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NATIONAL
COOPERATIVE
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PROGRAM

Calibration of Load Factors for LRFR Bridge Evaluation

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NCHRP REPORT 454

Calibration of Load Factors for LRFR Bridge Evaluation

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Portersville, PA

SUBJECT AREAS
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study to determine load factors for use in evaluating the load capacity of existing bridges. The report includes recommended values for load factors and presents the methodology and data used to calibrate the factors to provide appropriate safety margins. The material in this report will be of immediate interest to bridge engineers involved in bridge load rating and to engineers interested in the development of load and resistance factor rating procedures

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The *AASHTO LRFD Bridge Design Specifications,* which were developed under NCHRP Project 12-33, were adopted in 1994. These specifications represented a first effort by AASHTO to integrate knowledge of the statistical variation of loads and resistances into the design process. In developing the design specifications, considerable effort was made to keep the probabilistic aspects transparent to the designer, and no knowledge of reliability theory is necessary to apply the specifications.

During design, load capacity can be added to a bridge easily, and uncertainties in the magnitude of loads (and the resulting conservatism of design estimates) have only a small impact on construction costs. In contrast, the cost to strengthen an existing bridge can be very large, and, to avoid unnecessary expenditures, accurate estimates of loads are needed. In order to reduce the uncertainty of load estimates, a greater knowledge of the type, size, and frequency of vehicles using a particular bridge is needed. As a consequence, the application of reliability theory to bridge load rating is more complex and varied than the application of these principles to design, and rating engineers can benefit from a greater understanding of the basis for the load factors specified.

NCHRP Project 12-46, "Manual for Condition Evaluation and Load Rating of Highway Bridges Using Load and Resistance Factor Philosophy," was initiated in 1997 with the objective of developing a manual for the condition evaluation of highway bridges that is consistent with the design and construction provisions of the *AASHTO LRFD Bridge Design Specifications*, but with calibrated load factors appropriate for bridge evaluation and rating. The research was performed by Lichtenstein Consulting Engineers, Inc., of Paramus, New Jersey, with Dr. Fred Moses serving as a consultant for the development of load factors. This report fully documents the methodology and data used to calibrate the load factors recommended in the manual. The information in the report will assist bridge engineers in their rating practice and researchers in refining load factors as new data and analysis tools become available.

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This report was prepared as part of the activities for NCHRP Project 12-46 to develop a manual for evaluation of highway bridges using the LRFD safety philosophy. The principal investigator for this project was the firm of Lichtenstein Engineering Consultants, Inc., and this report was prepared as a subcontract to that project.

The writer wishes to acknowledge the help in the preparation of this report of Bala Sivakumar, Charles Minervino, and William Edberg of Lichtenstein Engineering Consultants, Inc.; Dennis Mertz of the University of Delaware; Michel Ghosn of the City University of New York; and the many reviewers from TRB, the project panel, and various state agencies.

CALIBRATION OF LOAD FACTORS FOR LRFR BRIDGE EVALUATION

SUMMARY

This report presents the derivations of the live load factors and associated checking criteria incorporated in the proposed *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* prepared for NCHRP Project 12-46 (hereafter referred to as the Evaluation Manual). A final draft of this Evaluation Manual was submitted early in 2000 to the project panel and the appropriate AASHTO committees.

These evaluation criteria, along with corresponding live load factors, are needed for performing the legal load rating analysis and the evaluation of permit loadings and postings, including site-specific data inputs. The material herein supplements the text and commentary in the proposed Evaluation Manual as it relates to load and resistance factor rating (LRFR).

This report presents the methodology and data used to calibrate the LRFR criteria for the proposed Evaluation Manual. This report supplements the derivations of the design factors developed for the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (Nowak, 1999).

Various additional applications are contained in the Evaluation Manual. These applications are not covered in the design specifications and include bridge rating for legal loads, posting guidelines, heavy truck permit review, bridge testing, and remaining fatigue life assessments.

Although the focus of NCHRP Project 12-33 was the calibration of the AASHTO LRFD Bridge Design Specifications, the focus herein is solely on the calibration of features unique to the evaluation process for existing bridges. For overall consistency, therefore, the philosophy in this report follows the existing approaches used in calibrating the load and resistance factors for the AASHTO LRFD Bridge Design Specifications.

The needs of bridge agencies and consultants have been considered herein. These needs have been addressed through the preparation of general guidelines in the Evaluation Manual. These guidelines apply to wide classes of existing bridges. The Evaluation Manual includes options to allow the incorporation of site-specific traffic, performance data, and target safety criteria when warranted by the evaluation needs of a particular bridge span.

This report will serve as a reference for future developments and modifications of the LRFR methodology for bridge evaluation as more data and improved analysis methods become available.

Chapters 1, 2, and 3 provide the goals of the study and the background material on reliability-based calibration, especially the recommended formats for bridge evaluation. The material is written for engineers who will use the Evaluation Manual. Relevant background on reliability methods is presented herein.

Chapter 4 describes the truck weight sample introduced by Nowak and used in the calibration of the AASHTO LRFD Bridge Design Specifications. This chapter shows how such data were used herein for developing the evaluation criteria. Methods for using site-specific data are emphasized.

Chapter 5 discusses the modeling of bridge safety, including nominal live load models, truck multiple presence probability, extreme load combinations, dynamic allowance, distribution factors, system factors, and safety index expressions. Chapter 6 provides the calibration of live load factors for legal load ratings for routine traffic, as well as the development of posting curves and the use of site-specific weigh-in-motion (WIM) data, when available. Chapter 7 extends the calibration to live load factors for permit analysis, including routine, special, and escorted vehicles. The live load factors and checking formats, for both single and multilane cases, are derived, compared, and summarized for presentation in the proposed Evaluation Manual.

Chapter 8 discusses field testing for rating bridges, while Chapter 9 outlines, for special cases, the direct use of safety indexes (beta values) in the rating process. Chapter 10 presents conclusions. References and Appendix A, which contains the standard normal distribution table, are also provided.

To the extent possible, this report refers to the final draft of the Evaluation Manual submitted by the research team to the NCHRP Project 12-46 research panel and the AASHTO Bridge Subcommittee. Changes subsequently made in the Evaluation Manual after being submitted by the Lichtenstein firm are not reflected herein. In addition to the final draft of the Evaluation Manual, readers of this report should also obtain the companion NCHRP Project 12-46 report (*Web Document 28*) prepared by Bala Sivakumar et al. of Lichtenstein Engineers. This report contains trial ratings, numerous bridge examples and comparisons of proposed and existing ratings, and various responses to questions raised in the preparation of the Evaluation Manual.

CHAPTER 1

INTRODUCTION

This report presents the derivations of the live load factors and associated checking criteria incorporated in the proposed *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* prepared for NCHRP Project 12-46 (hereafter referred to as the Evaluation Manual). A final draft of this Evaluation Manual was submitted early in 2000 to the project panel and to appropriate AASHTO committees. In addition, there is a companion project report (prepared by Lichtenstein Consulting Engineers, Inc.), which contains trial ratings, numerous bridge examples, and comparisons of proposed and existing rating results.

The evaluation criteria, along with corresponding live load factors developed herein, are recommended for the legal load rating analysis and the evaluation of permit loadings and postings, including the use of available site-specific data input. The material herein supplements the text and commentary in the proposed Evaluation Manual related to the Load and Resistance Factor Design (LRFD) factors.

A major goal in this report is to unify the reliability analyses and corresponding database used in the load and resistance factor rating (LRFR) and the recommendations for the Evaluation Manual compatible with the AASHTO LRFD bridge design specifications.

In addition, the following topics, unique to the development of the evaluation criteria, are also presented in this report:

- The derivations of the proposed live load factors using reliability methodology for the various categories of bridge ratings described in the proposed Evaluation Manual. (These derivations included the extension of the reliability methods utilized in the AASHTO LRFD Bridge Design Specifications [AASHTO, 1994] to the requirements for evaluation and rating of bridges);
- The traffic models and database used for calibrating the recommended live load factors in legal load rating for site-specific input of annual daily truck traffic (ADTT);
- An extension of the modeling of live load factors for the specific cases of checking of random traffic, routine permits, and special permit evaluation for heavy vehicles;
- The derivations and the implied safety criteria contained within the proposed allowable truck weight posting curve;
- How site weigh-in-motion (WIM) data, if available, can be incorporated in adjusting the load factors and ratings of specific bridge sites;
- An alternative rating procedure to the LRFD checking equations that directly uses the target safety indexes in calculating bridge ratings;
- Methods for extending the recommended live load factors to special cases that are not covered in the Evaluation Manual; and
- Areas for research and further data gathering.

CHAPTER **2**

BACKGROUND

In general, bridge evaluation, unlike bridge design, requires that engineers be more aware of the reliability analysis than is true during design. During the evaluation of bridges, the evaluation engineer will determine various different ratings. For example, in the Evaluation Manual, there is the design load rating, the rating for legal loads, and rating for permit loads. Also, there is greater flexibility in selecting factors in the Evaluation Manual, such as the target safety level for different permit categories. For these reasons, this report goes into detail regarding the reliability analysis and the calibration of load factors based on reliability analysis. The aim is not to be comprehensive in a description of structural reliability—there are many textbooks and articles devoted to this subject. Rather, the goal here is to present some basic material and to highlight issues unique in reliability analysis methods for bridge evaluation. The level of presentation, however, is aimed toward bridge engineers who will use the Evaluation Manual.

There has been considerable research and data gathering in recent years on highway bridge loadings and component resistances, especially in connection with the formulation of the recently adopted AASHTO LRFD specifications, which are reliability-based bridge design specifications. The LRFD specifications provide load and resistance factors that should lead to consistent target reliability levels for the design of components over a wide range of bridge span and material applications. The development of LRFD procedures for bridge design is similar to other LRFD developments such as the American Institute of Steel Construction (AISC) LRFD specification for buildings (AISC, 1996) or the American Petroleum Institute (API) LRFD format for offshore steel structures (API, 1992).

The designation of a reliability-based design format usually refers to procedures in which specification bodies consider the statistical distributions of loadings (e.g., dead, live, and environmental loads) and the statistical distribution of component strength (e.g., members, connections, and substructures). The reliability is calculated from these load and resistance distributions by specification committees who then formulate and recommend the specified load and resistance factors and associated design criteria.

2.1 RELIABILITY ASSESSMENT

To aid in visualizing the performance of a structural component, consider the simple component illustrated in Figure 1 with strength, R, and load, S. Both R and S are random quantities reflecting the uncertainty of their values at the time that the component is checked. The uncertainties may be described by statistical distributions, as shown in Figure 1, for both R and S. The component is safe, that is failure does not occur, as long as the realization of R, the resistance, exceeds the load, S.

Superimposing the two statistical distributions (as shown in Figure 1) gives a typical situation found in structural reliability analysis. That is, there is a slight overlap of the load distribution over the strength distribution. The amount of overlap of the two probability curves depends on the safety factor. Higher safety margins “push apart” the load and strength probability curves and reduce the overlap or probability of failure.

Typically, the load distribution, S, is based on assessing the largest load expected within the appropriate time interval of the analysis and R, is the corresponding strength. The probability of failure, P_f, may be expressed by integrating over the load frequency distribution curve as follows:

$$P_f = P[R < S] = \int P[R < s]\ f_s(s)\ ds \tag{1}$$

The notation, $P[\ ]$, should be read as “probability that,” while $f_S(s)$ is the load probability density curve or the probability value associated with load, s. Thus, the probability of failure is found by integrating or summing numerically over each value of load, s, the density function of load times the probability that R is less than the value, s. The probability of failure decreases if there is less overlap of the load and strength frequency curves (as illustrated in Figure 2*a* when there are higher safety factors). Further, since the area under a frequency curve is always one, there is lower failure probability if the frequency curves are steeper, as shown in Figure 2*b*. A sharp peaked frequency curve occurs if there is less uncertainty in the value of the variable, while a flatter distribution indicates a greater uncertainty. The relative shape of the distribution curves is best expressed by the standard deviation or, in a nondimensional form, by the coefficient of variation (COV) (which is the standard deviation divided by the mean value).

In general, the value of P_f increases with smaller safety factors and higher coefficients of variation (i.e., greater un-

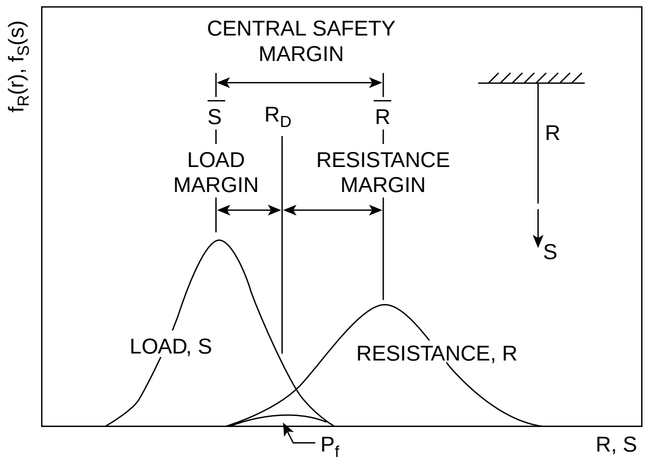

Figure 1. Basic reliability model and failure probability.

certainty). The shape of the probability curve (e.g., whether a normal, lognormal, or Gumbel distribution) usually plays a lesser role in the value of P_f computed compared with the safety factor and respective COVs.

In most structural safety models for calibrating design specifications, the reliability is highlighted rather than the probability of failure, where the reliability equals 1 minus the probability of failure.

In structural design, the reliability modeling, such as for the AASHTO LRFD Specification development, usually denotes S as the maximum lifetime load and R as the corresponding strength. For the evaluation calibration, however, it is necessary to consider intervals of life corresponding typically to periods between inspections. Data from the most recent inspection may help to reduce the strength uncertainty, while data from traffic surveys or bridge performance may reduce load uncertainty. In fact, at different stages in a bridge evaluation, the engineer may seek further site-specific data to reduce the uncertainties. Seeking further site-specific data becomes an option when the initial evaluation based on more general data input leads to an unsatisfactory rating.

Also, for evaluation, it is recognized that the statistical distributions are changing over time. Such change is illustrated in Figure 3, which shows possible load and resistance distributions when a structure is built and some period later. Typically, the load distribution on a bridge shifts to higher values because of increases in truck weights and traffic. The resistance distribution may shift to lower values because of possible deteriorations in the members.

These general descriptions serve to illustrate that reliability itself is a time-dependent variable, subject to influences of traffic, maintenance, and deterioration and also subject in analysis to modification by obtaining additional site data. This description should also help to explain why reliability levels used in evaluation obtained some years after a bridge is built are usually lower than reliabilities calculated for a new span. Economics are also tied into these comparisons, as inadequate reliability calculated at a design stage may be eliminated by increasing design member sizes, usually at a small percentage increase in structure cost. Low values of reliability at an evaluation stage may lead to costly bridge postings or replacements or, as recommended in the Evaluation Manual, the need to obtain more site-specific inspection and traffic data to reduce uncertainties and possibly raise the calculated rating to acceptable levels.

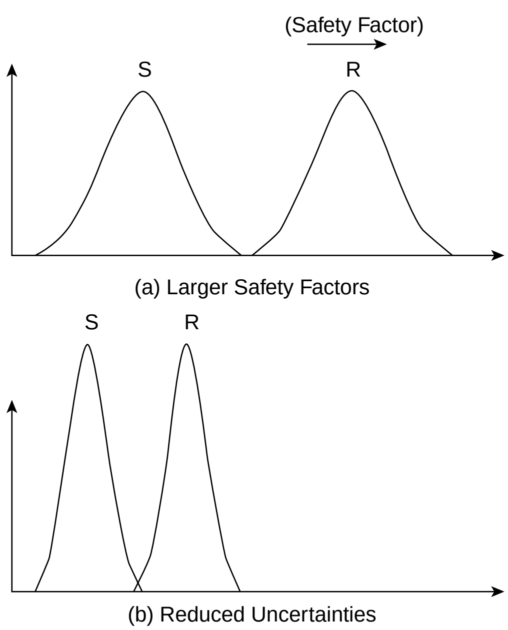

Figure 2. Illustration to achieve higher reliability.

2.2 CODE CALIBRATION

2.2.1 Calibration Goal

Code calibration refers to the process of selecting nominal load and resistance values and corresponding load and resistance factors for a specification. This effort is generally carried out by specification groups so that designer engineers are not concerned with this process. Most LRFD specifications appear strictly deterministic to designers—with the entire process of calculating reliabilities being totally transparent to the design operations. Keeping the reliability calculations out of the design process means that the statistical database as described above for loads and resistances need not appear as part of a specification.

2.2.2 Calibration Formulation

To calibrate a reliability-based structural design code, code writers generally use the following steps:

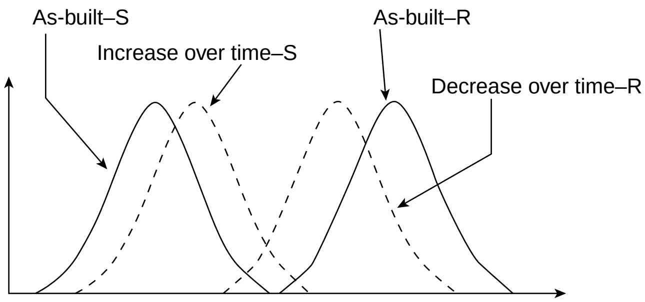

Figure 3. Changing reliabilities over time.

1. Define the limit states or conditions that are going to be checked. The limit states may be ultimate or service type with corresponding consequences.
2. Define the random variables that may affect the occurrence of a component or system limit state. These random variables usually include dead load effects associated with random material and geometric properties, as well as structural analysis modeling, live load effects associated with external traffic (including expected maximum truck weights, lateral bunching of vehicles on the span, and dynamic responses), and other environmental phenomena (e.g., wind, earthquake, collision, temperature, and scour). Other random variables include those that affect component and system resistance or strength capacity, such as material and geometric properties or uncertainties in strength analysis modeling.
3. Assemble a database for the various load and resistance random variables. The data should include, as a minimum for each variable, a COV (which is a measure of the scatter of the variable) and a bias (which is defined as the ratio of mean value to the nominal design value). In addition, if sufficient data exist or a predictive model can be validated, a random variable should be described by a particular probability distribution, such as a normal, lognormal, or extremal distribution. Such distributions can be fit using the bias and COV as input parameters. The bias of the random variable can only be determined after a fixed deterministic model or formula has been defined, such as a load model (e.g., HS20 or HL93) or the formula is given for checking the bending strength of an element. Because engineering checks compare member load effects with component capacities, the parameters of the load and resistance random variables (i.e., the bias and COV) must also reflect analysis and modeling uncertainties.

2.3 CALCULATION OF SAFETY INDEXES

In any code calibration, it is necessary to develop a calculation procedure for expressing the structural reliability or, conversely, the probability of failure. The calculation of the probability of failure is shown in Equation 1 in the expression for P_f. In general, however, the most commonly used procedures for expressing the safety include calculation of the safety or reliability index, often denoted as beta (β). The safety indexes or betas give a measure to the structural reliability or, conversely, the risk that a design component has insufficient capacity and that some limit state will be reached. Higher betas mean higher reliability.

An expression for the beta calculation can be found with simplified normal or lognormal approximations or by using available structural reliability computer programs that operate on a safety margin or limit state equation, often expressed by the variable, g. A limit state equation should express the margin of safety for any type of failure mode in a deterministic fashion such that it is clear from the value of the limit state variable, g, whether the component has survived or failed. For example, define the random variable g as a margin of safety as follows:

$$\begin{aligned} g &= \text{component resistance} - \text{load effects} \\ &= R - D - L \end{aligned} \tag{2}$$

where

R is the random resistance,
D is the random dead load effect, and
L is the random live load effect including dynamic amplification.

The component is safe if a realization of the load and resistance random variables (including the modeling uncertainties) lead to a safety margin where g is greater than 0 and the component fails if g is less than 0. Because R, D, and L are random variables, the magnitude of g is also random. As an approximation, one can consider the mean and standard deviation of the variable g to give a measure of the reliability. If the mean of g is large (a positive value means safe) and/or the standard deviation of g is small, then there is only a small probability that g will actually fall below zero or that failure will occur. A nondimensional safety index quantity, beta, which expresses a measure of this risk, can be written as follows:

$$\text{Safety Index, } \beta = \frac{\text{mean value of } g}{\text{Standard deviation of } g} \tag{3}$$

Thus, beta is the number of standard deviations that the mean safety margin falls on the safe side. The calculation of the mean and standard deviation of g as a function of the means and standard deviations of the load and resistance random variables is part of structural reliability analysis programs such as the RELY program (Baker, 1982). If the random variable describing the safety margin, g, were to follow a normal distribution, an exact value for the risk would come directly from the standard table of normal distributions given in books on statistics (see Appendix A). For example, a value of beta equal to 3 corresponds to a risk of 0.0013 or a reliability of

0.9987. This value is roughly a chance of failure of one in one thousand.

Even if the load and resistance variables are not normally distributed, the structural reliability programs generally introduce accurate equivalent normal approximations for these variables. In such cases, the betas lead to a probability of failure obtained from the normal distribution table that correlates closely to a failure probability that would be found by exact numerical integration or by simulation. Because of the limitations in assembling a precise statistical database for loads, modeling analysis, and resistance random variables, any errors introduced by the approximations in the beta reliability programs are usually relatively small. The validity of using the approximate values for the safety index from the reliability programs becomes evident when the calibration of target safety indexes for a specification is carried out as discussed below.

More important for the purposes of the Evaluation Manual, the failure probability expressed by the value of beta relates to what is called the "notional" probability of failure. The notional value of failure probability is calculated for a component and does not reflect possible failures because of gross blunders, lack of understanding of the technology, or human errors. In addition, failure probability calculations using betas may overestimate actuarial or true failure rates because of deliberate conservative design and specification decisions and potentially large structural system reserves that add conservative margins to the design. Typically in redundant designs, the failure probability of the system may be one order of magnitude smaller than the failure probability computed for a component. Thus, designers using a particular LRFD specification should not expect that the target beta values translated into probability of failure will actually correlate to observed failure rate statistics.

2.4 SELECTION OF TARGET SAFETY INDEX

After achieving a methodology and database to calculate the safety or reliability index, the next step is to select a *target safety index* for the code calibration. That is, choose as a goal the safety level that is warranted in the specification for the components. The aim in the subsequent calibration of load and resistance factors is usually to achieve *uniform* safety indexes so that, for any given component checked by the specifications, the calculated beta will be as close as possible to this target safety index. Existing working stress or load factor checking formats typically produce component designs that do not have uniform safety indexes. Hence, the advantage of the calibrated LRFD format from a reliability viewpoint is uniform safety indexes over different materials, spans, and load effects.

Typically, target beta values in the range of 2.0 to 4.0 are used in formulating LRFD design criteria. Ideally, the selection of the target beta should be an economic issue that reflects both the cost of increasing the safety margins and the implied costs associated with component failures.

An optimum cost target beta in a specification corresponds to a situation in which the marginal cost of further increasing the safety index is just balanced by marginal reductions in the risk-associated cost, which is the probability of failure times the cost of failure. The marginal cost of increasing the safety factor is much higher in the evaluation phase than in the design phase because an inadequate rating may lead to replacement or posting whereas, for new construction, higher safety margins may introduce very small cost increments. For example, to increase a design level requirement from HS20 to HS25 (25 percent increase in load capacity) may only be 1 to 2 percent of the total cost (Moses, 1989). To rehabilitate an existing bridge to raise its capacity by 25 percent may be very costly or impossible in some cases.

The cost of failure should be the same whether for new designs or associated with evaluation of existing structures. Given that the relative marginal costs for increasing capacity are higher in existing spans than for new designs, it is logical that the target safety indexes will be lower in evaluation than in design. This conclusion is consistent with AASHTO's historical use of lower margins in operating-level safety criteria used in bridge evaluation compared with higher safety levels (known as inventory) used in design of new spans.

Because some of the data needed for optimizing target safety indexes may be unavailable, such as the projected cost of failure, the optimization of costs may not be used by specification committees in selecting target safety indexes. Instead, an alternative approach to selecting the target reliability level is to consider past performance criteria. Average betas, calculated from a sample of past designs that are presumed to lead to good performance records, are gathered and averaged to prescribe a fixed target beta for future specifications.

The selection of targets on the basis of experience is an important feature of the proposed AASHTO Evaluation Manual. In past bridge practice in the United States, one level of safety margin, namely inventory, has been used for design and as an upper bound for bridge evaluation. A lower and less conservative safety margin, namely the operating level, has been most often used for decisions regarding posting and load limits. For example, the new *AASHTO LRFD Bridge Design Specifications* (AASHTO, 1994b) references a target reliability index of 3.5 while the AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (AASHTO, 1989) used a target in the range of 2.3 to 2.5, on the basis of operating level allowable stress and operating load factor ratings.

The difference between the risk corresponding to the notional target betas and the observed actuarial failure rate is a reason why LRFD codes usually avoid directly mentioning probability of failure criteria. For example, even the AASHTO LRFD specification does not directly mention risk values or target beta values. The betas, however, are discussed in several reports (e.g., Nowak, 1999).

The target safety index is used by specification bodies as an input parameter to calibrate a specification to achieve *uniform* reliability. As stated above, the target beta is based on average betas computed from a sample of past designs. If subsequent data and analysis show changes in the database (e.g., different biases or COVs) used to compute component safety indexes, the average values of beta will change. However, the calibration process is such that further subsequent calibration of the load and resistance factors with the new data will lead to only small changes in load and resistance factors. This situation highlights the "robustness" of the specification calibration when the target safety is based on past performance practices. (It is assumed that any change in the data or analysis leading to changes in beta values will lead to a new "target" beta based on the average computed betas from the sample of past designs.)

