NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT

301

LOAD CAPACITY EVALUATION OF EXISTING BRIDGES

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **301**

LOAD CAPACITY EVALUATION OF EXISTING BRIDGES

F. MOSES and D. VERMA
Department of Civil Engineering
Case Western Reserve University
Cleveland, Ohio

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:

Structures Design and Performance
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(Highway Transportation, Public Transit, Rail Transportation)

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WASHINGTON, D.C. DECEMBER 1987

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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Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

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FOREWORD

By Staff
Transportation
Research Board

This report contains guidance for the load capacity evaluation of existing steel girder and prestressed concrete highway bridges. The report includes recommended revisions to the evaluation requirements presently in the AASHTO *Manual for Maintenance Inspection of Bridges,* along with a companion commentary. This report can be used in conjunction with *NCHRP Report 292,* "Strength Evaluation of Existing Reinforced Concrete Bridges," to provide load capacity evaluation guidelines for the majority of existing bridge superstructure types. The contents of the report will be of immediate interest and use to bridge engineers, researchers, specification writing bodies, and others concerned with the load capacity evaluation of existing steel and prestressed concrete structures.

Steel and prestressed concrete highway bridges are presently evaluated and rated according to the requirements of the AASHTO *Manual for Maintenance Inspection of Bridges.* The *Manual* allows bridge evaluation based on two rating levels: an operating rating and an inventory rating. It does not provide any means, however, to account for the actual condition of the structure or to quantify other important factors that might be considered in the rating process. A more rational and flexible methodology for the evaluation of the load capacity of existing bridges is required.

The report concludes that bridges found to be deficient under the present rating procedures should be reevaluated using higher level methods. This higher level rating system should permit selection of safety levels in a rational manner based on the levels of effort expended on inspection, evaluation, and maintenance. The system should also take into account the states of deterioration and distress of the bridge and permit the owner to make informed decisions about the pay-off, in terms of higher load ratings, resulting from such measures as additional load control, inspection, and calculation effort.

NCHRP Project 10-15 was initiated in 1980 to develop an improved methodology for evaluating the structural capacity of existing reinforced concrete highway bridge superstructures. *NCHRP Report 292,* "Strength Evaluation of Existing Reinforced Concrete Bridges," summarized the findings of that study and presented specification recommendations for evaluating reinforced concrete bridges.

NCHRP Project 12-28(1), "Load Capacity Evaluation of Existing Bridges," was initiated in 1985, with the objective of providing evaluation guidelines for bridge superstructure types not covered in *NCHRP Report 292.* This report documents that work and also provides recommendations for revisions to the AASHTO *Manual for Maintenance Inspection of Bridges.*

The recommended specifications in this report are specifically for steel and prestressed concrete bridge superstructures. The proposed guidelines allow flexibility by providing a range of ratings depending on site conditions and the degree to which the evaluation is based on available information on traffic, bridge performance, and maintenance. The report also contains numerous examples demonstrating the use of the recommended specifications.

A second phase study was initiated in late 1987 with the objective of combining the evaluation recommendations from this report and *NCHRP Report 292* into a comprehensive recommended specification for load capacity evaluation of all major bridge superstructures. Additional objectives of the second phase are to work with several states in trial implementation of the recommended specifications and to develop a series of workshops and training aids on the use of the new evaluation procedures. It is expected that AASHTO will review the combined specification for consideration for adoption as a guide specification in 1988.

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LOAD CAPACITY EVALUATION OF EXISTING BRIDGES

SUMMARY

Evaluation of the load capacity of existing bridges is a major concern for highway agencies throughout the United States. Available funds must be expended in a rational strategy which allows for bridge inspection, maintenance, repair, posting, or replacement. To achieve a rational cost allocation policy, safety criteria must be consistent and uniform for all situations. A flexible approach is needed for the evaluator, which reduces the engineering effort for bridges that have excess strength capacity and, further, allows for greater effort to improve ratings in other cases, which may avoid unnecessary postings.

Structural reliability methods contain the necessary ingredients for a rational evaluation strategy. Safety is expressed in terms of a measure of the probability that the capacity will exceed the extreme load that may occur during the inspection interval. Structural reliability theory is now being used to formulate safety checking equations throughout the world. These procedures, explained in Chapter Two, are appropriate for rating bridges and lead to a load and resistance factor checking equation similar to AASHTO's load factor design methods. Safety is expressed in terms of the safety index (beta), which is the number of standard deviations (depends on uncertainties) contained in the expected margin of safety (depends on the load and resistance factors). It is shown that these safety indices correlate closely to the risk or probability that a bridge member loading will exceed its corresponding strength or capacity.

In the reliability modeling, the evaluator has two options to provide a target reliability level. One method is to use high safety factors that may then require posting or repair. The other approach is to reduce uncertainties. This can be done by more effort to obtain better site-specific truck data, improved analysis of the lateral load distribution, noting the deck roughness which affects bridge impact and recommending improved maintenance to control possible section loss during the inspection interval.

These reliability procedures provide a flexible evaluation strategy and eliminate the present AASHTO inventory and operating stress levels for posting which give rise to markedly different ratings by state agencies for the same situation, depending on the stress level they adopt. Instead, the results show a full range of possible evaluations ranging from a level exceeding the operating value to a level below the inventory value. The rating depends on the load and resistance factors selected which, in turn, depend on site traffic volume and potential truck overweight conditions, girder analysis used, observed deck smoothness, inspection effort and maintenance. All selections are intended to lead to the same reliability level because the factors were calibrated based on review of truck data from weigh-in-motion (WIM) analysis studies, bridge tests, and strength studies. A load prediction model is presented in Chapter Three. Thus, sites with higher truck traffic and reduced enforcement effectiveness are assigned higher load factors than sites with less severe traffic. These choices are carefully explained in Appendix A, which contains the recommended evaluation guidelines suitable for inclusion in the AASHTO *Manual for Maintenance Inspection of Bridges.* Calculation of nominal (unfactored) bridge member loadings is done with the present AASHTO rating vehicles. Thus, uniform reliability is maintained by varying the load factors with the severity of traffic at the site and the degree to which potential

information on member condition and impact is utilized. Impact is correlated to deck smoothness, while accuracy of girder distribution is correlated to the type of analysis (AASHTO tables, finite elements, or measurements) available to the evaluator. Similarly, the resistance factors are selected based on the amount of component deterioration, quality of inspection and maintenance, and most importantly on the presence of multiple load path redundancy.

The target reliability index is selected in the 2.5 range, based on a calibration with existing practice, which allows operating stress levels to be used. (Use of inventory stress levels leads to higher reliability. These higher levels are prescribed herein, however, only for structures that do not have redundancy. Comparisons of reliability levels are also given for load factor rating procedures.) The performance criteria selected (i.e., operating working stress level) for calibrating the target index does require acknowledgment from code writers, because the selected load factors are linked directly to the performance target reliability. A target reliability index in the range of 2.5 should be acceptable for evaluation, provided the structure has multiple load paths typically found in redundant bridge spans. For nonredundant structures, a higher index of 3.5 is recommended herein. These various reliability considerations are all transparent to the rating engineer who uses load and resistance factors in a deterministic manner similar to present load factor rating.

The proposed guidelines are specifically for steel bridges of different types and prestressed concrete spans. Reinforced concrete bridges are covered in *NCHRP Report 292.* The proposed guidelines allow flexibility by providing a range of ratings depending on site conditions and the degree to which the evaluation utilizes available information on traffic, bridge performance, and maintenance. The report contains numerous comparisons of bridges designed and checked with AASHTO HS-20 and other loading criteria and rated by the proposed guidelines as well as operating and inventory stress levels. These comparisons show that the proposed evaluation strategy gives a transition of rating factors varying from above operating levels to even below inventory levels, depending on the condition and redundancy of the structure, the intensity of the traffic at the site, the level of inspection, and the maintenance effort.

The proposed ratings reach or even exceed AASHTO operating ratings for good condition, redundant structures having high dead-to-live load ratios and low or moderately severe traffic. On the other hand, the proposed ratings may fall below AASHTO operating ratings for nonredundant, deteriorated spans, having a rough roadway surface and subjected to heavy traffic with numerous overloads.

For structures with multiple load paths, the proposed ratings exceed current inventory levels. However, they can fall below inventory levels for higher traffic and in the presence of deterioration.

The advantages of these procedures include:

1. A more rational rating criteria.
2. Consistency among different types of spans and materials.
3. A recognition of site condition as a major factor affecting the safety including traffic, member deterioration, and maintenance quality.
4. A range of flexible options for the engineer to improve the rating by improved data collection, rigorous lateral distribution analysis or tests, and recommendation of in-depth inspection and maintenance.
5. Recognition of the importance of redundancy as a factor affecting rating of existing bridges.
6. Reasonable consistency with safety levels achieved in present practices.
7. A basis for developing an overall bridge safety management system including posting, permit, maintenance, inspection, load enforcement, etc.

CHAPTER ONE

INTRODUCTION

GENERAL

The emphasis for the highway industry in the United States is now centered on maintenance, rehabilitation, and conserving the existing road network. Bridges are an important component of the highway system and, in part, because of their conservative design, bridges have been allowed to deteriorate over many years because of deferred maintenance and repairs. The Federal Highway Administration now estimates that more than 200,000 bridges are inadequate. Many of these are due to insufficient strength estimates that often lead to a posted or closed structure. In evaluating existing bridges, the engineer must consider options including posting, permit regulation, closing, rehabilitation, and replacement. Fortunately, the circumstances of the evaluation, such as past performance experience, shorter expected remaining bridge life, and the opportunity to perform further in-depth study on an existing structure, all provide greater flexibility for the evaluation calculations.

Most states utilize provisions of the AASHTO *Manual for Maintenance Inspection of Bridges* (henceforth referred to as the AASHTO *Maintenance Inspection Manual*) in their bridge evaluations. Different interpretations of these provisions, however, have led to a wide range of rating values for similar conditions. There is a need for a comprehensive framework that covers all bridge conditions, yet is flexible enough for the evaluation agency to consider its own needs and willingness to invest resources, when necessary, in the evaluation investigation. Many states are introducing bridge management programs, which provide for a long range bridge evaluation process that considers bridge condition, site traffic, maintenance and inspection costs.

Unlike pavement management programs, which can optimize all the pavement economic components, bridge management must relate to safety as well as economy. In the absence of definite criteria for safety evaluation, many bridge engineers have been reluctant to deviate from long established bridge calculation models (e.g., conservative inventory ratings) (*1*). A major development, however, for structural codes in recent years has been the introduction of structural reliability concepts to assist code writers to formulate checking models and safety factors. For example, the AASHTO load factor design (LFD) recognizes that smaller dead load factors and larger live load factors may provide more uniform reliability (*2*).

A comprehensive approach to safety may combine reliability concepts and data provided by the evaluator. The result will be a rating format based on the level of effort instituted by the evaluator. A range of dead and live load factors and strength resistance factors may provide the flexibility needed by the evaluator for specific conditions and yet assure that uniform reliability criteria are being met. An initial screening level can be developed which can be supplemented by detailed inspection and data gathering only if the initial rating is unacceptable. The reliability level to be satisfied should correspond to present standards of field performance based on actual case histories.

BACKGROUND

Currently, guidelines for evaluating existing bridges are provided in the AASHTO *Manual for Maintenance Inspection of Bridges.* Recommendations for introducing flexibility into the evaluation procedure are also given therein in general qualitative terms. Quoting directly from the manual, "... a higher safety factor for a bridge carrying a large volume of traffic ... especially if it includes many heavy loads...." This statement recognizes that higher safety factors are more appropriate for bridges subjected to heavy traffic. However, no specific safety factors are provided. Similarly, "factors of safety used in rating ... must provide for ... reasonably possible overloads ... lack of knowledge as to the distribution of stresses...." This statement recognizes that the safety factors should account for possible uncertainties in the rating procedure. These quotes and other statements do demonstrate the awareness of differences between design and rating and also that a rating can be influenced by a variety of controllable parameters (like safety factors and impacts) and should be used judiciously to introduce flexibility in the evaluation procedure. However, the checking methods provided in the manual do not take advantage of these statements. The intent then is to introduce in this report such procedures using reliability theory and based on recently collected data.

The opportunity to investigate a comprehensive approach to bridge evaluation is due to several circumstances. Recent code changes, such as the *Ontario Highway Bridge Design Code* introduced in 1979, have demonstrated that formal reliability methods can be used to calibrate safety factors based on uncertainty levels for all components of the design-evaluation process (*3*). Further, extensive field performance data are available ranging from in-service weigh-in-motion (WIM) to full-scale ultimate capacity determination (*4,5*). These data show that current design and evaluation parameters represent idealizations and approximations that, while broadly applicable, do not necessarily apply to specific conditions of the bridge being evaluated. The flexibility of acquiring and using site-specific information during the evaluation calculation becomes readily apparent.

The background to this project is the increasing interest in providing a flexible comprehensive approach to bridge evaluation which best utilizes the economic resources available and yet maintains consistent and definable criteria for bridge safety. An earlier related project, NCHRP Project 10-15, "Strength Evaluation of Existing Concrete Bridges," was initiated because of a gap in the evaluation of concrete bridges and because of an apparently large discrepancy between the observed and calculated live load capacities for both concrete slabs and girder bridges (*6*).

The final report from NCHRP Project 10-15 (*NCHRP Report 292*) introduced a reliability framework and allowed for a range of both load and resistance factors. The live load factors were

based on the analysis model and the information available on the truck traffic at the site including enforcement efficiency and traffic volume. Further, the resistance or capacity reduction factors were made dependent on the inspection report (amount of deterioration) and the maintenance effort intended to correct any deficiencies.

In addition to NCHRP Project 10-15, there have been other proposals to allow flexibility in the evaluation based on truck traffic and inspection data. A comprehensive proposal was made (Ref. *7*) in which the work specifically adjusts the allowable stress in five incremental levels between inventory and operating stresses based on site inspection and truck traffic data.

A major ingredient in reliability-based design is the target safety level. This may be established by economic justifications, but it can also be based on past performance investigations. A problem in the bridge evaluation investigation is the wide range of ratings that agencies may obtain for the same conditions, depending on whether they use (1) inventory or operating criteria (principally), (2) working stress or load factor, and (3) AASHTO HS or other rating vehicles. This factor complicates the establishment of an acceptable level for safety and requires input from the code writing group. Another criterion that should be used in the rating is the presence of redundancy or multiple load paths in the structure. This feature does appear in some individual state bridge policies such as New York (*31*).

OBJECTIVES AND SCOPE OF THE RESEARCH

With the development of a reliability framework and the necessary data base, the objective of this project is to provide a comprehensive evaluation document suitable for adoption in the AASHTO *Maintenance Inspection Manual* (*1*). The applications will concentrate on steel and prestressed concrete spans and consider the earlier work (NCHRP Project 10-15) on concrete bridges. The fundamental evaluation or rating equation will be in a load and resistance format (LRFD):

$$\phi R_n > \gamma_D D + \gamma_L L (R.F.) (1+I) \tag{1}$$

or solving for the rating factor, *R.F.*,

$$R.F. = \frac{\phi R_n - \gamma_D D}{\gamma_L L (1+I)} \tag{2}$$

Where R_n = nominal member capacity, D = dead load effect, L = nominal live load effect due to traffic loadings, I = impact value, γ_D = dead load factor, γ_L = live load factor, and ϕ = resistance or capacity reduction factor.

In LRFD applications, the load factors, γ, and resistance factors, ϕ, are related to their respective uncertainties and the overall target reliability or safety level. For example, the new AISC-LRFD code (*47*) shows in their commentary the following approximation:

$$\phi = (R_m/R_n) \exp(-0.55\, \beta\, V_R) \tag{3}$$

where R_m = true mean resistance, R_n = nominal resistance in the code's strength formula, V_R = coefficient of variation (standard deviation divided by mean—see Chapter Two for further definitions of statistical terms), and β = safety index or measure of reliability (typically, in the range of 2 to 4).

In the context of the output of the project, namely the rating equation, the following objectives must be met:

- A loading model (e.g., nominal design trucks) must match existing heavy traffic and provide uniform reliability over all bridge spans and geometries.
- A table of live load factors must be assembled which matches reasonable categories of truck traffic definition (e.g., heavy traffic, enforced weights, etc.) so as to produce consistent safety for different site conditions and methods of live load distribution analysis.
- A table of resistance factors for steel and prestressed members is needed which reflect uncertainties in resistance that may be related to factors available from inspection reports (e.g., condition surveys, deterioration, etc.).
- Criteria for expressing safety or reliability level and relating the criteria to past performance experience. The target reliability criteria should reflect the presence of redundancy or multiple load path definition as in the present AASHTO fatigue provisions.

To support these primary project objectives, the following studies were performed:

1. Formulate a structural reliability calculation including all aspects of the truck loading, analysis model, and member strength.
2. Assemble data, principally from available WIM and other field studies on truck weights, headways (multiple presence), truck configurations, volumes, impact, and girder and member force distributions—also, data on member resistances, especially steel and prestressed concrete spans.
3. Select a sample of bridges for comparison of safety indices. This will help in selection of the reliability target and also ensure that the loading model and load factors provide uniform reliability over a representative sample of bridges. Further, these bridges could serve as illustrations of the impacts of the proposed checking on actual rating factors.
4. Select a target reliability based on average safety indices computed for bridges which meet current acceptable performance levels such as operating stress level.
5. Calibrate the checking model. The tables of load and resistance factors will be selected to achieve a uniform reliability target by comparing calculated safety indices over a wide range of potential uses. The sensitivity of the results to possible deviations or errors in the data base must also be checked.

