling of this bridge that preceded these strength tests, the concrete compression strength was set at 2,500 psi (17.2 MPa), which was the only information available before the material test. In order to determine the impact of using the actual concrete strength on the rating process, the finite element model was revised to account

for the increased concrete compression strength (and the corresponding increase in stiffness) into the analysis of the bridge. Only a modest enhancement in the estimated bridge capacity in flexure was obtained, but a 34% increase was achieved in the shear capacity ratings for the girders using the results of Table 6.1.

Table 6.1 Compression Tests of Cores from RC Bridge (ID: 129-0045)

Source	Number	Average (psi)	Standard deviation (psi)	Coefficient of variation
Girder	14	4,880	603	0.12
Slab	7	4,698	573	0.12
Overall	21	4,820	586	0.12

6.2 RATING USING REALISTIC LOAD DISTRIBUTIONS

Girder distribution factors (GDFs) are used to distribute the traffic loads to the individual girders so that bridge design and evaluation can be performed on an individual member rather than a system basis. The GDFs are an important ingredient of bridge capacity evaluation. The GDFs are different in the AASHTO ASD and LF rating methods from those in the LRFR rating method; these differences are one reason for the complaints received in the survey (discussed in section 2.3) regarding the inconsistency in the bridge ratings obtained from these three methods. These differences were examined using finite element models developed for the four typical Georgia bridges

(summarized Chapter 4). Bridge responses captured by FE analysis (validated by the load tests, as discussed in section 4.3) were used to assess the *in situ* GDFs .

The GDFs for girder moment and girder shear are different. A comparison of the moment distribution factors from the different methods for the four tested bridges is tabulated in Table 6.2. The GDFs obtained from the FE models are substantially less than those calculated using any of the existing methods; accordingly, the load ratings calculated with the GDFs obtained by the FE models would substantially exceed the load ratings that are obtained by the existing methods in the *AASHTO MBE*. For the straight-approach reinforced concrete girder bridge, for example (first line in Table 6.2), the FE analysis of this bridge indicates that when load is placed on the bridge to maximize the moment in one of the interior girders, only about 41% of the applied load actually went into that interior girder. The girder distribution factor in the current LFR method however would require up to 60% of live load to be apportioned to that girder, while with LRFR, the percentage would be 69%. While the LFR procedure results in a rating factor of 0.75 for the HS-20 design load checking at Inventory level, the rating of this bridge using load distribution factor obtained from FE analysis for the same vehicle is 1.10, representing an increase of 47% [Wang et al, 2009; Ellingwood, et al, 2009]. Similar results were observed for the other three bridges considered in this dissertation.

6.3 RATING OF DEEP REINFORCED CONCRETE BRIDGE COMPONENTS

Out of the 2000 Georgia bridges that require posting according to the *AASHTO MBE*, more than 800 are governed by deficient shear ratings of their reinforced concrete pier caps. The posting of the steel girder bridge illustrated in Figure 4.6 (Bridge ID 085-0018) is based on the assessed shear capacity of its center pier cap. Accordingly, prior to

conducting the load tests, a series of independent FE analyses of the reinforced concrete pier cap for this bridge was undertaken. This pier cap behaves as a “deep beam,” in that its shear span is relatively short (its shear span/effective depth ratio, a/d , is approximately 1.0). In contrast, the shear capacity equations in the traditional bridge rating procedure (similar to those in *ACI Standard 318*, 2005,) are known to be valid for beams in which a/d is greater than approximately 3, but may underestimate the actual shear capacity of deep reinforced concrete beams, in some cases significantly (Hawkins et al, 2005).

Table 6.2 Comparison of the Moment Distribution Factors for Interior Girders

Bridge Type	LFR/ASR	LRFR	FEM
Concrete T	0.597	0.69	0.407
Concrete Skew T	0.757	0.73	0.482
Pre-stressed	0.818	0.85	0.521
Steel Girder	0.725	0.72	0.513

The Strut-and-Tie (S&T) model has been proposed recently as an alternative method for evaluating the shear capacity of reinforced concrete beams with short shear spans [e.g., Tang and Tan, 2004]. To determine whether an S&T analysis might enhance the rating of Bridge ID 085-0018, an independent rating analysis was performed. First, the current GDOT rating of the bridge and its current posted limits was confirmed by an

independent analysis. Next, the capacity of the pier cap in shear was assessed using both S&T and FE analyses. In the FE model, the shear capacity was assumed to be reached when yielding initiates in the steel reinforcement acting as the tie of the S&T mechanism over the support (see Figure 6.1) or the concrete compression strut crushes. The stress contours in Figure 6.1, obtained from the FE analysis of the pier cap, clearly show the development of arch action, which the S & T model captures. Table 6.3 compares the posting limits for this bridge for five GA/State Legal loads (cf Figure 2.3). The S & T shear strength model leads to ratings that range from 24% (HS-20) to 30% (Tandem) over the current method. In turn, the S & T shear strength estimates are less than the FE results, suggesting that the current posting limits for this bridge, which are based on traditional shear capacity calculations, are unduly conservative. Had the S & T model been used to determine the shear strength of the pier cap, the posted limits would have increased to 34 tons for HS20, 33 tons for Tandem and 44 tons for 3S2, as tabulated in Table 6.3.

To determine whether or not this conservatism is unique to this particular bridge, an analysis was performed of pile caps at two additional bridges identified through the Georgia DOT database – Bridge ID 083-0016 (built in 1966 for H-15 load; ADTT: 130) and ID 097-0032 (built in 1962 for HS-15; ADTT: 120) – where a/d is smaller than 1.5 and the shear capacity of the pile cap also governs posting. Table 6.4, developed using an HS-20 rating vehicle, reveals that the current rating procedure appears to result in excessively conservative posted loads for these two bridges as well. The ratings are particularly conservative when the point on the pier cap that supports the girder is close to the supporting piles, leading to a short shear span. Accordingly, the strut-and-tie

model appears to be more appropriate for assessing the shear capacity of existing bridge sub-structures for rating purposes.