As a further insight into selecting the target safety in a specification, an approach used in recent U.K. bridge evaluation codes (Das, 1997) presented acceptable historic failure rates for structures in light of other risks taken, such as industrial accidents, automobile and other travel risks, etcetera. These data were also compared with expected bridge failures in an historical period. It was noted that there are few if any known examples of bridge failures of the type considered in the design calibrations, namely the load effect exceeding the resistance. Most reported failures result from scour, seismic, and collision events. However, in U.S. experience, structure failure resulting from overloaded vehicles has occurred in posted bridges because of vehicles that clearly exceeded the posted loads.

Finally, the last point to be made in the context of selecting a target safety index is the interpretation of this index in the context of a bridge population. In the AASHTO LRFD development, for example, a truck loading database was used corresponding to a very heavy truck traffic volume and truck weight distribution (see below for more details). The average calculated value of beta with this database using a sample of past designs is given as 3.5 and also refers to this extreme loading situation. Bridge spans with lower traffic volumes or more typical truck weight histograms should have significantly higher safety indexes. Alternatively, the overall COV of the live load modeling could also include site-to-site variability in truck intensity. If site-to-site variability were done and the bias of the extreme loading intensity with respect to average site loading intensity were included, then the betas would have a different meaning. In that case, the average bridge span would have a safety index of 3.5 with some spans experiencing higher safety indexes and other spans lower safety indexes.

The AASHTO LRFD calibration report (Nowak, 1999) does not elaborate on whether the load intensity includes site-to-site uncertainty. Thus, it is not possible to judge the true meaning of the reported AASHTO target beta of 3.5—whether it is a value averaged over all spans or a value associated with a site having an extreme truck traffic intensity.

The approach adopted herein (which includes site-to-site variability) is to model the live load COV. Further, the evaluation live load factors are based, to the extent possible, on site-specific information such as traffic volume (ADTT) and, when available, also on truck weight intensity obtained by traffic surveys. The intent then is to maintain a uniform target safety index applicable to each specific span.

2.5 LRFD CHECKING FORMAT

The capacities of components are checked during either the design or evaluation process. In LRFD practice, a component is typically checked by an equation of the form

$$\phi R_n = \gamma_d D + \gamma_L L_n \tag{4}$$

where

ϕ is the resistance factor,
R_n is the nominal component resistance computed by a prescribed formula;
γ_d is the dead load factor;
D is the nominal dead load effect;
γ_L is the live load factor; and
L_n is the nominal live loading effect including impact prescribed by a load model such as HS20, HL93, or some other legal vehicle and/or uniform loading model.

For bridge loadings, the checking model must also specify number of lanes, positioning of loads, consideration of multispans, any treatment of support flexibility, combinations with other extreme loading effects (i.e., wind, scour and collision) and potential contribution to strength from system configuration and nonstructural components (i.e., deck and lateral bracing).

In design, a capacity, R_n, is found to satisfy the design check in Equation 4. In evaluation, the nominal resistance is estimated from inspection data and instead a rating factor, (R.F.) is multiplied by the loading term, L_n, which can be solved from

$$\text{R.F.} = \frac{\phi R_n - \gamma_d D}{\gamma_L L_n} \tag{4a}$$

Different and more detailed checking models may be appropriate for evaluation than those used in design, if inadequate ratings are found in the evaluation process. For example, in the evaluation, use of measured material properties, finite element analysis, or even load testing may be economically justified to raise a bridge's rating value.

2.6 CALIBRATION OF LOAD AND RESISTANCE FACTORS

The final step in developing an LRFD specification is to implement a table of partial load and resistance factors, including ϕ, γ_d and γ_L, which satisfy the target beta value. This implementation is often done by using the sample population of bridge component designs mentioned above for selecting the target betas. This sample should cover a range of different spans, dead-to-live load ratios, materials, etcetera. For this sample of components, the betas are computed for any assumed set of load and resistance factors. By an iterative process, choose the set of factors that produce the best combination of betas (i.e., the average beta falls closest to the target value with a minimum of deviation in the calculated beta for any of the samples).

The load and resistance factors found by this last step are tabulated in an LRFD design specification or an LRFR evaluation manual. In a design code, there may only be one set of factors based on data generalized for all design applications. In evaluation, however, a wide range of decisions may need to be made that use a different level of input of site-specific data. As more data are made available, there is reason to adjust the factors to reflect this new information. These issues pertaining to bridge evaluation are discussed in the following paragraphs.

2.7 EVALUATION ISSUES IN CALIBRATION

A major concern when calibrating the proposed AASHTO Evaluation Manual is the selection of the load and resistance factors for a broad range of site-specific applications, such as different traffic and live loading environments, as well as possible cases of deteriorated spans. For example, a random variable that must be described for different situations, including random as well as permit trucks, is the maximum truck traffic live loading. A number of approaches have been presented for modeling the extreme traffic live loading variable that, because of its inherent character, is different from other natural or environmental loadings, such as wind or earthquake. Truck traffic over the life of a bridge span is affected by political, economic, regional, and technological variables that are difficult to forecast. Traffic loading also generally increases over time because of regulatory and economic changes that are unknown at the time of design. Further, heavy truck traffic varies considerably from bridge site to site and from region to region. Data may be difficult to obtain because the controlling traffic load event on a bridge is very rare and usually involves trucks operating above the established legal limits. For example, at a heavy traffic site, the maximum lifetime vehicle loading effect may be the heaviest vehicle of more than 100 million individual truck events crossing the span.

In developing the AASHTO LRFD design specifications, Nowak used data from a very heavy truck population recorded in Ontario some 20 years ago (Nowak, 1999). By deriving load and resistance factors based on this unique population of vehicles, some assumptions are made that are affected by the maximum load projections. In fact, recently, the same site was used to repeat the original Ontario truck weight data acquisition, and the observations showed an increase in heavy truck load effects (Ontario General Report, 1997). Whether this increase is because of a load growth or changes in regulations or is merely a statistical fluctuation is difficult to determine.

Even with having more than 10,000 trucks weighed, the Ontario database is only a "snap shot" of that site's loading history. Uncertainties in projecting the maximum loading event for design or evaluation are still affected by heavy trucks that may have avoided the weighing operations, seasonal variations in truck data, and inaccuracies in the weighing operations. If the traffic at another site is "worse" than the Ontario weighing site, then, as discussed above, the betas computed will be lower than the proposed AASHTO target value. Conversely, for most sites with much milder traffic, the betas will be higher. Thus, the safety index reported in the AASHTO Design Specification leads to a target value based on the presence of a truck population consistent with the Ontario site. If truck traffic in general becomes more severe over time, then such load growth will consistently lead to lower betas than the calculated design betas.

For design applications, the variations in the truck weight input can be treated conservatively for economic reasons cited above, namely, the small marginal cost increases for new construction associated with higher design factors. For the bridge evaluation, however, a greater precision in describing the truck population data and resulting bridge loading effects are needed. It is also necessary when evaluating extreme loading events to reflect on the reserve capacity introduced by current design specifications.

The reasons for the existence of significant reserve strength in existing bridges are many. For example, in the case of a very heavy illegal truck crossing in Ohio several years ago, the vehicle weighed more than 550,000 lb and had traveled more than 100 miles crossing several bridges on secondary routes before being discovered by the State Police. To explain why the bridges on the route were undamaged, even with such a heavy loading, consider the inherent safety margins now present. A 60-ft span designed for an 80-kip design load (similar to HS20) has about equal design dead and live load bending moment effects. The live load factor in load factor design is 2.17, while the dead load factor is 1.3. Allowing the extra margin from the dead load to also be used to carry live load raises the live load margin of 2.17 by 0.3 to 2.47. Assuming only one lane was actually loaded when the heavy vehicle crossed, the capacity is raised by almost a factor of 2.0 to 4.94 (see Section 5.5, Distribution Factors). Further, the 30-percent impact factor present in design criteria may increase bridge capacity without really creating an overstress event (see Section 5.6, Dynamic Allowance). This further allowance raises the capacity from 4.94 to 6.42. Multiplying this factor by the 80-kip design load leads to a nominal capacity to withstand a live load of 514 kips. Considering the 1.13 strength bias used by Nowak for bending moment raises the expected bending capacity to 581 kips or more than this illegal superload weighed. Further significant increases in capacity may also arise because of section round-off and enhancements from nonstructural elements such as decks, guardrails, and sidewalks.

Before dismissing any potential distress resulting from heavy loads, consider that the actual maximum load effect event may result from the multiple presence of vehicles on a bridge and dynamic responses that are more likely to cause distress with repeated use. More importantly, many bridges are older and do not have the 80-kip design load mentioned in this example. Bridges that are older and deteriorated are definitely candidates for further distress in the event of a severe load, and carefully developed evaluation rules are needed to provide adequate uniform levels of safety for different sites and traffic conditions.

Despite the limitations stated above in the Ontario database used in calibrating the AASHTO Design Specification, the Ontario database will also be used herein for deriving the factors in the Evaluation Manual. The major reason is to provide consistency in the calibration process from the design to evaluation specifications. Adjustments in the Evaluation Manual to reflect site-specific traffic characteristics will be emphasized and considered wherever possible.

CHAPTER 3

OUTLINE OF DERIVATIONS

This report contains several analyses to meet the needs of the proposed Evaluation Manual. First, a description of the truck weight sample used by Nowak will be given. This sample population of heavy trucks will be used herein as a reference base in order to make the evaluation methodology in the AASHTO Evaluation Manual consistent with the reliability developments for the *AASHTO LRFD Design Specification.* It is important to introduce a reference truck population for evaluation. The requirements of a flexible evaluation specification is based on comparing a site-specific truck population with the reference traffic used for calibrating the rating live load factors.

The next step is to review traffic models for assessing multiple presence (i.e., the side-by-side occurrences of heavy trucks). These multiple presence events usually control the maximum live loading effect on a span. That is, the maximum loading effect on a bridge member may result from the single most heavy truck crossing (i.e., a single-lane event) or from two vehicles of lesser severity simultaneously crossing the bridge. Whether one- or two-lane loading cases govern depends on the expected maximum truck weights of one-lane and two-lane crossings and the relative values of the one- and two-lane distribution factors, g_1 and g_m. These distribution factors are discussed below using recent formulas (Zokaie, 1998) and adopted in the AASHTO Specifications (AASHTO, 1994). The new distributions show, in a relative sense, lower values for one-lane than two-lane distributions compared with previous "S/over" relationships of earlier specifications. These changes, discussed below, tend, for routine traffic, to make the multilane case govern the maximum loading effect. An exception is when one of the vehicles present on the span is a heavy permit truck vehicle.

Analyses are therefore given herein to provide maximum expected loading events for both multiple-lane and single-truck-lane events. The single-lane loading situations may control for permit loadings, low-volume bridge sites, and certain other bridge geometries (e.g., trusses). The two-lane loading governs other cases. For longer spans, platooning, or closely spaced vehicles in the same lane may also become important. Longer spans require lane load as well as the vehicle load effects, and the lane load magnitude has also been calibrated. Because the Nowak model used in the AASHTO LRFD calibrations assumed a very severe truck volume, shows how site-recorded annual daily truck traffic (ADTT) can be used to select a more optimum load factor for rating and evaluation.

The theoretical methods and the database for the calibration of live load factors for the Evaluation Manual are made consistent herein to the safety indexes (betas) generated by Nowak for the AASHTO Design Specifications (Nowak, 1999). This calibration provides a reference loading used for selecting live load factors in rating, postings, and permit checking. Further, the justification for recommending that the AASHTO legal vehicles be accepted as the nominal load model in the new Evaluation Manual is presented.

This report also provides the basis for selecting the posting curve (see Section 6 of the Evaluation Manual), which gives the allowable or posted truck weight as a function of the legal load rating factors. The basis for the recommended lower bound on ratings of 0.3 before a bridge should be considered for closure is described. Also, this report describes the use of site-specific WIM data in the selection of live load factors. These formulas are given in Section 6 of the Evaluation Manual. In addition, the basis for a method to directly use target betas for rating is discussed. This direct method is also summarized in the Evaluation Manual. The direct method is intended for very special cases and should be used only by engineers well versed in structural reliability applications.

CHAPTER **4**

TRUCK WEIGHT DISTRIBUTION

4.1 EQUIVALENT WEIGHT PARAMETERS

This section describes a reference distribution of truck weight statistics used to formulate a live load spectrum. In particular, this reference weight distribution is that proposed by Nowak and used in the calibration of the AASHTO LRFD live load factors (Nowak, 1999). This reference weight spectrum and calculation of expected maximum live load effect will be compared with site data in the calibration of live load factors in the evaluation. Subsequently in this report, truck volume data expressed by ADTT and a multiple presence model will be used to forecast the maximum loading event applicable during an evaluation time interval.

The first step in using the Nowak truck weight data is to develop equivalent statistical weight parameters in terms of the AASHTO legal vehicles, in particular the AASHTO 3S2 vehicle. The reason is that the AASHTO vehicles, rather than the HS20 or HL93 load models, form the basis of the evaluation loading model.

The data presented in Nowak's report (Nowak, 1999) are in the form of frequency distributions based on the largest 20 percent of the vehicle population. The original truck weight data taken at a site in Ontario in 1975 are not presented in the report. Instead, Nowak presented cumulative frequency distributions of bending moments for simple spans of different lengths based on the measured truck weights and dimensions. These frequency curves were obtained by finding the maximum bending moment of each Ontario truck for each span. Further, the curves, which are based on 10,000 data points, were extrapolated by Nowak to a full lifetime of some 75 million truck events, using normal distributions. The assumption of a normal variable to describe the truck weight scatter was also adopted herein.

For example, Table B.2 of the Nowak calibration report shows that for a 60-ft-long span, the mean (same as expected) maximum moment resulting from a single truck is equal to 0.72 multiplied by an HS20 moment effect. The expected maximum moment for a 1-day exposure with 1000 truck events is presented as 1.37 multiplied by an HS20 moment effect. To fit these events with the normal distribution requires the normal probability table, which relates probability level to the number of standard deviations (or variate) that a given probability value falls above the mean value. See Appendix A herein, which presents the standard normal distribution table.

For example, the corresponding normal variate for the 1/1000 level is 3.09. To convert values fitted by Nowak for the Ontario trucks with the HS20 model to the equivalent 3S2 vehicle weight parameters requires the ratio of the standard 3S2 moment effect to the HS20 moment effect. The standard 3S2 vehicle weighs 72 kips.

For example, in a 60-ft-long span, the moment of the HS20 is 403 kip-ft while the 3S2 model gives a moment of 309 kip-ft using published values (AASHTO, 1994a). Using the data in the previous paragraph, the mean truck moment for a 60-ft-long span is equal to 0.72 multiplied by 403. The AASHTO 3S2 vehicle produces a mean moment of $W/72$ multiplied by 309, where W is the mean 3S2 equivalent truck weight. Thus, solving for the AASHTO 3S2 equivalent mean weight gives the mean of the population weights (W) as

$$\text{mean, } \overline{W} = 0.72 \times \frac{403}{309} \times 72 \text{ kips} = 67.61 \text{ kips} \tag{5}$$

Similarly, the standard deviation of the truck weights (σ_W) is found from the 1-day expected maximum using the equations of the standard normal distribution as

$$\sigma_W = \frac{W_{100} - W_M}{t_{100}}$$

$$\text{or,} \quad \sigma_W = \frac{1.37 \ \frac{403}{309} \times 72 - 67.61}{3.09}$$

$$= 19.7 \text{ kips} \tag{6}$$

where σ_W = standard deviation of population weight
W_{100} = population weight at 1/1000 level
t_{100} = normal variate for 1/1000 level
W_M = mean value population

The Ontario truck weight data do not exactly describe a normal distribution, nor do the vehicles producing the maximum responses correspond exactly to 3S2 trucks. The computed values of the mean and sigma of the 3S2 equivalent weight distribution, W, will vary, depending on the span

TABLE 1 Determination of equivalent 3S2 vehicle from Nowak data

Span ft.	Average Moment* × HS20	Design Moments** HS20	3S2	Mean W kips	Standard Deviation σ_W (expected max.*) kips 1 day	1 month	75 years
40	.75	225	162	75.0	18.1(1.31)	17.8(1.46)	18.6(1.74)
60	.72	403	309	67.61	19.8(1.37)	18.8(1.52)	18.9(1.79)
80	.77	582	487	66.25	19.5(1.47)	17.9(1.60)	18.1(1.89)
100	.82	762	666	67.55	19.5(1.55)	17.8(1.68)	18.2(2.00)
120	.85	942	845	68.23	20.3(1.63)	18.3(1.76)	18.5(2.08)

*Taken from Nowak's Table B-2 (1999)
**Taken from (AASHTO, 1994a)
MEAN W and σ_W—See Eqns. 5 and 6, for example
expected one day max. = mean plus 3.09 sigma
expected one month max. = mean plus 3.99 sigma
expected 75 year max. = mean plus 5.33 sigma

chosen and the extreme period used to deduce the mean and standard deviation of the weights.

After some trial and error, as illustrated in Table 1, the upper 20 percent of the Ontario truck weight data were reasonably matched by a 3S2 population with a normal distribution and a mean of 68 kips and a standard deviation of 18 kips. These comparisons closely match the data taken from Table B-2 (Nowak, 1999). For example, matching the mean forecasted weight by Nowak with a normally distributed 3S2 produced values of 67.6, 66.3, 67.6, and 68.2 kips using spans of 60, 80, 100, and 120 ft, respectively. Examining the 1-month maximum projections by Nowak shown in Table 1, the equivalent value of σ_W is 18.8, 17.9, 17.8, and 18.3 kips for the same spans. Based on a large number of these comparisons, the mean, W, and σ_W of the equivalent 3S2 was chosen as 68 kips and 18 kips, respectively.

It should be emphasized that these weight parameters for the equivalent 3S2 vehicle fit the heaviest one-fifth of the truck weight population. It is assumed herein that the remaining trucks have no influence on the maximum loading events. This factor of "one-fifth" must be considered throughout this study in using traffic counts to extrapolate to the number of significant loading events.

4.2 MAXIMUM PROJECTED TRUCK WEIGHTS

The expected or mean maximum bending loading event using the Ontario weight data depends on the number of loading events. That is, for longer durations and higher truck volumes, the mean of the maximum single-truck loading event will increase. The extrapolation of measured traffic events to the maximum over some exposure period has received considerable study. Theoretically, this extrapolation can be done by raising the distribution of the individual events to a power corresponding to the number of events in the exposure period. This process leads to an exponential or Gumbel distribution for describing the maximum loading event. This approach has been used by Nowak as well as by researchers in the United Kingdom (Cooper, 1997), Switzerland (Bailey, 1996), and Spain (Crespo-Minguillon and Casas, 1996). It is easy to demonstrate that the COV associated with this extreme event distribution becomes quite small when the number of events is large, such as for traffic events. Typical reported results give COV values of only 1 percent to 5 percent, which is obviously too low for characterizing such an inherently random event as the maximum lifetime single truck weight event.

The reason for this picture is that the largest value sampled for a sequence of random events tends with smaller and smaller uncertainty toward the highest value present in the population sample of such events. For example, if one uses the original Ontario "raw" truck weight data of 10,000 truck events and samples from it one million times, the largest of the one million samples will almost always exactly equal the largest of the 10,000 recorded events. When one repeats the sampling of one million trucks over and over, the maximum value from each sequence is, therefore, almost always the same result or the largest recorded truck. Thus, the standard deviation (and COV) of the maximum value approaches zero. Even when the original 10,000-event sample is fit by a continuous distribution, the spread in the largest value of the million samples still approaches a narrow part of the tail of the fitted distribution with very low COV. This result creates a dilemma because a computer-based simulation of the maximum lifetime truck event shows a low COV, yet one intuitively "knows" that there is considerable uncertainty in forecasting such an extreme uncertain event as the maximum lifetime truck loading.

This dilemma must be resolved by incorporating additional modeling uncertainties in the extreme forecast. These modeling variables should include the uncertainty based on using the small sample size (10,000 in the above case); that is, every time one repeats and records a sample of 10,000 vehicles at a single site, the statistics will differ, especially the weight of the largest truck. As just stated, the largest truck recorded controls the distribution of the maximum extreme event when, for example, one million samples are drawn from the recorded sample.

More important for highway truck weight forecasting than the small sample size are the considerable site-to-site, seasonal, and other time variations in the truck weight description. These variations are not modeled in the use of only a single realization of data from one site. This limitation was recognized by Ghosn and Moses in developing their bridge loading model (Ghosn and Moses, 1986). Data for many sites were forecast independently, and this forecasting scatter was also incorporated in the COV of truck loading effects. Although not explicitly mentioned, such modeling uncertainty may also be included in Nowak's analysis of the data, because his overall COV of live load effects (which includes random variables for girder distribution analysis, truck weight occurrences, and dynamics) approaches very closely that given by Ghosn and Moses, namely a COV of about 20 percent (Nowak, 1999).

To be consistent herein with the Nowak methodology and data, the extreme event will be used to estimate the mean or expected maximum loading event in the exposure period. The corresponding COV of the maximum loading will include the modeling uncertainties just described. Forecasting the mean or expected maximum truck loading event at a site is relatively easy to perform, given that this expected maximum value closely corresponds to an individual truck weight fractile corresponding to $1/N$, where N is the number of load events in the exposure period.

For example, the traffic volume in Nowak's data is given as 1000 vehicles per day (considering only the top 1/5th of the traffic). A normal distribution is assumed for the truck weight variable. Thus, for 1 day and 1000 events, the expected maximum truck weight is the mean plus the corresponding number of standard deviations associated with a probability of 1/1000, namely 3.09. (See Appendix A for standard normal distribution table.) Using the mean weight of a 3S2 truck found above, namely, of 68 kips and a σ_W of 18 kips gives

expected 1-day maximum weight (3S2)

$$= 68 + 3.09 \times 18 = 123.62 \text{ kips} \quad (7)$$

Similarly, for the expected maximum design lifetime loading (75 years corresponds to 75 multiplied by 365 multiplied by 1000 equals 27 million heavy trucks) and for the longest permissible period between bridge evaluations (5 years equals 1.8 million heavy trucks), the expected maximum truck weights would be as follows using the normal variate terms:

expected 75-year maximum weight (3S2)

$$= 68 + 5.39 \times 18 = 165.0 \text{ kips} \quad (8)$$

expected 5-year maximum weight (3S2)

$$= 68 + 4.87 \times 18 = 155.7 \text{ kips} \quad (9)$$

Using the parameters for the 3S2, namely a mean and sigma of 68 and 18 kips respectively, and their corresponding bending moment values, the expected (mean) maximum bending moments for various return periods and different span lengths can be estimated. For example, the corresponding 1-day maximum moment for a 40-ft-long span would be (123.62/72) multiplied by 162 equals 278.1 kip-ft, where the moment on a 40-ft-long span due to a 72 kip 3S2 is 162 kip-ft (AASHTO, 1994a). From Table B-2, in Nowak's report, the 1-day maximum moment for a 40-ft-long span is given as 1.31 multiplied by HS20 moment equals 1.31 multiplied by 225 equals 294.78 kip-ft (Nowak, 1999), or about 6 percent above the value predicted from the 3S2 vehicle with mean of 68 kips and σ equals 18 kips. Several comparisons of expected or mean maximum bending moments for different spans and periods are given in Table 2 and show general agreement between

TABLE 2 Comparisons of predicted expected maximum moments using 3S2 vehicle with Nowak's simulated data (Table B-2) (Nowak, 1999)

	Time Period					
	two months Moment		two years Moment		75 years Moment	
Span FT.	Table B-2*	3S2**	Table B-2*	3S2**	Table B-2*	3S2**
40	338	319	356	335	369	349
60	629	609	661	639	681	665
80	954	960	1007	1008	1042	1048
100	1311	1313	1387	1378	1440	1433
120	1696	1616	1790	1749	1856	1818

*Equal to Coef. in Table B-2 × HS 20 moment (Nowak, 1999)
**3S2 Moment = [(68 + 18t)/ 72 kips] × 3S2 moment (AASHTO, 1994a)
where: for two months, t = 4.11; one year, t = 4.50; five years, t = 4.83.

Nowak's projections and values derived from the equivalent 3S2 vehicle. (Nowak projected his calculated bending moment spectra to very long return periods by linearly extending the graphs of the moments for each span on normal probability paper.)

These comparisons indicate that the mean value of 68 kips and a σ equals 18 kips for an equivalent 3S2 is a reasonable approximation to the Ontario truck weight statistical data. These reference values of weight intensity are utilized below to recommend adjustments in live load factors for bridge sites with available truck volume or weight data taken in WIM studies or for permit loads. Note that in Table B-18 of Nowak's report, the five-axle tractor trailers are shown with a mean of 65 kips and a COV of 0.26 (Nowak, 1999). The latter term provides a sigma of 17 kips. These values were not used herein; rather the best match for the 3S2 equivalent truck of Nowak's simulated bending moments were found to be 68 kips for the mean and 18 kips for sigma.

Other parts of the Ontario data that should be kept in mind in considering the accuracy of load projections are as follows:

- The data recorded is a 2-week sample. Any other 2-week sample would have a different outcome because of statistical variability and also seasonal influences on truck movements.
- Heavy trucks avoid static weigh stations, and the degree to which this avoidance occurred in the recorded sampling is unknown.
- Truck weights have changed over time. A repeat of the Ontario trial recently, some 20 years after the first weighings, showed increased truck weights in terms of the maximum bridge loadings (Ontario General Report, 1997).