ORGANIZATION OF REPORT

To assist a general audience of readers and because of its importance in deriving factors, a detailed description of structural reliability theory is given in Chapter Two and Appendix E. The simple fundamentals of probability are discussed as well as those aspects needed for describing and calculating the safety indices.

Chapter Three presents the details of the load and strength models incorporated herein. In detail, the chapter reviews the truck load model and the data available from various WIM and other studies.

Chapter Four incorporates the data and the reliability analysis into a reliability evaluation model. The selection of load and

resistance factors and calibration of target reliability is presented. A comparison with present rating formulas is also given. Chapter Five presents the conclusions and some discussion of implementation.

Appendix A presents a self-contained section of the guidelines intended for incorporation in the AASHTO *Maintenance Inspection Manual.*

Appendix B contains a commentary to the aforementioned guidelines.

Appendix C contains a set of examples comparing the existing and proposed procedures. These examples were obtained from the consultants working on this project or were collected from various highway agencies.

Appendix D contains further details of the WIM data base along with a description of the simulation model used for live load modeling.

Appendix E contains some mathematical aspects of probability and reliability theory. An introduction to system reliability theory as applicable to issues of bridge redundancy is also presented.

Appendix F contains a detailed but simplified procedure illustrating the use of the data base for calculating safety indices.

Appendix G gives some more advanced, yet simplified, methods for live load distribution that have been developed by other agencies and also by a consultant to this project.

Appendix H gives a summary of the data used for obtaining statistics of deteriorated steel members. Deterioration of prestressed members is also discussed.

Appendix I contains a description of maintenance and inspection strategies which serve as a guideline for choosing the appropriate options in the proposed specifications.

The final appendix (J) contains a glossary of the symbols used in this report.

CHAPTER TWO

STRUCTURAL RELIABILITY THEORY

INTRODUCTION

In assessing the performance of structures such as highway bridges the load intensity, load effect analysis, and strength parameters are not known with certainty. The intent of structural reliability theory is to characterize these uncertainties and allow for consistent and rational safety decisions. The application is not intended simply to predict the probability of structural failure but rather it aims at a systematic evaluation and adjustment of the safety factors in a structural design or evaluation code.

The developments in structural reliability have highlighted an awareness that safety factors must be calibrated to both the uncertainty present in estimating the safety margin and the consequences of a component failure. Inherent in these applications of reliability methods is that it should not interfere with the engineer's use of judgment and past experience or introduce complexities that increase the possibility of human errors. The design methods proposed herein are similar to other structural code modifications introduced recently in the United States and abroad. For example, it is known as ultimate strength design (USD) in the ACI concrete code, limit state design (LSD) in British and Canadian codes, and load and resistance factor design (LRFD) in the new AISC steel specifications. For this discussion, the most relevant application is the adoption by AASHTO of load factor design (LFD) which implicitly attempts to use probabilistic reasoning to specify a higher factor for live load than for dead load. The LFD changes, however, were not based on any specific probabilistic models or actual statistical data for either the loading, impact, distribution factors, or strength uncertainties. The result of the LFD factors while presumably better than the working stress factors in terms of uniform safety may still give inconsistent level of reliability for different bridge spans.

The LRFD format will be used herein rather than LFD because it provides even more flexibility in dealing with different bridge components including steel, concrete, timber, and so on. An LRFD format has been adopted in the recent *Ontario Highway Bridge Design Code* which applied formal reliability models to calibrate their respective partial safety factors (*3*). Two important aspects of LRFD should be noted. One is that the structure will continue to be analyzed by conventional elastic force distribution methods. The second item is that all the load and resistance factors are tabulated so that the engineer need not be concerned with any probability or reliability theory in carrying out his checking, either in design or in evaluation.

LRFD offers advantages over traditional allowable or working stress formats. In working stress design (WSD), the design check has the form:

$$(R)/S.F. = D + L + W \tag{4}$$

where R is the nominal strength or resistance, $S.F.$ is the specified safety factor, and D, L, and W are the nominal component force effects due to dead, live, and environmental loads, respectively. With only a single safety factor, the reliability may vary depending on the relative ratios of force effects and their relative uncertainties in the components due to external dead, live, and environmental impacts.

In the LRFD approach, the design check has the form:

$$\phi R = \gamma_D D + \gamma_L L + \gamma_W W \tag{5}$$

where R, D, L, and W may be the same *nominal* quantities as in the working stress design equation (Eq. 4). ϕ is the capacity reduction factor or resistance factor and γ_D, γ_L and γ_W are the separate load factors on dead, live, and environmental load effects. These separate or partial load and resistance factors must be calibrated by code writing committees to their respective uncertainties and overall to the target reliability intended for the particular component based on performance requirements. It is important to note that the reliability basis of the partial safety factors is transparent to the code user and the designer applies the LRFD check in a totally deterministic fashion. This application is a so-called level I format in contrast to level II or level III codes which involve the designer more directly in probabilistic analysis.

The remainder of this chapter considers the development of the reliability model and the methods for calibrating partial safety factors, including the use of judgment and past experience in the criteria specification. Appendix E discusses some of the more theoretical aspects of structural reliability.

PROBABILISTIC MODEL

A bridge component is said to be safe if it supports its load during the lifetime without reaching a limit state condition (for evaluation purposes, only strength rather than serviceability limit states will be discussed). A strictly economic optimum design is achieved when trade-offs in increased capacity (cost) are equally balanced by reduced likelihood of damage (risk). Factors of safety account for uncertainties in load, analysis, and strength. Uncertainties may be described as random variables. An example of a random variable is the truck weight. The distribution of truck weights taken from a recent weigh-in-motion study is shown as a discrete probability distribution in Figure 1. The most important characteristics of a random variable are the mean value or measure of central tendency and the standard deviation or measure of dispersion. Note that standard deviation is often called sigma. Sigma is the square root of the variance which is analogous to a moment of inertia of the probability frequency or density about the mean value. See Figure 2 for illustration of a continuous probability density curve.

In some applications the shape of the probability distribution is also important because extreme values are of interest. Keep in mind that as a result of the many random variables often present in a reliability analysis, a low failure probability, say one in a million (10^{-6}), can still be achieved by combinations of occurrence of each variable at much higher probability levels, e.g., 10^{-2} to 10^{-3} level. The extremely small likelihood that all variables fall beyond the extreme value contribute to making the overall probability of failure very small. This is accounted for in the probabilistic models which accurately perform the combinatorial aspects of failure prediction. This consideration helps to reduce the data needs because having to describe a single random variable at, say, a 10^{-6} level requires a great deal more statistical data to cover the extremes of the distribution.

To characterize a variable, two parameters are sometimes used. The bias is defined as the ratio of the mean value to the

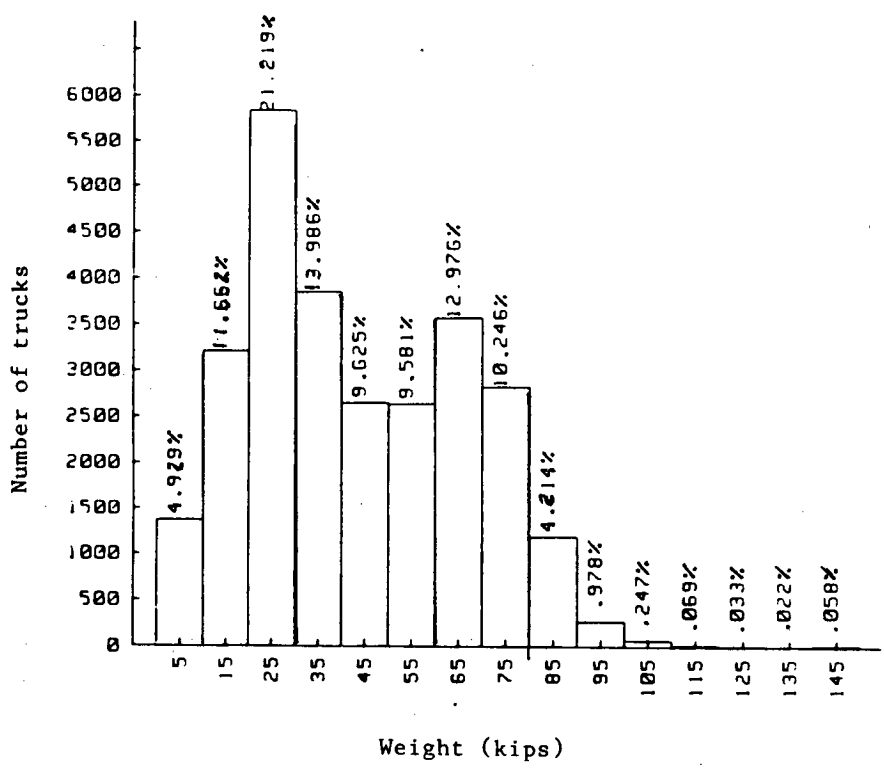


Figure 1. Gross weight histogram.

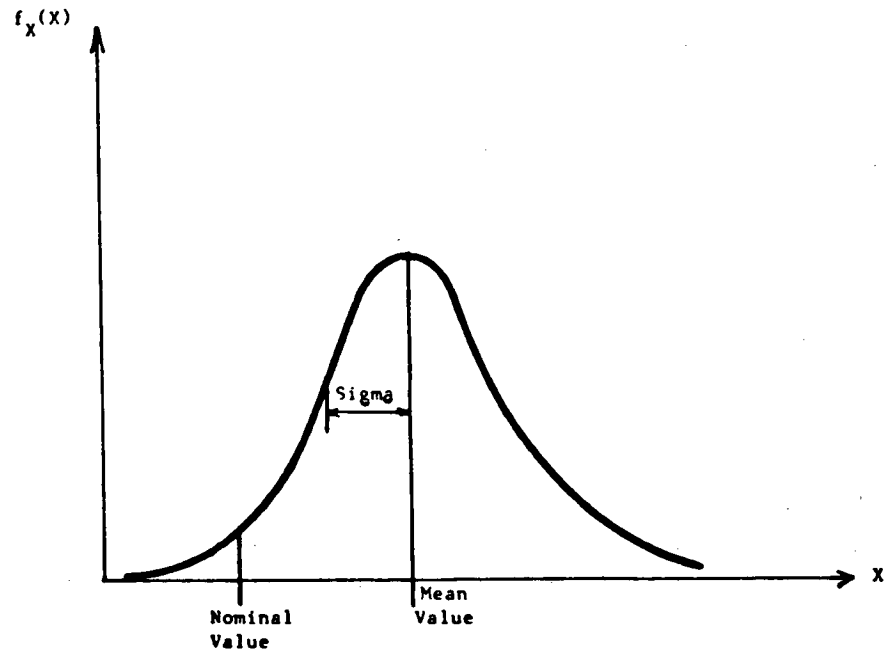


Figure 2. Probability density function.

nominal design value. For example, an A36 steel has a nominal yield of 36 ksi, but its actual mean value is close to 40 ksi. Hence, its bias would be 40/36 or 1.1 (or + 10 percent). A second parameter is the coefficient of variation, COV, which is the ratio of standard deviation divided by the mean value. It is usually expressed in percent. Again, for A36 steel, coupon results show a standard deviation of about 3 ksi. This gives a coefficient of variation, COV, of 3/40 or about 8 percent.

Probability of failure, P_f, is equal to one minus the reliability, R_o. P_f may be easily calculated only with some simplifying assumptions. For example, if the total load effect is taken as S where:

$$S = D + L + W \tag{6}$$

and D, L, and W are dead, live, and environmental load effect random variables. Then, for a single component, failure will occur when S, the total load effect, exceeds strength or resistance, R, or

$$P_f = Pr\,[R < S] \tag{7}$$

where Pr should be read as "probability". Equation 7 is illustrated in Figure 3. The two probability distributions for R and S are sketched on the same coordinate axis. The vertical coordinate is the probability function. Failure probability occurs because there is overlap of the two curves, i.e., when R is small and S is large. This basic model assumes that S defines the maximum lifetime loading and the corresponding resistance, R, at that time. Hence, only one sample event is considered with a realization of a single R and single S value. The component is safe if ($R > S$) and fails if ($R < S$). The probability of this event may be written in discrete sum form as:

$$P_f = \Sigma_i\, Pr\,[R = r_i] \times Pr\,[S > r_i] \tag{8}$$

i.e., failure probability is evaluated by looking at each discretized realization of R (i.e., the probability that $R = r_i$), and multiplying its probability by the probability that the load S exceeds this value of r_i. (R and S are assumed to be independent random variables.)

If the margin of safety is defined as g, this is another way of visualizing the failure probability. Let:

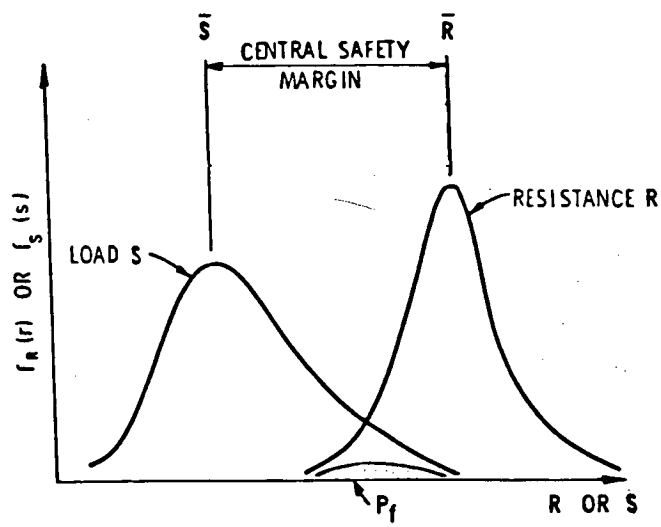


Figure 3. Failure probability model.

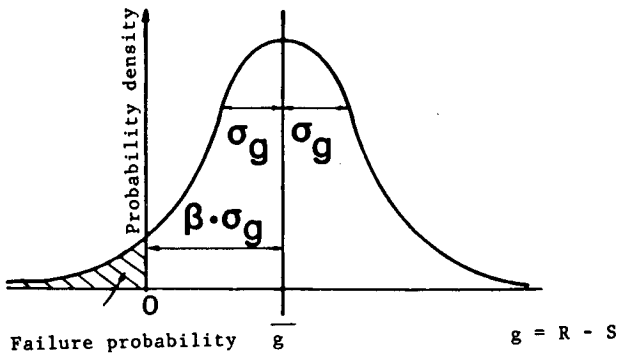


Figure 4. Failure probability and safety index.

$$g = R - S \tag{9}$$

Then,

$$P_f = Pr\,[g < 0] \tag{10}$$

Equation 10 is illustrated in Figure 4. The failure probability is the region where $g < 0$. The failure probability is simply in discrete form:

$$P_f = \sum Pr\,[g = g_i] \quad \text{for all values where } g < 0 \tag{11}$$

i.e., the failure probability is the sum of probabilities over the range where the margin of safety obtained is negative. For this model, it can be shown that the variable g has parameters:

$$\text{mean: } \bar{g} = \bar{R} - \bar{S} \tag{12}$$

Where the bar on top denotes the mean, and

$$\text{variance: } \sigma^2{}_g = \sigma^2{}_R + \sigma^2{}_S \tag{13}$$

where sigma, σ, is the standard deviation or square root of the variance.

From Figure 4, it is seen that the failure probability depends on both the mean of g, or $\bar{g}$, and its dispersion, σ_g. From a design perspective $\bar{g}$ is increased and P_f is reduced by increasing the safety margin (higher load factors or smaller resistance factor). Similarly, P_f is reduced if σ_g is decreased, i.e., by doing further analysis which leads to less uncertainty for any variable. This can be accomplished by obtaining more information on loads, better inspection, more accurate distribution analysis, and so on. Hence, this becomes an important factor in the subsequent development of partial safety factors for evaluation. These two probability measures (the mean and sigma) provide an exact measure of failure probability or risk, P_f, in the case where g is described by the standard normal probability distribution (also called Gaussian distribution). In this case,

$$P_f = \bar{\Phi}\,(-\bar{g}/\sigma_g) \tag{14}$$

where Φ is the normal probability function table which appears in any standard statistics text. For example, if $\bar{g}/\sigma_g$ equals 3, P_f is approximately one in a thousand events.