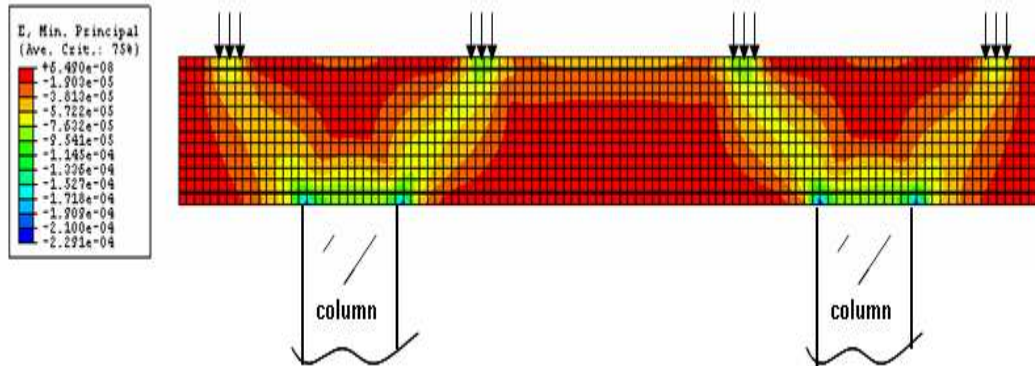


Figure 6.1 Development of Arch Action in Deep Beam

Table 6.3 Shear Ratings (tons) for RC Bridge (ID 085-0018)

Method	H20	HS20	Tandem	3S2	Log
GDOT Method¹	22	25	24	32	28
Strut and tie	30	34	33	44	38
Finite element	40	45	43	58	49

¹ The posted loads reported in this line of the table are taken from the GDOT database.

Table 6.4 Shear Ratings (tons) for HS20 Vehicle

Method	Bridge ID		
	085-0018	083-0016	097-0032
GDOT Method¹⁸	25	24	23
Strut and Tie	34	28	32
Finite Element	45	34	40

6.4 CLOSURE

This chapter explored the possibility for improving bridge load ratings by factoring site-specific knowledge (directly or indirectly obtained from in situ testing or inspection) into the bridge rating process. Three examples illustrated in the above sections indicate a significant gain in load ratings in level-two assessment. This level of analysis provides an incentive to obtain quantitative *in situ* measurements through modern non-destructive evaluation (NDE) techniques.

Material strengths *in situ* may be vastly different from the standardized or nominal values assumed in design. *In situ* material sample tests can be used to update load rating calculations. Concrete compression strength gain in a typical straight T-beam bridge leads to a 34% increase in the shear capacity rating of that bridge. For some other bridges, however, material strength might decrease due to aggressive attack from physical or chemical mechanisms. Material tests in these situations can help the bridge engineer obtain more realistic evaluations and reduce the likelihood of an unsafe rating. Decisions as to whether or not to conduct *in situ* material tests should be based on reliable

bridge inspection data, bridge economics and engineering judgment. Test procedure should be consistent with ASTM standards, and *in situ* sampling and interpolation of test results should have a reliability basis [Ellingwood et al, 2009].

The load tests and supporting analysis herein indicated that analytical approaches to bridge evaluation utilizing current girder distribution factors yield a conservative measure of actual load-carrying capacity. This conservatism is the result of assumptions made in the analysis regarding load sharing, composite action, support conditions and secondary member behavior. In view of the economic consequences of posting, it is apparent that if the customary rating practice suggests that a bridge is a candidate for posting, a more accurate structural analysis model should be employed to verify whether more accurate GDFs might change that decision.

The load-carrying mechanism in pier caps and other structural components that have short shear spans and behave as deep beams is better modeled by the strut and tie method than by the traditional ACI Standard 318 model. The use of this method is permitted by the LRFR option in the AASHTO *MBE*, and bridge engineers should be urged to adopt it. The rated capacity of the pier cap increases by up to 59% in the typical steel girder bridge examined when the S&T model is used. Investigations of similar bridge pier caps indicated that the level of conservatism is dependent on the dimensions of individual pier caps and the placement of the girders that they support.

CHAPTER 7

LEVEL THREE ASSESSMENTS: BRIDGE SYSTEM RELIABILITY

As part of the effort to develop the *AASHTO LRFD Bridge Design Specifications*, extensive databases were developed to describe the strength of individual bridge girders and vehicle live loads probabilistically [Nowak, 1999; Moses, 2001]. As noted previously, that research focused on the capacity of individual bridge girders; system effects were included only indirectly and approximately through new girder distribution factors that were developed in the course of the project. This approach was also adopted by AASHTO in the bridge rating methods found in the *Manual of Bridge Evaluation* that currently is in use. While component-based design of a new bridge provides adequate safety at reasonable cost, component-based evaluation of an existing bridge for rating purposes may be overly conservative and result in unnecessary repair or posting. In particular, it is preferable to perform load capacity ratings through a system-level analysis if decisions regarding bridge posting or road closure are at issue.

The level-three assessment introduced in this Chapter investigates bridge safety at the system level and presents an additional perspective on the (unknown) level of conservatism furnished by the current generation of reliability-based condition evaluation and rating procedures. This level of analysis could also provide an understanding of bridge system behavior when subjected to extreme load events which may have implications for the use of such methods in permit ratings for extreme vehicle loads.