It is for these reasons that Moses and Ghosn in their reliability analyses for design models considered both site-to-site variability and load growth as random variables that had to be incorporated in the reliability analyses (Ghosn and Moses, 1986). These two variables were not explicitly reported in the AASHTO-Nowak studies.

4.3 COMPARISONS OF SITE-SPECIFIC TRUCK WEIGHT DATA

A comparison of the Ontario truck weight data with different highway sites is now given. Rather than concentrate on the central portion of the weight distribution, which does not affect maximum loadings, the comparisons will be expressed in terms of an extreme fractile value, namely the 95th percentile of truck weight distribution. This parameter is denoted as W_{95} and is given in terms of the 3S2 AASHTO legal vehicle. In the study of Moses and Ghosn, this truck weight fractile was found by simulation studies to adequately model the severity of truck weights at a site (Ghosn and Moses, 1986). A review of a number of heavily traveled interstate sites with WIM data (Snyder, Likins, and Moses, 1985) showed the magnitude of W_{95} described as a random site-to-site variable to have a mean value of 75 kips and a 10-percent COV or a sigma value of 7.5 kips.

To compare these values with the Ontario data, the parameters found above, namely a mean of 68 kips and a sigma of 18 kips, must be used to provide the 95th percentile using the normal distribution. The parameters of the Nowak data correspond to the top one-fifth of the weight population, so that the 95th percentile of all trucks corresponds to the 75th percentile of the Nowak histogram of heavy truck weights. The corresponding variate for the 75th fractile is given in the standard normal distribution table as 0.68. Thus, the W_{95} value can be found as

$$W_{95} = \text{mean} + 0.68\sigma = 68 + 0.68 \times 18$$

$$= 80.24 \text{ kips} \tag{10}$$

That is, 95 percent of truck weights at the Ontario site weigh less than 80.25 kips (in 3S2 equivalents), while 5 percent weigh more. The next step determines how the Ontario W_{95} compares with the WIM data taken at various sites in the United States. (The WIM data were taken on behalf of an FHWA study and used sites preselected by a number of different states. No attempt was made to select a representative sample of traffic sites, although urban, rural, interstate, and primary routes were selected—all with relatively high volumes.) Using the mean and sigma from the combined W_{95} site-to-site data gives the nondimensional variate for the Ontario site as

$$t = \frac{80.24 - 75}{7.5} = 0.70 \tag{11}$$

The variate of 0.70, using the normal standard distributions table, results in a probability level of 0.76 or about a one in four probability of exceedance. In other words, the Ontario data when compared with the data cited by Ghosn and Moses from WIM sites is at a probability level that only one in four of the interstate or major route sites that were sampled have a more severe W_{95} value, or conversely, a more severe truck weight intensity. Thus, using the Ontario data as a reference for a load model provides an additional safety margin. The marginal bias is [80.24/75] or 1.07. The variability in site-to-site intensity is already included in the COV estimate of about 20 percent for live loads.

The Ontario data were used by Nowak to calibrate the AASHTO LRFD Specification to a safety index of 3.5 (Nowak, 1999). It can be concluded that, at least for the time the data were taken, the 3.5 target beta is likely to be higher at most heavy traffic sites (i.e., three out of four sites had less severe truck weight intensities). The volume of traffic selected by Nowak, namely an ADTT of 5000, also significantly exceeds most sites. The volume influence is discussed further below.

As stated above, recent Ontario truck weight data taken at the same site showed significantly higher truck weights. Whether these increased weights result from changes in regulations, laxer enforcement practices, greater allowance of permit vehicles, or simply a random occurrence event is not known. Nevertheless, use of these new data in a subsequent calibration did show lower target betas (Ontario General Report, 1997). These observations indicate a need to consider both site-to-site variability in truck weight populations and the change in weights over time.

It is, therefore, recommended in modeling the expected maximum live loading event to also consider a site-to-site random modeling variable representing the uncertainty of truck weight intensity. This variable was used in the studies of Ghosn and Moses and increases the overall load effect COV (Ghosn and Moses, 1986). Such a site variable leads to an interpretation of the safety index in the context of calibration such that the target reliability applies to the whole population of bridges. A target beta of 3.5 (which corresponds to about one in 10,000 from a normal distribution table) means that, for a population of bridges, the chance of any component failure is 1/10,000. In addition to the site-to-site variable for W_{95}, Ghosn and Moses included in their design model a load growth random variable. For an evaluation model, it is not necessary to include load growth because adjustments can be made periodically in evaluation factors based on current weight statistics. Using site-specific data in an evaluation reduces the overall COV of the maximum load estimate and, therefore, may increase the safety index, even if the average site intensity exceeds the level used in the calibration.

Despite having additional random variables of growth and site-to-site variations, the overall live load COVs reported by Ghosn and Moses (1986) are similar to those of Nowak, namely, 17 to 20 percent (Nowak, 1999). A breakdown of the main contributions to the COV are the site-to-site traffic variables (10 percent), dynamic impact random variable (10 percent), and girder distribution modeling random variable (10 percent). A comparison with similar breakdowns in the Nowak data could not be made because these values were not isolated in the calibration report. It would also be expected that the girder analysis modeling uncertainties may now be reduced with the recent introduction of the new Imbsen formulas, but this reduction has not been reported.

Based on comparisons with the AASHTO LRFD calibration truck weight data, an accurate equivalent weight distribution provides for a 3S2 AASHTO legal vehicle with a mean of 68 kips and a sigma of 18 kips. As was done by Nowak, a normal distribution of truck weights is assumed. These equivalent data will be used in the next chapter in subsequent calibrations of the live load factors for ratings. The justifications for continuing the use of the AASHTO legal vehicles for bridge rating are also presented in the next chapter.

CHAPTER 5

EVALUATION LIVE LOAD MODEL

This chapter outlines the variables that were considered in recommending a live load calibration model for the proposed AASHTO Evaluation Manual. These variables are the nominal loading model, the influence of multiple vehicle presence on a span (including random trucks), routine permits and special permits, girder distribution models, and dynamic amplification. This chapter also discusses the influences on reliability of system or on ultimate capacity and component deterioration. The chapter concludes by presenting the safety index model used for calibration of live load factors in the Evaluation Manual.

5.1 NOMINAL LIVE LOAD MODELS

Several considerations were involved in recommending the adoption in the Evaluation Manual of the AASHTO legal vehicles as the basis for the calculation of legal load bridge ratings. First, these legal vehicles are familiar to rating agencies and have been used for many years to determine if a bridge required posting for legal loads and to further select posting requirements readily understood by drivers. It was also shown in the AASHTO guide specifications for steel and concrete bridges (AASHTO, 1989) that uniform reliability over different spans could be achieved by using the legal vehicles as the nominal load effect calculation model (Moses and Verma, 1987). The AASHTO legal vehicles also have the added benefit that bridge ratings expressed as a nondimensional percentage can easily be converted to tons and reported as such in a recognized format that has been used for many years.

The AASHTO LRFD specification adopted the HL93 load model in place of the previous HS20 load model; it is observed herein that the AASHTO legal vehicles compare more uniformly with the HL93 model than with the HS load models. That is, the bending moment effects of the AASHTO legal vehicles have a relatively uniform ratio compared with HL93 for different spans. This ratio can then be accounted for in the selection of the live load factor.

Table 3 shows a comparison of bending moments in simple spans between the unfactored or nominal HL93 moments (called M_{LRFD}), HS20, and the AASHTO legal truck models. Table 3 shows that moments computed with the 3S2 vehicle for different spans have uniform ratios with M_{LRFD} with the ratio of moments close to an average value of 1.77. Over the span range of 40 to 120 ft, the moment ratios only deviate by about 2 percent from this value. On the other hand, the moment comparisons of M_{LRFD} and M_{HS20} deviate by up to 11 percent from the average value over the same span range.

Table 3 also shows comparisons of the moments using all three AASHTO legal vehicles and confirms that the AASHTO legal vehicles and the HL93 load model (M_{LRFD}) have similar moment variations with spans. In addition, Table 3 also shows that the simulated maximum 75-year lifetime moments (M75) presented by Nowak also have bias values with respect to the legal vehicles that do not significantly vary with span. This lack of variation is an indication that the truck configurations of the extreme weights in the Ontario truck sample may follow closely the configuration of the AASHTO legal vehicles. These results all suggest that, for purposes of evaluation, the AASHTO legal vehicles should correspond closely in format with the HL93 model derived by Nowak for the new LRFD specification.

The reason to continue using the AASHTO legal vehicles for evaluation rather than the HL93 model is that they are familiar to rating engineers and easily convert to tons of legal loading for reporting. The model of the legal vehicles is also easier to express in a posting format.

The recommended evaluation format in the Evaluation Manual uses the legal vehicles as the nominal live loading configuration of the trucks needed for computing the bending and shear effects. These load effects are then multiplied by the live load factors. The latter are derived from the calibration using the reliability indexes as reported herein in Chapter 6. (In this report, both 2- and 5-year evaluation intervals are discussed. A 2-year interval has been standard, but some agencies are permitting 5-year intervals in some applications.)

To gain an overview of the corresponding safety margins with the legal vehicle model, consider that the nominal HL93 is shown in Table 3 to produce moments that average 1.77 multiplied by the effect of a 3S2 vehicle (weight = 72 kips). In addition, bridges are now designed with the AASHTO LRFD live load factor of 1.75. Thus, the factored live load effect in LRFD design is 1.75 times the 3S2 configuration weighing 127.4 kips or a 3S2 weighing 223 kips. It was shown above that the expected maximum truck in a 5-year evaluation interval with the Ontario data would weigh about 156 kips. Thus, there is considerable margin between the maximum expected truck weight of 156 kips and the factored design value of 223 kips used to check components. This

TABLE 3 Comparisons of the simulated mean maximum lane moment, HL93, AASHTO legal vehicles and HS20 load models

span FT.	M_{LRFD}	M_{3S2}	$\frac{M_{LRFD}}{M_{3S2}}$	M_{HS20}	$\frac{M_{HS20}}{M_{3S2}}$	$\frac{M_{LRFD}}{M_{HS20}}$
40	588	324	1.82	450	1.39	1.31
60	1093	618	1.77	807	1.31	1.35
80	1675	974	1.72	1165	1.20	1.44
100	2323	1332	1.74	1524	1.14	1.52
120	3034	1690	1.80	1883	1.11	1.61
			ave. = 1.77		= 1.23	= 1.45

span FT.	M_{75}	M_{LEGAL}	$\frac{M_{LRFD}}{M_{LEGAL}}$	$\frac{M_{75}}{M_{LEGAL}}$	$\frac{M_{HS20}}{M_{LEGAL}}$
40	783	350	1.68	2.24	1.29
60	1444	618	1.77	2.34	1.31
80	2202	974	1.72	2.26	1.20
100	3048	1343	1.73	2.27	1.14
120	3917	1743	1.74	2.25	1.08
			ave. = 1.73		1.20

M_{LRFD}—HL93 design bending moment
M_{LEGAL}—Maximum of three AASHTO legal vehicles (AASHTO,1994a)
M_{3S2}—3S2 bending moment
M_{HS20}—Previous AASHTO load model (AASHTO, 1998)
M_{75}—Simulated mean maximum bending moment from Nowak Table B-11 (Nowak,1999).

comparison helps explain why evaluation ratings for legal loads in the proposed Evaluation Manual will significantly exceed 1.0 when checks are made for new LRFD-based designs.

In addition, the LRFD design specification (AASHTO, 1994b) considers one design vehicle loading in each of two lanes. Statistically, an event with simultaneous truck presence in multiple lanes usually corresponds to the maximum bridge loading event. Because this maximum expected event is not likely to occur with the expected maximum truck weight of 156 kips in each lane, the influence of multiple presences leads to even higher margins of safety (see Section 5.2). In addition, there are further margins of safety because of the dead load factor, the resistance and systems factors, and the resistance bias (i.e., the ratio of mean resistance to nominal strength).

Because of the considerable safety margin in design, it is concluded that there is considerable flexibility for many evaluation situations to reduce safety factors without leading to unacceptable levels of reliability. Such flexibility has long been recognized in U.S. bridge rating practices through the use of operating ratings for bridge management decision policies. The live load factors used for operating ratings are smaller than the design or inventory factors. The additional safety margins just cited make it possible to maintain in service bridges built many years ago to lower load standards or bridges that have suffered deterioration of capacity. This conclusion is particularly valid when a site's truck weight and volume intensity are not of the same magnitude considered in the design load model and when there are inspection data to justify the corresponding strength capacity.

5.1.1 Lane Loads

The relatively uniform ratio of moments of HL93 and AASHTO legal vehicles shown above are the reason why the legal loads can be used as a nominal loading model for evaluation. The comparisons in Table 3, however, do not show long spans or continuous spans in the negative moment regions. Because HL93 is the reference AASHTO LRFD load model obtained from a calibration, it was deemed advisable to adjust the legal load model for evaluation to make it uniform with the HL93 for longer spans and continuous spans. In the guide specification (AASHTO, 1989), a 200-lb/ft lane load was added to 75 percent of the AASHTO 3-3 legal vehicle to better match simulated vehicle loading events affecting longer spans.

In this study, the HL93 was used as a reference loading, and, by trial and error, a lane load was prepared and combined with the AASHTO vehicle to cover the longer spans and effects in continuous spans at the negative moment regions. The HL93 was selected as a reference because it is shown in the AASHTO LRFD design commentary that the HL93 also serves as an envelope loading to the various exclusion vehicles existing in a number of states. These "grandfather" vehicles weigh more than allowed by the federal bridge formula and are fairly common in other states as permit vehicles. Thus, selecting a lane load model to provide a uniform ratio with the exclusion vehicles seems prudent.

The lane loading in the Evaluation Manual is added to a portion of the AASHTO vehicles for longer spans and for continuous spans. In the latter case, the lane load is superimposed on a percentage of two AASHTO legal vehicles placed on each side of the support. The spacing and quantity of live load was varied in calculations performed by the research team until nominal loading produced a reasonably uniform bending moment ratio with respect to the HL93 for different continuous and long spans. The target ratio is similar to the ratios of HL93 and AASHTO legal vehicles shown in Table 3 for the simple spans. A uniform load effect ratio allows for the magnitude of the calibrated live factor to give a uniform safety margin and achieve the desired target reliability level as outlined for the simple spans.

5.2 MULTIPLE PRESENCE

The next step in the evaluation analysis after selecting the AASHTO legal vehicles as the nominal load model is the modeling of the multiple presence probabilities. These predictions relate to the expected maximum truck weights that are likely to be alongside in two lanes of a bridge. In many spans, the maximum lifetime truck loading event is likely to result from more than one vehicle on the bridge at a time. Because evaluation must consider also low-volume sites with a smaller probability of multiple presence as well as permit cases, the common assumption that truck loadings are governed by multiple presence cases will be checked herein. This check is done by comparing the expected maximum multiple presence load effect with that of the maximum effect of just a single truck present on the bridge as discussed in the previous sections.

Nowak, in the AASHTO LRFD development, assumed that side-by-side vehicle crossings occur as follows (Nowak, 1999). One out of every five trucks is a heavy truck described by the Ontario statistical parameters given above. Further, one out of every 15 heavy truck crossings occurs with two trucks side by side. In addition, of these multiple truck events on the span, one out of 30 occurrences has completely correlated weights. Thus, using the product of 1/15 and 1/30 means that approximately 1/450 crossings of a heavy truck occurs with two identical heavy vehicles alongside each other. For the design model, this assumption leads to the critical loading events either obtained from the simulations performed by Nowak or the approximations described herein. (With the above assumptions, the expected maximum lifetime loading corresponds to two heavy trucks side by side, each with a weight equal to the maximum expected truck in a 2-month interval. That is, a percentage of the total volume multiplied by the event probability of 1/450 gives a 75-year period divided by 450, or 2 months for the corresponding fractile for the expected maximum truck value. This simple analysis agrees closely with Nowak's simulation predictions.)

No field data on multiple presence probabilities and truck weight correlation, however, were provided in Nowak's study to support these truck crossing assumptions. It is necessary to reevaluate the multiple crossing probabilities because in evaluation there are many other types of situations, including low-volume roads and the presence of known permit loads. Some multiple crossing probabilities are available for validation from the studies in Ohio by Moses and Ghosn (1983). It is also assumed in the following calculations that there is no known a priori correlation between weights of trucks in adjacent lanes. This assumption is probably conservative because statistical correlation of weights in adjacent lanes may actually be negative, given that it is lighter rather than very heavy trucks that can use the passing lane.

According to Nowak's calibration of the AASHTO design LRFD for a 75-year design life, the expected maximum side-by-side loading event will be two trucks, each with a weight corresponding to the maximum vehicle weight in a 2-month exposure. Nowak's conclusion is based on computer simulations that use the Ontario data and his multiple presence assumptions stated above. The corresponding truck weights in each lane can be estimated by taking a weight fractile corresponding to the maximum of the total traffic volume divided by a factor of 450. Given that the maximum lifetime event of the total traffic corresponds to the maximum event in 75 years, the return period for the expected side-by-side event will be 2 months (i.e., 75 years divided into 450 intervals leads to a 2-month interval). Such simplified yet accurate modeling is used herein rather than simulation methods to estimate the maximum loading events for different site traffic situations.

5.3 EXTREME LOAD EVENTS

The results of the side-by-side model led Nowak to the HL93 loading with its corresponding live load factors in the AASHTO design specification (Nowak, 1999). The expected maximum 2-month truck weight event from Table B-2 of Nowak's report using the Ontario weight data is approximately 1.5 times the corresponding HS 20 loading. The 1.5 ratio varies slightly depending on the span. In terms of truck weights, the HS20 averages 1.23 times the AASHTO 3S2 in moment effects, which gives 1.23 times 72, or 88.6 kips in 3S2 equivalent (see Table 3), so that the expected maximum lifetime load event is 1.5 HS20 or 133 kips in each lane.

Although Nowak's conservative truck traffic and multiple presence assumptions may not cause a major cost impact in the design load specification (which assumes all bridge designs are exposed to a very severe loading regime), these assumptions can have a significant effect on bridge evaluation for which most bridges see a much less extreme loading in terms of truck weights and volume. Existing bridges often have been designed to lower standards than the new LRFD-HL93 criteria, and evaluation standards must recognize this fact or many such bridges will need to be posted or closed.

The cost to increase structure capacity for design is not significant. For example, raising the HS20 to HS25 level (25-percent load increase) was reported to cost less than an average of 2 percent (Moses, 1989). However, at the evaluation stage, bridges with inadequate ratings must be strengthened at a very large cost, posted (which can cause major inconveniences), or replaced.

For the evaluation criteria considered in this report, the multiple presence model will be calibrated on both observations from WIM data and supported by a more detailed traffic modeling. It will be assumed that trucks move freely in the traffic stream and that the distribution of the truck weights in the passing lane is the same as for the general traffic. This assumption is conservative. The total load on the bridge is the superposition of the individual lane loads that simultaneously occur. Only two-lane bridges are considered in this probabilistic modeling because multiple presence data for these cases are available. Further, specifications usually use reduction factors for more than two lanes, so that the two-lane case controls.

Let W_1 be the truck load in Lane 1 and W_2 be the truck load in Lane 2. The total load, W_T, is then

$$W_T = W_1 + W_2 \tag{12}$$

If each load is assumed to be independent, then, from probability theory (Thoft-Christensen and Baker, 1982) the statistical parameters of W_T mean value are

$$\overline{W_T} = 2\overline{W_1} \tag{13}$$

where $\overline{W_1}$ is the mean of W_1 and also of W_2 (equal to 68 kips above), and standard deviation,

$$\sigma_{W_T} = \sqrt{2}\,\sigma_{W_1} \tag{14}$$

where σ_{W_1} is the standard deviation of W_1 and also of W_2 (equal to 18 kips above).

For multiple presence cases, the expected or mean maximum load must be estimated by first determining the expected number of side-by-side events. That is, the expected maximum value of the random total weight, W_T, corresponds to a fractile of $1/N_{mp}$, where N_{mp} is the number of multiple presence events in the exposure period. N_{mp} is seen below as an important parameter for deriving load factors for evaluation and is a function of the ADTT value. The number of multiple presences, N_{mp}, is expressed as

$$N_{mp} = (\text{ADTT}/5) \times 365 \times \text{years} \times P_{s/s} \tag{15}$$

where ADTT is divided by 5 because the truck weight histogram provided by Nowak only contains the top one-fifth of the truck weights. The rest of the truck population is assumed to have negligible effect on maximum loadings. $P_{s/s}$ is the probability that a truck in the right lane is accompanied by another truck in the passing lane so the side-by-side crossing of the bridge causes total load superposition. For the extreme 5000 ADTT case, Nowak suggests a value for $P_{s/s}$ of 1/15 (Nowak, 1999). For a 2-year exposure, then,

$$N_{mp} = 365 \times 2 \times 1000 \times (1/15) = 48{,}667 \tag{16}$$

Using the standard normal distribution table in Appendix A gives a variate of t equal to 4.09 (i.e., 1/48667 = 0.000021, or 1 – 0.000021 = 0.999979, which shows a value of $t = 4.09$ from the standard normal distribution table). Substituting for the expected maximum truck load (W_{Tmax}) using the parameters of W_T in Equations 13 and 14 gives mean,

$$\overline{W}_{Tmax} = \overline{W}_T + t\sigma_{W_T} = 2W_1 + t\sigma_{W_T} \tag{17}$$

or

$$\overline{W}_{Tmax} = 2 \times 68 + 4.09 \times 1.414 \times 18 = 240 \text{ kips} \tag{17a}$$

This value, for $\overline{W}_{Tmax} = 240$ kips, is the expected or mean maximum loading event in 3S2 equivalents acting in both lanes during a 2-year exposure under this severe loading case. For a 75-year design exposure, the number of side-by-side events would be 1.83 million and $t = 4.87$. Substituting in Equation 17 gives the expected maximum lifetime load of

$$\overline{W}_{Tmax} = 2(68) + 4.87 \times 1.414 \times 18 = 260 \text{ kips} \tag{18}$$

This value of expected equivalent load effect in two lanes of 260 kips should be compared with Nowak's data mentioned above. Equation 8 shows that the maximum expected lifetime load effect is 165 kips for a single lane. For two lanes, Nowak uses a reduction in exposure so a vehicle with a 2-month return period will simultaneously be present for each lane. The 2-month maximum truck weight is reported by Nowak as 0.85 times the single lane expected maximum lifetime load effect acting in each lane, which gives 0.85 × 2 × 165 = 280 kips total (Nowak, 1999). This total can be compared with the 3S2 equivalent parameters by using a 2-month exposure, so that $N = 1000 \times 60$ days = 60,000, and $t(N) = 4.15$. The weight in each lane is then

$$W = 68 + 4.15 \times 18 = 143 \text{ kips} \tag{19}$$

Thus, in two lanes, the expected loading effect would be twice this value, or 286 kips, which compares well with the value of 280 kips. The difference between these estimates and the value of 260 kips shown in Equation 18 results from the modeling herein, which assumes truck weights to be independent, whereas Nowak's modeling assumes every 30th vehicle crossing corresponds to the identical heavy vehicles traveling in each lane. More important than these differences, however, are the influences in both calculations of using the exaggerated multiple presence probabilities (one in 15 being side by side) and the heavy truck spectrum present in the Ontario database.

For design applications, such expected load events, although large (i.e., 143 kips simultaneously in each lane), are well below the factored design load for HL93 mentioned above, which was 223 kips in each lane. For the AASHTO Load Factor Design (LFD) in the Standard Specification (AASHTO, 1998), the factored loads in 3S2 equivalents would be

$$\begin{aligned} \text{LFD} &= 2.17 \times 1.23 \times 3\text{S2} = 2.67 \times 3\text{S2} \\ &= 2.67 \times 72 = 192 \text{ kips per lane} \end{aligned} \tag{20}$$

where the weight of a 3S2 is 72 kips. Table 3 shows the average ratio of HS20 moment to 3S2 is 1.23. Thus, the LFD is carried out at a factored load equivalent to 192 kips in each lane compared with the 223 kips in HL93 design model. These values are both well above the expected loading event of 143 kips per lane.

For a 5-year exposure period corresponding to a maximum inspection interval, the expected one-lane maximum load is given in Equation 9 as 156 kips. Reducing this value by Nowak's two-lane reduction factor of 0.85 gives 132.6 kips in each lane. Alternatively, using the formulas for the traffic model gives a value of $N_{mp} = 122{,}000$ for 5 years, and $t = 4.30$. Substituting in Equation 17 gives the expected maximum live load effect as

$$\overline{W}_{\text{Tmax}} = 2 \times 68 + 4.30 \times 1.414 \times 18 = 245.5 \text{ kips} \tag{21}$$

This value amounts to 122.7 kips per lane, or slightly below the 132.6 value just mentioned. In current AASHTO operating ratings (AASHTO, 1994a), the load factor is only 1.3 times the AASHTO legal vehicles compared with AASHTO's 2.17 load factor times HS20 in design. The smaller load factor reduces the capacity in terms of 3S2 loading from 192 kips per lane in Equation 20 to 1.3 times 72, or only 93.6 kips per lane. The expected load event for the traffic just cited, namely 122.7 kips per lane, would exceed this capacity. For this reason, both in the AASHTO guide specifications (AASHTO, 1989) and in the proposed Evaluation Manual, the load factor proposed for rating severe traffic situations should be higher than the 1.3 value (AASHTO, 1982 and 1994a). As shown below, the Evaluation Manual recommends a live loading factor of 1.8 for the most severe traffic situations.