A convenient measure of $\bar{g}/\sigma_g$ is denoted as the safety index, or beta, as it is commonly called because of the general use of this Greek symbol. Thus, safety index, or β, is defined as:

$$\text{Safety Index, } \beta = \frac{\text{Mean value of safety margin, } \bar{g}}{\text{Standard deviation of } g,\ \sigma_g} = \frac{\bar{g}}{\sigma_g} \tag{15}$$

β may be expressed (as can be seen visually in Figure 4) as the number of standard deviations that the mean of g falls on the safe side. If g is a normal distribution, the risks taken from a normal probability table are:

Safety Index, β	Risk, P_f
1.5	0.07
2	0.023
2.5	0.006
3	0.001
3.5	0.0002
4	0.00003
5	10^{-7}
6	10^{-9}

These values of β in the table cover the range of most applications of structural reliability. Betas in the typical range of 2 to 4 have been reported (target β's are discussed below in the Calibration Section). Although Eq. 14 is exact only if g is a normal distribution, the following observations have been made in the structural reliability theory (*9,10*):

1. g will in fact be a normal distribution if both R and S are normal. The tendency of R and S to be normal becomes more exact if the variables are composed of a sum of other variables. R and S are exactly normal if they are a sum of normal variables, for example, S in Eq. 6 is a sum of D, L, and W.
2. A similar expression to Eq. 14 is exact if g is a log normal distribution (a probability function that is derived from the normal and uses the same Gaussian table). This will occur if R and S are products of variables as, for example, a resistance depending on a product of material, fabrication and other uncertainties, or a load being a product of variables associated with intensity, analysis, and the like.
3. The simplified expression for g, namely $R - S$ as in Eq. 9 can be generalized to any situation in which g is a failure function, i.e., a realization of $g > 0$ corresponds to a safe occurrence; and $g < 0$ failure, and:

$$g = g\,(X_1, X_2, \ldots\ldots X_n) \tag{16}$$

where the variable g now denoted as the failure equation is a function of a set number of random variables $X_1, X_2, \ldots, X_n$. In this case the use of Eq. 14 for risk (P_f) is exact if the g function is linear and all X_i are random normal variables (may be correlated). If, however, the variables are not normal, a

mapping must take place to change the function g to be a function of equivalent normal variables. Similarly, if g is not a linear function, it can be linearized about a point, denoted as a design point (or failure point), which is the point corresponding to the most likely failure event (example, high load and low resistance). This design point is defined by realizations of the variables, X_1, X_2, etc., which is the failure point X_1^*, X_2^* ... closest to the mean value. Figure 5 shows such a formulation for two variables X_1 and X_2. Computer programs are available in a National Bureau of Standards report to calculate β's for general distributions of random variables ($X_1 \ldots X_n$) and failure function g (*11*). The program automatically outputs the safety index, β. Comparisons of risk from the foregoing table for different β's show good agreement with results of more exact numerical integrations or simulations (*10*). The programs mentioned are often referred to as AFOSM—Advanced First Order Second Moment—reliability programs and will be used herein exclusively for calculating beta. Any form of distribution for the variables may be used, and the output of the program is a safety index which accurately measures the true reliability. Also outputted is the design point X^* which helps to define the most likely region of failure. Analysts should note whether the statistical data base is adequate in the neighborhood of this design point for each variable.

Modeling the safety index with β rather than dealing directly with risk, e.g., probability statements such as 0.001, helps in comparison of safety factors in a code and moreover avoids the possible legal obstacles in specifying actuarial failure rates. It should also be emphasized that the safety index pertains directly to structural components and not overall system failure (*12*). It also does not include failures that are the result of human errors. The latter are, in fact, usually responsible for most structural failures and include such events as computational error, misapplication of structural models, ignoring failure modes and accidents, such as collision or fire and deliberate overload among others. The methodology herein is intended to model probabilistically the "normal" variabilities associated with practice. In general, safety factors do not "protect" against the human type errors. The importance of these safety factors, however, in allocating costs and making evaluation decisions has emphasized the need to make them as rational as possible. The control of "other" causes of failure, such as human error, must be done by quality assurance, review checks, and other enforcement procedures operating independently of the specified component safety factors.

The target for a reliability-based design procedure and the magnitudes of the safety factors are established by a method of code calibration discussed subsequently. It should be noted that safety modeling for bridge codes may differ from other applications such as buildings. This is because of expected increases in bridge loads over the relatively long life of a structure, 50 to 100 years. Further, uncertainty over availability of maintenance and inspection funds may lead to faster deterioration. Thus, if one examines data corresponding to a new bridge construction, engineers have been conservative and betas may be relatively high in the range 4 to 5. This is illustrated in Figure 6, which shows that the failure of concern to us is actually an overlap of a future (higher) load distribution with a future (lower) strength distribution. The betas for the latter case may only be in the range of 3 to 3.5. This difference partly explains why lower factors of safety may be appropriate for evaluations compared to design since the period between evaluations may be relatively static with respect to load changes and maintenance allocations.

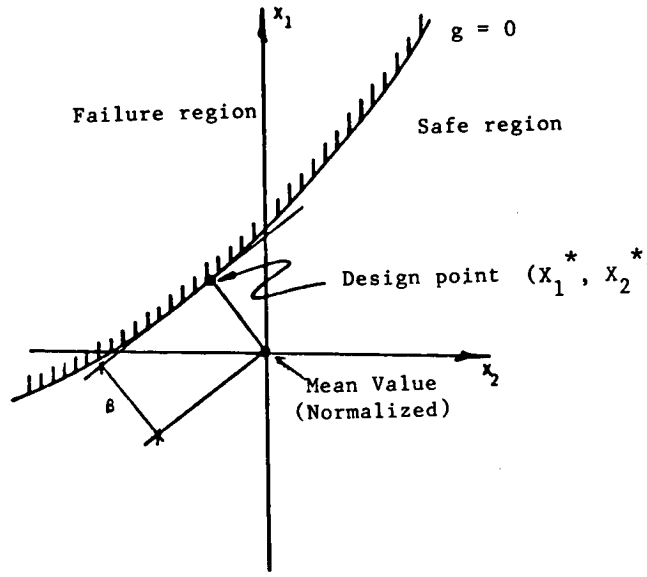


Figure 5. Calculation of safety index.

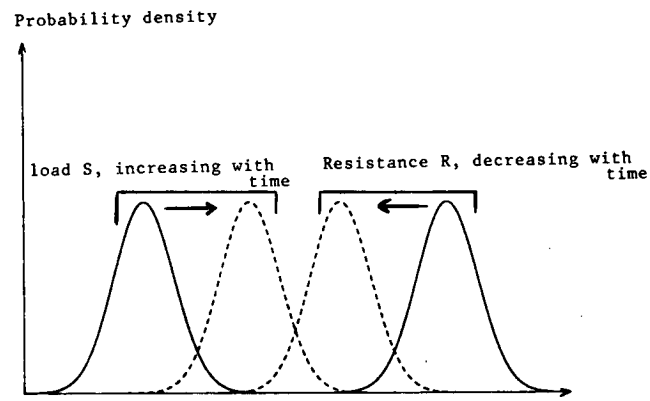


Figure 6. Overlap of future higher load with future lower strength.

CODE APPLICATION

In applying a reliability format, instead of asking designers to design each component for a prescribed target beta, code writers must select and tabulate the load and resistance values to achieve a uniform or consistent average risk level over the full range of code utilization, e.g., bridge spans, materials, load limits, and so forth. This makes the reliability basis transparent to the designer and avoids confusion. The LRFD procedure is inherently more flexible than WSD because the additional partial safety factors permit the code writers to "tune" the code to avoid any large scatter in the resulting betas. This is especially important in bridge evaluation because it is desirable to provide

options to the evaluator, which trade more beneficial rating factors at the expense of more information obtained through intensive levels of evaluation efforts.

The first aspect of all reliability-based procedures is the determination of a target reliability or beta levels. The second task is to select load and resistance factors to achieve the target over the full range of code utilization with as little scatter or deviation from the target as possible. The scatter occurs because the LRFD check (Eq. 5) uses the same load and resistance factors for all applications so the betas may still vary from component to component. Any scatter in beta, however, should be considerably less than the scatter that now occurs in WSD application.

In some applications, target betas were selected to satisfy economic considerations combined with applying engineering judgment. For example, a target of $\beta = 3.5$ was selected in the recent Ontario code (*3*). In the AISC-LRFD steel specification (issued in 1986), target betas selected were 3.0 for members for dead, live, or snow loading; 2.5 for members with wind loading; and 1.75 for seismically loaded members (*8*). The lower β's for wind and seismic loading were selected because they were consistent with analysis of previous specifications, and present building performance shows no apparent lack of adequate safety. Betas, β, of 4.5 were chosen for connections to ensure system integrity. These higher betas also reflect a judgment that the cost of increasing connector reliability is lower than the cost of increasing corresponding member reliability. This variation in target betas in the final code document was a change in the original philosophy of trying to achieve constant reliabilities. Similarly, Moses, in an LRFD development for the American Petroleum Institute for the design of fixed steel offshore platforms promoted a philosophy which allowed existing design and performance criteria (WSD) to directly influence target reliabilities independently for each component (*16*).

The aim in LRFD is then to reduce the scatter of β's within a category (e.g., steel members, connections, piles) but not necessarily make all the target β's uniform. This accounts for possible inherent variabilities that may be overlooked in the modeling and also accepts the notion that on the average the β's with existing specifications are acceptable and economic.

CODE CALIBRATION

The steps in performing a code calibration may be summarized as follows:

1. Assemble a representative sample of components for each category. This means different spans, geometry, number of lanes, traffic etc. These should be selected from both existing bridges and hypothetical or generic designs.
2. Compute safety indices for each example.
3. Select a representative value of β as a target. If past judgment and experience indicate that structures are overly conservative, the target may be reduced. Similarly, if there have been failures due to load exceeding resistance or other indications that the safety margin is insufficient, target β's should be raised.
4. Choose by iteration, load and resistance factors, so that the target is obtained for the sample with the least amount of scatter.
5. Perform sensitivity studies on the data base, i.e., vary parameters for which data are insufficient and subjective estimates had to be made. Changing parameters will usually affect the target beta, but the important thing is to determine if the load and resistance factors are affected.

At this stage, some compromises and adjustments may be necessary in the partial factors. This is because the same load factors must be applied to all material types. Relative adjustments in target betas may also be warranted if changes in behavior models (for example wheel load distribution) are improved with new analysis tools. Also, consideration should be given to adjusting the target betas based on system considerations. For example, if all the member reliabilities are the same, the reliability of a parallel girder system is much higher than the reliability of a series system such as a statically determinate truss (see Appendix E).

SELECTION OF LOAD AND RESISTANCE FACTORS

As outlined earlier the selection of partial safety factors is made to achieve an average target beta with as small a scatter as possible. Some considerations in this regard are as follows:

1. Selection of partial factors must be done in conjunction with the nominal formulas for load effects, analysis, and component capacities. Any changes in nominal design values affect the bias and, therefore, the safety index. Thus, selection of a live load factor must consider the design loading model, e.g., HS-20, AASHTO rating vehicle, or other loading formulas and the method of wheel load distribution (e.g., S/D, Ontario charts, finite element modeling), and impact value (e.g., AASHTO, Ontario, measurements). Changes in any of these nominal load terms must be done in conjunction with changes in the live load factors because they both influence beta. Similarly with the resistance (e.g., yield bending limit versus plastic moment), the nominal strength equation and the resistance factor must be simultaneously selected to achieve the target betas. Usually, resistance formulas contain conservative bias which affect directly the necessary resistance factor.
2. Selection of partial factors should minimize the scatter in betas. Usually, a trial and error search with appropriate round-off intervals (e.g., 0.05 increments in γ or ϕ) are made to attain the target β while reducing any scatter about the target.
3. In addition to a representative sample of actual components, a general study of a range of hypothetical components should also be carried out. This search, referred to as a generic population, is necessary to ensure that acceptable betas are implied for all possible users of the code (spans, geometries, materials, lanes) without regard to their frequency of use.

The next chapter applies the reliability modeling to the assessment of bridge safety. Each variable is identified with its data base and then combined into the safety index program. An illustration of the code calibration is given for a bridge design example.

CHAPTER THREE

ASSESSMENT OF LOAD AND STRENGTH VARIABLES

INTRODUCTION

An evaluation of a safe loading level for a bridge encompasses external loads, calculation of load effects (such as moment and shear) and the strength of components and systems. Safety factors account for uncertainties in any of these parameters by providing a margin between calculated load effect and nominal structure resistance. The magnitude of the safety factors needed is a function of the safety or reliability level that should be attained and the degree of uncertainty in the knowledge of any of the variables. Chapter Four will analyze the reliability and calculate load and resistance factors to meet the target safety goals. This chapter reviews the variables and the data base used to describe uncertainties in the safety model.

It is important to note that the safety factors are applied to nominal values. For example, the live load factor is applied to a calculated member load effect. The latter is based on some specified load model, such as a truck vehicle with given axle spacings, axle load percentages and gross load. Safety is affected by both the load factor and the specification of the nominal loading such as the gross vehicle weight in the calculation model. Similarly, resistance is affected by both the capacity reduction factor (resistance factor) and the formula which describes the nominal strength (e.g., elastic or plastic ultimate strength).

LOADS

The most important load effects for evaluation are the dead and live load effects in the superstructure. Thermal, wind, seismic, and deformation loads may be neglected in this analysis because presumably the structure has withstood these effects for a long period and the likelihood of occurrence of extreme values during the relatively short inspection interval is small.

Dead Loads

Dead load can usually be predicted more accurately than live loads (vehicles). This assumes that accurate records have been kept and the as-built condition agrees with the available drawings. Changes of a permanent nature made since the original construction, such as light standards, utilities, and width changes, must be noted. Uncertainty about such major changes cannot be incorporated into the dead load factor without creating a penalizing situation for all structures. Instead, it is important that systematic record-keeping assure that calculations contain the most recent dead load modifications. Otherwise, a situation exists of possible human errors not covered by the specification safety factors.

In general, dead load uncertainty has been studied by comparing variability in material density and dimensions of typical steel and concrete components. Furthermore, the uncertainty of dead load effects has been estimated by comparing results of simplified calculations usually used in practice with more elaborate models. Given the relatively smaller dead rather than live load uncertainty, it is not deemed necessary to do more elaborate analysis of dead load effects. In this study, the values were selected as follows:

Dead Load: bias = 1.0
and coefficient of variation = 10 percent (17)

(The bias is defined as the ratio of the true mean value divided by the nominal value available for the evaluation. The coefficient of variation, or COV, is the standard deviation divided by the mean (see Chapter Two).)

Nowak subdivided dead load variables into steel, concrete, asphalt, and other weights (fixtures, curbs, pipes). Typically a COV of 4 percent is used with steel and 8 percent for concrete to represent material density and dimensional tolerances (*17*). For asphalt, the COV was found from measurements in Ontario to be as high as 25 percent. The COV for other deck materials is similar to steel and concrete. In addition to these variabilities, the analysis of dead load effects has some scatter. This is due to simplified assumptions on distribution of load and the idealized assumptions of the calculation model. Typically, 5 percent may be assigned to the analysis (*18*). This means that in 95 percent of the cases the true dead load response will be within plus or minus 10 percent the calculated value. Material and analysis uncertainty may be combined in a root mean square sum, i.e., for steel components:

$$V_D = \sqrt{V^2_M + V^2_A} = \sqrt{0.04^2 + 0.05^2} = 6.4 \text{ percent} \quad (18)$$

where V_D is used to denote the coefficient of variation, COV, of the dead load effect and V_M and V_A the material and analysis COV respectively. Rather than overly complicate the situation with different factors for each material, a single dead load factor is proposed for all material and overlay weights. The 10 percent COV cited in Eq. 17 is an average value and is factored into the overall dead load consideration using sample site data giving the percent of overlay weight in the total structural dead weight. The proposed guidelines do suggest, however, increasing the nominal overlay weight by 20 percent (about a one sigma level) in the absence of adequate site data regarding the true thickness.

As an illustration of the differences in adopting variable dead load factors for each type of dead load component and a single dead load factor, an example from Casagoly and Dorton (Ref. *34*) is considered below. The table shows the different components of dead load along with their load factors for both nominal and measured overlay thicknesses as evaluated by the Ontario code and also by the proposed evaluation procedure.

The last row shows an "equivalent" overall dead load factor that can be used.

Item Nominal	Dead Load Factors—Ontario Nominal	Measured*	Proposed Nominal	Measured*
Asphalt—234 lb/ft	1.5	1.2	1.2 (increase nominal 20%)	1.2
Concrete Slab-568 lb/ft	1.2	1.2	1.2	1.2
Steel-160 lb/ft	1.1	1.1	1.2	1.2
Equivalent Overall Factor	1.26	1.18	1.26	1.20

* Applicable to the case where wearing surface thickness is measured

In this example, the proposed dead load factor agrees precisely with Ontario in the case where nominal asphalt thickness is used and is within 2 percent for the case where the thickness is measured. These differences in the equivalent dead load factor for the proposed procedure and Ontario are thus quite small as long as the percentages of component weights do not change very much from this example. This is because some allowance has been made in the overlay uncertainty for a 20 percent increase in nominal thickness when cores are not available to give good accuracy on thickness. This is equivalent to a 1.44 dead load factor on overlay weight.

Live Load

The variabilities of live load include vehicle weights, vehicle positions, and the uncertainties of live load calculation effects. It is recognized by both researchers and in the present AASHTO *Manual for Maintenance Inspection of Bridges* (henceforth referred to as the AASHTO *Maintenance Inspection Manual*) that live loads will have considerable uncertainty because of site sources of heavy vehicles, total truck volume, and likelihood of several heavy vehicles being on the bridge at the same time (*1*).