In this Chapter, bridge ratings are examined through a bridge system reliability analysis, and the possibilities of incorporating proof load tests and successful service

performance history into the rating analysis are explored. In order to determine the additional level of conservatism (if any) that arises from system behavior, finite element-based system reliability analyses of all four study bridges were conducted. These analyses allow the characteristic values of the variables used in the condition evaluation to be changed on the basis of the outcomes of a proof load test or service-proving load.

7.1 VIRTUAL PROOF LOAD TESTS OF SAMPLE BRIDGES

A properly conducted proof load test can be an effective way to evaluate the structural performance of a bridge as a system and to update the bridge load capacity assessment in situations where the analytical approach at the first and second levels produces low ratings, or structural analysis is difficult to perform due to deterioration or lack of documentation (Saraf and Nowak 1998). However, a proof load test represents a significant investment in terms of capital, time, and personnel, and the tradeoff between the information gain and the risk of damaging the bridge during the test must be considered. Therefore, proof tests are rarely conducted by the State DOTs for rating purposes (Wang, 2009).

One of the key conclusions from the bridge modeling and tests in Chapter 4, in which bridge response measurements obtained from the load tests of the four bridges were compared with the results of finite element analyses of those bridges using ABAQUS [Simulia, 2006], was that finite element modeling is sufficiently accurate that it can be used to conduct “virtual” load tests of the majority of similar bridges. These virtual load tests can provide the basis for developing recommendations for bridge ratings using structural reliability principles. As noted in the introductory section above, such recommendations require the bridge to be modeled as a structural system in order to

properly identify the performance limit states on which such recommendations are to be based.

7.1.1 Push-down Analysis

To identify such performance limit states and to gain a realistic appraisal of the conservatism inherent in current bridge design and condition rating procedures, a series of static “pushdown” analyses of the four bridges identified in Chapter 4 was performed to determine their actual structural behavior when loaded well beyond their design limit. In a pushdown analysis, two rating vehicles are placed side by side on the bridge in a position that maximizes the response quantity of interest in the evaluation (maximum moment, shear, deflection, etc.). The loads are then scaled upward statically and the performance of the bridge system is monitored. The dead weight of the bridge structure is included in the analysis. The response initially is elastic. As the static load increases, however, elements of the bridge structure begin to yield, crack or buckle, and the generalized load-deflection behavior becomes nonlinear. If the bridge structure is redundant and the structural element behaviors are ductile, substantial load redistribution may occur. At some point, however, a small increment in static load leads to a large increment in displacement. At that point, the bridge has reached its practical load-carrying limit, and is at a state of incipient collapse.

The static pushdown analyses initially were performed using the FE platform ABAQUS, with random material properties determined by their respective mean values to obtain a “best estimate” of bridge capacity. In all four cases, two HS-20 vehicles were placed side by side on two adjacent lanes at a point so as to maximize the elastic moment in the girders. These vehicle loads were scaled upward by a load factor until the point

was reached at which the load-deflection plot indicated the onset on nonlinearity in the bridge structural system. The dead load was held constant throughout the pushdown analyses. An illustration of the static pushdown analysis of the reinforced concrete T-beam Bridge ID 129-0045 is presented in Figure 7.1. The point of initial flexural yielding occurs at a moment corresponding to approximately 4.31 times the two HS-20 design loads, at a deflection of approximately 1.4 in (36 mm), which is equal to approximately 1/345 times the span. The ultimate live load capacity of this bridge, defined as the point at which a small increase in load causes a large increment of displacement or the FE solution failed to converge, is approximately 4.8 times the applied HS20 loads. It is interesting to note from Figure 7.1 that this 51-yr old bridge shows a considerable degree of ductility in behavior. The level of load imposed during the diagnostic load test by the four fully loaded trucks is also shown in Figure 7.1; the diagnostic test load was approximately 1.3 times the two side-by-side HS20 loads (in terms of maximum moment). Clearly, the capacity of this bridge is substantially in excess of what a girder-based calculation would indicate.

Similar pushdown analyses performed on the other three bridges yielded the results summarized in Table 7.1. The elastic ranges of all four bridges are in excess of four times the design load level, indicating the level of conservatism associated with traditional design and rating procedures. The ultimate capacity, as shown in the 5th column of the table, is related to the number of the beams. The Skewed T-beam Bridge and the Prestressed Concrete Bridge both have five beams and clearly have more reserve capacity than the other two bridges which have four beams each.