5.4 TRAFFIC MODEL

The previous section showed how the side-by-side probability for heavy truck crossings denoted as $P_{s/s}$ can be used to find N_{mp}, the number of multiple presence events. In turn, N_{mp} is used to find the expected maximum loading event in 3S2 equivalents during the exposure period. A probability value for $P_{s/s}$ equal to 1/15 for two trucks in side-by-side positions was provided by Nowak without reference to a database. It will be assumed herein that this magnitude of side-by-side probability is valid only for the extreme traffic situations corresponding to the 5000 ADTT value considered for the AASHTO LRFD extreme live load calibration (Nowak, 1999). No data were available to either support or refute this assumed value.

In Nowak's analysis, it is assumed that 1/15 of the "heavy" vehicle passages are accompanied by a side-by-side "heavy" vehicle event. Given that "heavy" vehicles are taken as 1/5th of the total truck population, the 1/15 assumption actually translates such that any random truck is side by side with another truck during 1/3 of all truck passages on a bridge, because there are five times as many total trucks as there are heavy trucks.

Such a multiple presence ratio, namely one out of three, is extremely high, as is also the assumption that one out of 30 passages for heavy trucks is also accompanied by a side-by-side equally heavy truck. The data provided by Moses and Ghosn from WIM tests observed multiple presence factors of only 1 to 2 percent for interstate sites, even with ADTT of above 2000 (Moses and Ghosn, 1983). Multiple presence probability is affected by traffic speed, road grade, weather, traffic obstacles, truck platooning, and other driver and truck characteristics. A simple traffic model will now be illustrated to estimate multiple presence for traffic volumes as a function of ADTT. The model shows much lower estimates of side-by-side events than either the Nowak assumptions or the observed WIM values. However, this traffic model does provide a baseline for recommending side-by-side predictions as a function of ADTT.

The $P_{s/s}$ estimates from the model below are quite low because the model ignores principally the platooning of trucks in a traffic stream. The WIM data cited will therefore be used to calibrate this model. The model assumes vehicles move freely in the traffic stream in both right and passing lanes. Assume an average truck speed of 60 mph (88 ft/sec.), and that 80 percent of the trucks are in the right lane with an average truck length of 60 ft. Assume also that trucks move freely in the passing lane. The average time between trucks in the right lane is

$$\begin{aligned} \text{Average time (seconds)} &= \frac{24 \text{ hr} \times 60 \text{ mins} \times 60 \text{ sec}}{0.80 \text{ ADTT}} \\ &= \frac{108{,}000}{\text{ADTT}} \end{aligned} \tag{22}$$

where ADTT is the truck volume. The average spacing in feet between the start of one truck to the start of the next truck in the right lane is

$$\begin{aligned} \text{Average spacing (feet)} &= 88 \text{ ft/sec} \times 108{,}000/\text{ADTT} \\ &= 9{,}504{,}000/\text{ADTT} \end{aligned} \tag{23}$$

The average number of slots between trucks, assuming an average 60-ft truck length, is

$$\text{Average number of slots between trucks} = [9{,}504{,}000/\text{ADTT}]/60$$

$$= 158{,}400/\text{ADTT} \quad (24)$$

Because trucks are assumed to move freely, there is no correlation between the presence of trucks in the right lane and the passing lanes. For every truck in the right lane, there are assumed to be 0.25 trucks in the passing lane. The expected probability, $P_{s/s}$, of a side-by-side occupancy simultaneously in the passing lane for any truck in the right lane would be the chance of a truck alongside, which is 0.25 (the ratio of trucks in the passing lane and the right lane), divided by the average number of slots between trucks in the passing lane or

$$P_{s/s} = \frac{0.25}{158{,}400/\text{ADTT}} = 1.58 \times 10^{-6} \times \text{ADTT} \quad (25)$$

Thus, for the ADTTs of 5000, 1000, and 100, $P_{s/s} = 0.008$, 0.0016 and 0.0001578, respectively. These side-by-side probabilities are much smaller than the 0.33 value given by Nowak for an ADTT of 5000 and also smaller than the multiple presence of 1 to 2 percent given by Moses and Ghosn for ADTTs of over 2000, where all trucks are considered, not just heavy trucks. A major simplification in these calculations ignores the platooning or bunching of heavy trucks. For example, if we assume trucks move in groups averaging five in number, then the values of $P_{s/s}$ should increase by a factor of 5. The predicted number of side-by-side events is still below the values presented in the Nowak report. The only field-measured $P_{s/s}$ data available pertain to the 1000 ADTT level, which can conservatively be taken as 1 percent, or about seven times the $P_{s/s}$ from this simplified traffic model.

Thus, for purposes of calibration of the Evaluation Manual, herein, the following assumptions will be used.

For ADTT equal to 5000, use Nowak's value of 1/15 for the side-by-side probability. This value maintains consistency between calibration of the Evaluation Manual and calibration of the published AASHTO LRFD design specifications.

For ADTT equal to 1000, use a multiple presence probability value equal to 1 percent. The value of 1 percent corresponds to the values found by Moses and Ghosn for even heavier traffic volume and also is conservative with respect to the simple traffic modeling just provided.

For ADTT equal to 100, use $P_{s/s}$ equal to 0.001. This value is consistent with subjective field observations and conservative with respect to the prediction model. For such low traffic volumes, platooning of trucks in the same lane should be minimal.

Corresponding to these multiple presence percentages, Table 4 presents the number of loading events and the corresponding expected maximum truck weights for both one and two lanes found from Equations 15 and 17. The one-lane case without multiple presence is also included because the maximum load effect event on some spans may arise from a one-lane loading. The number of one-lane loading events is much greater than in the two-lane case, so it is possible for the one-lane event to be more significant. The respective distribution factors (D.F.s) should be used for the one- and two-lane cases. Using the previous AASHTO distribution factors, the one-lane case would typically be a D.F. equal to S/7 compared with the two-lane D.F. of S/5.5. However, the use of the new distribution formulas in the LRFD specifications developed by Imbsen have dramatically changed the ratios of one- and two-lane distributions for most spans. This development is discussed in the next section.

Table 4 shows that, for an ADTT of 5000, the two-lane expected (mean) maximum live loading effect in both lanes (W_T) using the Ontario truck weight statistics is 240 kips. For an ADTT equal to 1000, W_T is 217.5 kips and for 100 ADTT, W_T is only 173.9 kips. These differences in W_T with ADTT will be used in Chapter 6 to adjust the live load factor for legal load rating of bridges as a function of ADTT. In Table 4, the expected maximum live loads show greater influence with ADTT for the two-lane case than for the one-lane case.

5.5 DISTRIBUTION FACTORS

It is necessary to introduce the D.F.s in assessing the maximum load events in multi-girder bridges. Recent changes in lateral distribution factors in the AASHTO specification (i.e., the Imbsen formulas) have changed the nominal girder bending moment effects for specific truck loading models. The previous AASHTO formulas or so-called "S-OVER" equations (e.g., S/5.5 for a two-lane or S/7 for a one-lane loading) have been replaced by more precise equations based on grillage analysis and reflect the span length and lateral stiffness. The new formulas should also reduce the analysis uncertainties. Although not yet carried out, this reduction in load effect COV may in future calculations be incorporated in the safety index values.

Whether the one-lane or two-lane load effects will control the evaluation rating depends on the ratio of one- and two-lane distributions and the maximum expected truck weights in the respective one- and two-lane cases. Assuming the uncertainties are the same for both single-lane and multiple-lane loadings, a lower safety index will correspond to the higher expected girder load effect. Attention must be given in these comparisons to the fact that the AASHTO specification for one-lane loads already includes a "1.2 multiple presence factor" built into the distribution factor to account for differences in expected maximum load effects. In the comparisons herein, it is assumed that the 1.2 factor is removed and the analysis is represented by the distribution factors without any built-in conservative bias. Removing the multiple presence factor allows a consistent calibration and tabulation of live load factors for all spans.

Consider an example prepared by Bala Sivakumar of a simple span, composite steel stringer bridge (65-ft span). The Imbsen distribution factor for two or more lanes is 0.631,

TABLE 4 Projections of maximum loadings using Ontario truck data

Two Year Values

		Two Lanes			**One Lane**		
ADTT	$P_{S/S}$	N	t	W_T	N	t	W_1**(kips)**
5000	1/15	48667	4.09	240.1	730,000	4.70	152.6
1000	0.01	1460	3.20	217.5	146,000	4.35	146.3
100	0.001	14.6	1.49	173.9	14,600	3.81	136.6

Five Year Values*
$t_{(ADTT)}$

	Two Lane Case		**One Lane Case**	
ADTT	N	t	N	t
5000	121,667	4.3	1,825,000	4.9
1000	3650	3.5	365,000	4.5
100	36.5	1.9	36,500	4.0

*to be used with WIM data—See Chapter Six, Eqns. 36 and 37.
For one lane, $N = 365 \times \text{ADTT} \times 2/5.0$; t corresponds to an exceedance value of $1/N$, and $W_1 = 68 + t \times 18$.
For two lane case, $N = 365 \times \text{ADTT} \times 2 \times P_{s/s}/5$; t corresponds to an exceedance value of $1/N$ and $W_T = 2 \times 68 + 1.414t \times 18$

while the distribution using the formulas for one-lane loaded is 0.46. This latter factor must be corrected by dividing by 1.2 to obtain the actual distribution factor of 0.38 without the "built-in" multiple presence factor. The "1.2" factor, which is statistical in nature, reflects the greater likelihood of heavier individual truck weights in one-lane situations, but this difference is already considered in the loading models herein. Thus, for the 5000 ADTT case in Table 4, the expected maximum loading in 2 years is 240 kips in two lanes or 120 kips per lane. Using the factor of 0.631 gives an expected maximum girder effect equal to 120 multiplied by 0.631, or 75.7 kips. For the one-lane loading case, Table 4 shows an expected maximum one-lane loading of 152.6 kips and, multiplying by the one-lane factor of 0.38, gives 58.0 kips. Thus, in this case, the two-lane load case controls the critical loading event.

For the 100 ADTT case, Table 4 shows the corresponding two-lane expected load effect on a girder as $(173.9/2) \times 0.631 = 54.9$ kips, while for one lane it is $136.6 \times 0.38 = 51.9$. These two cases are much closer. For spans, in which g_1 is closer in value to g_m, the single-lane situation does control the maximum loading event. Tables 5a and 5b present further comparisons of one- and two-lane distribution factors using analysis data prepared by Sivakumar. Also shown in Table 5b are data comparing the distributions taken from the formulas with direct grillage analysis. It is seen that the Imbsen formulas are generally conservative compared with the grillage analysis.

Comparing the ratios of g_m/g_1 shows an average of about 1.69 for the examples in Table 5b and is much higher than would be expected using the previous AASHTO distribution ratios, namely (S/7)/(S/5.5) which equals 1.27. Table 5a shows that, for the eight examples shown, the average ratio of the new and the previous AASHTO distribution factors for the two-lane case is 0.87, while for the one-lane case it is 0.65. These changes are significant for several reasons. The overall drop in distribution factors explains why bridges designed with the new LRFD load models do not have section sizes that significantly exceed existing HS20 designs, despite the more severe HL93 load model. For the purposes of evaluation, the reduction in distribution factors will raise the bridge ratings somewhat for the same checking procedures. From a reliability point of view, higher ratings are justified. The new analyses are more accurate and should have a lower scatter when comparing computed distributions with true values that would be found from field measurements.

Even more significant for the evaluation process, however, is the large reduction in the one-lane distributions for multi-girder bridges. The one-lane factors are used extensively for fatigue life calculation and for permit checking. The reduced D.F. for one lane will appear then to significantly increase fatigue lives (AASHTO, 1990) and also will allow heavier permits than existed previously.

5.6 DYNAMIC ALLOWANCE

Much attention has been given in both bridge design and evaluation to the dynamic increment of live loading occurring as a result of the inertia and dynamic response effects of a vehicle crossing a bridge span. The simplified AASHTO formulas used in the past, in which impact factor was span length

TABLE 5a Comparisons of girder distribution factors—New LRFD distribution factors vs. previous AASHTO specification (data provided by Bala Sivakumar)

	Imbsen Formula Prev. AASHTO Spec.				Ratios***	
Span (Spacing-ft)	g_m*	g_1**	g_m = S/11	g_1 = S/14	**Multi Lane**	**Single Lane**
1. 65′ steel stringer (7.33)	.631	.383	.666	.524	.95	.73
2. 80′ composite steel (8.0)	.595	.355	.727	.571	.82	.62
3. 125′ composite steel (7.83)	.635	.365	.712	.559	.89	.65
4. 80′ prestressed I-girder (8.5)	.724	.428	.773	.607	.94	.71
5. 50′ Reinf. conc. T-beam (7.83)	.727	.452	.712	.559	1.02	.81
6. 120′ prestressed I-girder (9.0)	.732	.416	.818	.643	.89	.65
7. 161′ composite steel (13)	.840	.448	1.182	.929	.71	.48
8. 90′–90′ Cont.steel I-girder (10)	.698	.403	.909	.714	.77	.57
					Average = .87	.65

*multi-lane distribution in terms of truck load effect per girder based on new Imbsen formulas (Zokaie)
**one-lane distribution in terms of truck load effect per girder based on new Imbsen formulas without 1.2 multiple presence factor
***Ratio = Imbsen D.F./Previous AASHTO D.F.

TABLE 5b Comparisons of girder distribution factors (data provided by Bala Sivakumar)

	Imbsen Formula			Grillage Analysis		
Span (Spacing-ft)	g_m*	g_1**	g_m/g_1	g_m*	g_1**	g_m/g_1
[General*(S)	S/11	S/14	1.27-Previous AASHTO Spec.]			
1. 65′ steel stringer (7.33)	.631	.383	1.65	.61	.35	1.73
2. 80′ composite steel (8.0)	.595	.355	1.68	.65	.35	1.90
3. 125′ composite steel (7.83)	.635	.365	1.74	.62	.30	2.07
4. 80′ prestressed I-girder (8.5)	.724	.428	1.69	.68	.35	1.95
5. 70′ prestressed adj. boxes (4)	.272	.155	1.76	.35	.18	2.0
6. 50′ Reinf. conc. T-beam (7.83)	.727	.452	1.61	.68	.40	1.71
		average =	1.69			

S = girder spacing
*multi-lane distribution truck load per girder based on Imbsen formulas
**one-lane distribution truck load per girder without 1.2 multiple presence factor

dependent, have given rise in some specifications to detailed dynamic analyses of bridge frequencies and dynamic models of various truck configurations. The most significant contributor to dynamic response was found to result from irregular bridge deck surface and from the bump at the end of the bridge (Moses and Ghosn, 1983). Both phenomena are difficult to forecast during design, and, in such cases, the dynamic impact is not heavily dependent on vibrational characteristics of either the bridge or the trucks.

Some errors by researchers in interpreting past bridge test results also led to greater than necessary concern for the dynamic portion of loading. Several researchers used electronic or numerical filtering techniques to estimate the dynamic response portion of strain measurements (Bailey, 1996). Although this approach is accurate for longer spans, such approaches give erroneous results for short and medium spans because the spacing of the axles creates a static oscillation in the bridge strain record. This is especially true where the truck axle groups, rather than gross weight, play the major part in the strain record; the former will have peaks and valleys corresponding to when each axle group is over the center of the span. These maxima and minima in the strain record are not the result of dynamics and would appear even at crawl speeds. Filtering methods may erroneously identify this static oscillation as a dynamic response, when, in fact, it is purely a static response.

Careful examination of strain records is necessary in order to sort out true dynamic bridge vibration from such static oscillations resulting from axle spacings. When this more careful but subjective approach is used to measure the dynamic impacts in the field, the dynamic responses become smaller than had been reported previously.

It is generally recognized that the deck surface influences any dynamic characteristics that are present. A truck crossing a smooth surface will cause little dynamic response to the bridge. Because of this influence, the Evaluation Manual allows impact values ranging from 0.1 to 0.3 based on deck condition, which may be characterized during the inspection. For design, a larger value of impact is appropriate because deck condition is not known. Consequently, several specifications, including the Ontario Bridge code (Ontario Highway Bridge Design Code, 1993), have simplified the dynamic response calculations and removed the formulas that equate impact to bridge frequency and other dynamic parameters.

Dynamic response is important for the fatigue portion of loading because it affects every truck crossing. The assessment of the lifetime maximum loading event, however, is less dependent on bridge dynamics for several reasons:

1. It is known from bridge test data and analysis that the impact percentage decreases with heavier truck weights.
2. Further, the maximum truck loading event is usually associated with a multiple-presence crossing, and the overall dynamic load as a percent of static effect reduces even more because the dynamic responses of the bridge resulting from each truck will not likely coincide.
3. The increased stress response because of dynamic behavior exists for a very short time interval, perhaps a fraction of a second depending on bridge frequency. High strain rates of loading simultaneously cause an increase in the yield point of steels, which offsets part of the dynamic portion of the response record.
4. If dynamic response were to cause a bridge to exceed the yield limit, the dynamic response would be further controlled because calculations show that bridge damping is an important factor in dynamic behavior and material yielding significantly raises the damping level.
5. Slight changes in positioning a vehicle laterally on the bridge may lead to changes in peak strain observed that more than offset the contribution resulting from dynamic effects. This effect of transverse vehicle placement has also made it difficult to measure dynamic strains in field tests.

In summary, dynamic response modeling may often be overestimated. For design, specifying a constant value of impact for all bridges (e.g., 33 percent for the HL93 vehicle portion of the loading in AASHTO) is sufficiently accurate and serves simply to further increase the overall safety margin. This increase in safety margin is important, especially where pressures to increase legal loads have made the subject of justifying the existing safety margins germane. Accurate dynamic response of bridges is more relevant in the evaluation process, where the condition of the approach and the deck surface can be inspected. In the proposed Evaluation Manual, the impact varies from 0.1 to 0.3, depending on the roadway surface condition, and is the approach contained in the AASHTO Guide Specification (AASHTO, 1989). See *NCHRP Report 301* for further explanation of these values (Moses, 1989). The added load effect of an impact factor in the checking formulas in the Evaluation Manual may lead to a recommendation for improving the deck surface in order to raise the ratings. This action will have a positive effect on the bridge's durability and fatigue life. For this reason, simplified impact formulas, such as those suggested herein, which are fixed percentages based on approach and deck condition, are warranted.

Exhaustive surface measurements of deck elevations, accompanied by dynamic analysis of the bridge for various truck types, are usually not warranted for the reasons discussed above. As stated in the Evaluation Manual, bridge measurements can be made to estimate dynamic responses. However, special care must be taken in such a study and the discussion cited above must be fully taken into consideration. These considerations include use of heavy trucks for the test vehicles, careful positioning of vehicles to obtain strain repeatability, and accurate interpretation of strain records to estimate dynamic values. (See also NCHRP, 1998)

5.7 STRUCTURE SYSTEM CAPACITY AND MEMBER CONDITION

In LRFD design specifications, the nominal calculated resistance is multiplied by a resistance factor, ϕ, associated with a given type of component. The resistance factor depends on the resistance bias (i.e., ratio of mean to nominal strength value) and the corresponding component strength uncertainty (i.e., COV). The higher the bias, the larger will be the factor ϕ, while a greater resistance uncertainty reduces ϕ. The values of ϕ are found during the calibration process and depend also on the target safety index.

In the proposed Evaluation Manual, the component resistance capacity is further factored by a system factor ϕ_s and a member condition factor, ϕ_c, in addition to the resistance factor for the component type, ϕ. These additional two factors are now described.

5.7.1 System Factor, ϕ_s

The proposed Evaluation Manual recognizes that a span's ultimate strength, rather than solely the component strength, should be the basis for load rating. System redundancy was considered in fixing the rating criteria for the AASHTO guide specifications (AASHTO, 1989, and Moses and Verma, 1987). There are several reasons for including system reserve or ultimate bridge capacity in the rating. First, system reserve helps to justify the reliability targets inherent in the AASHTO operating stress levels. As pointed out above, safety indexes are calculated for individual component or member limit states. If the inventory or design reliability index corresponds to a beta of 3.5, the operating betas will be lower on a comparative scale. These indexes were found in the NCHRP 12-28(1) study (Moses and Verma, 1987) to average around 2.5 for working stress and 2.3 for load factor rating. Such lower beta values for rating, if formally interpreted as actuarial risks, are inconsistent with the needs for public safety. Furthermore, observed bridge failure rates surely indicate higher safety indexes than implied by the values calculated for operating ratings for members.

The explanation offered in the NCHRP 12-28(1) study was that system or ultimate strength betas were higher than component betas, typically by an increase in β of 1.0. Allowing operating levels of rating only for spans with known redundancy would then ensure that operating ratings of 2.5 for components actually implied a system beta of 3.5, which represents adequate reliability. In the AASHTO guide specification (AASHTO, 1989), the definition of redundancy was left to the engineer and the wording was simply the same as appearing in the AASHTO design specifications under the fatigue section (AASHTO, 1998).

In the proposed Evaluation Manual, the quantification of redundancy is excerpted from the recent study by Ghosn and Moses appearing in *NCHRP Report 406*, which defines redundancy more carefully and considers the details of member and span geometry. On a comparable scale, bridges that satisfy the legal load rating requirement will have safety indexes against system failure closer to 3.5, rather than the operating member target beta of 2.5. (Again, these values are notional values and are not to be interpreted actuarially.) The system contributions are incorporated in the Evaluation Manual in the rating equation through the value of ϕ_s.

Values of ϕ_s below 1.0 show a limited redundancy, while values above 1.0 mean a highly redundant geometry. *NCHRP Report 406* also shows how a direct analysis for a specific span can be carried out to find the system factor. The use of system properties in the evaluations leads to more uniform safety indexes among different spans with respect to collapse. The evaluation does not consider service levels of performance in defining requirements for system redundancy—only ultimate strength.

As computer programs capable of performing nonlinear structural analysis become more readily available, it is expected that bridge evaluation engineers will develop even more precise expressions for ultimate strength. Nonlinear analysis, for example, has become routine in evaluating existing steel offshore oil platforms using analyses of three-dimensional rigid frameworks with flexible foundations under static, wind, wave, current, and earthquake loadings (Kriger et al., 1994). The redundancy calibration (Ghosn and Moses, 1998) for bridges established criteria (based on reliability methods) to enable the results of such nonlinear analyses to be readily incorporated in the bridge-rating process.

The system factors provided in the Evaluation Manual taken from *NCHRP Report 406* (Ghosn and Moses, 1998) are intended for bending moment checks. System factors for shear are not provided, and a uniform value of $\phi_s = 1.0$ should be used for checking shear. Shear failures in members are often brittle, so the presence in a system of parallel members may not provide added shear capacity through redundancy. Bending failures are usually ductile, so redundancy does add to system bending capacity.

In many LRFD design specifications, the target beta for brittle type modes, such as shear or buckling, are made higher than for ductile modes, such as bending, so that brittle type failures are not likely to control the structure reliability. It is not clear whether higher targets for shear were prescribed for the AASHTO LRFD design specifications, although use of conservative shear formulas for concrete members has been reported. In principle, the target beta level for any brittle type limit state, such as shear, should exceed the target system beta for bending, which was used to calibrate the system factors.

5.7.2 Member Condition Factor, ϕ_c

The proposed Evaluation Manual incorporates a condition factor, ϕ_c, based on member condition at the time of inspection. The value of ϕ_c decreases with increasing member deterioration: This approach follows the same recommendations given in the AASHTO guide specification derived in NCHRP Project 12-28(1) (Moses and Verma, 1987). The influence of member deterioration is to reduce the component's legal load rating. The condition factor is not intended to replace adequate inspection data. The nominal resistance, R_n, should be the best estimate of component strength, including section loss and other strength deteriorations. However, there should not be a double penalty applied to deteriorated sections such that a conservative estimate of section loss and a member condition factor both reduce the factored strength in the rating check.

The factor, ϕ_c, given in the Evaluation Manual is meant to recognize that for a deteriorated section, there is greater uncertainty in estimating the true strength than for an as-built member. As stated above, the resistance factor, ϕ, should decrease with larger component strength uncertainties. In addition to greater strength uncertainties, ϕ_c accounts for possible further deterioration that may occur over the next inspection cycle. Data to support the recommended values of ϕ_c are given in *NCHRP Report 301*. The aim is to select a value of ϕ_c that keeps the safety index for deteriorated components at the same level as the target safety index adopted in the calibration of the evaluation factors.

5.8 SAFETY INDEX EXPRESSIONS

The previous sections discussed the variables in the reliability modeling needed for calibration. This section elaborates on the safety index models for the calibration of live load factors in rating and permit processing.

In general, the statistical parameters describing the resistance random variables are independent of the loading description. The resistance uncertainties are the same values given by

Nowak for the AASHTO LRFD design calibration, except for the system and condition factors described in the previous section.

The dead load uncertainty is also the same given by Nowak, except that the uncertainty of the overlay portion of the dead load need not be increased—provided adequate detailed field inspection of overlay thickness is carried out.

In describing maximum bridge live load, the uncertainty of the truck loading depends in turn on the uncertainty of the lateral distribution analysis, the uncertainty of the dynamic amplification, the scatter of the truck weight spectrum in a site-to-site comparison, and the randomness associated with an extreme loading event simulated from vehicles chosen from the truck weight spectrum. This live load characterization suggests that the overall COV of the maximum live load effect random variable remains relatively constant for different exposure periods and traffic ADTT. Thus, the load COV parameters given by Nowak may also be used in the evaluation calibration. The emphasis herein therefore focuses on the expected (mean) maximum live load effect and its dependence on site-specific data.