A considerable amount of load data has been assembled in recent years. Some of the truck weight and classification statistics are not relevant because they were collected at static weigh stations. Many heavy vehicles avoid such stations because of the penalties involved. Such stations report only a very small percentage of violations compared to values of 20 to 30 percent cited in other studies. Similarly, data have been collected by Nowak (*32*) on enforcement citation weights. Unfortunately, the data do not contain any reference to percentage of overweights in the traffic stream nor the type of roadway. A more reliable source of truck weight data is weigh-in-motion (WIM) information which monitors the highway stream in a manner undetectable to drivers (*19*). More of these data are becoming available especially in connection with pavement monitoring and the Strategic Highway Research Program (SHRP). These data show a wide range in degree of enforcement and the pattern, frequency and magnitudes of overloads. One factor not distinguished in the data, however, is the number of permit vehicles. By all indications, such permits are becoming more frequent (especially in the 80 to 120 kip range) and also heavier with numerous permits above 150 kip. It has been found, moreover, that along with heavy (legal) permits there will often be illegal, overloaded trucks moving essentially with the same weight magnitudes (*20*).

Also missing from the data base is information on the degree of compliance with posting signs. The WIM data cited earlier have not been taken at posted areas, although such has recently been proposed. This represents an important consideration for establishing the posting level for bridges that have rating factors, *R.F.*, less than one.

Live Load Model

Based on the WIM data taken using the bridge system, several variables have been isolated to provide a simplified model for simulating the maximum expected live load effects (*21*). This formulation is then related to data that may be available for a specific site. In this formulation, the expected maximum bridge response is expressed as:

$$M = a\ W_{.95}\ m\ H\ I\ g \tag{19}$$

where M = predicted maximum load effect (e.g., moment, shear, axial force); a = constant which relates the load effect to a reference loading model taken herein to be the representative AASHTO rating vehicles; $W_{.95}$ = characteristic truck weight at a site; m = influence of dominant vehicle type and configuration at a site (axle spacing and truck type variable); H = influence of multiple presence, such as side-by-side and following vehicles, and H also includes the effect of the extreme weight portion of the truck weight spectra exceeding $W_{.95}$; I = impact allowance (same as AASHTO $1 + I$); and g = girder distribution or other structure analysis variable.

Each of these variables is now discussed along with presentation of the appropriate data base. Examples of application then follow.

$W_{.95}$—Characteristic Truck Weight. The extreme load effect will depend on the shape of the truck weight histogram especially at the extreme tail portion of the curve. See Figure 7 for one example of a truck weight histogram. Note that the emphasis

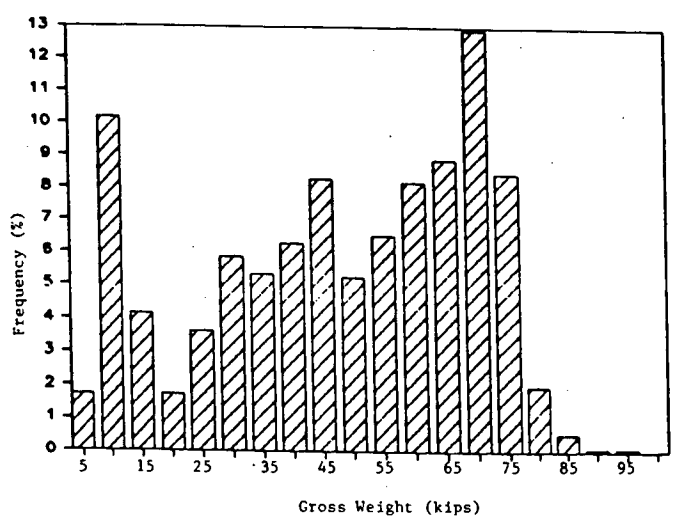


Figure 7. Truck gross weight histogram, Hope, Arkansas. (Ref. 4)

on extreme weights contrasts with bridge fatigue, for example, in which damage contribution is dominated by all the vehicles present in say the upper ⅓ of the histogram. In many instances, however, this conclusion is somewhat tempered by the fact that the extreme loading is caused by the simultaneous influence of more than one vehicle (see discussion of H below). The overall load effect (as opposed to a single axle or vehicle) that controls most structures depends on the tail end of the histogram and is not just due to the largest vehicle that may appear. The latter is difficult to estimate from conventional statistics, because there can easily be more than one million truck crossings on a busy roadway in one year (ADTT > 2,500).

The quantity of truck weight data needed to estimate with high confidence the mean of the largest of one million crossings requires too many data points that are impossible to obtain. Consider, however, the following simplified model. If two trucks simultaneously cause the maximum load effect, one need only estimate the largest vehicle out of a much smaller number of trucks. The explanation is that multiple side-by-side events represent only about 1 percent to 2 percent of all crossings. As shown in Appendix E, this may lead to the maximum occurrence of 10,000 events (i.e., equal to 1 percent times one million trucks). For these 10,000 events, the maximum expected event is closely related to the event in which each truck is similar in weight to each other. This will correspond to a probability level of the square root of 10,000 or one in a hundred assuming independent vehicles in each lane. A similar observation using extremal models is made in Appendix E assuming a normal distribution for truck weight. This assessment leads to a more manageable amount of data to collect.

To establish the maximum load effect on a bridge over a period of time, one needs to obtain the gross truck weight histogram as described above. Because the maximum load effect is characterized by the presence of two vehicles side-by-side, the probabilities of placement of these vehicles are required, which can be obtained from what is known as the headway (or arrival) histogram. If an "event" is defined as the presence of one or more vehicles on a bridge, the probability distribution of the load effect due to an event can be determined by a combination of all possible weights and headways determined from the weight and headway histograms. Next, the distribution of load effect for an event must be projected so that the maximum load effect over a period of time may be determined. This projection is accomplished by "raising" the aforementioned distribution to a power equal to the number of vehicle crossings in the projection period. To clarify this procedure, it is illustrated by using data obtained from a site in Ohio (I-90). First, the traffic is divided into two basic truck types (*39*)—singles and combinations. Gross weight histograms for both are shown in Figure 8. Next, the headway histograms are assembled and smoothed as shown in Figures 9(a) and 9(b).

These four headway histograms correspond to the four different positioning of two successive trucks on the bridge. These four different cases are: (1) a truck in the right lane is followed by a truck in the right lane, denoted by (1,1); (2) a truck in the right lane is followed by a truck in the left lane, denoted by (1,2); (3) a truck in the left lane is followed by a truck in the right lane, denoted by (2,1); and (4) a truck in the left lane is followed by a truck in the left lane, denoted by (2,2).

These positions are also shown pictorially in the corresponding histograms. These histograms are subsequently smoothed, an example of which is shown in Figure 9(c).

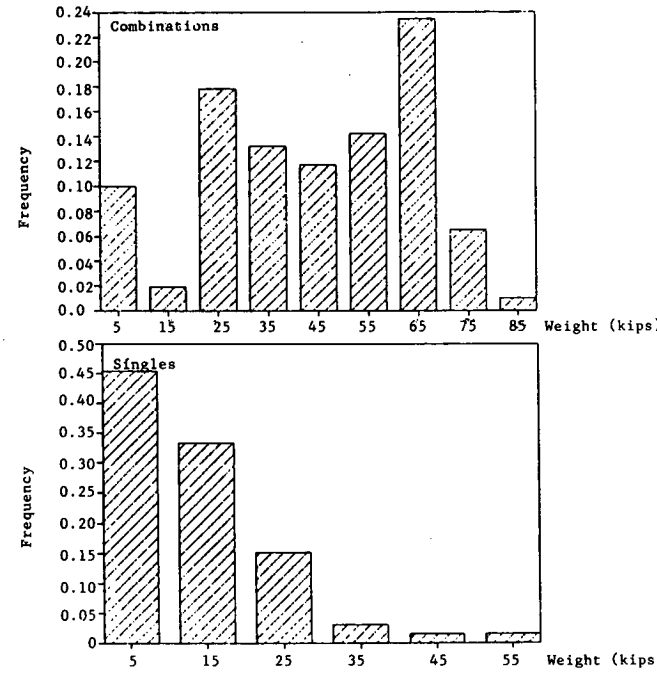


Figure 8. Gross weight histograms for I-90.

From these curves, the distribution for the maximum load effect (moment on a 100-ft span) is simulated and is shown in Figure 10. (Further details on this simulation are provided in Appendix D.) Also shown are the distributions of the load effects for different projection periods ranging from 1 month to 50 years. At this point, it must be noted that this load effect is based just on the population of combinations. This is seen (*40*) to be accurate for longer spans (greater than 100 ft). For shorter spans, the effects of single trucks are also considered.

The expected maximum load effect is presented as the median (50th percentile) of these distributions and is given in Table 1. For example, the expected maximum load effect (combined in two lanes) during a 2-year interval is equal to 3,209 kip-ft. For convenience of illustration the results can be modeled with respect to vehicle loading weight rather than bending moment. Thus, the maximum expected (mean) load effect for 2 years is equivalent to a total load in both lanes simultaneously of 192 kip on the bridge. The uncertainty of these maximum load effects is quite small and is explained by extremal probability theory in which the maximum of a series of samples increases with the number of samples, but the standard deviation decreases. The coefficient of variation, COV, of the values in Table 1 is thus on the order of 3 to 8 percent for the case where the weight and headway histograms are known.

Another convenient way of looking at this result is in terms of a value that is indicative of the truck weights at a particular site. Typically, this value is taken to be the 95th percentile value of the gross weight histogram, i.e., only 5 percent of trucks exceed this level. For this particular (I-90) site, the $W_{.95}$ value for combinations is 69 kip. Therefore, the maximum load effect (in terms of $W_{.95}$ and vehicle weight) varies from 2.2 to 3.0, as shown in Table 1. This amplification in $W_{.95}$ is termed the headway factor H. A detailed discussion of this headway factor is given later on in this section.

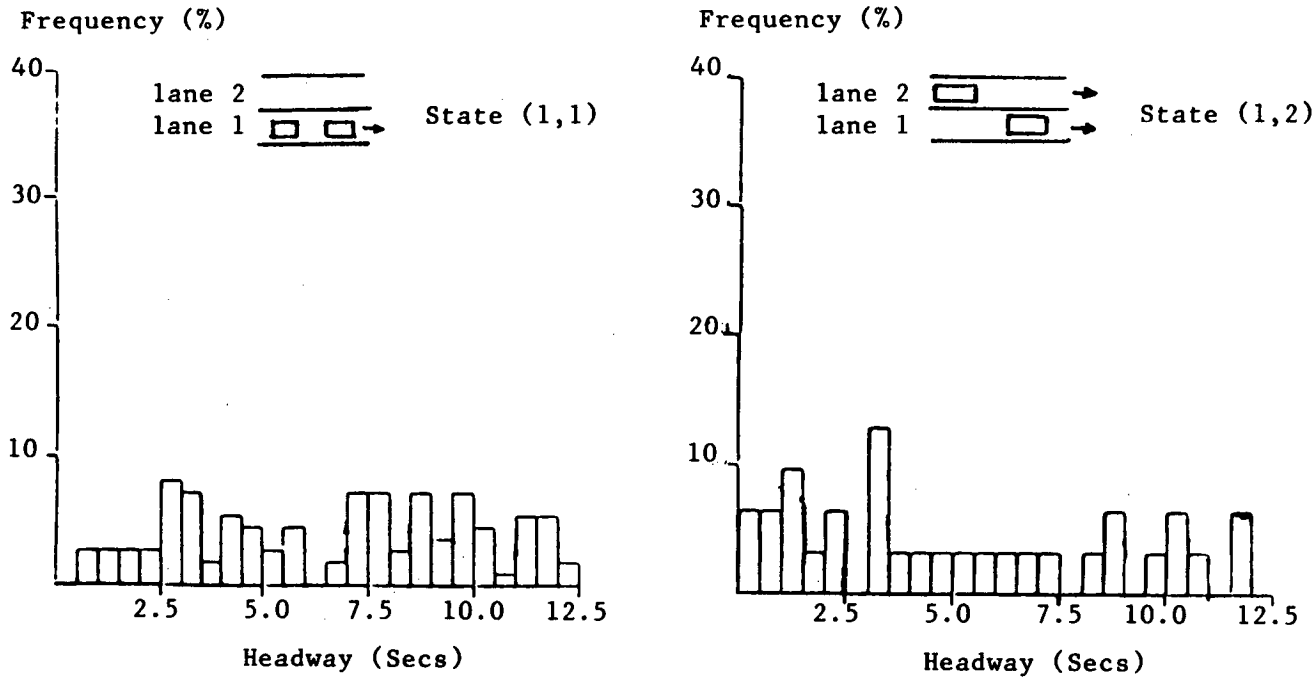


Figure 9(a). Headway histograms (I-90).

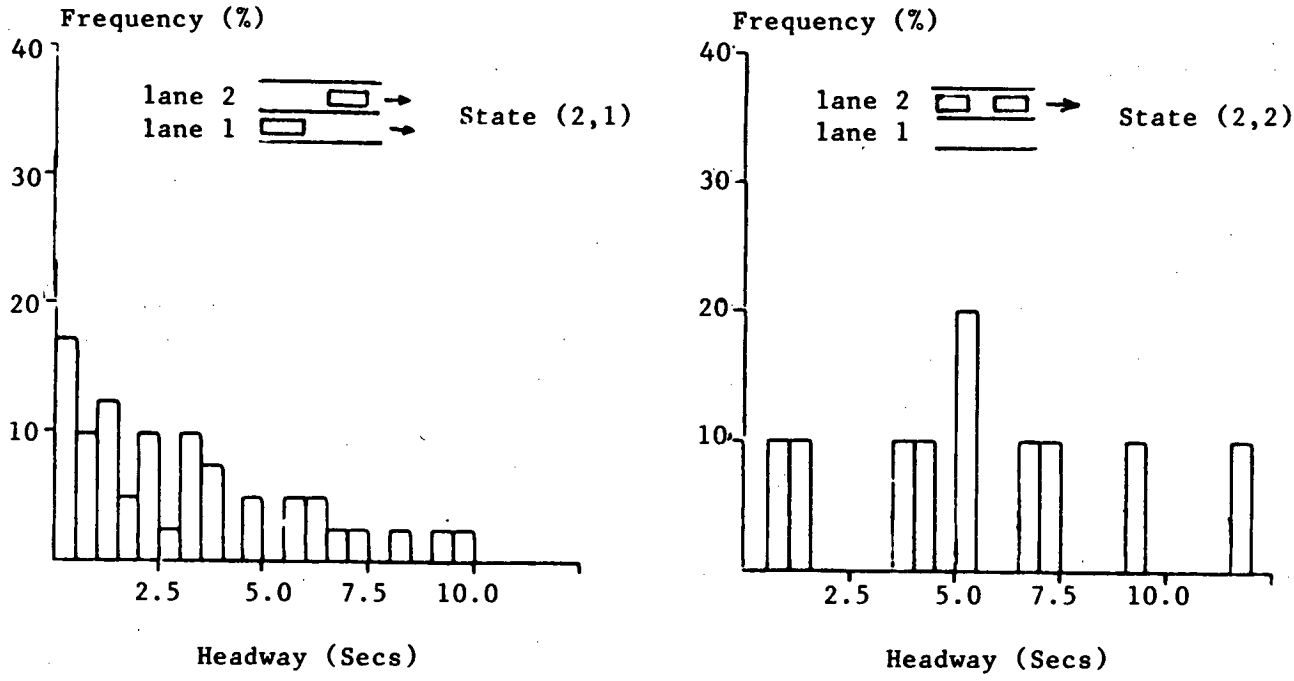


Figure 9(b). Headway histograms (I-90).

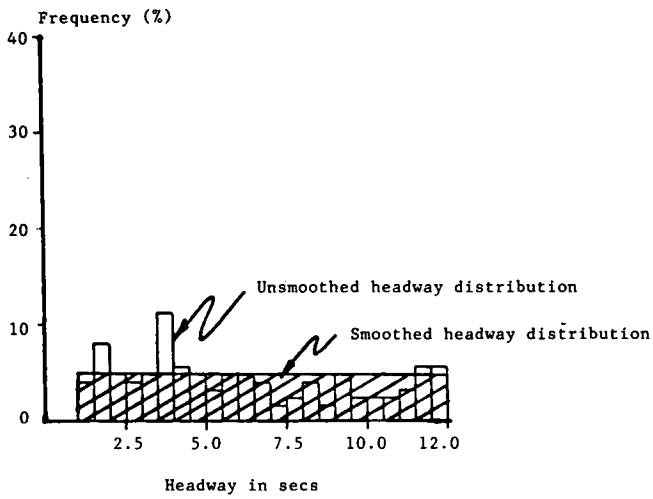


Figure 9(c). Example of a smoothed headway histogram.

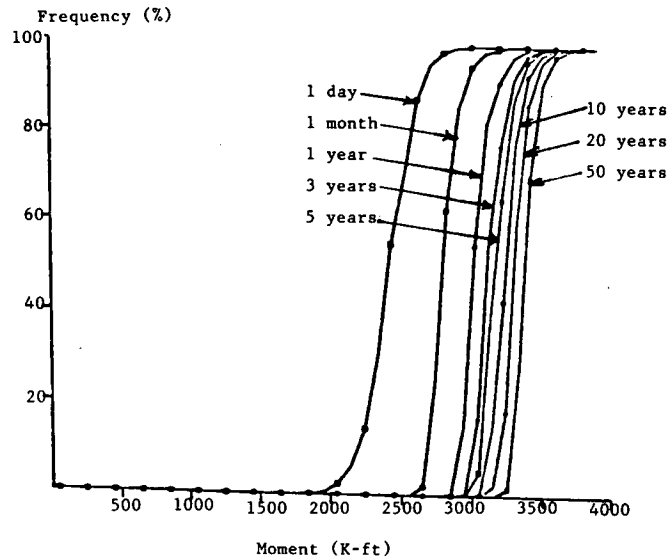


Figure 10. Projected maximum load effect for I-90 (span = 100 ft, combinations only).