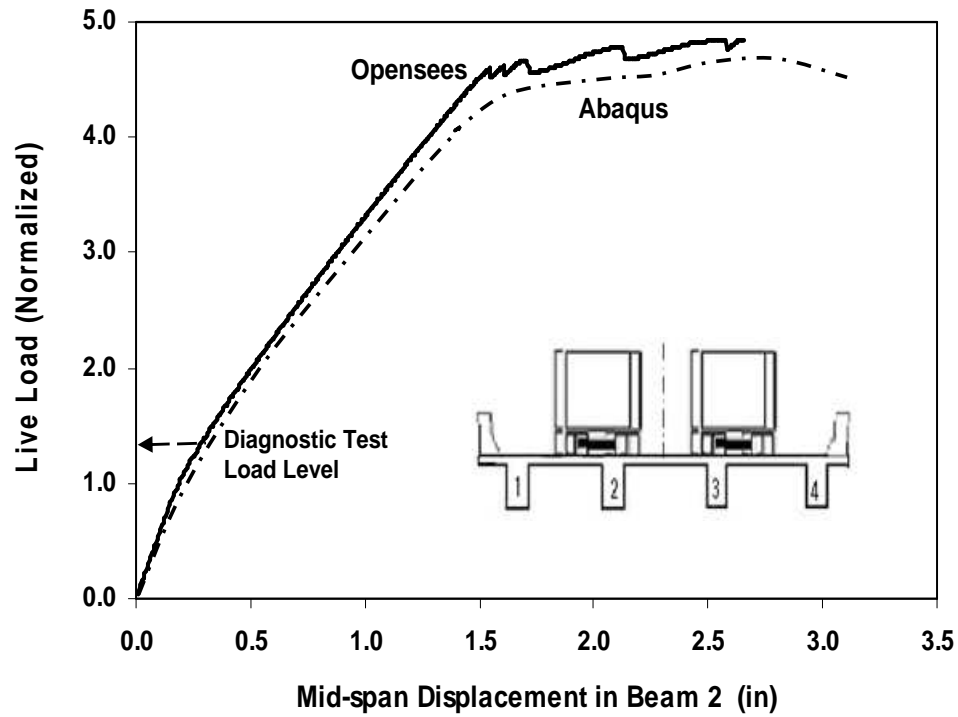


Figure 7.1 Push-down Analysis of RC Bridge (ID 129-0045)

7.1.2 Finite Element Analysis Based Bridge System Reliability

To accelerate the FE-based reliability analysis, FE models of the four sample bridges were developed using the open-source platform, OpenSees (McKenna, et al, 2007). The more detailed ABAQUS models, which had been validated in the load tests, were employed to confirm the bridge structural behavior predicted by the OpenSees models as the system was loaded beyond its design limit. Using the RC T-beam Bridge again as an example, Figure 7.1 illustrates the consistency achieved between the ABAQUS and the OpenSees models through a complete *push-down* analysis, in which the bridge is loaded well into the inelastic range. Following this validation, the system performance of the sample bridges was characterized statistically by propagating the

uncertainties in material strengths, stiffnesses and geometry through the OpenSees FE models using a Latin Hypercube Sampling technique (Imam and Conover, 1980) to achieve efficient coverage of the sample space with a manageable number (40) of FE analyses. The random variables involved in these FE analyses are described by the statistics defined in the LRFD databases as mentioned previously (Chapter 2). The limit state of performance was taken as the point at which the bridge system exits the elastic range, as identified from its load-deflection curve (cf Figure 7.1); that definition is believed to represent the limit of usability/repairability of the bridge, but does not represent life-threatening behavior if the structure possesses even a moderate amount of ductility (cf Figure 7.1).

The flexural capacities so determined from this system reliability analysis were rank-ordered and plotted on lognormal probability paper, as illustrated in Figure 7.2 for the straight approach RC bridge (ID 129-0045). The lognormal distribution provides a good fit to these data. The mean and coefficient of variation in the system capacity of this bridge (at first yield) are 4.31^1 times the applied two HS-20 loads and 15%, respectively. The variability is of the same order as the individual girder capacities (Nowak, 1999), but the larger mean is characteristic of the beneficial system effects in a system reliability assessment. Bridge system resistance distributions for other sample bridges are summarized in the last column of Table 7.1. When used in a reliability assessment with the same statistical load models used to develop the *LRFD Bridge*

¹ Note that the mean value of bridge capacity obtained from the probability distribution of system capacity is virtually identical to the bridge system capacity estimated in 7.1.1 with a deterministic analysis with all parameters set equal to their mean values. One of the tenets of modern fragility analysis holds that the mean response of an engineered system can be approximated by the response estimated by setting all parameters equal to their mean values. Similar results have been found in seismic and wind fragility modeling of building structures. Mathematically, the observation is tantamount to expanding the limit state in a Taylor Series about the means and truncating the terms of order higher than unity.

Specifications, one obtains a system reliability index of 3.51 for the RC T-beam Bridge, which is comparable to the safety level stipulated for a new bridge in AASHTO LRFD. The rating factor based on the *system* capacity [cf Eq (2.6)] for the HS-20 vehicle at Operating level is 1.74, presenting a 87% increase in rated load capacity comparing to that calculated at the component level as stipulated in the *AASHTO MBE*. The comparison of the ratings at system level and at the component level for other sample bridges is tabulated in Table 7.2. It may be appropriate to factor in this additional conservatism in bridge evaluation on a case-by-case basis, depending on the consequences of the rating exercise.

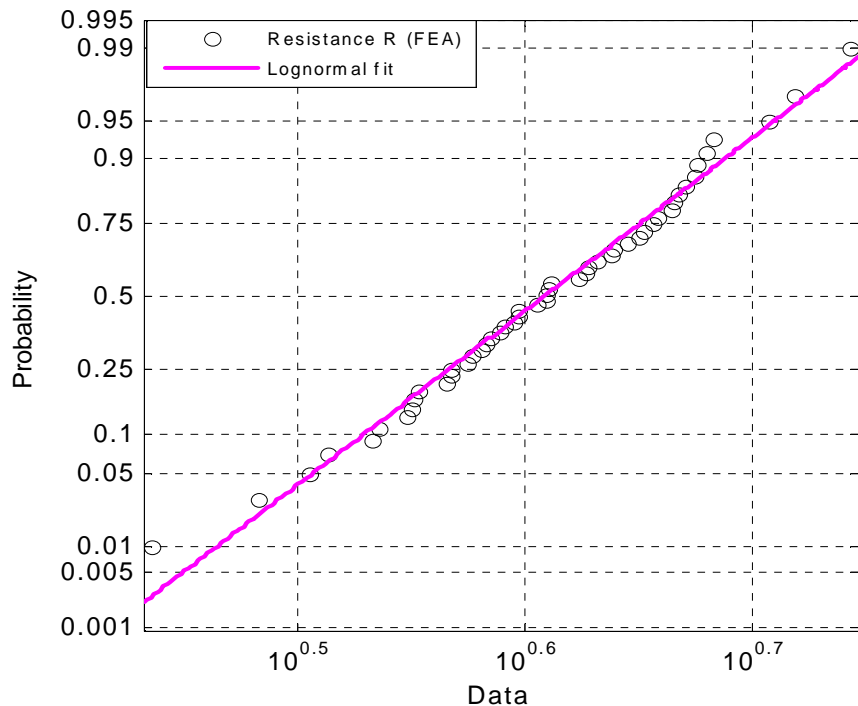


Figure 7.2 Lognormal Fit of System Resistance of the RC Bridge (ID: 129-0045)