In a properly calibrated LRFD format, the dead load, live load, and resistance factors are chosen so that the calculated safety index closely approaches the target reliability index. This selection process means that the same beta is produced for different components and span lengths. Nowak, in fact, calibrated the AASHTO LRFD design factors over a wide range of spans, including short spans where the live load effect is significantly greater than the dead load effect in the girders.

The load and resistance factors calibrated by Nowak are also robust in nature. That is, even if some of the input data to the safety index model (e.g., load and resistance bias, COVs, and distribution functions) were changed, the load and resistance factors selected by Nowak would still lead to uniform safety indexes for different components and spans. That is, the average betas with the LRFD format would still agree with the average beta for the sample designs used to fix the target beta. Of course, changes in input data would change the average beta, but the uniformity of betas and consistency with past practice would still be observed. This situation is valid because the reference for the calibration target is not an a priori safety index, but rather a target deduced from the bridge sample of past practice. See Nowak (1999) Appendix E. Changes in statistical parameters by a small amount will still produce uniform betas, even if the target changes. This robustness is a characteristic of LRFD calibration and is discussed above. [See also an example of this robustness in the calibration of the API LRFD Specification for Offshore Structures (Moses and Larrabee, 1988).]

Thus, for the purpose of consistency between the design and evaluation live load factors, the beta calculations do not have to be repeated for each calibration of the evaluation factors. Uniform betas will be achieved in evaluation by adjusting the live load factor in direct proportion to the calculation of the expected maximum traffic live load based on site data. This adjustment is analogous to a change in the bias parameter of the live load. To simplify this calibration for different live load effects, the case where the live load effect dominates over dead load effects will be used as a reference case for modifying the live load factor for the various other evaluation options required.

To illustrate this calibration for a component with live load only, Equation 4 reduces to

$$\phi R_{\mathrm{n}} = \gamma_L L_{\mathrm{n}} \tag{26}$$

The safety index may be derived from the margin of safety expression, Equation 3. To illustrate the calibration of γ_L, variables L and R are both approximated as lognormal distributed random variables. (See other LRFD formats such as AISC-LRFD, in which the lognormal format for the safety index is widely used in structural reliability methods.) The expression for beta is

$$\beta = \frac{L_{\mathrm{n}}[\bar{R}/\bar{L}]}{[V_R^2 + V_L^2]^{1/2}} \tag{27}$$

where the numerator is the mean safety margin and the bar on top denotes a mean value and the capital V denotes the COV (Thoft-Christensen and Baker, 1982).

If V_R and V_L are now assumed to be equal to the values assumed by Nowak, then beta depends only on the ratio ($\bar{R}/\bar{L}$), which is the mean safety factor or the mean resistance capacity divided by the mean or expected maximum live load effect. The maximum load will be found below from the traffic model and truck weight database and depends on ADTT. Equation 27 shows that the required live load factor needed to reach the target safety index should be in direct proportion to the value of the expected maximum live load effect. For smaller expected live loads, the live load factor can be reduced, while, for larger expected live loads, the live load factor should be increased.

Using Equation 26 and the definition of the bias, namely mean divided by nominal value, the expression for the mean safety margin is

$$[\bar{R}/\bar{L}] = \frac{B_R B_n}{\bar{L}} = B_R \frac{\gamma_L L_n}{\phi \bar{L}} = \frac{B_R \gamma_L (W_n) g}{\phi \bar{L}} \tag{28}$$

where

B_R is the component resistance bias (i.e., mean divided by nominal value),
γ_L is the live load factor,
W_n is the nominal weight of the rating vehicle,
g is the lateral distribution factor for the girder being checked,
ϕ is the resistance factor, and
L is the expected mean maximum live load.

From Equation 28, the mean safety margin remains constant, and thus β remains constant if γ_L is fixed in direct proportion to the expected mean maximum live load effect (L). This relationship is used throughout the remaining part of this report to calibrate the live load factors. That is, γ_L is selected for the various rating and permit cases so that it is directly proportional to the calculated expected maximum mean live load effect. A reference case to fix this relationship of γ_L to achieve the target beta is given in Chapter 6.

Thus, for higher mean values of the expected live loading, there will be proportionately a higher live load factor required to maintain the target safety index. Similarly, if the mean live load effect is reduced, then the live load factor can be reduced accordingly. This relationship will be used in the next sections to adjust live load factors for different site parameters, such as ADTT- and WIM-based data, and for routine and special permit checks.

CHAPTER **6**

CALIBRATION OF EVALUATION FACTORS

6.1 REFERENCE CRITERIA FOR CALIBRATION

This chapter presents the Evaluation Manual's method of calibration of live load factors in the context of LRFD- or reliability-based specifications. This calibration is carried out with the safety index concept and corresponding models on load and resistances described above.

In general, load and resistance factors may be calibrated in a number of distinct ways. In the AISC-LRFD (AISC, 1996) or API-LRFD (API, 1992) texts, for example, each resistance component (e.g., member in bending or type of connection) was calibrated separately with its own unique component target beta. A component-by-component calibration means that the average beta for each component with the new LRFD load and resistance factors equals the average beta for that component type deduced using the sample of bridge members designed with the existing design working stress design (WSD) factors.

The advantage of the LRFD format is that, after calibration, there should be much smaller deviations in computed betas from the target beta compared with the betas with the previous format based on WSD rules. The Evaluation Manual benefits even more from the uniformity inherent in betas in the LRFD format because of the wide variety of site-specific cases that must be calibrated.

In a second approach to calibration of betas, all the components of a certain group (e.g., all steel members in bending, compression, or shear) are simultaneously calibrated with a single common target safety index extracted from the existing design sample for these same components. Different targets, however, may apply to connections, foundations, or other materials. These different targets may be based on consideration of the subjective marginal economic costs of higher safety factors, the consequences of component failure, or traditional design practices (e.g., specifying that a connection shall be stronger than the member).

In the third approach, a single target beta is prescribed for all components. This target may be extracted from safety index calculations using the existing design sample and experience represented by the WSD format.

In the fourth method, a set of LRFD factors is calibrated using a target beta based on actuarial risk values and economic criteria. This fourth approach suggests that the betas and the corresponding risk values are actuarially correct and that design standards should balance explicitly the construction costs and consequences of failure. Such an approach requires not only a model for assessing risks, but also a consequence cost model reflecting the damages resulting from overloads. The optimum risk (and corresponding safety factors) occurs when marginal cost increases for higher safety factors just balance the reduction of risk costs equal to the cost of failure multiplied by the probability of failure.

Last, a fifth calibration approach is to determine a table of LRFD factors, which are based on a full range of system and component failure consequences. These consequences are expressed in terms of damage cost and also the marginal cost factors required to increase component reliability. This last approach, which essentially is an optimization of costs and consequence risk, also considers the importance of the bridge failure in terms of life and property loss.

The recommended approach herein for calibration of the live load factors in the Evaluation Manual is described as follows. The approach is suitable for an evaluation manual that requires flexibility in specifying load factors to reflect different site conditions and the amount of site data retrieved. The issues of marginal costs and optimal target risks are considered, but not explicitly. Because a wide variety of engineers in different rating organizations must use the same rules, the evaluation format must be clear and unambiguous and lead to similar results by different investigators. Furthermore, the AASHTO LRFD Evaluation Manual must relate back to the AASHTO LRFD design specifications and provide a clear relationship of the reliabilities and design margins in current design practices with those in evaluation.

To simplify the calibration criteria for the Evaluation Manual, it will be assumed that the calibration carried out by Nowak for the AASHTO-LRFD design rules, namely the dead and live load factors and resistance factors, along with the expressions for nominal load and resistances, does produce a consistent and uniform reliability format. The AASHTO-LRFD calibration reported a target reliability index of 3.5 (Nowak, 1999). As pointed out above, this index applies to a very severe traffic loading case. Furthermore, the target index is actually achieved with a live load factor somewhat smaller than the 1.75 live load factor recommended in the AASHTO LRFD load factor table. Adjustments to develop an evaluation-oriented live load factor will further be made herein to consistently reflect the site realities and economic considerations of evaluation criteria versus design criteria.

The ADTT value at a site, usually known from traffic planners, was shown above to be an important variable in calculating the expected maximum live load effect. Chapter 5 shows that the required live load factor produces a target beta that depends directly on the expected or mean maximum live load effect. Using the terms and assumptions given in Chapter 5, the live load factors will be adjusted from a reference case to satisfy a fixed target safety index (see Equation 28). The reference case will be the severe traffic loading case in the LRFD design calibration. Thus,

$$\gamma_L = \text{expected maximum live load effect} \times \gamma_{L,\,\text{ref}} \div \text{reference expected live load effect} \qquad (29)$$

where

$\gamma_{L,\,\text{ref}}$ = reference live load factor

Substituting for the reference expected live load effect, which for two lanes and 5000 ADTT was found to be 240 kips (in 3S2 equivalents), and the corresponding live load factor derived in Section 6.2.2, which is 1.8, gives the general expression for the evaluation live load factor for any specific application as

$$\gamma_L = \begin{array}{c}\text{expected maximum}\\ \text{live load effect}\end{array} \left[\frac{1.8}{240\text{ kips}}\right] \qquad (30)$$

Equation 30 ignores the relatively small influence on the safety index that occurs when there is change in the nominal dead-to-live-load ratios. This ratio changes in design applications for different span lengths. For evaluation, the ratio also changes for different site conditions. However, if a checking format is properly calibrated, as was done by Nowak for the AASHTO Design LRFD, then this influence on safety index for different spans and dead-to-live-load ratios is usually small. [See Figures F-4 to F-10 (Nowak, 1999)].

For the very heavy 5000 ADTT traffic case, the most severe load effect was given in Equation 17 as 240 kips, equivalent to two 120-kip vehicles moving side by side. In addition, Equation 30 assumes that the COV of strength capacity and load effects are constant for all applications. The recommended live load factors for evaluation are, therefore, consistent with the data used in calibrating the AASHTO-LRFD specification.

To obtain the reference live load factor of 1.8 shown in Equation 30 for the most severe traffic case, the live load factors for evaluation are adjusted from the design live load factor to account for the following considerations:

1. The exposure period in evaluation is different than the exposure period for design. The shorter period reduces the expected maximum live load effect given the same truck weight population and ADTT.
2. The target reliability for evaluation should be reduced compared with the target selected for design. This reduction in reliability target is a consequence of economic considerations and of the lower cost of increased design load factors compared with the relatively high cost of postings, rehabilitation, or replacements caused by inadequate ratings. These economic issues have long been reflected in the AASHTO rating philosophy, through less conservative safety margins for the operating rating levels and by ignoring some service limits in rating.
3. It is unnecessary in rating to include any deliberate conservatism that was added to the LRFD load factors. Such “rounding-up” is clearly mentioned in the AASHTO calibration report (Nowak, 1999) and may reflect a need to account for possible future load growth that is not explicitly modeled in the calibration. Again, the costs associated with such “round-up” is small. It was not deemed appropriate to refer to the specifications in future load growth cushions because the presence of these allowances may only serve to increase the pressures to raise truck weight limits. For purposes of evaluating existing spans, the load growth is not relevant, and the “round-up” allowance should be removed because of its cost.
4. In addition to other items affecting the load factors, there must be a conversion of the design factors, which are based on the HL93 load model, to the evaluation rating model, which uses the AASHTO legal vehicles.
5. Any site information known at the time of evaluation should be used. During evaluation, the site traffic ADTT is available, along with other parameters of the truck traffic. The performance of the bridge under traffic loadings, including deflections and other service-related observations, may be known. In addition, the construction details (such as mill reports and as-built dimensions) are known. Because the calibration report (Nowak, 1999) did not explicitly spell out the individual contributions in the overall estimate of approximately 20 percent for the maximum live load effect COV, the influence of each item of information that becomes available in the evaluation cannot be explicitly used. Nevertheless, the added knowledge of the bridge performance at the time of evaluation should be used in a qualitative manner to avoid requiring excessive conservatism in evaluation compared with the design practice.

These influences on the live load factors are considered in recommending the live load factors for rating. The basic approach fixes the live load factor for the most severe reference traffic case and adjusts the live load factors based on the corresponding expected maximum live loading events. This approach will ensure a consistency between LRFD design criteria and evaluation criteria. In Chapter 7, the approach will be extended to permit truck evaluations.

6.2 RECOMMENDED LIVE LOAD FACTORS FOR RATING

6.2.1 Design Load Check

In the proposed LRFD Evaluation Manual, the inventory rating uses the live load factor of 1.75 applied to the HL93 nominal loading. This load check is the same as the current AASHTO LRFD live load design requirement and allows for a comparison of the bridge capacity at the time of rating with new bridges at the time of construction. The site inspection data affecting this check would be field information, which may change the resistance part of the analysis because of deterioration, and possibly also the deck condition, which would influence the overlay thickness estimate and the dynamic impact factor.

In addition, to provide some relative comparison with present operating levels, a check with a live load factor of 1.35 is also prescribed. This check is applied to the HL93 nominal loading. This “operating” check for the design load may serve some agencies as a possible screening criterion to determine which spans require more intensive site data analysis for the legal load checking. The reduction of live load factors from 1.75 to 1.35 reduces the safety index, as described in the following paragraphs. The index for design is given as 3.5, while a target in the range of 2.3 to 2.5 (lifetime loading) corresponds to AASHTO’s existing operating criteria.

In the study for the AASHTO guide specification (*NCHRP Report 301*), it was found that the safety index, based on the working stress operating criteria, was 2.5 and, based on the load factor rating, the average safety index was 2.3. In *NCHRP Report 301* (Moses and Verma, 1987), the 2.5 target was used, while in the final recommendations for the guide specification, based on panel and other inputs, the 2.3 target beta was used. In AASHTO’s Condition Evaluation Manual (AASHTO, 1994a), for the load factor rating provisions, the live load factor for rating is reduced by a factor of 5/3, to 1.30, compared with the design load factor of 2.17.

Readers should note the discussion given above regarding the meaning of safety index and the notional probabilities of failure. The betas are actually parameters to assist in the calibration of uniform reliability and should not be interpreted in actuarial failure probability terms. As new data and analysis tools become available, the uncertainties will change, and, hence, the betas will change. However, the load factors calibrated herein based on direct correlation to performance criteria with historical AASHTO rating criteria may remain fixed. Because of the inherent robustness of the calibration procedure, the benefits of new data and analysis to determine accurate actuarial failure rates will become apparent only when evaluation methods directly take on explicit risk cost and benefit methods in the context of optimizing ratings.

TABLE 6 Statistics for safety index computations

Case	Bias	COV	Distribution
Dead Load	1.04	0.08	Normal
Live Load	1.00	0.18	Lognormal
Resistance	1.12	0.10	Normal

In order to provide a screening level for the “operating” case that is conservative, the 2.5 target beta will be used. A drop from a design target index of 3.5 to a screening target of 2.5 represents about one order of magnitude increase in the notional risk of failure. Note, however, that failure refers to a component and not a system and is biased here toward a severe traffic spectrum. To compare the inventory (beta = 3.5) criteria and operating (beta = 2.5) criteria, a study was made of live load factors for different target safety indexes, using a database similar to Nowak’s (Nowak, 1999). The statistics used are provided in Table 6.

A live load bias of 1.0 and COV of 0.18 for the live load is in the range of Nowak’s data. As shown in Chapter 4, the expected maximum load effect in terms of an AASHTO 3S2 vehicle is 260 kips using the traffic modeling herein. Dividing the total load by two provides an estimate of the expected maximum single-lane load equal to 130 kips. The nominal HL93 load, also in terms of a 3S2 vehicle is shown above as 1.77 multiplied by the 3S2 vehicle, or (1.77 multiplied by 72 kips) equal to 127.4 kips per lane. Using this value gives a bias [i.e., mean divided by nominal effect, close to 1.0 (i.e. 130/127.4)].

Using these data, the safety indexes were computed over a wide range of nominal live-to-dead-load ratios that typify short- to long-span bridges, namely, live/dead ratios from 0.50 to 2.0 The calculated safety indexes are similar to those reported by Nowak (Nowak, 1999), but do differ because of respective models and methods of computing betas. For the target of 3.5, the required live load factors ranged between 1.65 and 1.77, depending on the live-to-dead-load ratio. For the target of 2.5, the required live load factors ranged between 1.28 and 1.35. Thus, a conservative live load factor for screening equal to 1.35 was recommended for an operating check using the HL93 loading. In most spans, the ratings will increase from the operating design load check (Section 6.4.3 of the Evaluation Manual) to the ratings obtained for the legal loads (Section 6.4.4 of the Evaluation Manual).

6.2.2 Legal Load Ratings

The live load factors for legal load rating represent the most important factors calibrated in this rating process. These factors will determine if a bridge has adequate capacity to safely carry legal loads such as the AASHTO legal vehicles derived from the federal bridge formula.

A single set of legal load ratings is obtained using the Evaluation Manual procedures. This situation is unlike previous AASHTO bridge evaluation practices, which provided both inventory and operating rating levels. The nominal loading will be the AASHTO legal trucks or other legal vehicles operating in the respective jurisdictions. The target reliability for calibrating the rating factors are similar, on the average, to target reliability using the historical AASHTO operating levels of safety. To ensure adequate safety against collapse, the system factors in the Evaluation Manual are adjusted so that statically determinant members will have increased safety indexes approaching those of the traditional inventory or design level.

The live load factors for legal load rating recommended in the Evaluation Manual are not intended for rating "legal" vehicles whose weights and dimensions significantly exceed the federal bridge formula because of "grandfather" exemptions. Such vehicles should be checked as routine permits using the load factors derived in Chapter 7 of this report.

The calibration of the legal load factors begins with the design-factored loading of 1.75 multiplied by the HL93 nominal load model and then makes several adjustments to reach the recommended values for rating. First, consider the most severe evaluation traffic category for routine traffic, namely the 5000 ADTT. The proposed live load factor for rating in this case closely agrees with the recommended AASHTO guide specification for the most severe load category, namely a specified live load factor of 1.8 multiplied by the nominal AASHTO legal load effects.

The adjustments start with the nominal HL93 load effects multiplied by the design live load factor of 1.75. Adjust the live load factor for evaluation as follows:

- Reduce the 1.75 to 1.6 since Nowak states that 1.6 is acceptable on the average (see Tables F-4 and F-5 in Nowak report), but the specification adopted 1.75 to be "more conservative." Using 1.6 reduces the live load factor by a ratio of 0.91 (i.e., 1.6/1.75). The target safety index is meant to be an average value for different span geometries and not a lower bound on the safety level. LRFD factors are implicitly selected to meet average beta targets and do not satisfy the target beta for every span. Procedures to accomplish such a task require an iterative selection of load factors and are referred to as Level II design procedures in the reliability literature (Melchers, 1987).
- Reduce the design target beta level from 3.5 to the corresponding operating level of 2.5 mentioned above in connection with the design load check. Several parametric analyses mentioned in Section 6.2.1 above indicate this reduction in beta corresponds to a reduced load factor ratio of about 0.76 (i.e., 1.35 divided by 1.75).
- Reduce the live load factor to account for a 5-year, instead of a 75-year, exposure. Using Nowak's projections of expected maximum load effect for different durations (Table B.14) produces a reduction of roughly 0.94.
- Compare the nominal HL93 bending effects with those of the nominal AASHTO legal vehicles given in Table 3 to show an average ratio of 1.73.
- Compare the proposed live load factor for rating with the guide specification live load factor of 1.8, by considering that the guide used a target for beta corresponding to a load factor rating (LFR) of 2.3. This further reduces the live load factors by a ratio of 1.80/1.95 = 0.92. See *NCHRP Report 301* (Moses and Verma, 1987) and the AASHTO guide specification (AASHTO, 1989) for comparisons.
- Use a resistance factor for the LRFD of 1.0, which is the value given in the AASHTO LRFD design specifications. However, for deteriorated members, a lower resistance value is recommended.

Balancing all these different considerations means that the factored design load transforms to a factored rating load as

$$\begin{aligned} 1.75 \text{ HL } 93 &= 1.75 \times 0.91 \times 0.76 \times 0.94 \times 1.73 \times 0.92 \\ &\quad \times 3\text{S}2 = 1.81 \times 3\text{S}2 \end{aligned} \tag{31}$$

The value recommended for the Evaluation Manual is provided in Table 6.4.4.2.3-1 as 1.80. This 1.8 factor is for the worst traffic category, namely 5000 ADTT, and is consistent with the LRFD design code and the guide specification. For agreement with the guide specification, a further 0.95 resistance value would be used. In the Evaluation Manual, however, the resistance is reduced only when inspection indicates capacity deterioration. Note also that the bias of the Ontario W_{95} data compared with average WIM-based data in the United States is given in Chapter 4 as 1.07. This bias was not part of the adjustments in this calibration. This additional margin should be mentioned whenever the beta level is discussed related to risk of failure for an individual span.

To judge these safety margins, note that the recommended factored live load for evaluation for this worst traffic case (5000 ADTT and Ontario truck weight spectrum) is equivalent to 1.8 multiplied by 72 equal to 130 kips of truck load (in a 3S2 configuration) in each lane. In addition, there is a safety margin arising from the dead load factor and resistance bias that makes the span able to carry additional overloaded vehicles safely. Thus, there is considerable margin for reducing the factored effects for less severe traffic cases and for carrying overweight permits.

Chapter 5 shows that the expected maximum live load equals a total of 240 kips in 3S2 equivalents acting in *both lanes* under the severe traffic condition. As derived above, a factor of 1.8 multiplied by 3S2 provides a target beta consistent with acceptable AASHTO operating levels of experience for this load situation. Because the required load factor is proportional to the mean maximum live load in Equation 30, any reduction in the expected maximum loading leads to a proportional reduction in the required live load factor.

For an ADTT of 1000, Table 4 shows that the maximum expected live load effect acting in both lanes is 217 kips. The live load factor for this case is then

$$\gamma_{L,1000}(\text{ADTT} = 1000) = 1.8 \times 217/240 = 1.62 \qquad (32)$$

The Evaluation Manual proposes using 1.60 for the 1000 ADTT category.

For ADTT = 100, it was shown in Table 4 that the maximum expected live load effect acting in both lanes is 174 kips, or 86.5 kips of load per lane. The live load factor for this case would be

$$\gamma_{L,100}(\text{ADTT} = 100) = 1.8 \times 174/240 = 1.31 \qquad (33)$$

However, in the 100 ADTT case, a one-lane load case may govern as explained in the next paragraph. For this reason, the recommended live load factor for 100 ADTT is given as 1.4 in the Evaluation Manual.

6.2.3 One-Lane Bridges

The next step is to extend the derivation of live load factors to the one-lane case using the expected maximum one-lane loading events given in Table 4. Using Equation 30, the following expression gives the live load factor:

$$\gamma_{L,\text{one lane}} = \frac{1.8}{120} \times \begin{array}{c}\text{expected maximum}\\ \text{one lane live load}\end{array} \qquad (34)$$

For one-lane, it was shown in Table 4 for the 100 ADTT case that the maximum expected live load event is 136.6 kips. Substituting in Equation 34 gives a load factor of 2.05. Similarly, for the 5000 ADTT, the live load factor for one-lane is 2.30, and for 1000 ADTT it is 2.20. The one-lane live load factor does not decrease significantly with the traffic volume as in a two-lane case because side-by-side presence does not occur in the one-lane case and only the largest individual sample truck from the entire sequence of truck events is important. The number of samples is large, even for the 100 ADTT case. For one-lane bridges, the one-lane load factors are applied to the legal loads along with the distribution factor for one-lane, namely, g_1.

Using the old S/5.5 and S/7 formulas, the ratio of distribution factors for one and two lanes is 1.27. With this ratio, load effects for one- and two-lane events are similar. For the 5000 ADTT volume, the factor of 1.8 multiplied by the ratio 1.27 equals close to the one-lane factor of 2.30. However, using the ratio of multilane and one-lane distribution factors given in Table 5, it is clear that the multilane case governs. In Table 5, the average ratio of D.F. values for two- and one-lane cases is close to 1.70. It is only for the low traffic case of 100 ADTT that it is possible for a single-lane case to govern. For example, if the ratio of two-lane and one-lane D.F. is below 1.55 for a particular span, then for the 100 ADTT case, the single-lane control (i.e., 2.05, given above, divided by 1.55 equals 1.3, the factor found in Equation 33). To make it unnecessary to check both single-lane and two-lane loading effects in a two-lane bridge, the 1.3 factor for 100 ADTT found in Equation 33 is increased to 1.4 in the recommended live load factors for legal load ratings in the Evaluation Manual. The 1.4 factor should cover all reasonable ranges for the ratio of single- and two-lane D.F. values.

Single-lane live load factors for legal load ratings are then only necessary for situations where the bridge contains a single traffic lane. The live load factors for legal load evaluation would then be 2.30 for 5000 ADTT, 2.20 for 1000 ADTT, and 2.10 for 100 ADTT.

For simplification, a single live load factor of 2.3 could be applied for all traffic sites for the one-lane check.

Using a lower bound on the live load factor of 1.40 for all two-lane traffic cases also helps to raise the minimum level of safety, regardless of traffic volume. Such an increase in safety would cover cases in which the inspection interval would be longer than shown in Table 4. Another example would occur when individual overweight trucks in the tall of the distribution are not adequately enveloped by the Ontario data used for the LRFD design specification.