The foregoing simulation has been repeated for data from several sites as well as cumulative truck data developed by combining data from different sites but similar roadway category, e.g., state, rural, interstate (*4*). These results are summarized in Table 2, assuming the same number of vehicle crossings for each data set for the same bridge span. From Table 2 it can be seen that the expected maximum vehicle loading on a short bridge corresponds to between 214 and 231 kip, or equivalent to two side-by-side trucks each almost 50 percent above the legal load limit ($2 \times 80 \times 1.5 = 240$ kip).

Using headway data from sites in Ohio as representative (*33*) of actual headways and gross truck weight histograms recorded by the WIM system throughout the United States (*4*), one can establish the projected live load effect for a large number of sites assuming different volumes of traffic. Such a procedure has been followed in Appendix D, which shows that maximum loading does vary significantly among different sites. This is expected because of the nature of the truck sources, degree of enforcement, permit issuance, and other geographic and economic factors. These data vary considerably among sites, especially if the vehicles are separated into two categories, namely, singles and combinations. At an average site, combinations, mostly FHWA category 3S2, will make up more than 75 percent of the trucks. However, for short spans the singles may be the most critical vehicle. It is noted from the data in Appendix D that there is more variability in the $W_{.95}$ among the singles than the combinations. The ratio of the maximum expected loading to the $W_{.95}$ value is accounted for by the variable H or headway effect.

H—Headway Influence. As shown in Table 2 the maximum equivalent loading effect is typically 2.5 to 3.5 times the individual $W_{.95}$ value. This amplification is represented in the variable, H. Keep in mind that $W_{.95}$ is a characteristic of a single vehicle and the quantity of interest is the expected total load effect. The source of this amplification in H is due to several factors in the following order of importance.

Table 1. Median maximum load effect by simulation on a 100-ft span site, I-90. (Ref. 40).

Interval	Median Maximum Load Effect (K-ft)	Median Maximum Load (kips)	COV	H, Headway Factor
1 day	2489	148.5	7.7%	2.2
1 month	2881	172.0	4.2%	2.5
1 year	3108	185.6	3.9%	2.7
2 years	3209	191.6	3.8%	2.8
5 years	3257	194.4	3.8%	2.8
20 years	3387	202.2	3.2%	2.9
50 years	3462	206.7	2.9%	3.0

Combinations

P=1%

volume = 2000 ADTT

Table 2. Simulated maximum load effect for different road categories (span = 60 ft, ADTT = 2,000).

Roadway Category	Simulated Maximum Load Effect (kips)	Average $W_{.95}$ (Kips)	H-Median (Amplification)
Interstates	213.9	76.4	2.80
Non-Interstates	229.6	82.0	2.80
U.S. Routes	223.6	89.4	2.65
State Routes	230.8	81.3	2.84

WIM reference data (4)

Volume assumed = 2000 trucks/day

P, Percent side-by-side = 1%

Projection Period = 2 years

1. Side-by-side influence—For a multilane bridge, the likelihood of frequent side-by-side occurrences that cause loading superposition is important. Field data show that under normal traffic movement between 1 percent and 2 percent of truck crossings will occur with two side-by-side trucks with arrivals close enough together to cause simultaneous load superposition (*19*). Thus, for a two-lane structure with 2,000 ADTT there will be between 14,600 (0.01 × 2,000 × 365 × 2) and 29,200 simultaneous truck crossings in a 2-year interval. Because of this large number there is also a high probability of two very heavy vehicles in each lane.

Field data (*19*) show that the characteristics are the same for heavy traffic in the right lane and the passing lane because heavy vehicles have sufficient power with today's modern engines to pass slower moving vehicles. The critical event is thus one of simultaneous occurrences rather than a single heavy vehicle in one lane. This conclusion takes into account also the different girder distribution factors associated with single and multiple events for typical parallel stringer cases.

2. The magnitude of the extreme load event is also dependent on the shape of the truck weight histogram in the extreme region (see Figure 7). Figure 11 shows an example of a histogram with occurrences recorded out to 150 kip. Figure 12 shows the five worst singles and combinations for this site. The axle weights and configurations show that these vehicles are illegal overloads. Appendix D contains many examples of simulations for H, the ratio of the expected extreme event to the $W_{.95}$ value for the same hypothetical number of simultaneous occurrences. The values of H range from 2.26 to 4.19 for single vehicles and 2.20 to 3.62 for combinations. The variability of H deduced from different sites can be reduced by first combining H with $W_{.95}$ and grouping sites according to truck volume and traffic enforcement effectiveness. This is done subsequently and also presented in detail in Appendix D.

3. Truck Volume—As the volume increases, there is both more likelihood of extreme events in the population and also more multiple presence occurrences. The increase in maximum expected loading with volume is given in Table 3 with data from several sites. Typically, for large volume sites the increase in H will be 10 percent for a factor of tenfold increase in volume (see Appendix D for more examples).

4. For bridges with spans above 50 ft the expected maximum load increases because of the possibility of vehicles closely spaced in front or behind the main combination of side-by-side vehicles. The data for headways are based on field measurements (*19*). Figure 13 shows the expected increase in load effect as a function of span for several sites as obtained from a simulation. The ratio plotted is the median moment from an exact simulation to the median moment from a simulation, which ignores following and preceding vehicles on a bridge. This shows the increase in moment just due to the span length effect mentioned. It is seen that for shorter spans under 60 ft the increase in load effect is less than 5 percent and the dominant loading is due to side-by-side vehicles. For longer spans the combinations control because there is still not enough space to "insert" additional vehicles. Also, as the number of "slots" for vehicle placement increases, there is smaller probability that all slots will be occupied by extreme weight vehicles. The curve shown in Figure 13 is used in subsequent load modeling.

Thus, it is seen that both H and $W_{.95}$ are important variables determining the projected live load effect. So both these factors can be combined into one term written as $HW_{.95}$. This term basically gives the projected live load effect in terms of the vehicle weight, i.e., it is the projected live load effect divided by the appropriate (moment, shear, etc.) normalized influence coefficient.

As shown above, there are large variations in both H and $W_{.95}$ from site to site. This implies that there is also a large site-to-site variation in $HW_{.95}$, as can be noted in Appendix D (Tables D.1 and D.2). Rather than lump all sites together as may be done for new designs, which increases the variability, one has the option, in evaluation, to group sites based on volume, highway classification, type of hauling, and degree of real or perceived truck weight enforcement.

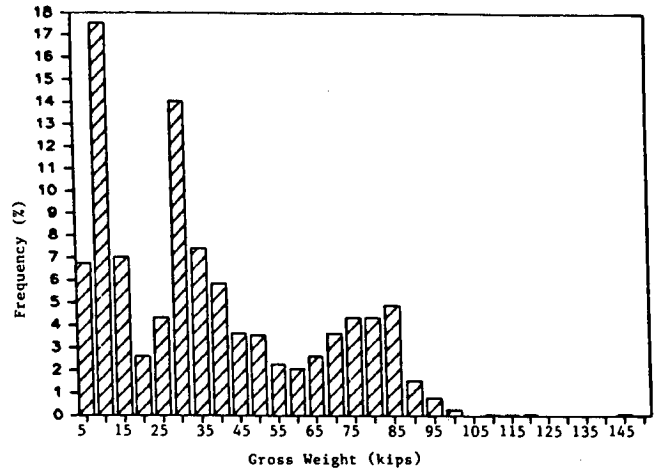


Figure 11. Gross weight histogram, Huntsville, Texas.

Table 3. Maximum load effect (kips) due to volume of traffic (combinations).

	VOLUME - ADTT		
Site [4]	500	2000	5000
1	165	168	171
2	215	220	228
3	172	176	178
4	220	228	233
5	223	232	243
6	223	232	238

P = 1% side by side occurrence rate
Projection period = 2 years

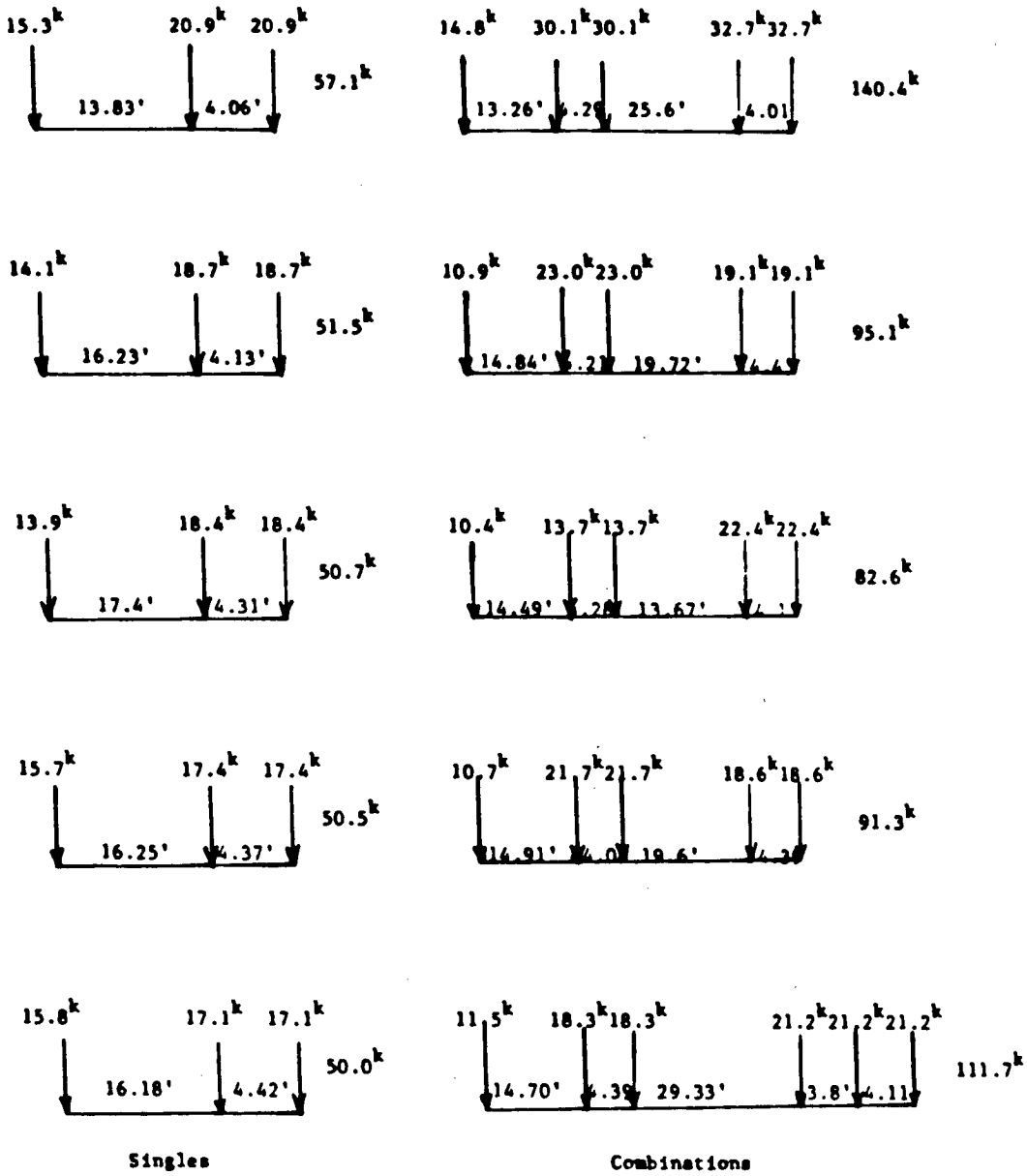
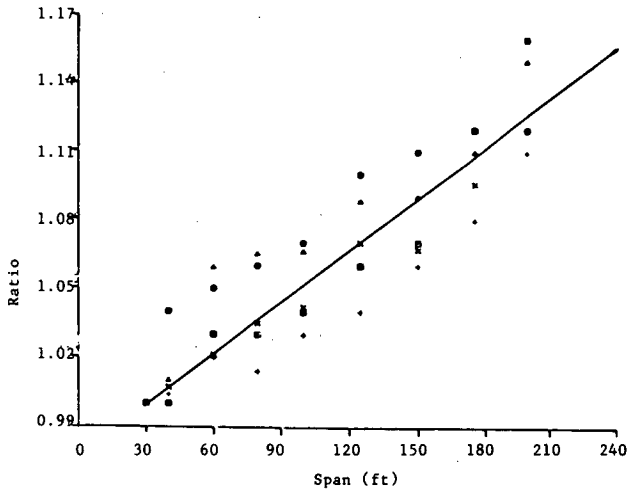


Figure 12. Five worst singles and combinations, Huntsville, Texas.

First, one has to decide on the major factors which influence the severity of traffic at any site. Use of both the load model and simulation program indicates that the main factors are the degree of enforcement and the volume of traffic. The degree of enforcement is directly related to the 95th percentile of the weight histogram, i.e., a site with no apparent weight control will be expected to have a higher value of $W_{.95}$ as compared to a site which has some sort of weight restriction. The volume of traffic is directly related to the headway factor, i.e., a high traffic volume site has higher probabilities of side-by-side occurrence which lead to higher values of the projected live load effect. Thus, a combination of the terms H and $W_{.95}$ captures the essence of the live loading due to traffic at a site.

The maximum legal weight of combinations is 80 kip, whereas for a single vehicle it is 50 kip in most jurisdictions. Therefore, the maximum legal live load effect (which is characterized by at least a side-by-side occurrence) is 160 kip and 100 kip for combinations and singles respectively. Arbitrarily 180 kip is chosen for combinations and 100 kip for singles as the cutoff points for determining the degree of enforcement of a site based on the projected live load effect. The intent is to ensure that the possibility of the exceedance of these limits is small, in other words, the possibility of an enforced site being actually unenforced is minimal. Because combinations are more common and, therefore, are more likely to be heavily loaded, the cutoff point for combinations is higher than that for singles. Similarly, cat-



Ratio = simulated moment / convoluted moment

Simulated moment: includes the effect of following and preceding vehicles

Convoluted moment: includes the effect of only side by side vehicles

Figure 13. Increase in moment with span.

Table 4(a). $HW_{.95}$ values from WIM data base.

BRACKETED FIGURES ARE COV's (1% SIDE BY SIDE OCCURRENCE)

SINGLES	ADTT < 100	ADTT < 300	ADTT < 500
Unenforced	115^K(10%)	122^K(10%)	125^K(10%)
Enforced	88^K(8%)	96^K(8%)	99^K(8%)
COMBINATIONS	**ADTT < 500**	**ADTT < 2000**	**ADTT < 5000**
Unenforced	205^K(7%)	217^K(10%)	224^K(10%)
Enforced	168^K(4%)	174^K(5%)	180^K(6%)

Table 4(b). Summary of $HW_{.95}$ data.

BRACKETED FIGURES ARE COV's

Traffic Control Single Trucks	Traffic Volume Light	Heavy
Unenforced	120 (10%)	125 (10%)
Enforced	92 (8%)	100 (8%)
Combinations	**Light**	**Heavy**
Unenforced	210 (10%)	225 (10%)
Enforced	170 (5%)	180 (6%)

- Unenforced weights based on legal limits in U.S., i.e. bridge formula.
- Light Traffic, singles ADTT less than 300 trucks/day.
- Light Traffic, combination ADTT less than 2000 trucks/day.

egorizations for volume of traffic are based on actually observed ADTT (*40,49*). An ADTT of 5,000 for combinations and 500 for singles (*49*) is taken to be heavy volume traffic. Medium volume traffic is arbitrarily chosen to correspond to a 300 ADTT for singles and 2,000 ADTT combinations. Similarly, light volume traffic corresponds to a 100 ADTT for singles and 500 ADTT for combinations. Thus, any site having an $HW_{.95}$ value of 180 kip or greater for combinations (based on light volume traffic 500 ADTT) can be treated as unenforced. Similarly, for singles, an $HW_{.95}$ value of 100 kip or greater is taken to be unenforced based on light volume traffic (100 ADTT). Using this criterion as the basis for categorization, average values for the maximum loading effect along with its scatter within its category can be established. Such an approach has been followed in Appendix D, which leads to average values for a 2-year projection period as shown in Table 4(a) along with their coefficients of variation. To avoid too many categories, the six categories, as given in Table 4(a), were reduced to the four, as given in Table 4(b), which summarizes the live loading data in terms of $HW_{.95}$. Using this categorization, one can also establish the average values of $W_{.95}$ for each of the enforcement and volume categories, as given in Table 5.