Table 7.1 Analysis of Bridge Capacity, Determined as the Point of First Yield

Bridge ID	Type	Design load	Elastic Load factor on HS20	Ultimate Load factor on HS20	Elastic System Resistance Distributions (mean, cov)
129-0045-0	RC – T – straight – not posted	H-15	4.31	4.80	LN(4.31, 0.146)
015-0108-0	RC – T – skewed - posted	HS-15	4.50	5.34	LN (4.50, 0.150)
223-0034-0	Prestressed – straight – not posted	HS-20	5.94	6.87	LN (5.94, 0.108)
085-0018-0	Steel girder – straight - posted	H-15	5.37	5.71	LN (5.37, 0.111)

**Table 7.2 Load Rating at Component Level vs System Level
(HS20 Operating Rating)**

Bridge ID	Type	Component level rating	System Level Rating	Percentage of increase
129-0045-0	RC – T – straight – not posted	0.93	1.74	87%
015-0108-0	RC – T – skewed - posted	2.00	2.61	31%
223-0034-0	Prestressed – straight – not posted	1.72	2.94	71%
085-0018-0	Steel girder – straight - posted	1.14	2.54	122%

7.2 RATING USING SERVICE-PROVEN LOAD HISTORY

Many older bridges have performed well in service without any indication of damage, but yet have been rated as structurally deficient without considering their satisfactory performance over the years under the ever-increasing traffic volume and truck weights. These bridges generally were designed for lower loads but for higher factors of safety and, if well maintained, may have reliability levels that are equal to or higher than those in modern construction. Surviving an extended service load history which is stochastic in nature provides evidence of structural reliability that may be comparable to what might be learned from a proof load test (Ellingwood, 1996; Stewart and Val., 1999). Satisfactory service history should be considered, especially for old bridges, in designing in-service inspection programs and in making decisions for updating analytical ratings and load postings. The AASHTO *MBE* does not provide a mechanism for updating structural resistance for service-proven bridges.

A proof test of a bridge enables the lower tail of the resistance distribution to be truncated at the level of the maximum load carried as shown in Figure 7.3. In contrast, for a service-proven bridge, the magnitude of the maximum load carried by the bridge during its service history is unknown; however, it can be determined statistically using the weigh-in-motion data described earlier (e.g., Nowak, 1999). For a structure surviving a sequence of random vehicle loads, the magnitude of which is described by the probability distribution function $F_Q(r)$ determined using weigh-in-motion data, the revised strength $f_R''(r)$ can be written as (Ellingwood, 1996):

$$f_R''(r) = \frac{F_{Q^*}(r)f_R(r)}{\int_{-\infty}^{\infty} F_{Q^*}(r)f_R(r)dr} \quad (7.1)$$

where $f_R(r)$ and $F_{Q^*}(r)$ are the prior probability density function of resistance and the cumulative load distribution function of the maximum load to occur during the service period of interest, respectively. This updated density can be used in a structural reliability assessment to determine the beneficial effect of successful service performance.

To illustrate the benefit of prior successful bridge performance on rating, consider the concrete T beam bridge (ID: 129-0045), which gave 51 years of serviceable performance. Prior to considering the benefit of successful bridge performance, the use of the mean and COV of bridge capacity presented by Nowak [1999] for new bridges in Eqs (2.1) and (2.2) leads to the prior safety index $\beta = 2.54$. The updated distribution of resistance, as determined from Eq 7.1 by Monte Carlo simulation, is illustrated in Figure 7.4. As a result, with the updated resistance, the estimated bridge failure probability decreases and reliability index increases as the successfully service life of the bridge increases, as illustrated in Figure 7.5. This increase in reliability translates to an increase in bridge capacity rating factors, as indicated in Figure 7.6. Rating factors for this bridge with respect to HL-93 design loading at inventory level, prior to and after considering the 51-year successful service life of this bridge, are summarized in Table 7.3. These results indicate a 16% increase in flexural ratings and a 40% increase in shear ratings by considering the 51-year service load history. Note that the rating in shear increases more than that in flexure. The COV of the prior shear resistance is much larger than the COV

of prior resistance in flexure; knowledge of successful performance causes the effect of the larger COV on the updated reliability to be diminished.

7.3 CLOSURE

The level-three analysis discussed in this Chapter has presented a structural system reliability approach for bridge evaluation by proof load testing, which can be the basis for possible improvements to the component-based LRFR option in the *AASHTO Manual of Bridge Evaluation*. By using a FE-based bridge system reliability analysis, ratings for the RC bridge ID: 129-0045 has increase by 86% percent comparing with the component level analysis. Even just by considering this bridge's 51-year successful service life alone, ratings have increased by 16% in flexural and 40% in shear.

FE models for “virtual” proof load testing possess great potential in modern load ratings, particularly in investigating the conservatisms that appear to be inherent in traditional girder-based rating calculations and in avoiding the extensive cost and risk associated with “real” *in situ* proof load testing. The feasibility of using finite element analysis, validated through either systematic field inspection or through diagnostic load tests, to conduct “virtual” proof load tests of bridge systems and support the improvement of bridge evaluation practices, has been demonstrated in this Chapter.