The proposed lower limit of 1.4 for the two-lane live load factor in the Evaluation Manual is actually close to the load factors specified in the previous AASHTO operating criteria, namely 1.3, in load factor rating and 1.33 in working stress operating rating. In both cases, the dead load factor is 1.3, which is higher than the Evaluation Manual's dead load factor of 1.25, so the overall ratings may be similar. (The Evaluation Manual maintains the same dead load factor and resistance factors as given in the AASHTO LRFD bridge design specifications.) Thus, it is concluded that the existing AASHTO rating factors may attain the target safety indexes only for low traffic volumes (100 ADTT). Further comparisons are needed, however, to take into account the new AASHTO distribution factors and impact allowances in making a complete comparison in this regard.

6.3 POSTING ANALYSIS

The previous section developed live load factors for rating legal loads. When the computed rating factor is less than 1.0, a bridge may need to be load posted. A posting equation and curve are recommended in the Evaluation Manual to convert the legal load rating factors to an allowable truck weight. One possible practice is to use the rating factor and multiply it by the legal load to give the posting load. This simple relationship, however, ignores the reliability analysis concepts used herein.

Some agencies have recognized the increasing uncertainties associated with posting by rating the bridge with operating stress levels and then selecting posting weight levels using the more conservative inventory levels. In addition, some agencies consider local traffic conditions in selecting posting levels. The aim in the recommended posting curve in the Evaluation Manual is to present a rational consistent approach that selects posting weights on the basis of reliability estimates consistent with the philosophy of the Evaluation Manual.

Posting is a complex topic because the Evaluation Manual recommendations overlap with various local jurisdictional and legal requirements that are not necessarily consistent. Examples include the question of signage and whether it shows one, two, or three vehicles and the degree to which any extra "cushion" is needed in the posted values. Clearly, posted bridges present the largest and perhaps the sole source of bridge failures of the type where live load effects because of vehicles exceed capacity and the bridge span collapses. Yet, if the posted bridge fails and the driver of the overloaded vehicle is found, then the agency is relieved because the truck owner is at fault and perhaps will pay for the bridge replacement. If the signage has too much cushion, however, drivers may stop respecting the posted signs, and overloading may then be more likely.

The posting analysis recommended herein starts from the point that the legal load R.F. value has been determined by the evaluation procedures outlined in the Evaluation Manual. In this case, there is a single rating factor produced. The new Evaluation Manual does not produce both inventory and operating values as in the past, but rather only one recommended rating value that considers site loadings, resistance on the basis of inspection data, and bridge redundancy.

The philosophy recommended herein is that if the R.F. is greater than 1.0, then the bridge is satisfactory for that legal vehicle check. If, however, the R.F. is lower than 1.0 for any of the AASHTO legal vehicles, then the bridge should be posted for that vehicle type, rehabilitated, or replaced.

One exception is that a reinforced concrete bridge that has been carrying normal traffic without signs of distress need not be posted, provided it is being inspected regularly. The same rule applies to a posted concrete bridge, namely, the posted value for a concrete bridge should not be reduced on the basis of the computed rating factor if the bridge shows no sign of distress. Instead, the bridge should be inspected frequently.

The justifications for these actions for concrete bridges are that tests and field experience have shown that, for reinforced concrete bridges (especially T-beams and slab bridges), there is considerable reserve capacity beyond the computed value and that such spans show considerable distress (e.g., spalling and deflections) before severe damage or collapse actually occur.

For prestressed and steel bridges, whenever a rating factor is lower than 1.0, the bridge should be posted. It is recommended in the Evaluation Manual that a single posting load not be applied whenever a meaningful evaluation as developed in the Evaluation Manual has been carried out. The wide variety of vehicle types cannot be effectively controlled by any single posting truck model. A single posting based on a short truck model would be too restrictive for longer truck combinations, particularly for short-span bridges. A single posting load based on a longer combination would be too liberal for short trucks for almost any span combination.

Experience has shown that the three AASHTO legal vehicles, namely Type 3, 3S2, and 3-3, adequately model short vehicles and combination vehicles in general use in the United States. The AASHTO vehicle configurations were also shown in Table 3 to provide consistent bias with respect to the new HL93 load models. Exceptions to this assumption concerning the coverage of the AASHTO legal vehicles may include short but heavy special vehicles and longer train combinations, but these vehicles, in most cases, would be treated as permit vehicles.

6.3.1 Posting Curves

It is recommended herein to use the three rating factors based on the three AASHTO legal trucks for posting decisions. The three rating factors for each AASHTO rating vehicle can be used to develop the posting loads for single-unit vehicles, tractor-trailer or tractor-semitrailer combinations, and vehicle trains, respectively. The guidelines given in the Evaluation Manual are intended to be of assistance to authorities responsible for establishing posting limits.

If, for any of the AASHTO legal vehicle types, the R.F. is between 0.3 and 1.0, then the posting curve presented in the Evaluation Manual should be used to fix the posting weight for that vehicle type. The recommended curve in the Evaluation Manual is a linear relationship giving R.F. versus posted weight. The curve provides the legal weight of the AASHTO truck when the R.F. equals 1.0 and goes down to 3 tons when the R.F. equals 0.3. A straight line connects these two points for each of the three rating vehicles. Although it may be desirable to refine these curves and perhaps substitute nonlinear relationships, it was judged that the available data for analysis were not sufficient to refine the curves.

The Ontario Bridge code uses a curve for fixing the posted load for each of the Bridge codes three vehicle types versus the R.F. for that check (Ontario Highway Bridge Design Code, 1993). However, the proposed Canadian Bridge code does not use these curves and instead selected a straight line (CSA, 1990). In neither code, however, is any statistical database given to support the selection of the shape of the posting curves. Therefore, it seems reasonable to recommend a curve.

The proposed posting format includes the existing AASHTO requirement that in no instance shall a bridge remain open to vehicular traffic when the computed capacity for a single unit vehicle falls below 3 tons. This traditional rule, although often stated and commonly applied, indicates neither the method of evaluation (e.g., working stress or load factor ratings), nor the allowable stress level to be applied, either inventory or operating. Nevertheless, it is reasonable that a 3-ton lower limit be applicable.

The analysis adopted herein identifies the corresponding rating factor that meets a target reliability level and provides a span with a 3-ton vehicle capacity. It is shown below that the 3-ton level corresponds to a rating factor of 0.3. If the R.F. falls below 0.3 for any vehicle class, the span should be closed to that vehicle type. By coincidence, this 0.3 lower limit also agrees with the recommendation in the proposed Canadian code (CSA, 1990), although no justification is given therein, except to mention that the lower limit relates to the weight of empty vehicles. This argument does not seem relevant given that the weight of vehicles that may be affected by the lower limit is not just based on empty trucks of the AASHTO types, but also on vehicles such as fire trucks, garbage haulers, and school buses.

As shown in the Evaluation Manual, the lower bound R.F. equal to 0.3 limit is adopted in conjunction with the posting graph. That is, if the computed R.F. is less than 0.3, the legal capacity for that vehicle is zero. Above an R.F. equal to 0.3, the posting load approaches the legal load as R.F. increases to 1.0. The 0.3 lower R.F. limit may seem high, but the limit relates to a safety level computed with a single criteria for the live load factor corresponding to an operating and not inventory level. The lower limit of R.F. equal to 0.3, at which the bridge will be closed, was derived on the basis of several variables that change the uncertainties of the reliability calculation for posted bridges compared with unposted situations. The rating factor of 0.3 may even, in some cases, be similar to existing bridge closing criteria based on inventory levels of stress.

6.3.2 Posting Derivation

The Evaluation Manual recommends posting loads that drop off more quickly than the rating factor. Thus, the selection of posting loads relative to the numerically calculated rating factor is conservative and is intended to cover the several changes in the distribution of the random variables affecting the reliability analysis. The uncertainties in estimating the maximum live load on a posted bridge differ markedly from those in normal traffic situations with legal truck traffic. The following differences should especially be noted:

1. The statistical distribution of gross vehicle weights at a site with a posted structure will differ from that at a site with no postings. It is likely that, proportionately, there will be a greater percentage of vehicles at or exceeding the posted limit compared with the numbers close to or exceeding the legal limit on an unposted bridge. An allowance for potential overloads is contained in the posting curve presented. Any overload allowance or safety "cushion" is not intended, however, to be used by a driver as a justification for subverting legal posted signs.

   Although little actual field data exist with which to make the necessary projections, evidence exists in many jurisdictions that overloaded trucks have caused bridge failures on posted spans. These failures or severe damage occur because drivers deliberately ignore the posted sign or else are unaware of the meaning of the displayed warning. Posting signs must be unambiguous and widely disseminated to drivers. Exceptions to posting signs for seasonal or production requirements or for other reasons should not be encouraged because such exceptions may lead drivers to ignore the potential consequences of crossing a posted bridge with an overloaded vehicle. An allowance for potential overloads is recommended in the rating factor curves presented herein, along with a safety margin intended to maintain a reasonable reliability target level.

   To provide a deliberate overload cushion, the posting curve considers an additional margin of 10,000 lb at the level where R.F. equals 0.3 (the point at which the span must be closed to that vehicle type). Thus, for a vehicle posting at the minimum level, namely 6000 lb (3 tons), it is assumed for this analysis that the vehicle actually weighs 16,000 lb. This weight cushion of 10,000 lb introduces an expected load bias by a ratio of 16000/6000, or 2.67. However, in the reference equation for rating bridges (Equation 30), there is already an expected load bias based on the expected maximum load effect of 120,000 lb/lane for heavy traffic compared with the 80,000-lb legal loading. This ratio corresponds to a load bias (mean/nominal) of 1.5. (That is, vehicles of 120,000 lb in each lane are shown above to correspond to the maximum expected loading event). So the added load bias for a posted situation is increased herein by a factor of 2.67/1.5, or 1.78. This additional live load bias will be used in the reliability comparisons below.

2. The percentage distribution of the gross vehicle weight to the individual axles may change as the gross legal weight decreases. A vehicle could satisfy both the posted gross and the individual axle combination limits and still cause a load effect in excess of that assumed in the rating factor calculation. In the rating analysis, an assumed standard axle distribution is used (e.g., see AASHTO legal vehicles). This acute load distribution on the axles has been incorporated in the recommended posting curve.

   Load effects were compared by using different spans and unbalanced loads of a Type 3 vehicle. The comparison showed that as the gross load decreases, the load effect could increase up to 10 percent by unbalancing or load shifting the gross weight to a different ratio than assumed in the Type 3 vehicle. This ratio of increased moments would be higher for a combination vehicle. For example, instead of equal load on each tandem, a driver on a posted bridge could put all the payload on one tandem, causing a greater load effect than would occur using equal loads on each tandem. Because the vehicle most likely to cause a bridge closing is the shorter

Type 3 vehicle, a 10-percent modification ratio will be used in the formulation below (i.e., raise the live load factor by 1.1).
3. The dynamic load allowance or impact percentage will generally increase as the gross weight of a vehicle decreases. This increase in associated dynamic allowance is reflected in estimating the lower limit of the posting loads. For the dynamic effect, increasing the impact by 10 percent should take care of the higher impacts associated with lower gross weight.
4. It is recommended that the reliability level inherent in the posting curve be raised for lower posting loads to achieve higher reliability targets. For lower posting loads, there is a need for a greater precision for the posting calculations, and there is a greater likelihood of overloads occurring (based on evidence of historical events). Also, given that posted bridges are often older and have deteriorated spans, the reliability level for calculating the lowest acceptable rating should be raised at the lower posting loads. At the lower posting levels, achieving reliability targets closer to design or historic inventory levels is preferred to the higher operating reliability levels that are characteristic of other practices in the Evaluation Manual. This consideration of the reliability targets is reflected in the posting curve by raising the live load factors by the ratio of design value (1.75) to operating value (1.35) or a ratio of 1.3. As shown above, an increase in live load factor by a ratio of 1.3 raises the beta level by about 1.0.

To reach a lower acceptable bound on the rating factor, assume that a bridge is closed if the posted load reaches 3 tons, as determined by the computed R.F., using the methods in the Evaluation Manual. For a 3-ton, Type 3 vehicle (legal weight 25 tons), modifying the rating to account for inventory, instead of operating live load factors (an increased ratio of 1.3), increased dynamic allowance (a ratio of 1.1), a load shifting allowance (a ratio of 1.1), and an additional 10,000-lb load overweight allowance cushion (arbitrary value, which leads to an additional live load ratio of 1.78) gives overall the following rating:

$$\begin{aligned} \text{R.F.} &= [3\,\text{ton}/25\,\text{ton}] \times 1.3 \times 1.1 \times 1.1 \times 1.78 \\ &= 0.34 \end{aligned} \tag{35}$$

That is, a 3-ton vehicle would correspond to a computed R.F. of 0.34 in order to accommodate the higher uncertainties and to achieve the desired reliability target, considering the factors just raised.

The recommended lower limit for the rating factor before a bridge closing should be considered is therefore given in the Evaluation Manual as 0.3. This lower value also agrees with the proposed Canadian specification. For the other two AASHTO vehicles, the 0.3 minimum for R.F. should also be applicable. The same posting curve, which goes between a legal load for an R.F. equal to 1.0 down to 3 tons at an R.F. equal to 0.3 should then be used for all legal vehicles. When the R.F. for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the span. When the R.F. falls below 0.3 for all vehicle types, then the span should be considered for closure.

6.4 USE OF WIM TRUCK WEIGHT DATA

Much of the uncertainty in predicting the maximum bridge loading results from site-to-site variations in the intensity of truck traffic. Often, only site volume is available in the form of ADTT from agency planners and, hence, is used to select the live load factor for the legal rating, as recommended in Section 6.2.

When both truck weight and traffic volume data are available for a specific site, then appropriate load factors should be derived using this information. Measured truck weights at a site should be obtained by accepted WIM technology. In general, such data should be obtained from systems capable of undetected weighing of all trucks without having heavy overweight vehicles bypass the weighing operation. Several procedures have been presented for characterizing the expected maximum live load. This characterization uses various upper fractiles of the measured truck statistics at a site (e.g., the 95th or 99th percentile). If possible, the presence in the traffic stream of permit vehicles should also be extracted from the weight population in computing the statistical parameters of the truck weight distribution. Ignoring the permit trucks in estimating the maximum loading effects is appropriate when calculating the rating for the legal loadings.

On the basis of the previous sections, a simplified procedure for deriving the live load factors when WIM data are available is given in the Evaluation Manual. The procedure follows the same analysis used above for deriving the live load factors for legal ratings. In order to obtain an accurate projection of the upper tail of the WIM weight histogram, only the largest 20 percent of all truck weights should be considered as the basis for fixing the truck weight spectrum and for extrapolating to the largest loading event. A sufficient number of truck samples should be taken to provide accurate parameters for the weight histogram (see Section 6.4.1). Any characteristic of a data sample, such as the 95th percentile weight fractile (designated as W_{95}) or even a simulated projection of the maximum loading event, is itself a random variable. That is, if the measurement or sampling process is repeated, it is likely that a different projection will be obtained. The uncertainty of this sampling variable is reduced as additional data are obtained. Such sampling variables are called statistical, modeling, or subjective variables in the reliability literature (Melchers, 1987).

It was shown above that the live load factor should be proportional to the expected maximum loading event. The live load factor of 1.8 is deemed satisfactory for the 5000 ADTT traffic, with an expected maximum total loading of 240 kips in the two lanes. Thus, the reference mean girder loading for calibration is equal to 120 kips per lane multiplied by the two-lane distribution factor. The live load factor for measured site data should be in the same ratio as the maximum expected loading from the measured WIM and truck traffic field data for the particular site compared with the 120-kip-per-lane reference value. Thus, for a two-lane loading case, the live load factor (γ_L) should be taken as proportional to the expected maximum loading event divided by the reference load case of 240 kips. Using Equation 30 and substituting for the expected maximum live loading event gives

$$\gamma_{L,\text{ two lane}} = 1.8\,\frac{2W^* + t_{(\text{ADTT})}1.41\,\sigma^*}{240} > 1.30 \tag{36}$$

where

- W^* = the average (mean) truck weight for the top 20 percent of the weight sample of trucks,
- σ^* = the standard deviation of the top 20 percent of the truck weight sample, and
- $t_{(\text{ADTT})}$ = fractile value appropriate to the maximum expected loading event as given in Table 4 (for the two-lane case, the fractile value depends on the number of side-by-side occurrences, which depends on the ADTT).

Equation 35 provides a lower bound of 1.30, which is less than the specified 1.4 factor for the 100 ADTT traffic sites given in the Evaluation Manual. A smaller factor is reasonable because the direct use of a site's WIM weight database will reduce the uncertainties in a predicted maximum loading event compared with the use of the Ontario data presented by Nowak.

The nominal factored design loading, 6, is then found as

$$L_n = \gamma_{L,\text{ two lane}} \times \text{AASHTO legal load effect} \times \text{impact} \times g_m \tag{36a}$$

For the single-lane loading case, the expected maximum live loading event depends on the number of individual truck passages, which also depends on the ADTT. The fractile value depends on the traffic volume. The maximum loading in the girder is obtained by multiplying the expected maximum single-lane vehicle event by the single-lane distribution factor, g_1. The reference load effect remains the same, which is 120 kips in one-lane. Thus, the single-lane live load factor should be taken as

$$\gamma_{L,\text{ one lane}} = 1.8\,\frac{\left[W^* + t_{(\text{ADTT})}\,\sigma^*\right]}{120} > 1.8 \tag{37}$$

The nominal factored design loading, L_n, is then based on

$$L_n = \gamma_{L,\text{ one lane}} \times \text{AASHTO legal load effect} \times \text{impact} \times g_1 \tag{38}$$

A lower limit of 1.8 is arbitrarily placed on the one-lane factor, based on experience with WIM data. In checking a span, the larger of the one- and two-lane load effects shall determine the rating factor for the component. Distribution values for the single- and multiple-lane cases must be based on structural analysis or field measurements without any "built-in" multiple presence modifications. The measured site statistical parameters, W^* and σ^*, should be substituted in the equations for the load factors. These equations are summarized in Section 6 of the Evaluation Manual.

6.4.1 WIM Data Requirements

The amount of WIM data needed for forecasting depends on whether the single-lane or multilane load situation governs the maximum loading effects. For a site when two lanes govern, the W_{95} fractile should be accurate for extrapolating to the maximum loading event because the maximum loading event is controlled by two vehicles on the bridge. Single vehicles with extraordinary weights probably will not affect the extreme loading event. For a site where the one-lane loading event case governs, more data are needed to project the extreme truck loading event, because the extreme loading is controlled by the heaviest single vehicle event. Consideration in the data should be given to seasonal, weekly, and daily variations to ensure that the maximum controlling loading event is captured in the database.

Determining the quantity of WIM data or the number of measured truck weights needed for assessing the site data is difficult. Quality of data is more important than quantity. An agency should ensure that the sample data are unbiased and that no avoidance of weighing operations by overloaded trucks is occurring. Also, if possible, factor out of the data sample any permits that may be in the traffic stream. For an accurate estimate, special seasonal, weekly, or even daily fluctuations should be collected.

As for the number of truck samples required, the projections use only the top 20 percent of trucks in the sample weighed, as per Nowak. To gain a good estimate of parameters after throwing away 80 percent of data, the following estimates may be used. For high-traffic sites, probably one 24-hr equivalent weighing period would be sufficient. This sampling period would include at least four hundred trucks in the upper 20 percent. For a low-volume site, perhaps 2 days would be needed to have enough data to incorporate any heavy vehicles corresponding to local industries that may affect the high end of the truck weight distribution.

CHAPTER 7

PERMIT VEHICLES

The Evaluation Manual recommends evaluating permit requests consistent with the safety philosophy, in the AASHTO design specifications and the Evaluation Manual's proposed legal load rating requirements. Two types of permits are considered—routine and special. In general, routine permit limits typically will be as high as 50 percent above legal, or 120 kips. In some jurisdictions, routine permits may be allowed for vehicles up to 150 kips. Special permits cover heavier trucks and so-called superloads. Routine permit checks may also be used in evaluating so-called "grandfather" exemptions when such vehicles are in common service and significantly exceed the federal bridge formula.

In the case of routine permits, there is random traffic alongside the permit vehicle. The frequency that a permit of a certain weight level crosses a bridge may vary in different jurisdictions. For this reason, the Evaluation Manual recommends a conservative analysis approach that leads, in the case of routine permits, to a two-lane distribution analysis that places two identical permit trucks simultaneously crossing the bridge. The live load factors, however, for such a check are reduced compared with legal load rating values to account for the small likelihood of simultaneous crossing events and also the lesser likelihood that a permit truck will be significantly overloaded. Special permits, on the other hand, are assumed to cross the span without another truck alongside so that the analysis uses a single-lane distribution factor. In this case, the live load factor is increased to reflect the probability that there is some contribution from a vehicle alongside the permit vehicle. The probability that there is a vehicle alongside the permit vehicle is a function of how many times a permit vehicle will cross the span.

For simplifying the permit-checking calculations, the bridge analysis assumes that the routine permit vehicle is acting in both lanes, which allows the use of the new AASHTO distribution factors. For special permits, the one-lane loading analysis with the AASHTO one-lane distribution factor is recommended. It is unnecessary, therefore, in the rating analysis to consider any cases with a permit vehicle in one lane and some other vehicle in the other lane.

Because of the influence of random traffic, the live load factors for routine permit checking depend on both the ADTT value and the upper weight limit for the class of permit vehicles being checked. The live load factor increases with greater traffic volume because there is a higher chance of heavier vehicles alongside the permit vehicle as it crosses the bridge. The live load factors decrease with increasing permit weight because the influence of random vehicles alongside the permit vehicle decreases as the weight of the permit vehicle increases.

Because special permit vehicles usually make few crossings, random traffic alongside the permit truck will probably have a limited influence on the live load factors. Given that regulations among agencies vary with regard to permit frequency, the live load factors for special permits should depend on the expected number of passages of the permit vehicle in the inspection interval. For simplification, the proposed Evaluation Manual does not explicitly express the live load factors as a function of the number of passages. The results in this report, however, may be used to express the live load factors as a function of the number of passages; in this case, the special permits should be grouped into equivalent load effects in estimating the number of passages.

That a permit vehicle moves across a span without bridge damage on one occasion is not a "proof" that the permit vehicle may proceed for an unlimited number of passages. For each passage, the maximum load effect in the span is affected by different realizations of the various random variables, including the truck weight alongside the vehicle, the dynamic amplification of the bridge, and even different girder distributions because of varying lane positions of the permit vehicle. Hence, the calibration of the permit live load factors incorporates all these uncertainties.

It is the intent in the Evaluation Manual to prescribe uniform procedures for reviewing permit loads. The aim is to use LRFD methods that establish a uniform target reliability consistent with other provisions of the Evaluation Manual as well as procedures inherent in AASHTO LRFD design specifications. The target reliability is generally the operating level consistent with past AASHTO practices. Some agencies, however, may wish to raise the reliability for some permit cases, such as superload permits. The commentary in the proposed Evaluation Manual discusses this situation. Higher reliability levels may be imposed by raising the live load factors.

The analysis for permit loads proceeds in a manner similar to the calculation of the legal load rating factors—The exception being that a known vehicle is moving in one of the lanes. It is assumed that inspection data are available to establish the nominal resistance side of the checking equation. The dead load factors, impact factor (except where noted), and the component resistance factors are the same values used in the legal load rating. Live load factors for permit loads are determined by comparing the expected maximum live load effect with the reference case cited in Chapter 6. The reference case is for the extreme traffic volume of 5000 ADTT, which produced an expected truck load event per lane of 120 kips. To maintain the reliability targets, this event required a factored checking load of 1.8 multiplied by the AASHTO legal vehicle (e.g., the 3S2). This live load factor was identified in the legal load rating as the requirement to meet the target reliability for the most severe traffic situation.

7.1 ROUTINE PERMITS

Routine permits are issued by most agencies whenever all the bridges on a route satisfy some capacity limit. Often, in the past, such permit criteria were based on the operating rating factors of bridges on the route. For example, as an illustration of such historical criteria, if a bridge's operating rating exceeded 150 percent, then the bridge was allowed to carry routine permits up to 120 kips. Because the proposed Evaluation Manual does not contain inventory and operating levels, a permit analysis of the type recommended in the Evaluation Manual should be used in place of such historical rules based on operating ratings. The various classes of permits applicable in a given jurisdiction must be analyzed with the permit rating analyses in the Evaluation Manual, using the permit live load factors provided in the Evaluation Manual.

The permit live load factors in Table 7 reflect the variety and number of possible permit situations. The precision obtained depends on how the results are presented and the number of independent groups of permits and traffic situations. As the permit frequency and/or the ADTT increase, the live load factor should also be increased. A study for Ohio DOT presented simulations of permit effects inserted into a traffic stream characteristic of different locations developed from the WIM database (Moses and Fu, 1990). A follow-on study was reported by Fu and Hag-Elsafi for New York State DOT that considered a more specific database and traffic characterization (Fu and Hag-Elsafi, 1997).

The New York study proposed a live load factor of 1.35 for routine permits under 130 kips and 1.05 for routine permits above 130 kips. Both cases use a two-lane loading distribution factor. A two-lane analysis, in effect, places the permit vehicle alongside a second permit of equal weight. Thus, for the higher permit weights, there is an additional safety margin because of the assumption in the distribution analysis that both lanes are carrying the same permit vehicle. It was shown by Fu and Hag-Elsafi that a conservative reliability level is obtained in all cases considered (Fu and Hag-Elsafi, 1997). Their ADTT and permit frequency were based on historical records kept by the permit bureau in New York. Neither one-lane load cases nor the influence on the permit live load factors because of ADTT values were noted.