This enables us to determine if a site is unenforced or enforced, just by using a weight histogram (which is relatively simple to establish from planning studies or other reference to truck weight data for the site such as type of industry). Using values from Table 5, sites having a $W_{.95}$ greater than 80 kip for combinations and/or 46 kip (rounded off to 50 kip) for singles can be treated as unenforced. That is, the site is categorized as enforced if fewer than 5 percent of trucks exceed the legal weight limit. Also, three traffic volume categories corresponding to an ADTT of 100, 300, and 500 for singles and an ADTT of 500, 2,000, and 5,000 can be treated as light, medium, and heavy volume sites. Again, the volume categories are reduced from three to two, i.e., to heavy and light volume traffic. Therefore, an ADTT greater than 2,000 for combinations and 300 for singles can be regarded as heavy.

Summarization gives the following:

Combinations

Heavy volume	> 2,000 ADTT
Light volume	< 2,000 ADTT
Unenforced	$W_{.95}$ > 80 kip
Enforced	$W_{.95}$ < 80 kip

Singles

Heavy volume	> 300 ADTT
Light volume	< 300 ADTT
Unenforced	$W_{.95}$ > 50 kip
Enforced	$W_{.95}$ < 50 kip

Subsequently, with the data on these variables, safety indices will be derived in Chapter Four for each category and load factors selected to provide a uniform target reliability.

m—Vehicle Configuration. Because of the wide variety of legal vehicles that occur in different parts of the United States, it is useful to relate a given load data base to representative vehicles, such as the AASHTO rating vehicles. It is possible, moreover, to simulate extreme truck events by using the actual measured truck weight and axle configurations. In the more

Table 5. $W_{.95}$ for sites with different degrees of enforcement (bracketed figures are COV's).

	$W_{.95}$ kips	
	SINGLES	COMBINATIONS
UNENFORCED	46 (16%)	80 (8%)
ENFORCED	33 (14%)	69 (5%)

Results summarized from data in Appendix D

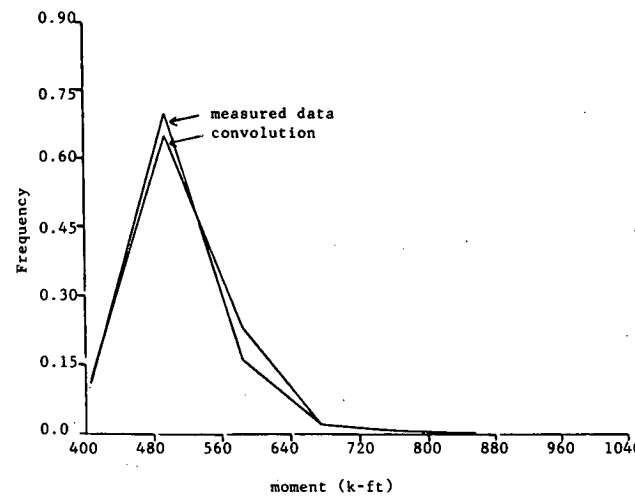
Table 6. Average *m* values for different simply supported spans using AASHTO rating vehicles (combinations only).

	SPAN				
Site	30'	60'	90'	120'	200'
1	0.92	0.99	0.99	0.99	1.00
2	0.87	0.94	0.96	0.97	0.98
3	0.92	0.97	0.98	0.98	0.99
4	0.88	0.94	0.96	0.97	0.98
5	0.87	0.93	0.96	0.97	0.98

$$m = \text{Average of: } \frac{\text{Vehicle normalized response (weight = 1)}}{\text{Response of controlling AASHTO rating vehicle (weight = 1)}}$$

precise case, the measured population is used to calculate a distribution of bending moments for hypothetical bridge spans. In the simplified case a distribution of truck weight is convoluted independently with the variable *m*. This variable is described by taking a ratio of the load effects of each measured vehicle and dividing by the effect of the vehicle in the load model (i.e., AASHTO rating vehicle). For a given measured set of data only single vehicles in excess of 30 kip and combinations in excess of 60 kip are used for the comparison. Table 6 illustrates *m* value from different sites. If the *m* values are close to 1.0, it means that the AASHTO rating vehicle model is a good average representation of the measured loads for different span lengths. As the span increases, the effect of the vehicle configuration and axle weight distribution decreases and the *m* value approaches 1, which shows that for longer spans the vehicles can essentially be treated as point loads. For spans less than 60 ft, the values of *m* in Table 6 below 1 indicate that on the average the AASHTO rating vehicles are conservative when compared to measured truck traffic. Using data from a large number of sites instrumented by the WIM system, statistical values of *m* can be established for both singles and combinations. Such a procedure has been followed and average values and coefficients of variation, COV, for *m* have been determined and are given in Table 7 (for further details refer to Appendix D).

Figure 14 shows the frequency distribution of bending moment at the center of a 60-ft span obtained by direct calculation with a measured data set compared to results using the variables *m* and *W* treated as if they were independent variables. The two distributions in Figure 14 are similar, illustrating that our model which combines *m* and *W* as independent variables does provide accurate results.



measured data: m, W simulated directly from data

convoluted data: m, W independent

Figure 14. Distribution of bending moment using convolution and measured data.

Table 7. Average *m* values for AASHTO rating vehicle from WIM study as a function of span (COV based on WIM truck spectra data).

	30'	40'	50'	60'	70'	80'	90'	100'	110'	120'	130'	140'	150'	160'	170'	180'	190'	200'
SINGLES	0.89	0.93	0.94	0.94	0.96	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.99	0.99	0.99
COV%	8.4	8.8	7.7	6.1	4.7	4.8	4.4	4.2	4.3	2.7	1.9	1.7	1.5	1.6	1.6	1.5	1.5	1.4
COMBINATIONS	1.0*	1.0*	0.95*	0.92	0.9	0.91	0.92	0.93	0.94	0.95	0.95	0.95	0.96	0.96	0.96	0.97	0.97	0.97
COV%	11.0*	11.0*	11.0*	11.0*	11.0	8.2	7.5	5.7	5.8	5.4	5.0	4.6	3.6	3.4	3.2	3.9	3.3	3.2

* values adjusted to reflect WIM data scatter

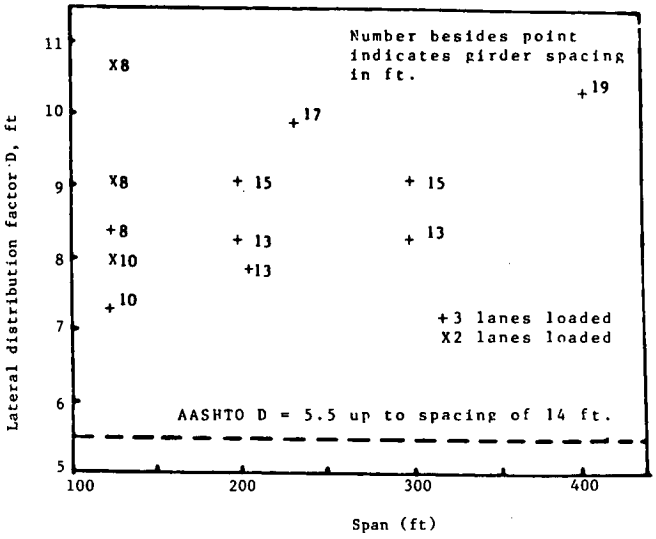


Figure 15. Calculated girder distributions. (Ref. 22)

g—Girder Distribution. This variable relates to the uncertainties of the structural analysis, especially any bias and scatter which derives from the use of simplified girder distribution factors. Present studies sponsored by NCHRP and FHWA on improved wheel load distributions should improve our knowledge on this variable (*36*). Several field measurement projects and finite element analysis have suggested that present AASHTO factors (e.g., $S/5.5$ for slabs on steel girders) may be conservative. Figure 15, taken from a recent AISI publication, illustrates possible conservatism for steel bridges with distributions shown in the figure as small as $S/10.5$ with loads in all lanes (*22*). Similarly, Table 8 compares some field measurements of the percent of total moment carried by a single girder with the predicted AASHTO values. It should be noted that any systematic bias in g is equivalent to an additional safety factor and should not be removed simply because rigorous analysis suggests it is biased. In the reliability calibration being developed herein, safety margins may be reduced only if uncertainty is reduced or some inconsistent or nonuniform bias is removed. The reason why a bias by itself has no effect on calibration is that it also shifts the target beta implicit in present practices. Bridge safety is always adversely affected when any conservative bias is removed. In fact, it may have been historically deliberate to include part of the overall safety factor in the analysis (g), because it is less obvious and does not promote heavier trucks in the same way as, say, increasing the nominal loading from HS-20 to HS-25. Keep in mind that the HS-20 vehicle is only 72 kip. With a safety factor of 1.82 as in working stress design the limit state condition should occur at a load of 72×1.82 or 131 kip. Clearly, many vehicles exceed this level without any apparent bridge damage and this reflects other safety factors implicit in the design including conservative values of g, impact, and dead loading.

The present AASHTO $S/5.5$ factor would indicate that a single girder may carry $1.45 \times$ wheel line ($S = 8$ ft) or 36 percent of a two-lane loading. The measurements indicate that AASHTO is generally conservative although not usually as conservative as implied by the AISI finite element studies given in Figure 15. A possible explanation is that AASHTO values assume the most severe positioning of the vehicles in their respective lanes. It should also be noted that field studies often show much lower stresses than calculated values because of uncertainties regarding the section modulus. Most older bridges have noncomposite designs, i.e., no positive shear connectors. Yet, in fact, actual behavior is usually composite. See Table 9 which compares measured stresses and calculated stresses where the latter already incorporates the measured distribution factors. In several instances it has been observed that, even after full composite action is assumed, measured stresses are still below calculated values. This reflects added section influence due to deck width, overlay, parapets, curb, deck steel, and other participation in the section not normally accounted in calculations. Furthermore, stresses are reduced in simple spans by effective end restraints due to end walls or even axial forces due to frozen bearings.

Table 8. Comparison of field measurements with AASHTO girder distribution. (Refs. 19, 21)

Site	Field Measurements*	AASHTO
1	0.66	0.72
2	0.64	0.52 (nonconservative)
3	0.56	0.67
4	0.54	0.72
5	0.56	0.78

*Includes a one sigma bias to reflect measurement scatter

In the present study, such additional strength capacity due to added section modulus cannot normally be included in the rating. One reason is that added composite section may in fact be lost at higher overloads as friction is overcome by the shear requirements. Secondly, field testing is only recently being made available to aid routine rating.

It is anticipated that for the near future, rating calculations will be carried out with specified AASHTO distribution factors similar to present values. However, the engineer should have the option to do a more rigorous analysis and this is provided in the proposed guidelines. Elaborate modifications to these factors for routine design have been shown only in the recent Ontario provisions. Comparisons of the Ontario distribution graphs derived from accurate structural analysis procedures and

Table 9. Comparison of measured and computed stresses due to test truck.

Site	Average Measured Stress (Ksi)	Computed Stress (Ksi)	
		Composite	Noncomposite
1	5.03	5.13	7.61
2	3.87	5.65	7.92
3	5.57	6.95	9.91
4	3.26	3.93	5.87

Reference (20)

the AASHTO values are given in Table 10. These show that the average bias (conservative) is about 10 percent for a slab on steel multigirder system (spacings 5 to 10 ft and spans above 50 ft). This agrees nicely with limited field results taken by the researchers at Case Western Reserve University. (The variability of this quantity is illustrated in Table 10 from the analysis by showing the lower-bound encountered). Scatter is primarily due to location of the critical girder with respect to the traffic lane and various diagonal and bracing details.

As in the AASHTO derivation, the Ontario charts assume the worst position of the vehicle with respect to maximum loading. The vehicle lateral spacings are also smaller than those that usually occur in a moving situation. Also given in Table 10 are some suggested g values that are function of the span length. Although only slightly more complicated than present AASHTO values, these g values give less scatter.

For the purpose of the present study, the bias in g using AASHTO nominal values will be taken as 0.90 with a COV taken as 13 percent for steel members. Similar values have also been reported in NCHRP Project 12-26 which establishes a large data base of bridges for estimating wheel load distribution factors. Distribution factors for 30 bridges were calculated both by using a sophisticated finite element model (GENDEK5) and a simplified method (MSI-multisurface interpolation). Subsequently, distribution factors for 323 bridges were determined by using MSI. The true bias can be found by relating the bias of the GENDEK5 values to MSI which, in turn, is related to the AASHTO distribution factors. The final values of the bias and coefficient of variation according to structure type are given as:

Structure Type	Bias-g	COV-g
T-Beam	1.05	13%
P/S Concrete	0.96	8%
Steel I Girder	0.90	13%

A value of g above 1.0 means the present AASHTO lateral distribution is unconservative for these types. These values for steel and prestressed (P/S) concrete are used in the subsequent reliability modeling and the derivation of their corresponding resistance factor, ϕ. Any modifications to present AASHTO distribution factors which result in a systematic shift in lateral distribution factor will require adjustment in the proposed ϕ values.

The COV can be reduced by finite element, better analysis or measurements. If a refined analysis is used directly, the bias is effectively removed. The benefit of better analysis should be used in reducing uncertainty—not in changing bias that is already considered in the calculation of the safety indices. The table below shows the suggested V_g data on g (for steel members) if different analyses are used. These reflect accuracy of finite element models of structural behavior.

Analysis Used	Bias, B_g	COV, V_g
1. AASHTO values (for steel girders)	0.9	13
2. Some simplifying assumptions made and conventional methods of analysis used	1.0	10
3. Finite elements, orthotropic plate theory, and other sophisticated methods with very few simplifying assumptions	1.0	7
4. Field measurements	1.0	4

Note, the scatter in g reflects load distribution to members—but not the stress calculation which includes further uncertainties such as section modulus or end fixity. The latter affect the stresses but to a lesser degree the load distribution. Section modulus uncertainty is especially large when a noncomposite behavior is assumed as in the case of many older structures. (The authors believe that even if tests demonstrate a composite behavior, these cannot be relied upon unless the test loads exceed some factored live loading. Otherwise, there is the possibility of losing the composite action at some high load level with possible catastrophic consequences.)

The data on g do vary with structure geometry. For example, for a two-girder system or a truss, the distribution of loading may be more accurate. There are also some agencies that presently use a distribution factor which divides total load equally to each girder. In an ultimate limit state condition, the distribution will be more balanced. This is not considered in this study except to advocate that redundancy be a consideration in specifying the safety index in a manner similar to the present AASHTO fatigue requirements. There is evidence to suggest that a parallel girder system will have adequate reserve capacity to permit some lateral redistribution of loads.

I—Dynamic Impact. The final loading variable recognized herein is the dynamic amplification. This variable has received considerable attention in recent years, both analytically and experimentally. The AASHTO specification equation has a maximum 30 percent value which decreases with span length. The Ontario code has increased the maximum to 40 percent, with impact value based directly on the bridge natural frequency rather than the span length (there is some relationship between these two quantities). Some field measurements seemed to reinforce these values. The authors, however, and others have refuted such high impact values using field testing. The main parameters affecting impact are the bridge approach, bumps, and other pavement roughness. Expected impact values for evaluation may therefore be lowered for several reasons:

1. Impact decreases with vehicle weight. Some reported tests were conducted with light trucks giving rise to larger impact values. The recent study of Ontario Survey of Commercial Weights (1975–82) also shows a clear reduction in impact with vehicle weights.
2. Because maximum loading occurs with several vehicles on the bridge, it is unlikely that all dynamic responses will be equally high and also in synchronization with each other.
3. Tests with high impacts (*48*) were achieved by deliberately creating rough surfaces, with boards or other items set on the pavement. By specifying a range of impact values in the evaluation based on deck surface conditions resurfacing (to avoid posting) may be encouraged and, hence, the realization of lower impact values.

Table 11 shows some examples of impacts extracted from the Case Western Reserve University (CWRU) measured data base. These values are obtained by looking at the strain record and the portion of record showing oscillations. Refer to Figure 16, which explains how impact is determined. It should be noted that automated data processing to record impact by analyzing strain records is not available for spans under 120 ft because

Table 10. Comparison of *D* values from rigorous analysis (Ontario specifications) and AASHTO.

SPAN (FEET)	NO. OF LANES IN BRIDGE	AASHTO D VALUE (FT)	D VALUES OBTAINED BY RIGOROUS ANALYSIS (FT) FOR INTERNAL GIRDERS*	
			MEDIAN	LOWER BOUND
10-15	2	5.5	4.8	4.6
15-20	2	5.5	4.9	4.7
20-30	2	5.5	5.1	4.8
30-50	2	5.5	5.5	5.2
50-300	2	5.5	5.9	5.7

* Values above 5.5 mean AASHTO value is more conservative.

Table 11. Examples of measured impact values. (Ref. 20)

SITE	RANGE OF MEASURED IMPACT	AVERAGE
1	2.5 - 5.5%	3.3%
2	3.2 - 13.7%	9.1%
3	5.4 - 14%	10.4%
4	3.4 - 15.4%	5.7%
5	4.6 - 22.1%	11.6%

Note: The random variable used herein is I which modifies the static moment. (Unlike AASHTO in which 1 + I modifies static moment). Thus, the mean of our variable I ranges from 1.1 to 1.3 with a COV of 10% for different deck surface and approach conditions. A 10% COV for smooth surface means in fact there is a 5% chance that the impact will exceed 1.28 or 28% impact during the critical event.

part of the apparent strain oscillation is due to static response related to wheel spacings. This partly explains why some researchers have reported high impact values. Another cause of high reported impacts is that authors have compared static crawl runs with normal speeds and based impact on comparison of peak stress in each case. However, changes in vehicle lateral distribution will also affect peak stresses and may confuse the impact value.