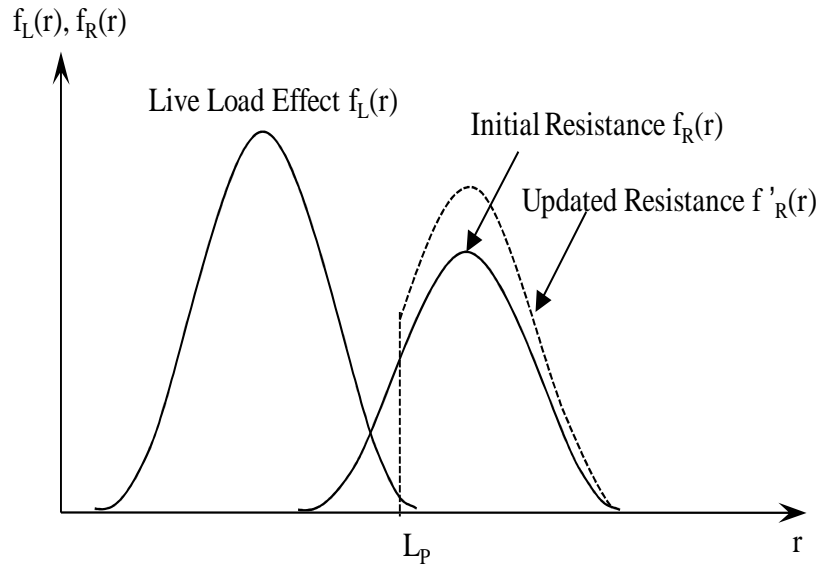


Figure 7.3: Structural Reliability Models for Bridge Proof Load Test

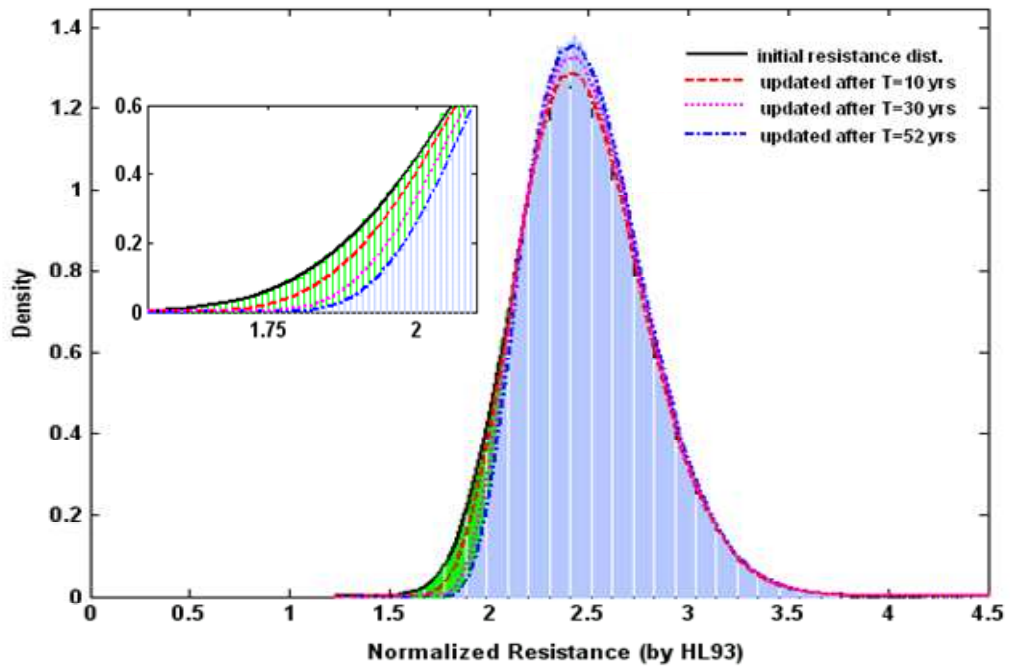


Figure 7.4 Influence of Service Load on Updated Distribution of Structural Resistance for the RC Bridge (ID: 129-0045)

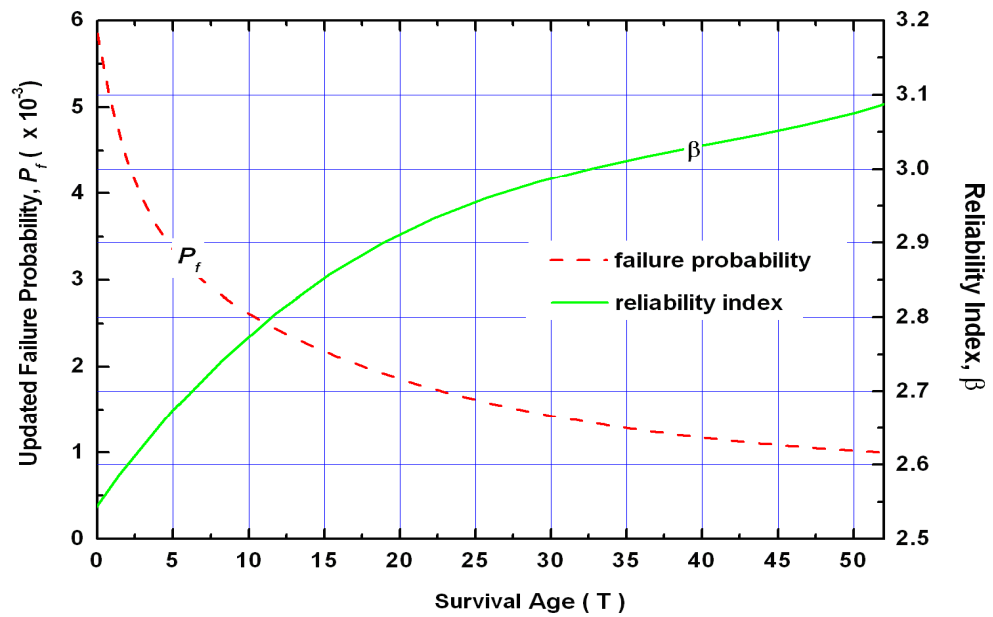


Figure 7.5 Updated Failure Probabilities and Reliability Indices for RC Bridge (ID: 129-0045)