TABLE 7 Two-Lane Routine Permits, Minimum Live Load Factors, and Two-Lane Checking

					Live Load Factor - $\gamma_{L,\text{Two Lane}}$			
N_p	Y	ADTT	N_R	W_R	P = 80k	P = 125k	P = 150k	P = 200k
10	2	100	36.5	103	1.24	.98	.91	.82
		1000	73	108	1.27	1.01	.93	.83
		5000	4487	119	1.34	1.05	.97	.86
	5	100	91	109	1.28	1.01	.93	.83
		1000	183	114	1.31	1.03	.95	.85
		5000	1217	125	1.38	1.08	.99	.88
100	2	100	365	118	1.34	1.05	.96	.86
		1000	730	122	1.36	1.07	.98	.87
		5000	4867	132	1.43	1.11	1.01	.90
	5	100	912	123	1.37	1.07	.98	.87
		1000	1825	127	1.39	1.09	1.00	.88
		5000	12167	136	1.46	1.13	1.03	.91

N_p - number of permits per day
side by side prob.-$P_{s/s} = 0.005, 0.01, 1/5$ for ADTT = 100,1000, and 5000 respectively.
number of side by side events, $N_R = N_p \times P_{s/s} \times 365 \times$ years
$W_R = 68 + t(N_R)\,18$
$t(N_R)$ - constant from normal prob. Table for probability level of $(1 - 1/N_R)$
total weight, $W_T = P + W_R$, where, P-permit weight
For, γ_L See Eqn. 39
e.g., line 1: $N_R = 36.5$; $t(N_R) = 1.92$; $W_R = 103$; $W_T = 103 + 80 = 183$

Possible contradictions must be considered using any set of proposed permit rules if the results are applied without further clarifications. For example, as shown in the previous chapter, a bridge should be posted if the rating factor (R.F.) is lower than 1.0. The severe 5000-ADTT traffic case uses the proposed live load factor of 1.8. (or an R.F. of 1.0 requires a live load capacity of 1.8 multiplied by a 3S2 truck or 1.8 multiplied by 72 kips, which equals 130 kips in each lane). Thus, the bridge would need to be posted for legal loads if it had a factored live load capacity of less than 130 kips. Yet, if the agency uses the just-stated New York requirements, it could accept a permit truck of, for example, 90 kips, because a factor of only 1.35 is needed (i.e., a factored load of 90 multiplied by 1.35 equals 21 kips in each lane).

A set of rules causing a posting limit for trucks below the legal truck limit of 80 kips for routine traffic (because the span does not satisfy the legal load rating check) and at the same time accepting a routine permit vehicle of 90 kips is not a reasonable practice. Such situations were eliminated in the Evaluation Manual wherever they arose.

The illustration just given of a posted bridge allowing overload permit trucks is, in fact, not illogical. The situation described arose out of New York's proposed regulations based on New York's simulation of maximum expected live load effects. The 1.8 live load factor for legal load rating was derived with Nowak's data and produces a maximum expected load effect of 240 kips (or 120 kips in each lane) for random traffic and the 5000-ADTT traffic volume. By inserting a permit vehicle of 90 kips in the traffic stream and checking for the event when the permit crosses the bridge, the expected maximum loading effect is lower compared with the maximum random traffic event. To state it simply, putting a permit vehicle of 90 kips into the simulation does not affect the critical live load effect. The heavy traffic, including the many overloads in Nowak's truck database, controls the maximum load event. Hence, the bridge is "safer" when the permit crosses than when the extreme overloads in Nowak's random truck data are present based on the simulation. The solution to a situation where overloads control the legal load rating is not to approve higher permits, but to better enforce the existing truck weight regulations.

To avoid having a specification rule that approves a permit on a posted bridge, the live load factors in the Evaluation Manual are linked to the same ADTT values as are the reference values for the legal load rating. The live load factors for routine permits proposed in the Evaluation Manual converge for low permit loads, to the same live load factors for legal load rating. The Evaluation Manual then allows lower live load factors for the larger permit loads. The live load factors are constant up to 100 kips and then are reduced by interpolation down to where a permit load of 150 kips is applied. It is expected that agencies should select some upper weight limit on routine permits above which the category of special permits should be used.

7.2 PERMIT RELIABILITY ANALYSIS

The selection of live load factors for routine permits was done as follows. Three categories of site traffic were used, namely, ADTT of 5000, 1000, and 100. The site truck weight statistics were the same as the Nowak data, namely a mean of 68 kips and a sigma of 18 kips for the top 20 percent of the truck weight population. The same multiple-presence modeling used above for routine traffic for the three ADTT categories, namely a probability of a truck alongside the permit, was 1/15, 0.01, and 0.001, respectively. Both 2-year and 5-year exposures were considered. The number of permits was taken as 10 and 100 permits per day to cover the likely range. The upper value of 100 permits per day, even with an ADTT of 5000, corresponds to a permit percentage of 2 percent, which is higher than the data reported in the New York study. A range of routine permit weights of 80 kips, 125 kips, 150 kips, and 200 kips were examined.

For each traffic case, the expected number of multiple presences were estimated and the corresponding weight fractile and the expected maximum truck weight of the alongside vehicle computed. This alongside truck weight was combined with the permit weight. For example, the most extreme traffic has an ADTT of 5000, 100 permits per day, and 5-year exposure. The expected number of multiple presences with a permit vehicle in one lane and a heavy random vehicle selected out of the Ontario weight data in the other lane is equal to 12,167 side-by-side passages (5 years multiplied by 365 days multiplied by 100 permits per day multiplied by 1/15). The normal distribution fractile corresponding to this number of occurrences (i.e., a probability value of 1/12167 equals 8.22 multiplied by 10^{-5}) is 3.77, from Appendix A. The expected alongside weight corresponding to this fractile is 136 kips (i.e., 68 plus 3.77 multiplied by 18). This value is independent of the weight of the permit vehicle. Assuming a permit weight P of 125 kips, the total maximum expected two-lane loading weight is 261 kips (i.e., 136 plus 125).

The same reference safety margin given for calculating the live load factor for rating legal vehicles is allowed for calculating the live load factors for routine permits. Using the same reference safety margin produces the target reliability level (i.e., operating level).

The 1.8 live load factor multiplied by a nominal 3S2 in each lane (weight of 72 kips) provides the target reliability when the expected maximum truck load effect is 240 kips for both lanes. In the permit checking, the live load factor is multiplied by the weight of the permit truck, whereas, in the reference case, the live load factor multiplied by the weight of the 3S2 vehicle. To keep the same safety margin, the ratio of a 3S2 vehicle (72 kips) and permit truck weight, P, must be inserted in Equation 30, or

$$\gamma_{L,\text{two lane}} = 1.8\frac{W_T}{240} \times \frac{72}{P} \tag{39}$$

where

W_T = expected maximum total weight of permit and alongside vehicles and
P = weight of permit vehicle in checking equation.

The routine permit checking uses the multilane distribution factors that were used for deriving the reference level safety factors for random traffic. For the example just cited from Equation 39, the required live load factor equals 1.13 [1.8 multiplied by (261/240) multiplied by (72/125)]. (See Table 7.)

Table 7 presents the results of the live load factor calculations for the range of parameters mentioned above. Live load factors increase with ADTT, permit rate, and exposure period, and decrease with permit weight. These factors apply only to an analysis with the two-lane distribution values and a permit vehicle present in each of two lanes. The results in Table 7 were used to select the factors in the Evaluation Manual. The load factors recommended in the Evaluation Manual are upper bounds to the values in Table 7 and reflect also the possibility of one-lane check governing as discussed in the next section. These factors were tested in several parametric and bridge sample studies, and results were reported by Bala Sivakumar.

7.2.1 One- and Two-Lane Distribution Permit Checks

The previous paragraphs cite a two-lane distribution analysis when checking routine permits. The total load (i.e., the permit and the expected maximum alongside vehicle), is placed on the span for checking. Because the nominal permit vehicle weight is used in the permit check, the live load factor in Equation 39 has been adjusted for the total weight on two lanes of the bridge to satisfy the reference safety margin. The live load factor generally decreases as the weight of the permit vehicle increases.

Alternatively, in some permit cases, a more accurate solution is obtained when a single-lane distribution factor is used and the weight of the alongside vehicle is "superimposed" onto the permit vehicle weight and assumed to be acting in the same lane. Because the vehicle in the checking equation is the permit vehicle, the live load factor is adjusted to account for the added load effect of the alongside vehicle. For example, a very high permit weight and a small expected alongside vehicle weight suggest that a single-lane case should be used to govern the checking. In general, the live load factor increases for larger ADTT and for the number of times that the permit vehicle is expected to use the span.

Two approaches to treat different weight vehicles in the two lanes were compared. The first approach, described in the previous section, uses a sum of the truck weights in each lane and multiplies the sum by the two-lane distribution factor. The second approach uses a formula presented by Zokaie (Zokaie, 1998). The second approach follows a suggestion developed by Modjeski and Masters for PennDOT. (Zokaie's formula is presented as Equation 42.)

In order to check whether the one- or two-lane case should control, it is necessary to compare the single-lane distribution factor, g_1, with the two-lane distribution factor, g_m. It is important in making this comparison to remove the implied multiple-presence term given as "1.2," which is built into the AASHTO LRFD formulas for g_1. The values of one- and multiple-lane distribution factors without the multiple-presence corrections are presented in Table 5 for several typical beam types and spans.

In the first approach, the live load factor to reach the target reliability is given in Equation 39 as

$$W_T = P + W_R \tag{40}$$

where

P = the weight of the permit vehicle and
W_R = expected maximum vehicle weight alongside the permit.

The factored live load effect in a girder used for component checking, L_n, is given as

$$L_n = \gamma_{L,\text{two lane}} P g_m \tag{41}$$

where g_m is the two-lane distribution factor.

In the second approach, the format presented by Zokaie gives the equivalent load effect (W_1) due to vehicles of weight P (permit) in one lane and W_R in the other lane as

$$W_1 = P g_1 + W_R (g_m - g_1) \tag{42}$$

where g_1 is the single-lane distribution factor. In Equation 42, the equivalent load effect can be seen as the contribution of the permit weight as a single-lane effect and then treating the alongside load W_R first as if it were present in both lanes and then subtracting a one-lane effect. Equation 42 is approximate because the distribution factors in the formulas for g_m and g_1 relate to the maximum girder effect, which is not necessarily the same girder for both the single-lane and multilane loadings, nor is the position of the vehicle in the transverse direction necessarily the same to produce maximum single-lane and multilane load effects.

Some examples presented by Zokaie showed the accuracy of this approximation. The purpose of Equation 42 is to analyze multilane loads with different vehicles in each lane by using the published AASHTO distribution formulas without having to perform a new grillage analysis. The load effect W_1 may be considered as a single-lane maximum girder effect. To find the required live load factor for the permit loading with a single-lane distribution factor, use the reference load effect in Equation 30 and maintain the same reference ratio of nominal girder resistance (R_n) to expected maximum girder live load effect ($\bar{L}$). From Equation 28, the reference ratio for the girder load effect is

$$\frac{R_n}{\bar{L}} = \frac{1.8 \times 72\, g_m}{120\, g_m} \tag{43}$$

where the reference live load factor is found above as 1.8 for the severe traffic case corresponding to the maximum expected equal loads of 120 kips in each lane. The terms g_m in Equation 43 are given simply to emphasize that girder resistance and girder load effects are being compared. To maintain this same ratio of resistance divided by expected load when a single-lane load is applied, the live load factor for the single-lane should be found from

$$\frac{R_n}{\bar{L}} = \frac{1.8 \times 72\, g_m}{120\, g_m} = \frac{\gamma_{L,\ \text{one lane}} \times P g_1}{W_1} \tag{44}$$

where the permit weight P is multiplied by the live load factor and distribution factor g_1 to obtain the girder resistance. W_1 is the maximum expected one-lane girder loading. Solving for the one-lane live load factor by substituting for W_1 from Equation 42 gives

$$\gamma_{L,\ \text{one lane}} = \frac{1.8\,(72)\,[P g_1 + W_R(g_m - g_1)]}{(120)\ P g_1} \tag{45}$$

The factored live load effect for component checking, L_n, is then given as

$$L_n = \gamma_{L,\ \text{one lane}}\ P g_1 \tag{46}$$

The load effect using the two-lane distribution, Equation 41, and one-lane distribution, Equation 46, should be compared to see which case governs. For example, Table 5, which compares g_m and g_1 for several typical bridges, has an average value of g_m/g_1 of about 1.7. This ratio based on grillage analysis is considerably higher than that found with previous AASHTO ratios of g_m and g_1 using the “S over 5.5” and “S over 7” values for multilane and one-lane distributions. For comparison, let the load effect ratio (c) be written as

$$c = L_n(\text{two lane})/L_n(\text{one lane}) \tag{47}$$

In addition, let a equal g_m/g_1 and b equal W_R/P. Using Equations 40 through 46 and the nondimensional parameters, a, b, and c leads to

$$c = \frac{0.5\,(1 + b)\,a}{1 + b\,(a - 1)} \tag{48}$$

Equation 48 can be used to compare one-lane and two-lane load effects. For example, for $a = 1.7$ and $b = 0.5$, $c = 0.94$, which means the one-lane case governs. For $a = 1.4$ and $b = 1.1$, $c = 1.02$ and the two-lane case governs.

Table 8 presents the results of the live load factors for the one-lane case, assuming the average ratio of g_m/g_1 is equal to 1.7. Table 8 presents results for the controlling live load

TABLE 8 Two-Lane Routine Permits, Minimum Live Load Factors, and One-Lane Checking Case

					Live Load Factor-γ_L for One-Lane Check*			
N_p	Y	ADTT	N_R	W_R	$P = 80$k	$P = 125$k	$P = 150$k	$P = 200$k
10	2	100	36.5	103	2.05	1.70	1.60	1.47
		1000	73	108	2.10	1.73	1.62	1.49
		5000	4487	119	2.20	1.80	1.68	1.53
	5	100	91	109	2.16	1.74	1.63	1.49
		1000	183	114	2.16	1.77	1.65	1.51
		5000	1217	124	2.25	1.83	1.70	1.55
100	2	100	365	118	2.20	1.79	1.67	1.53
		1000	730	122	2.23	1.82	1.69	1.54
		5000	4867	132	2.33	1.88	1.75	1.56
	5	100	912	123	2.24	1.82	1.70	1.54
		1000	1825	127	2.28	1.85	1.72	1.56
		5000	12167	136	2.37	1.90	1.77	1.59

N_p - number of permits per day
side by side prob.-$P_{s/s}$ = 0.005, 0.01, 1/15 for ADTT = 100,1000, and 5000 respectively
number of side by side events, $N_R = N_p \times P_{s/s} \times 365 \times$ years
$W_R = 68 + t(N_R)\ 18$
$t(N_R)$ - constant from normal prob. Table for probability level of $(1 - 1/N_R)$
For: γ_L, one lane, See Eqn. 45
[assume $g_m/g_1 = 1.7$, for Table 7]
e.g., line 1: $N_R = 36.5$; $t(N_R) = 1.92$; $W_R = 103$;

$$\gamma_L = \frac{1.8 \times 72\ [80 + 103\ (1.7 - 1.0)]}{120 \quad \times \quad 80} = 2.05$$

* To estimate average equivalent two-lane live-load factor, divide one-lane factor by 1.7

factor for different permit percentages, exposure periods, ADTT, and permit weights. The load factors increase with traffic and permit volume and decrease with the weight of the permit vehicle. In order to convert the one-lane live load factors shown in Table 8 into equivalent live load factors for two lanes (i.e., produce the same nominal girder load effects), the factors shown in Table 8 should be divided by 1.7. This latter value is the average ratio of g_m/g_1 used for computing the factors in Table 8. Results of both Table 7 for two-lane cases and Table 8 for one-lane cases were combined to select the recommended live load factors for the Evaluation Manual presented herein in Table 9. Different ratios of g_m/g_1 were also considered in making the selection of factors.

It is noted from a number of comparisons that, for most cases of routine permits, the two-lane case distribution governs. For the special permits, the one-lane case is recommended. Thus, in the Evaluation Manual, the two-lane distribution is used for routine permits. For special permits, the one-lane distribution is used, which places the permit vehicle in one-lane for the checking. In both cases, the corresponding live load factors are adjusted to make the factors accurately represent any load effect resulting from alongside vehicles.

To avoid having a special permit made acceptable for a bridge that is posted or for which routine permits are restricted, the range of special permits should start at the level at which the routine permits stop. Above some recognized weight limit, all permits should be considered special (or escorted). Given that the relative contribution of alongside vehicles decreases as the permit weight level increases, the corresponding live load factor also is decreased. The next section presents the analysis of the special permit load factors that were recommended for the Evaluation Manual.

7.3 SPECIAL PERMITS

For the special permit case, it is recommended for greater accuracy to use a one-lane distribution analysis. As described above, the method for selecting the live load factors for maintaining the target reliability is to provide the reference level of the ratio of mean resistance to mean maximum live load effect. This method applies also to the special permit case.

TABLE 9 Recommended Table of Live Load Factors**

Evaluations					
Load Rating - Legal Loads					
		D.F.	**ADTT***	**Load Factor***	
		two lane	5000	1.8	
			1000	1.6	
			100	1.4	
Permit Checks					
				Load Factors Permit Weight*	
Permit Type	**Traffic**	**D.F.**	**ADTT**	**80–100 kips**	**>150 kips**
Routine	mix	two lane	100	1.4	1.10
			1000	1.6	1.20
			5000	1.8	1.30
Special			**ADTT**	**Number of Crossings**	**Load Factor for Permit Check**
	escorted	one lane	—	—	1.15
	mix	one lane	100	1	1.35
			1000	1	1.40
			5000	1	1.50
Special	mix	one lane	100	less than 100	1.30
			1000	”	1.40
			5000	”	1.45

* Interpolate the load factor considering ADTT and permit weight.
** See Tables 6.4.4.2.3.-1 and 6.4.5.4.2-1 in Evaluation Manual for legal load rating and permit load rating, respectively.

Applying this method assumes the target reliability is satisfied with this ratio and that the uncertainties for special permit loading are the same as for routine traffic. This is a conservative assumption, given the wide array of variables in the reliability modeling and the fact that site-to-site uncertainty in estimating the maximum truck weight event and corresponding load effects will be greater for random traffic than for special permits.

In the analysis of the one-lane loading, the expected maximum load effect, W_1, is found from the weight contribution of the permit vehicle, P, plus the influence of the maximum expected weight in the adjoining lane denoted as W_R. The required load factor to maintain the reliability level must satisfy

$$\gamma_{L,\ \text{one lane}} = 1.8\,\frac{72\ W_1}{P\,120} \tag{49}$$

where

$$W_1 = P + W_R \tag{50}$$

Substituting Equation 50 into Equation 49 for W_1 is equivalent to using a conservative ratio of g_m/g_1 equal to 2.0 in the Zokaie formula in Equation 42. This substitution results in live load factors that are conservative for the one-lane checking case. The results of the calculations for the special permit load factors are detailed in Table 10 for a range of ADTT, the number of permit crossings, and the permit weight. The influence of the passing lane and, hence, ADTT is small until the number of repetitions of the permit vehicle exceeds about 10. The values in Table 10 provided the recommended factors in the Evaluation Manual (Table 9 herein) for the special permit category.

The permit factors in this report are intended to assist the needs of the various agencies wishing to go beyond the recommendations in the Evaluation Manual. Table 10 presents the special permit cases in terms of ADTT, a broad range of permit weights, and the expected number of permits. Several factors must be considered. For low-permit weights, the expected maximum load effect can result from random traffic, rather than from the permit loading. Thus, for the 80-kip level of permit, the span should be controlled by the live load factors from the legal load rating level (i.e., 1.8, 1.6, and 1.4) for the three levels of ADTT using the two-lane loading factors. As permit load increases, the relative effect of alongside vehicles decreases and that is why the load factors are shown to decrease with permit weight.

For the case of a single special permit, there is a very small probability of any influence from a random alongside vehicle. As the number of times that the permit vehicle is allowed to cross the span increases, the expected maximum alongside weight from any crossing increases. When the number of

TABLE 10 Special Permits, Minimum Live Load Factors, One-Lane Checking Controls

Traffic Mix	D.F. One Lane	W_R	ADTT	Number of Crossings- Total / Eval. Period	Load Factors - One Lane / Permit Weight - kips: 80	125	150	200
		.3	100	1	1.08	1.08	1.08	1.08
		.7	1000		1.09	1.09	1.08	1.08
		4.5	5000		1.14	1.12	1.11	1.10
		3.4	100	10	1.13	1.11	1.10	1.10
		6.8	1000		1.17	1.14	1.13	1.12
		45.3	5000		1.63	1.47	1.41	1.32
		34	100	100	1.54	1.37	1.32	1.26
		68	1000		2.00	1.67	1.57	1.45
		87	5000		2.25	1.83	1.71	1.55
		83	100	1000	2.20	1.80	1.68	1.53
		91	1000		2.31	1.87	1.74	1.57
		107	5000		2.52	2.00	1.85	1.66

N_R = number of crossings $\times P_{s/s}$, where the latter is 1/15, 0.01 and 0.001 for ADTT = 5000, 1000 and 100, respectively. Find W_R and W_T as in Table 8, assuming $g_m/g_1 = 2.0$, and $W_1 = P + W_R$
For, γ_L,one lane, see Eqn. 49

example: $P = 200$ k, 1000 crossings and 5000 ADTT case:
$N_R = 1000 \times 1/15 = 66.7$, $t(N_R) = 2.17$ and $W_R = 68 + 18(2.17) = 107$ and,
$W_1 = 200 + 107 = 307$;

$$\gamma_L,\ \text{one lane} = \frac{307 \times 1.8 \times 72}{200 \times 120} = 1.66$$

crossings of the special permit vehicle reaches a significant number of repetitions, then the live load factors for routine permits (two-lane loading) should be used.

As noted above, some agencies and consulting firms have analyzed special permits accounting for the special permit vehicle in one lane supplemented by a legal vehicle in the adjacent lane. As shown in Table 10, on the order of 100 crossings of the special permit vehicle are required before the expected alongside vehicle has a weight equal to that of a legal vehicle. Thus, for the situations presented in the Evaluation Manual, it is not necessary to model any permit movements with an analysis of one permit load in one lane and a legal vehicle in the other lane. For routine permit evaluation, a permit is placed in each lane and the two-lane distribution factor is used. For special permits, a single permit is placed in one lane with no other vehicle in the second lane. Use of the corresponding recommended live load factors takes account of the contributions from vehicles in the second lane.

Table 10 shows that, for large numbers of crossings of the permit vehicle in the range of 1000 crossings, the load factors in Table 10 increase and reach the corresponding values for routine permits. Note that the load effect for the routine permits is based on two-lane loading, while for special permits, the one-lane loading distribution factors applies. For a single-crossing event, the alongside weight is negligible and is similar to an escorted permit.

This aspect of the analysis (i.e., that the special permit vehicle with only a single crossing acts for the purposes of estimating the maximum load effect as though it were a controlled escort) may be difficult for some agencies to accept. However, the aspect of the analysis does agree with probability notions, although agencies may choose to "reward" escorted crossings or "penalize" non-escorted crossings. Rewarding escorted crossings may be done for traffic safety and for enforcing speed and/or lateral position requirements for the permit vehicle as it crosses the bridge.

One alternative considered is whether to raise the target beta for special permit crossings on the basis of economic cost/benefit grounds. Agencies should consider whether there should be some increase in the required target reliability because the benefits to the public of only one crossing by a permit vehicle may not be worth the added risk. In Chapter 6, it was shown that an increase in load factor by about 1.35 will raise beta about 1.0 for random traffic loadings. It is recommended that decisions be based on the reliability analysis rather than on adding conservatism in the distribution analysis (i.e., using multilane factors) because such analysis factors add varying amounts of increased safety, depending on span geometry, and may be inconsistent with the overall goals of a uniform reliability level for the system.

The Commentary in the Evaluation Manual mentions raising live load factors for special vehicle cases such as superloads. The live load factors can be increased at the discretion of the agency. The proposed increase in factors given in the commentary for such cases raises the reliability levels to the design or inventory level in a consistent manner. Regarding superloads, these loads may represent the largest load that a bridge has yet seen in its lifetime. Checking superloads is unlike checking routine permits where the bridge has likely carried such load levels in the past. Because of the confidence resulting from such past "proof-tests," a higher reliability for heavy superloads may be warranted. The increased factor mentioned in the Evaluation Manual commentary is to increase γ_L for the superloads from a value of 1.15 to 1.35.

Another issue is that a special permit vehicle will pass over many bridges, and, from a system point of view, an agency may be concerned with any one of the bridges being damaged. However, such highway system considerations in terms of the risk of *any* bridge failing are not considered in either the AASHTO LRFD design specifications or any other aspects of the Evaluation Manual. The aim in this report is to present methodology—highway agencies can be expected to adopt those policies that work best in their jurisdictions.

7.3.1 Short Spans and Long Combination Vehicles

The Evaluation Manual noted an inconsistency in long combination permit vehicles checked for crossing short-span bridges. For example, if a tridem from a short vehicle is of the same weight as that of a tridem from a combination vehicle and the tridem alone controls the short-span load effects, then the two vehicles should be rated the same. A problem may arise because the recommended live load factor decreases for heavier routine permit weights, as shown in Table 9. This reduction is based on vehicle gross weight and accounts for vehicle presence alongside the permit. For simplicity, the alongside influence was based on gross weight contributions to load effects.