That impact is related directly to roadway roughness is seen by examining the strain records for three similar bridges in Ohio (Figures 17(a), 17(b) and 17(c)). It is noted that the first two bridges have relatively significant dynamic response, while the third bridge has a smooth record. The latter bridge is on the Ohio Turnpike and had a very smooth deck and approach. The other two sites had average to rough decks. Unfortunately, there is not enough data to allow a clear delineation of impact with pavement roughness. Furthermore, in addition to roughness, impact can be increased by bridge approach and grade.

For the safety analysis that follows, three categories of roughness will be used. The same COV is used even though the roughness increases. This reflects the possibility that evaluators may be biased conservatively with regard to their interpretation of the degree of site roughness:

Smooth surface, mean, I = 1.10, COV = 10%
Medium surface, mean, I = 1.20, COV = 10%
Rough surface, mean, I = 1.30, COV = 10%

These values are also supported by analysis of the dynamic bridge tests performed by the Ontario Ministry of Transportation and Communications (MTC) (*52*) and Nowak and Zhou (*26*). It has also been noted (*35*) that the average impact of 1.1 as obtained from WIM studies falls in the middle range of the data plotted from the MTC tests. Thus, all the above data do indicate that higher impact values may be overly conservative for evaluation purposes. In the guidelines (Appendix A), some instructions are provided for selecting categories for a given roadway.

RESISTANCE

The uncertainties in steel member behavior arise because of variability in material yield, fabrication (including dimensional tolerances), and accuracy of strength prediction theories. For a steel beam, Galambos has recommended a bias of 1.1 and a COV of 13 percent (*18*). This was based on a review of rolled beam sections and plate girders. The bias arises because the nominal yield used by the designer is less than the true mean yield. (This bias, which favors higher reliability, should not be included if field samples are used to test for the yield and an average value is selected for the yield stress.)

The issue is complex because strain rate, direction of loading, and thickness of test samples all affect the yield stress. A 10 percent bias for structural steel, however, seems to be an upper-bound for most situations. A 12 percent COV covers the variabilities cited earlier with the overall V_R determined from:

$$V_R = \sqrt{V^2_M + V^2_F + V^2_P} \tag{20}$$

where V_M = COV due to material; V_F = COV due to fabrication; V_P = COV due to scatter in prediction theory; and V_R = overall resistance COV.

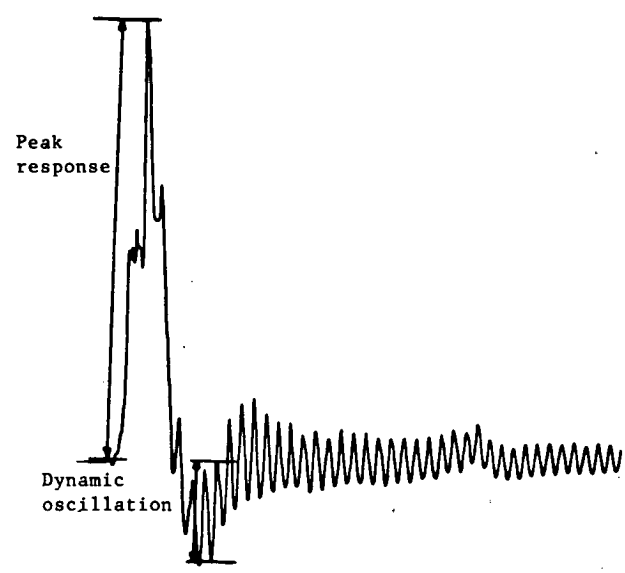


Figure 16. Strain record.

Additional statistics on resistance of structural steel and prestressed concrete members collected by Ellingwood (*11*) are given in Table 12 which gives both the bias and coefficient of variation for these elements. The COV depends on the type of failure mode being analyzed (bending, shear, compression) and the type of component (rolled beam, composite beam, welded girder) being considered. Most test data are for new specimens and indicate scatter in the range of 10 to 14 percent. For older structures, the scatter may be larger because of different sources of material, fabrication techniques, and the effects of age and environmental condition. In the case of a member in which there is some partial corrosion and loss of section a further uncertainty is to estimate any further losses that may occur prior to the next inspection interval. All of this leads to a somewhat subjective estimation of the resistance COV. It should be noted that safety indices are quite sensitive to V_R because there is no "averaging" of effects as in the case of the loading "side" of the equation which contains both dead plus live loading. Based on these considerations, the following data will be used in subsequent safety index calculations:

- New condition, steel members: bias = 1.1, COV = 12%.
- Partially corroded material and some slight loss of section: bias = 1.05, COV = 16%.
- Severely corroded material and noticeable loss of section: bias = 1.0, COV = 20%.

Table 12. Typical resistance statistics for structural members. (Ref. 11)

Designation	Bias	COV
Tension member limit state - yielding	1.05	0.11
Tension member limit state - tensile strength	1.10	0.11
Compact Beam, uniform moment	1.07	0.13
Beam-column	1.07	0.15
Plate girders flexure	1.08	0.12
Axially loaded column (based on AISC Code)	1.08	0.14
CIP pretensioned beams	1.06	0.08
CIP posttensioned beams	1.04	0.095

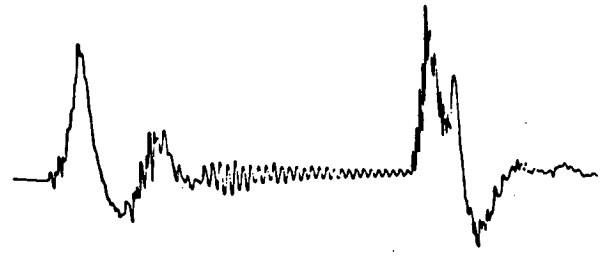


Figure 17(a). Sum of strain records from Drake road.

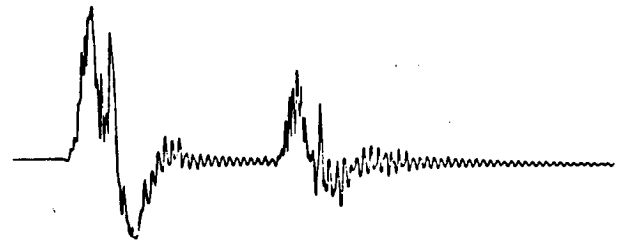


Figure 17(b). Sum of strain records from I-90.

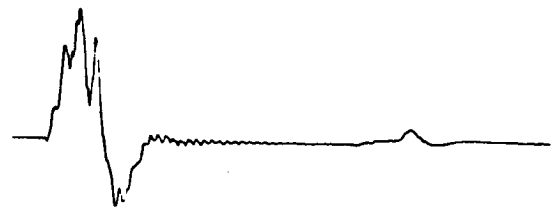


Figure 17(c). Sum of strain records from I-80.

The aforementioned values have been estimated from a large body of data collected by ASTM and later analyzed by Albrecht and Naeemi (*27*). Further details and calculations are presented in Appendix H.

For new condition prestressed members, the situation is slightly more complex because the bias and COV are strongly affected by the prestressed reinforcement index and the grade of construction, pretensioned vs. post-tensioned. In some codes, it is proposed to use different factors directly on steel and concrete properties (see National Building Code of Canada). Rather than overly complicate the situation, the values given in Table 10 are used to obtain a bias of 1.05 and a COV of 9 percent.

Determining the statistics for deteriorated prestressed concrete members will enable us to calculate the capacity reduction factors appropriate for this case. Explicit statistical data are not available at present, the main reason being that the major cause of problems with such members is damage due to vehicles (*30*) and not deterioration (inasmuch as they are relatively new as compared to similar types of steel bridges). Further discussion is deferred to Chapter Four where this issue is resolved, to a certain extent.

If a very detailed and thorough inspection is conducted, the section properties can be obtained accurately, which leads to a reduction in the COV and corresponding increase in the capacity reduction factor. Conversely, if the inspection is not detailed or if plan dimensions or condition surveys are used, there is an

added uncertainty in the amount of deterioration present (if any), which would lead to a decrease in the value of ϕ.

Similarly, incentives for vigorous maintenance and penalties for intermittent maintenance can be introduced by raising or lowering the capacity reduction factor. Unfortunately, there are no available data relating the degree of inspection/maintenance to the COV and bias of the resistance. One alternative available is to assume certain values of ϕ and do a back-calculation to see if the values of COV implied are reasonable (refer to Chapter Four).

To gain an insight into the value of V_R, it should be noted that for a bias of 1.0, a 10 percent COV means there is a 5 percent chance that the strength is less than 83 percent of the nominal calculation and 1 percent chance the strength is less 77 percent of nominal. For a 20 percent COV, there is a 5 percent chance the strength is less than 67 percent of the nominal and 1 percent chance that it is below 53 percent of the nominal. Part of V_R relates to possible further future section losses prior to the next inspection. Note that in these instances the nominal resistance should include best estimates of section loss. If one is conservative in the nominal estimates, then one may be imposing an implicit conservative bias and combined with an increase in V_R may together be excessively conservative. Unfortunately, it is difficult to find data in this area, and the sensitivity study cited must be conducted to test the range of partial safety values that are realized with different assumptions regarding the data (see Chapter Four).

APPLICATION OF RELIABILITY TO BRIDGE DESIGN MODEL (*23,33*)

In the long term, an evaluation procedure should be related to the design philosophy to avoid possible contradiction between design and evaluation checks. In a related study, Moses has been involved in developing reliability analysis for bridge design. At present, there is also an FHWA project at the University of Michigan under Professor A. Nowak on this subject. The goal is to select nominal load models, load factors, nominal strengths, and capacity reduction (resistance) factors in a manner that provides consistent levels of safety. These goals do not imply uniform reliability for all components because it may be desirable and economical to provide different levels of safety based on redundancy, bridge span (cost), and importance. Thus, any safety level differences should be done deliberately by comparing safety indices.

A common starting point for introducing reliability criteria is to ask what is the desirable or target safety index. In the Ontario code a beta goal of 3.5 has been explicitly mentioned, while in the AISC a goal of 3.0 appeared in the original references. In the authors' viewpoint, there may be too many imponderables to suggest a specific target, and past experience combined with engineering judgment should be part of the selection of standards. The data base should be applied to existing design practices to determine the implicit safety levels in current practice and to review any discrepancies that cannot be rationally justified. As an example, the study by Ghosn and Moses evaluated a safety index from nominal strength, R_n, produced by the AASHTO design procedure (*24,33*). (In the case of evaluation R_n already exists from inspection.)

For example, with AASHTO-WSD design:

$$R_n = 1/0.55\,[D + L_n(1+I)] \tag{21}$$

where L_n is the nominal loading calculated from HS-20 and I the AASHTO impact for the span. Similarly, with AASHTO-LFD design:

$$R_n = 1.3\,D + 1.3\,(5/3)\,L_n(1+I) \tag{22}$$

Note that the bias values on R are different in WSD and LFD because LFD directly allows the plastic sections for checking compact sections, while WSD does not account for it. Assuming a 14 percent increase in plastic section means that WSD, in fact, has an additional 1.14 bias on R_n compared to LFD.

To simplify the nominal dead load, a formula due to Hansell and Viest (*25*) is used which was derived from examining a number of typical steel spans:

$$D = 0.0132\,(L_n + I) \times \text{span length (ft)} \tag{23}$$

Thus, for a 60-ft span the dead load maximum moment is almost 80 percent of the HS-20 nominal maximum moment. (This equation will be further checked in this study using sample bridges to be determined from several states.) To review the safety indices implied by WSD and LFD, β was calculated using the level II program for a range of simple spans. These beta values are plotted in Figure 18.

It is seen from Figure 18 that the safety index for WSD ranges from a low of about 2.5 for short spans to about 4.4 for long spans. This is not justified by any logic regarding failure cost. Presumably, the failure consequences, including public inconvenience and possible loss of life, should be the same for a short and long span bridge. The incremental cost to raise the capacity of a short span bridge is much less than that of a long span; therefore, in fact, higher betas for short spans could be desirable and optimum. The recognition of the "excessive" safety margin for longer spans was the concern that inspired LFD. As seen in Figure 18 the safety indices for LFD are more uniform than for WSD but also are not smooth with β ranges between 2.5 and 3 for HS-20. The impact of higher design loads instituted by many states such as an HS-25 criterion is also shown in Figure 18. The betas for this criterion are increased because L_n is increased by 25 percent. These betas, however, are also not uniform or consistent.

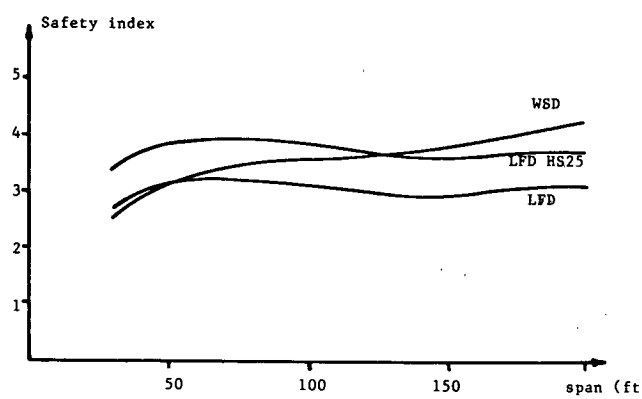


Figure 18. Safety indices with current AASHTO design.

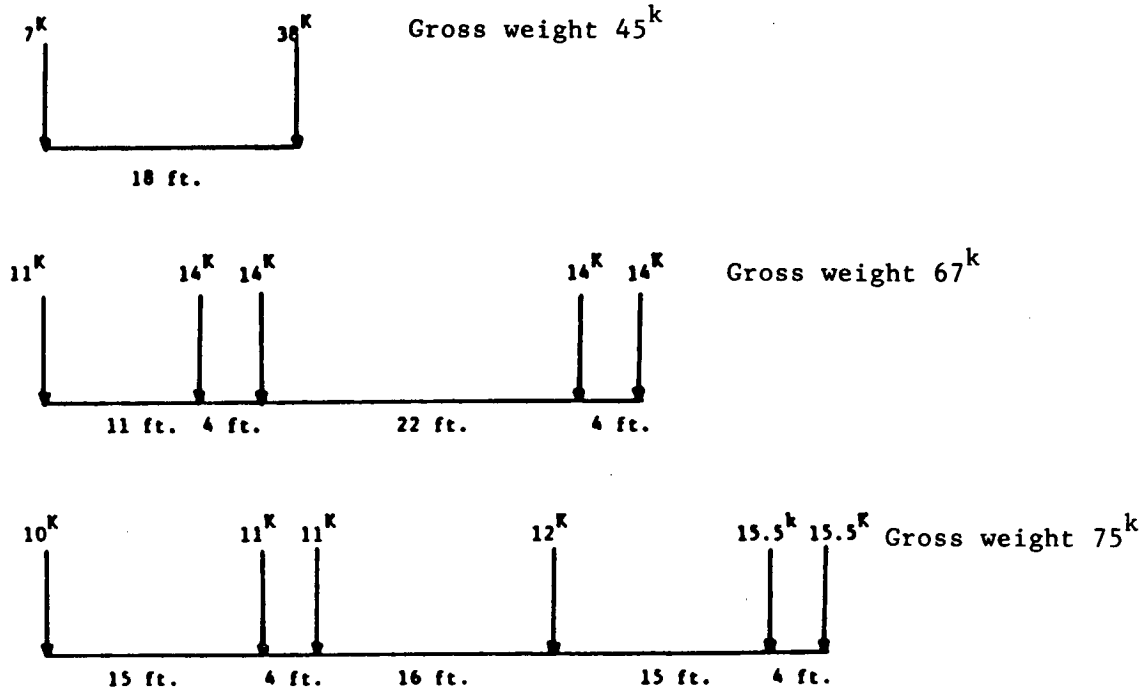


Figure 19. Proposed design vehicles. (Ref. 33)

Clearly, present design criteria including HS-25 do not appear to provide any consistent reliability. Before revising the criteria, however, a sensitivity study should be done to see if perhaps other interpretations of the data may yield different conclusions. This was done for each of the variables in the loading model and also including a load growth variable which is highly subjective (*21*). The results of these sensitivity studies also showed that present AASHTO provisions did not lead to uniform reliability. Similarly, the reserve capacity inherent in system performance was introduced. For example, parallel girder systems often have greater reliability because of higher section capacity after including the entire deck and the redistribution of loading to less loaded girders. These were also compared in the opposite sense to structures that were statically determinate (no load redistribution is possible) and further may be subject to deterioration due to severe environmental condition and lack of maintenance. By assuming such a pessimistic and optimistic range for system performance, the beta calculations were performed with the following changed parameters:

Resistance bias = 0.8 and COV = 15% (pessimistic)
Resistance bias = 1.4 and COV = 15% (optimistic)

As expected, the betas are drastically affected by the changed bias. For example, lower safety levels should be expected if the structure is not maintained. It was observed, however, that the betas were still nonuniform over different spans for the AASHTO provisions.