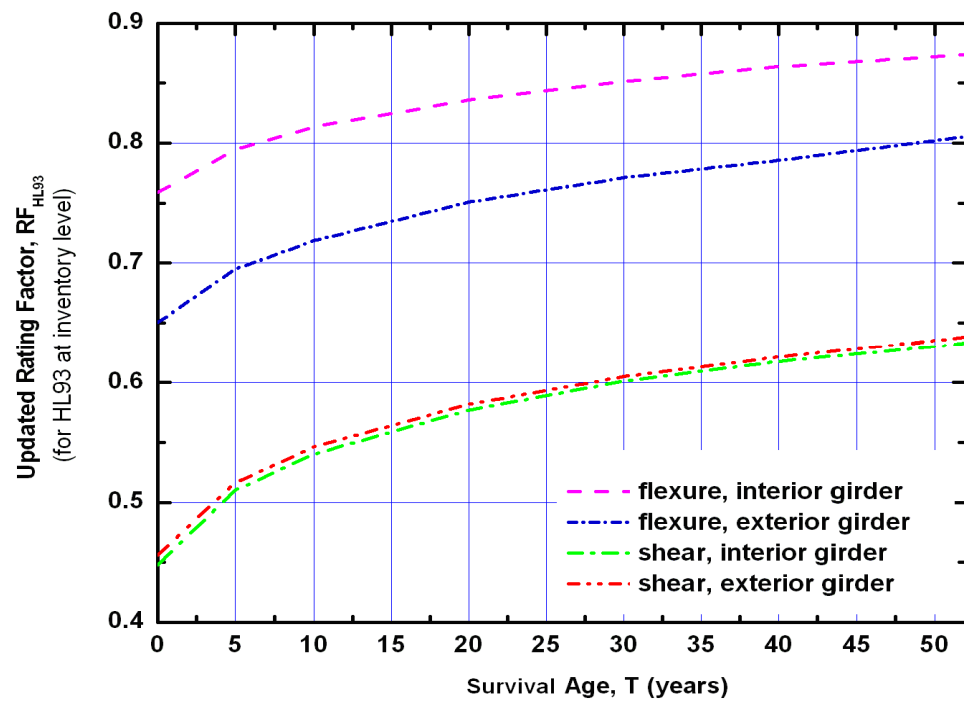


Figure 7.6 Updated Rating Factors Respect to HL-93 at Inventory Level for the RC Bridge (ID: 129-0045)

Table 7.3 Comparison of Rating Factors Computed Before and After Considering Service Load History for RC Bridge (ID: 129-0045)

	Rating Factor			
	Flexure		Shear	
	Interior girder	Exterior girder	Interior girder	Exterior girder
Before updated by service load history	0.75	0.65	0.45	0.46
After updated by service load history	0.87	0.81	0.63	0.64

CHAPTER 8

SUMMARY, CONCLUSIONS AND FURTHER WORK

8.1 SUMMARY

The proposed bridge rating framework developed in this dissertation addresses condition assessment and evaluation by analysis, load test, or a combination of the two methods, depending on the circumstances. Consistent with the *AASHTO LRFD Bridge Design Specifications*, they have a sound basis in structural engineering and structural reliability principles, allowing ratings to be updated as changing circumstances (traffic demands, additional data, material deterioration, and other factors) warrant. The research included the following four major activities: review and critical appraisal of existing bridge rating procedures; bridge load testing; advanced bridge performance analysis using finite element modeling; and structural component and system reliability analysis to more closely couple the bridge rating to the *in situ* performance objectives.

Bridges that are typical those of current concern in rating and posting were selected for load testing and analysis. Finite element models of these bridges were developed to assist the design of the load tests and in the interpretation of the results. The bridge load test results, in turn, were used to validate and improve the finite element modeling. The measured bridge girder deflections in all cases were in good agreement with those predicted by the FE model. This validated analytical and experimental combined approach was then used to develop reliability-based framework to improve the current bridge rating process.

A three-level bridge rating framework was developed to provide bridge engineers with several rating options. At the simplest level, the results of visual inspection are incorporated in a format similar to that in the *AASHTO Manual of Bridge Evaluation*. The higher levels involve more in-depth rating analyses that facilitate a customized rating of an individual bridge by integrating *in situ* information into the capacity rating metric. This study provides reliability-based methodologies and technical tools for performing ratings systematically. Such *in situ* information can be obtained from material testing, condition rating records, load tests, successful service life performance, and system reliability analysis. The higher level ratings emphasize the importance of learning from inspections in rating a given bridge and provide clear incentives to obtain quantitative *in situ* measurements through field inspections, load tests and other modern non-destructive evaluation technology.

8.2 RESEARCH CONCLUSIONS

Application of the proposed evaluation framework to existing steel, reinforced concrete or pre-stressed concrete bridges indicated that posting requirements based on current bridge evaluation practices, which do not incorporate available site-specific knowledge in any detail, can be unduly conservative from a structural reliability viewpoint. The proposed improvements recognize the uniqueness of an individual bridge and take advantage of accessible *in situ* information to the extent feasible to produce bridge ratings that provide for public safety without undue economic impact on the community served.