To avoid this dilemma, it was recommended that the gross weight of the vehicle used to interpolate for the load factor be that portion of the vehicle that is on the span when the maximum live load effect occurs. Axles and groups of axles that are not on the span when the maximum moment or shear is computed should not be used as part of the weight total in selecting the live load factors from the Evaluation Manual.

CHAPTER **8**

BRIDGE TESTING

The Evaluation Manual makes explicit reference to field testing as an aid to the bridge rating process. Two types of testing may be considered, depending on the capacity limitations noted during the rating calculations. The first type of test is a diagnostic test to support a more precise load distribution to the individual components. Such a test is needed when structural models, including grillage or finite element methods, cannot accurately predict behavior because of uncertainties in member properties, boundary conditions, and influence of secondary members. A field study with diagnostic models helps to improve or validate a structural analysis model.

The second type of test is a “proof test” and provides information about the strength of the bridge. The test is especially needed when components may have “hidden” details such as unknown reinforcement in concrete spans and unknown bracing contributions in steel structures or have boundary conditions and member interaction effects that cannot be easily modeled.

Specific information on personnel qualifications for performing tests, procedures for conducting a test, how to interpret the output, and how to calibrate results to reliability targets are provided in a recent bridge testing manual study (NCHRP, 1998). This study, which is referenced in the Evaluation Manual, also contains a vast list of bridge tests and examples of rating bridges using test results. An appendix in the bridge testing report (NCHRP, 1998) shows how field tests improve accuracy of bridge performance assessment and reduce the uncertainties of load effect analysis, dynamic response, and strength variables. These data can then be used to modify the load and resistance factors in the LRFD rating formulas to achieve the target reliability indexes.

CHAPTER 9

DIRECT USE OF BETAS IN RATING

An alternative rating procedure that allows a direct use of safety indexes (betas) in the bridge-rating decision process may be useful in certain situations. In general, design specification organizations have avoided recommending involvement by designers in applying safety indexes in selecting design parameters. The reason is that design is basically a production process in which a specification provides nominal strength and loading formulas and, with the aid of safety margins (in either the traditional safety factor format or the reliability-based load and resistance factor format), a design checking procedure. Checking procedures lend themselves to computerization and to consistency among different designers. In the LRFD format, only the code writers deal with probabilistic analysis and with associated questions of target safety and optimum risk levels.

In evaluating existing structures, however, there is a growing interest in having engineers perform a direct risk assessment to determine the future course of rehabilitation investments and the balance of replacement costs with continued operation. Structural examples include evaluation of existing offshore platforms and aging aircraft. Both of the latter cases have received considerable research and structural reliability applications, and the term “geriatric structures” is entering the engineering vocabulary. In the evaluation of existing structures, there is ample opportunity to evaluate the tradeoffs of risk and costs analytically.

For bridge evaluation, the following situations may exist that would lead an agency to consider a direct use of reliability methods by the evaluating engineers:

- Bridges whose loss would represent significant economic consequences, such as long spans and suspension bridges, in which costs and consequences for a variety of threats (e.g., deterioration, live load, earthquake, scour, and collisions) must be considered simultaneously). Such bridges are not typically considered in the specifications.
- Evaluation of bridge types not covered by the standard specifications.
- Bridges whose live loading characteristics may differ markedly from the descriptions contained in the Evaluation Manual. For example, bridges that must carry a special type of overloaded vehicle or bridges whose principal traffic is trucks (which changes the loading effects from those considered herein). Other examples may be spans in which ADTT is exceptionally high or in which there are many more frequent multiple-presence situations because of traffic lights, bridge geometry, and access conditions. In addition, long-span bridges may require different live load models than considered herein. Typically, a long-span live load event will be affected by “trains” of trucks in a single lane, as well as by having bridge components with much higher dead-to-live-load ratios than used in the calibration of the AASHTO LRFD specification.
- Bridges with material properties markedly different from those considered herein. Such material properties may relate to experimental materials, such as plastics or use of epoxies for attachments or levels of deterioration, and material distributions markedly different from those discussed in the Evaluation Manual. These situations would also cover cases where a site-applicable test program has been conducted.
- Bridge spans controlled by analysis predictions that lead to distributions of uncertainty of load effects greatly different from those reflected in the Evaluation Manual.
- Bridge types for which a significant body of field experience has been collected, either favorable or unfavorable, that suggests the computed reliability index should incorporate such data. Examples from other fields include offshore platforms where Bayesian probability methods have utilized field observations after hurricane events to update or improve the values of the computed safety indexes.
- A direct risk assessment of a bridge may be useful when the bridge owner is using such risks as part of an overall highway and bridge safety management system. In such cases, structural risks of the type reflected herein are notional values applicable to a particular industrial perspective. Combining and manipulating risks from different sources may not always lead to appropriate balance. For example, many risks result from human errors and from unknown technological factors not expressed in safety index calculations. The treatment of only the notional risks may not lead to an optimum solution.

A direct use of risk analysis in the bridge assessments should be carried out only by engineers familiar with the basic methodology of structural reliability technology. The applicable statistical database should be based on sufficient observations and measurements such that there is opportunity to calibrate these data to observed bridge performance. The engineers involved in such ratings should demonstrate experience in the derivation of LRFD specifications for structural design and evaluation criteria.

The direct rating approach with safety indexes will be explained herein as follows. The aim is to solve for the safety index of a given bridge component. Associated with this calculation will be the rating factor so that a relationship between safety indexes and rating factors can be obtained. A component rating is acceptable if the computed safety index exceeds a prescribed level.

These safety indexes are called notional values in the structural reliability literature. The corresponding risk values provide only the risk that load effect exceeds resistance for the specified limit states considered. Such risks do not include the following possibilities:

- Failures because of gross negligence in loading and/or construction,
- Failures because of human errors such as errors in computation,
- Failures in modes that are ignored by the evaluator, and
- Failures in modes that are poorly understood technologically.

Reliability procedures are not substitutes for a limited technological understanding or a limited applicable database. In order to apply safety index methods, a limit state failure function must be available. The existence of a failure function clearly implies that the technology is well understood and there is no debate that, given a realization of the random variables, all engineers will agree whether the component has survived or failed.

A first-order reliability format may be sufficient, although advanced reliability formats may be necessary. The safety index, β, may be written as in Equation 27 as

$$\beta = \frac{Ln\,\overline{R}/\overline{S}}{\left[V^2{}_R + V^2{}_S\right]^{1/2}} \tag{51}$$

This lognormal format allows beta to be calculated from the mean load effect, $\overline{S}$, the mean resistance, $\overline{R}$, and their respective coefficients of variation, V_S and V_R. Solving for the load term, gives

$$\overline{S} = \overline{R}\ \exp\left[-\beta\left[V^2{}_R + V^2{}_S\right]^{1/2}\right] \tag{52}$$

For rating a span, the component load effect is composed of dead and live load (including impact), so that substituting gives

$$\overline{S} = \overline{D} + (\text{R.F.})\ \overline{L} = \overline{R}\ \exp\left[-\beta\left[V^2{}_R + V^2{}_S\right]^{1/2}\right] \tag{53}$$

Equation 53 allows for a direct solution of the rating factor, given the mean values of resistance, dead load, and live load; their respective coefficients of variation; and the target safety index. Alternatively, given these same statistical parameters, the safety index for the component can be computed directly from Equation 51.

To pursue this analysis, each random variable must be considered separately to determine each random variable's respective bias and COV. The resistance variable is intended to cover natural variability of materials, fabrication uncertainties, and professional judgments. Professional judgment pertains to the method of calculating component strength and reflects how experimental tests compare with calculated predictions. Three variables (i.e., material, fabrication, and test variations) must all be included in the statistical parameters (bias and COV) of the resistance variable, *R*.

Typical resistance values for new construction in steel and prestressed concrete are given in the AASHTO calibration report (Nowak, 1999). Bias values on the order of 1.1 and COVs in the 10- to 15-percent range are typical for the overall resistance random variable, *R*, of common structural types. A lognormal distribution is usually described for resistance. For other material or component applications not covered in that calibration report, users must either provide their own test data or else find relevant tests in the literature.

Dead loads consist of the effects of permanent weights on the structure actually present at the time of evaluation. The dead load random variable must reflect both the uncertainty of the weight of the components and the uncertainty of the calculation of the dead load effects on the member being checked. A normal distribution was used by Nowak for describing the dead load uncertainty. While there are some data reported (Nowak, 1999) to consider the bias and COV of the material weights, there are few data to substantiate the dead load analysis uncertainty. One important issue in modeling the dead load variable is to use site data on asphalt overlay and soil density. The use of site data could significantly reduce the respective uncertainties compared with values used at the design stage. Readers should consult the Nowak report for more dead load data.

The live load variable should include a number of factors, such as truck weight data and distribution of truck types, dimensions, load distributions to the axles, dynamic allowance uncertainties, and girder distribution uncertainties. Some work published offered values in a range of applications (Ghosn and Moses, 1986). For example, a COV of 10 percent was used to cover site-to-site variations in the truck weight variable, W_{95}. This COV could be reduced if data were obtained at a site from WIM studies. The COV of distribution factors (i.e., the analysis random variable) for different bridge types ranged from 8 to 13 percent, depending on respective field measurements. However, analysis COV can usually be reduced as more sophisticated structural analysis is performed, such as the Imbsen formulas and grillage or finite element analysis. The COV of dynamic behavior has also been given. (See Ghosn and

Moses, 1986, and Nowak, 1999.) The principal physical variables affecting the dynamic response are surface roughness and support bump, which are properties of the bridge being evaluated.

A model for multiple presence of heavy vehicles on the bridge span is also needed to predict maximum live load effects. The presence of permit vehicles must also be reflected, as outlined in Chapter 7. These individual live load variables should be combined in a consistent manner to produce an overall bias and COV for the live load effects on a component. Such calculations may include simulation (Nowak, 1999) or analytical models (Ghosn and Moses, 1986). Simplified formats of the type described herein may be applied if the user is familiar with the basic assumptions and with the applicability of the assumptions to the site being investigated.

Nowak described the live load uncertainty with a normal distribution. This distribution fit nicely into his calculation model for the safety index. Moses and Ghosn, however, used a log-normal description of the live load effect because this random variable depends on a product of independent random variables, including truck weight spectra, analysis, and dynamics. The lognormal distribution is appropriate when the variable being modeled is a product of other random variables. The log-normal model is frequently used in structural reliability to describe load effects, including live loads, wind, and wave. Simply changing the live load effect random variables from normal to lognormal in a reliability program *reduces* the calculated safety index in the LRFD design calibration by about 0.3 compared with the target of 3.5 using a normal model for live load. Thus, it is is very important in the direct approach to rating using betas to consider such questions as the selection of distribution type. Otherwise, consistency with respect to the LRFD design and evaluation models will be lost.

In addition to component reliability, the direct application of safety indexes as a measure for bridge rating should encompass system reserves and redundancy issues. These analyses require either advanced, nonlinear structural assessment programs or the use of simplified tables presented in the NCHRP redundancy project (Ghosn and Moses, 1998). If risk tradeoffs are being contemplated because of the calculated safety indexes (including the setting of priorities for bridge rehabilitation), it is essential that bridge system, and not just component capacity, be considered. In the redundancy project, system analyses included for each bridge example (a) the ultimate capacity to resist collapse due to overloads, (b) the bridge response, which leads to loss of functionality (e.g., intolerable displacements), and (c) damage mitigation (i.e., the ability to withstand collapse in the event a bridge suffers a fatigue, collision, scour, or other type of damage scenario). A bridge to be denoted as redundant should be checked and shown to be satisfactory for all three analysis cases.

The criteria for determining acceptability of a rating using the direct use of betas provided in this chapter should be a satisfactory reliability index. Before an acceptable target beta is fixed, it should be validated with past performance information. Betas have served as notional measures of safety and should not be confused with past actuarial experiences. There are several areas in this report where factors are calibrated using conservative assumptions of performance. If all such conservative assumptions were to be replaced by their unbiased values, it is likely that the safety indexes reported herein as target values would be much higher. The removal of conservative assumptions can only be carried out when more data and performance experience is available. Also, it is important to maintain consistency between any risk-based evaluation methodology and the corresponding design methodology now contained in the new AASHTO LRFD specifications.

CHAPTER **10**

CONCLUSIONS

A consistent approach has been presented to calibrate live load factors for the proposed AASHTO Evaluation Manual. The aim of the calibration has been to achieve uniform target reliability indexes over the range of applications, including design load rating, legal load rating, posting, and permit vehicle analysis. As much as possible, the database of the recently approved AASHTO LRFD Design Specifications has been utilized. The loading database has been based on an extreme truck weight spectra (Nowak, 1999). The factors recommended herein are consistent with this database. If truck weights continue to increase, then the factors herein should receive renewed investigation. No set of evaluation factors for bridges will protect against extreme heavy truck overloads or the failure to properly inspect and maintain the bridge. The factors recommended herein are intended to be a part of an overall bridge management system that considers proper load enforcement and bridge maintenance policies.

The goal in the calibration effort for the Evaluation Manual has been to use the state of the art in structural reliability modeling and bridge data. Further work needs to be done. Such work should be aimed at improving these models and incorporating additional field studies and bridge performance assessments. Such investigations should lead to evaluation criteria that allow bridge agencies to consistently perform tradeoffs of risk and costs, particularly in regard to site data acquisition and performance monitoring tools. Evaluation of bridges is an ongoing activity for which the benefits of increasing the projected safe life of the bridge may greatly exceed the costs of improved monitoring, inspection, and evaluation technologies.

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APPENDIX A

NORMAL DISTRIBUTION TABLE

Table of Standard Normal Probability $\Phi(x) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{x} \exp\left(-\frac{1}{2}x^2\right)$

x	$\Phi(x)$	x	$\Phi(x)$	x	$\Phi(x)$
0.0	0.500000	0.40	0.655422	0.80	0.788145
0.01	0.503989	0.41	0.659097	0.81	0.791030
0.02	0.507978	0.42	0.662757	0.82	0.793892
0.03	0.511967	0.43	0.666402	0.83	0.796731
0.04	0.515953	0.44	0.670031	0.84	0.799546
0.05	0.519939	0.45	0.673645	0.85	0.802338
0.06	0.523922	0.46	0.677242	0.86	0.805106
0.07	0.527903	0.47	0.680822	0.87	0.807850
0.08	0.531881	0.48	0.684386	0.88	0.810570
0.09	0.535856	0.49	0.687933	0.89	0.813267
0.10	0.539828	0.50	0.691462	0.90	0.815940
0.11	0.543795	0.51	0.694974	0.91	0.818589
0.12	0.547758	0.52	0.698468	0.92	0.821214
0.13	0.551717	0.53	0.701944	0.93	0.823814
0.14	0.555670	0.54	0.705402	0.94	0.826391
0.15	0.559618	0.55	0.708840	0.95	0.828944
0.16	0.563559	0.56	0.712260	0.96	0.831472
0.17	0.567495	0.57	0.715661	0.97	0.833977
0.18	0.571424	0.58	0.719043	0.98	0.836457
0.19	0.575345	0.59	0.722405	0.99	0.838913
0.20	0.579260	0.60	0.725747	1.00	0.841345
0.21	0.583166	0.61	0.729069	1.01	0.843752
0.22	0.587064	0.62	0.732371	1.02	0.846136
0.23	0.590954	0.63	0.735653	1.03	0.848495
0.24	0.594835	0.64	0.738914	1.04	0.850830
0.25	0.598706	0.65	0.742154	1.05	0.853141
0.26	0.602568	0.66	0.745373	1.06	0.855428
0.27	0.606420	0.67	0.748571	1.07	0.857690
0.28	0.610261	0.68	0.751748	1.08	0.859929
0.29	0.614092	0.69	0.754903	1.09	0.862143
0.30	0.617911	0.70	0.758036	1.10	0.864334
0.31	0.621719	0.71	0.761148	1.11	0.866500
0.32	0.625517	0.72	0.764238	1.12	0.868643
0.33	0.629300	0.73	0.767305	1.13	0.870762
0.34	0.633072	0.74	0.770350	1.14	0.872857
0.35	0.636831	0.75	0.773373	1.15	0.874928
0.36	0.640576	0.76	0.776373	1.16	0.876976
0.37	0.644309	0.77	0.779350	1.17	0.878999
0.38	0.648027	0.78	0.782305	1.18	0.881000
0.39	0.651732	0.79	0.785236	1.19	0.882977

(*continued on next page*)

Table of Standard Normal Probability $\Phi(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{x} \exp\left(-\frac{1}{2}x^2\right)$ (*Continued*)

x	$\Phi(x)$	x	$\Phi(x)$	x	$\Phi(x)$
1.20	0.884930	1.70	0.955435	2.20	0.986097
1.21	0.886860	1.71	0.956367	2.21	0.986447
1.22	0.888767	1.72	0.957284	2.22	0.986791
1.23	0.890651	1.73	0.958185	2.23	0.987126
1.24	0.892512	1.74	0.959071	2.24	0.987455
1.25	0.894350	1.75	0.959941	2.25	0.987776
1.26	0.896165	1.76	0.960796	2.26	0.988089
1.27	0.897958	1.77	0.961636	2.27	0.988396
1.28	0.899727	1.78	0.962462	2.28	0.988696
1.29	0.901475	1.79	0.963273	2.29	0.988989
1.30	0.903199	1.80	0.964070	2.30	0.989276
1.31	0.904902	1.81	0.964852	2.31	0.989556
1.32	0.906582	1.82	0.965621	2.32	0.989830
1.33	0.908241	1.83	0.966375	2.33	0.990097
1.34	0.909877	1.84	0.967116	2.34	0.990358
1.35	0.911492	1.85	0.967843	2.35	0.990613
1.36	0.913085	1.86	0.968557	2.36	0.990863
1.37	0.914656	1.87	0.969258	2.37	0.991106
1.38	0.916207	1.88	0.969946	2.38	0.991344
1.39	0.917736	1.89	0.970621	2.39	0.991576
1.40	0.919243	1.90	0.971284	2.40	0.991802
1.41	0.920730	1.91	0.971933	2.41	0.992024
1.42	0.922196	1.92	0.972571	2.42	0.992240
1.43	0.923641	1.93	0.973197	2.43	0.992451
1.44	0.925066	1.94	0.973810	2.44	0.992656
1.45	0.926471	1.95	0.974412	2.45	0.992857
1.46	0.927855	1.96	0.975002	2.46	0.993053
1.47	0.929219	1.97	0.975581	2.47	0.993244
1.48	0.930563	1.98	0.976148	2.48	0.993431
1.49	0.931888	1.99	0.976705	2.49	0.993613
1.50	0.933193	2.00	0.977250	2.50	0.993790
1.51	0.934478	2.01	0.977784	2.51	0.993963
1.52	0.935744	2.02	0.978308	2.52	0.994132
1.53	0.936992	2.03	0.978822	2.53	0.994297
1.54	0.938220	2.04	0.979325	2.54	0.994457
1.55	0.939429	2.05	0.979818	2.55	0.994614
1.56	0.940620	2.06	0.980301	2.56	0.994766
1.57	0.941792	2.07	0.980774	2.57	0.994915
1.58	0.942947	2.08	0.981237	2.58	0.995060
1.59	0.944083	2.09	0.981691	2.59	0.995201
1.60	0.945201	2.10	0.982136	2.60	0.995339
1.61	0.946301	2.11	0.982571	2.61	0.995473
1.62	0.947384	2.12	0.982997	2.62	0.995603
1.63	0.948449	2.13	0.983414	2.63	0.995731
1.64	0.949497	2.14	0.983823	2.64	0.995855
1.65	0.950529	2.15	0.984222	2.65	0.995975
1.66	0.951543	2.16	0.984614	2.66	0.996093
1.67	0.952540	2.17	0.984997	2.67	0.996207
1.68	0.953521	2.18	0.985371	2.68	0.996319
1.69	0.954486	2.19	0.985738	2.69	0.996427

(*continued*)

Table of Standard Normal Probability $\Phi(x) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{x} \exp\left(-\frac{1}{2}x^2\right)$ (*Continued*)

x	$\Phi(x)$	x	$\Phi(x)$	x	$1-\Phi(x)$	
2.70	0.996533	3.20	0.999313	3.70	0.999892	
2.71	0.996636	3.21	0.999336	3.71	0.999896	
2.72	0.996736	3.22	0.999359	3.72	0.999900	
2.73	0.996833	3.23	0.999381	3.73	0.999904	
2.74	0.996928	3.24	0.999402	3.74	0.999908	
2.75	0.997020	3.25	0.999423	3.75	0.999912	
2.76	0.997110	3.26	0.999443	3.76	0.999915	
2.77	0.997197	3.27	0.999462	3.77	0.999918	
2.78	0.997282	3.28	0.999481	3.78	0.999922	
2.79	0.997365	3.29	0.999499	3.79	0.999925	
2.80	0.997445	3.30	0.999517	3.80	0.999928	
2.81	0.997523	3.31	0.999533	3.81	0.999930	
2.82	0.997599	3.32	0.999550	3.82	0.999933	
2.83	0.997673	3.33	0.999566	3.83	0.999936	
2.84	0.997744	3.34	0.999581	3.84	0.999938	
2.85	0.997814	3.35	0.999596	3.85	0.999941	
2.86	0.997882	3.36	0.999610	3.86	0.999943	
2.87	0.997948	3.37	0.999624	3.87	0.999946	
2.88	0.998012	3.38	0.999638	3.88	0.999948	
2.89	0.998074	3.39	0.999650	3.89	0.999950	
2.90	0.998134	3.40	0.999663	3.90	0.999952	
2.91	0.998193	3.41	0.999675	3.91	0.999954	
2.92	0.998250	3.42	0.999687	3.92	0.999956	
2.93	0.998305	3.43	0.999698	3.93	0.999958	
2.94	0.998359	3.44	0.999709	3.94	0.999959	
2.95	0.998411	3.45	0.999720	3.95	0.999961	
2.96	0.998462	3.46	0.999730	3.96	0.999963	
2.97	0.998511	3.47	0.999740	3.97	0.999964	
2.98	0.998559	3.48	0.999749	3.98	0.999966	
2.99	0.998605	3.49	0.999758	3.99	0.999967	
3.00	0.998650	3.50	0.999767	4.00	0.316712	E-04
3.01	0.998694	3.51	0.999776	4.05	0.256088	E-04
3.02	0.998736	3.52	0.999784	4.10	0.206575	E-04
3.03	0.998777	3.53	0.999792	4.15	0.166238	E-04
3.04	0.998817	3.54	0.999800	4.20	0.133458	E-04
3.05	0.998856	3.55	0.999807	4.25	0.106885	E-04
3.06	0.998893	3.56	0.999815	4.30	0.853006	E-05
3.07	0.998930	3.57	0.999821	4.35	0.680688	E-05
3.08	0.998965	3.58	0.999828	4.40	0.541254	E-05
3.09	0.998999	3.59	0.999835	4.45	0.429351	E-05
3.10	0.999032	3.60	0.999841	4.50	0.339767	E-05
3.11	0.999064	3.61	0.999847	4.55	0.268230	E-05
3.12	0.999096	3.62	0.999853	4.60	0.211245	E-05
3.13	0.999126	3.63	0.999858	4.65	0.165968	E-05
3.14	0.999155	3.64	0.999864	4.70	0.130081	E-05
3.15	0.999184	3.65	0.999869	4.75	0.101708	E-05
3.16	0.999211	3.66	0.999874	4.80	0.793328	E-06
3.17	0.999238	3.67	0.999879	4.85	0.617307	E-06
3.18	0.999264	3.68	0.999883	4.90	0.470183	E-06
3.19	0.999289	3.69	0.999888	4.95	0.371067	E-06

(*continued*)

Table of Standard Normal Probability $\Phi(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{x} \exp\left(-\frac{1}{2}x^2\right)$ (*Continued*)

x	$\Phi(x)$		x	$\Phi(x)$		x	$1 - \Phi(x)$	
5.00	0.286652	E-06	6.00	0.986588	E-09	7.00	0.128	E-11
5.10	0.160827	E-06	6.10	0.530343	E-09	7.10	0.624	E-12
5.20	0.996443	E-07	6.20	0.282316	E-09	7.20	0.361	E-12
5.30	0.579013	E-07	6.30	0.148823	E-09	7.30	0.144	E-12
5.40	0.333204	E-07	6.40	0.77688	E-10	7.40	0.68	E-13
5.50	0.189896	E-07	6.50	0.40160	E-10	7.50	0.32	E-13
5.60	0.107176	E-07	6.60	0.20558	E-10	7.60	0.15	E-13
5.70	0.599037	E-08	6.70	0.10421	E-10	7.70	0.70	E-14
5.80	0.331575	E-08	6.80	0.5231	E-11	7.80	0.30	E-14
5.90	0.181751	E-08	6.90	0.260	E-11	7.90	0.15	E-14

The **Transportation Research Board** is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's mission is to promote innovation and progress in transportation by stimulating and conducting research, facilitating the dissemination of information, and encouraging the implementation of research results. The Board's varied activities annually draw on approximately 4,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation.

The National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Bruce M. Alberts is president of the National Academy of Sciences.

The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. William A. Wulf is president of the National Academy of Engineering.

The Institute of Medicine was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Kenneth I. Shine is president of the Institute of Medicine.

The National Research Council was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purpose of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both the Academies and the Institute of Medicine. Dr. Bruce M. Alberts and Dr. William A. Wulf are chairman and vice chairman, respectively, of the National Research Council.

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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