The load tests and supporting analysis indicated that utilizing current girder distribution factors can yield conservative measures of actual load-carrying capacity. This

conservatism is the result of assumptions made in the analysis regarding load sharing, composite action, support conditions and nonlinear behavior, as well as the differences in material strengths. In view of the economic consequences of posting, it is apparent that if the customary rating practice suggests that a bridge is a candidate for posting, a more accurate structural analysis model should be employed to verify whether more accurate GDFs might change that decision.

The load-carrying mechanism in reinforced concrete pier caps that have short shear spans and behave as deep beams is better described by the strut and tie model than by the traditional ACI Standard 318 model. The use of this new capacity calculation method is permitted by the LRFR option in the AASHTO *MBE*. A preliminary investigation of similar bridge pier caps indicated that the level of conservatism is dependent on the dimensions of the individual pier caps and the placement of the girders that they support.

Routine bi-annual inspections play an important role in bridge condition assessment by providing *in situ* data to support the real-time bridge reliability estimate and to assist the decision making regarding suitable maintenance. These inspections are mostly completed using visual inspection techniques, do not result in quantitative estimates of deterioration, and therefore are difficult to incorporate in a strength updating process based on modern structural reliability assessment methods. This study provides a method for linking qualitative visual inspections to quantitative reliability-based load rating. A new set of condition factors was developed to couple the rating procedure more closely to the results of bridge inspections

A properly conducted proof load test as well as bridge's successful service history can be effective way to assess the structural behavior of a bridge as a system and to

update the bridge load capacity at the system level, especially in situations where the analytical approach at the first and second levels produces questionable ratings, or structural analysis is difficult to perform due to deterioration or lack of documentation. FE analyses conducted as “virtual” proof load tests possess great potential in modern load rating, particularly in eliminating the conservatisms that appear to be inherent in traditional girder-based rating calculations and in avoiding the risk and cost associated to a “real” *in situ* proof load test. The feasibility of using finite element modeling, validated through either systematic field inspection or through diagnostic load tests, to conduct “virtual” proof load tests of bridge systems and support the improvement of bridge evaluation practices, has been demonstrated in the study.

Experience in conducting the load tests suggested that basing the performance assessment of an existing bridge on global response measurements, such as displacement, as opposed to local responses, such as strain, minimizes the likelihood that spurious local non-homogeneous or material behavior may lead to false conclusions regarding structural system safety. It was also observed that redundancy in measurements, through multiple gauges at a single location and gauges at multiple locations in a single element, is essential to accurate condition assessment and should be utilized whenever practical.

8.3 RECOMMENDED FUTURE WORK

In the course of the research conducted in this dissertation, several topics worth further investigation have been identified:

In bridge rating by the LRFR method, the performance based evaluation philosophy is embedded in the load and resistance factors. Each load or resistance factor should have a sound reliability basis and should enable performance objectives that are

consistent with current practice to be achieved. The system factor appearing in the LRFR rating equation (Eq 2-6) should account for the reserve capacity and redundancy of the bridge in the rating calculation, but is based on a component level rather than system level of analysis. The system reliability analysis conducted herein demonstrates the inadequacies of this component-based approach. The development and basis of the system factors in LRFR should be more closely examined.

The proposed multi-level rating approach clearly highlights the learning process of a given bridge through field inspections. Current bridge inspection programs seldom do an adequate job of reflecting *in situ* condition quantitatively, making it difficult to incorporate the results into the bridge capacity rating process. The use of modern non-destructive evaluation (NDE) techniques, such as Carpenter hammer sounding, Schmidt rebound hammer and ultrasonic pulse velocity, should be encouraged in bridge inspection activities, as they may permit a revised estimate of *in situ* strength to be used in the bridge rating calculations. The benefit and the cost of introducing these technologies should be considered in designing in-service inspection programs and in maintenance decisions. Inspection guidelines corresponding to these NDE techniques should be established to coordinate with the updating of rating calculations.

While each existing bridge structure is unique, many bridge types may share common failure mechanisms and may be governed by a relatively few parameters, especially when a group of bridges is constructed from a limited numbers of standard designs. It is therefore possible to classify bridges by identifying these governing parameters and their variation within the population and to develop bridge type-specific strategies for load rating and condition assessment. The type-specific strategy could

provide significant advantage for inspection and load rating of bridges shearing common material, similar geometry and detailing, and the same critical behavior mechanisms.

Much of the development of the reliability assessment of bridges has focused on the performance of the bridge superstructure. Less effort has been spent on studying reliability of bridge substructures (piercap, piers and columns) and connections. Bridge deficiencies may become apparent under seismic, hurricane and other extreme events, in addition to traffic demands. Efforts should be increased to collect data on the bridge substructure/connection behavior under extreme events in a time-dependent manner. These results should be included in condition assessment framework expressed in terms of reliability.

A decision engine to supplement the tiered rating framework is necessary for authorities to maintain the functionality of the bridge system in a cost-effective manner. If a bridge is rated below the minimum acceptance level, the assessing engineer should examine the options of possible maintenance strategies on a future cost (or life-cycle cost) basis. The bridge authority must have an internally consistent decision-making system which interprets the rating results, estimates the cost of each alternative strategy, examines the overall objective of bridge management in terms of risk, and assigns priorities for bridge maintenance. The bridge condition assessment process will be able to determine not only the structural adequacy of the bridge at the present time, but will provide technical support for financial risk management strategies and future maintenance options.

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