The major reason for the lack of uniformity of beta is the HS design vehicle and present AASHTO dead and live load factors. These do not provide a uniform comparison with existing truck configurations. Figure 19 shows a set of vehicles proposed by Ghosn and Moses as a design model (*24, 33*). The vehicle weight and configuration are based on the extreme weight vehicles observed in the WIM load spectra study and are similar to the present AASHTO rating vehicles (*1*). After calibrating new load and resistance factors, the betas obtained with this new model are presented in Figure 20. It can be seen that the betas are more uniform with a maximum beta of 3.6 and a minimum of 3.44 over spans 30 to 200 ft. Similarly, the sensitivity studies were repeated with different growth terms and resistance parameters. Again, it showed that the betas are more uniform than the values obtained with either the working stress design or the load factor design. (*33*). Hence, one gains confidence with the data and the recommendations for the new design load model and corresponding calibrated load factors. Thus, the developments herein to establish load and resistance factors for an evaluation format can subsequently be included in the bridge design formats now under study by FHWA, NCHRP, AASHTO, and others. Although it is desirable to have consistency in the reliability formats between the design and evaluation codes, this is not necessary because achieving the goals outlined in this report establishes a more rational evaluation guideline.

The next chapter formulates the bridge evaluation model. It is similar to the bridge design model just described (in Eqs. 21 to 23) except for the following differences:

1. No load growth is considered because evaluation covers a relatively short inspection interval period.
2. A range of load factors is needed to cover the use of load information that may be used during the evaluation including weight statistics, volume, impact, and improved girder analysis.
3. A range of resistance factors is needed to reflect the inspection data corresponding to the uncertainty of component and system capacity.

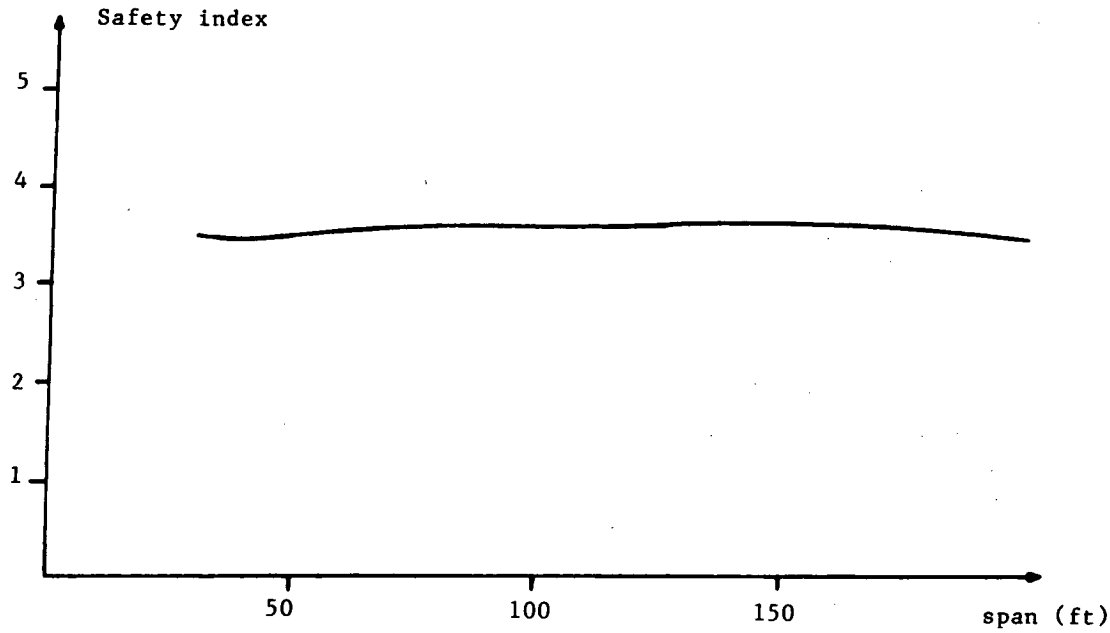


Figure 20. Safety indices with proposed design ($\gamma_D = 1.4$, $\gamma_L = 2.85$, $\phi = 1.0$). (Ref. 33)

CHAPTER FOUR

RELIABILITY MODEL FOR EVALUATION

The reliability model for evaluation uses the theory and discussion of the variables outlined in Chapters Two and Three. The initial calibration will be done with a generic study of reliability calculations over a range of simple spans. After selecting the loading model and corresponding load and resistance factors the model is also checked with specific bridge examples covering a range of spans, support geometries, lanes, traffic conditions, and rating factors.

This chapter supports the values selected in the evaluation guidelines. The latter are presented in Appendix A in a specification form which can be adopted for inclusion in the AASHTO *Manual for Maintenance Inspection of Bridges* (henceforth referred to as the AASHTO *Maintenance Inspection Manual*).

SAFETY INDEX CALCULATION

The safety margin equation is:

$$g = R - D - L \tag{24}$$

Using the live load model described in the previous chapter, this gives

$$g = R - D - amHW_{95}Ig \tag{25}$$

To calculate the safety indices, it is necessary to establish statistical values (bias and COV) for the resistance, the dead load, and each of the parameters in the live load model. This has been accomplished in Chapter Three. Table 13 gives a summary of this data base. All that needs to be done is to determine nominal resistance and dead load values, which can then be substituted in Eq. 25 to determine the safety index β (using available programs).

The proposed rating equation in an LRFD form is

$$\phi R_n = \gamma_D D + \gamma_L L\,(1+I)\,R.F. \tag{26}$$

where ϕ is the resistance factor, γ_D is the dead load factor, and γ_L is the live load factor. Therefore, bridges that are just safe, according to this equation, have a rating factor, *R.F.*, equal to 1.0. This enables us to calculate the nominal resistance for generic spans which is then given as:

$$R_n = \frac{\gamma_D}{\phi} D + \frac{\gamma_L}{\phi} L\,(1+I)\,R.F. \tag{27}$$

Similarly, the nominal dead load effect is estimated using an

equation developed by Hansell and Viest (*25*) which is given as:

$$D_n = 0.0132\, L(1+I) \text{ span} \tag{28}$$

The input to the safety index calculation routine requires mean values that can be obtained from the nominal values in Eqs. 27 and 28 by multiplying with the bias (given in Table 13). Detailed stepwise calculations following this general procedure are given in Appendix F.

Examining Eq. 26 shows that the nominal resistance for these generic spans is influenced by the choice of the load and resistance factors which, in turn, affects the safety index. This is the key step in the calculation, i.e., load and resistance factors are selected such that a particular level of safety (target safety index) is achieved consistently for these generic spans. The indices will vary with span because of different dead to live load ratios since the same load and resistance factors must be used for all spans. Hence, factors must be selected that on the average meet the target safety indices.

Establishing the target safety index is another important step in the calibration procedure, the discussion of which will be deferred to the next section.

SELECTION OF TARGET RELIABILITY

It is assumed in this report that bridges should have a constant level of reliability. This is expressed by the safety index (equal to the number of standard deviations that the mean safety margin falls on the safe side of the limit state equation. See Chapters Two and Three and Appendix E for further elaboration). A constant reliability assumes that it is not desirable to use such features as bridge replacement cost, traffic, function, and material type as reasons for adjusting the reliability level. Redundancy is important, however, and this is the only basis, herein, for adjusting the target betas—in other words, higher betas for nonredundant components, as compared to redundant components, because of obvious differences in failure consequences. (For a brief discussion of this aspect refer to Appendix E.)

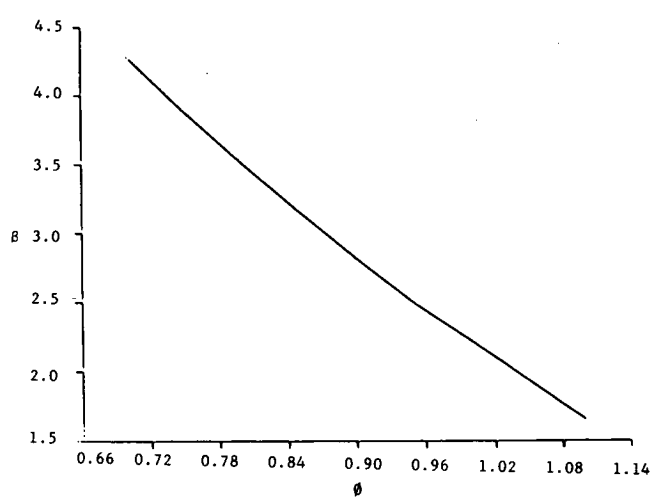


Figure 21. ϕ vs. β for unenforced, light traffic.

Table 13. Summary of data base for evaluation.

(a) RESISTANCE (STEEL)

CONDITION	BIAS	V_R
1. Good condition	1.1	12%
2. Slighty corroded, some possible loss of section	1.05	16%
3. Heavy corrosion	1.0	20%
RESISTANCE (P/S)		
1. Good condition	1.05	8%

(b) DEAD LOAD - nominal for generic simple spans

Dead Load = 0.0132 x (L_n + I) x span (feet)
Bias = 1.0
COV = 10%

(c) 'a' - deterministic factor to compute max. simple span moment

span(ft)	30	40	50	60	80	100	160	200
a(k-ft)	4.46	6.92	9.42	8.5	13.36	16.75	31.75	41.75

(d) 'm'

span(ft)	30	40	50	60	80	100	160	200
m(singles)	0.89	0.93	0.94	0.94	0.96	0.97	0.98	0.99
V_m(singles)	8.4%	8.8%	7.7%	6.1%	4.8%	4.2%	1.6%	1.8%
m(combinations)	1.0*	1.0*	1.0*	0.92*	0.91	0.93	0.96	0.97
V_m(combinations)	11%*	11%*	11%*	11%*	8.2%	5.7%	3.4%	3.2%

*Values for m based on WIM studies except for * values adjusted to reflect WIM data scatter

(e) $H\,W_{.95}$

	LIGHT TRAFFIC	HEAVY TRAFFIC
SINGLES		
UNENFORCED	120^K (10%) (Category 3)	125^K (10%) (Category 4)
ENFORCED	92^K (8%) (Category 1)	100^K (8%) (Category 2)
COMBINATIONS		
UNENFORCED	210^K (10%) (Category 3)	225^K (10%) (Category 4)
ENFORCED	170^K (5%) (Category 1)	180^K (6%) (Category 2)

Light Traffic: ADTT under 2000 trucks/day for combinations; under 300 trucks/day for singles
Heavy Traffic: ADTT over 2000 trucks/day for combinations; over 300 trucks/day for singles

(f) Girder distribution, g

Method of analysis	BIAS B_g	COV V_g
1) AASHTO Distribution Values (for steel girders)	0.9	13%
2) AASHTO Distribution Values(for P/S)	0.96	8%
3) Conventional Structural Analysis	1.0	10%
4) Finite elements, orthotropic plate theory, also for statically determinate systems	1.0	7%
5) Field measurements	1.0	4%

(g) Impact, I

Roughness	Mean	V_I
Smooth	1.1	10%
Medium	1.2	10%
Rough	1.3	10%

All variables assumed to be lognormally distributed

The authors believe that uniform reliability assumes a bridge failure may precipitate damage or loss of life, which has the same impact whether it is a long-span bridge on a heavily traveled route or a short-span bridge with little traffic. The function of the bridge can, however, be accounted for in the model. This is done by increasing the target beta based on function and then adjusting the appropriate safety factors. Figure 21, for example, shows the impact of different betas on the resistance factors. From this curve it appears that a 10 percent reduction in capacity reduction factor, ϕ, increases beta by about 0.6. From Chapter Two, this is equivalent to about a fivefold reduction in risk.

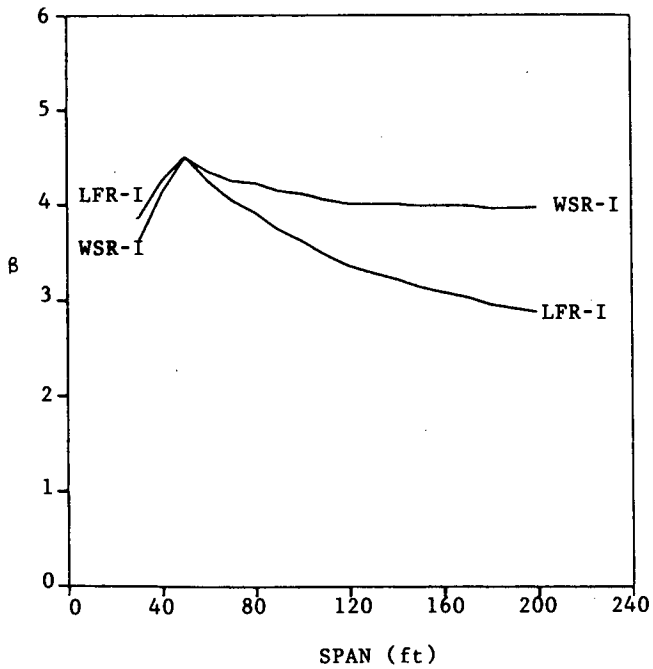


Figure 22. β for inventory checks (unenforced heavy traffic).

Formulating a reliability-based code often resolves into two problems: (1) assembling the statistical data base, and (2) selecting a target beta for the code. The two issues become intertwined if one decides, as is often done, to select a target based on current performance experience and engineering judgment (this is an alternative to selecting a target on the basis of economics or equivalent insurance costs).

In the case of most structural design, the evaluation of present standards is reasonably direct. That is, if all designers produce similar structures from the same present code, there is a basis for defining "current design." If one is satisfied with the performance and safety record of the current design, then the calculated betas implicit in the current design can be used as the target beta. (The reason for changing to a reliability basis is still present as, for example, in the new AISC-LRFD. Namely, present design reliability may be satisfactory "on the average" but still has too much safety index scatter, i.e., for different materials, components, or dead to live load ratios. The goal in a new design format change is to achieve more uniform reliability.)

The problem in bridge evaluation is that there is no "current design practice." In some sense, the situation is similar to bridge design in which a few states are independently moving towards HS-25 or design for permit vehicles and away from the standard AASHTO HS-20 design. In evaluation, there are at least four different procedures for posting rating in use in different states. These include the working stress WSR operating (WSR-O) and inventory levels (WSR-I) and the load factor LFR operating (LFR-O) and inventory (LFR-I) levels. Also, states may apply different distribution factors or nominal loadings (e.g., AASHTO legal vehicles, HS-20 or other vehicles). Presumably the nominal loading or vehicle configurations reflect their own state laws so it can be neglected for this discussion.

For the present calibration it is necessary to make some decisions about the beta values implied in each of these rating procedures. Figure 22 compares the betas for a set of hypothetical simple spans with sections determined that just satisfy an $R.F. = 1$ based on WSR-I using present procedures. The live-load loading category is assumed as unenforced, heavy traffic and the nominal loading is taken to be the HS-20 design vehicle. The betas are compared to spans with an $R.F. = 1$ using LFR-I. It can be seen that the WSR-I betas are in the range of 3.8 to 4.6 for most spans, while LFR-I betas vary between 3.0 to 4.6. If the betas are examined for operating stress criteria (for unenforced light traffic), which also provide a rating factor equal to 1.0, the betas are much lower as in Figure 23 (a). The betas range from 2.2 to 3.4. Also shown are the betas for LFR-O checks, which are typically lower than those for WSR-O, and range from 2.0 to 3.0. The target beta (based on past practice) therefore depends on what criteria are used. Figures 23 (b) and 23 (c) show the betas for all the four traffic categories using WSR-O and LFR-O (see Table 14 for the definition of these traffic categories). A large variation in the safety indices is noted, depending on the severity of traffic (and hence the projected live load effect). Thus, to select a target safety index one has to decide on the method of rating (WSR or LFR), the level of rating (inventory or operating), and the severity of traffic (unenforced or enforced, light or heavy volume traffic).

The general philosophy adopted in selecting the recommended rating guidelines is to select a target beta by relying on some statements in the AASHTO *Maintenance Inspection Manual.* Namely, if a bridge is frequently inspected and the loads are reasonably controlled (which corresponds to unenforced, light traffic), one can do assessment with the operating stress level. (Furthermore, the many states using these criteria have not found an unacceptable average failure rate.) For this reason, reference beta values have been selected similar to the operating WSR as the target beta for situations where these criteria are satisfied. Using this approach and the data base in this chapter, the live load factors should be calibrated to average beta values extracted from Figure 23. Assuming spans less than 100 ft is most common; a target beta in the vicinity of 2.75 would be appropriate. If it is assumed that all the bridges in this range have acceptable safety levels, a target beta of 2.5 may be selected. Consequently, this safety index value of 2.5 is chosen as the target in this report. Note that Figure 23 shows beta with a peak at a 50-ft span. The shape of Figure 23 was investigated further by doing a simplified reliability calculation and looking carefully at each term in the analysis affecting the mean safety margin and the scatter. See Appendix F for this analysis.

As mentioned before, redundancy is the only basis for adjusting the target safety index. So, for nonredundant components, a higher safety index would be desirable. (Refer to Appendix E for detailed explanations.) It is assumed herein that the original design or inventory level (for unenforced, heavy traffic) is suitable for both redundant and nonredundant components. Hence, for good condition nonredundant elements, the target safety index will be derived from current inventory levels that are shown in Figure 22 for both working stress and load factor (for unenforced and heavy volume traffic). An average value of 3.5 is chosen.

It should be noted that one calibrates to different levels of traffic for redundant and nonredundant elements. This is because calibration to the original design or inventory level should

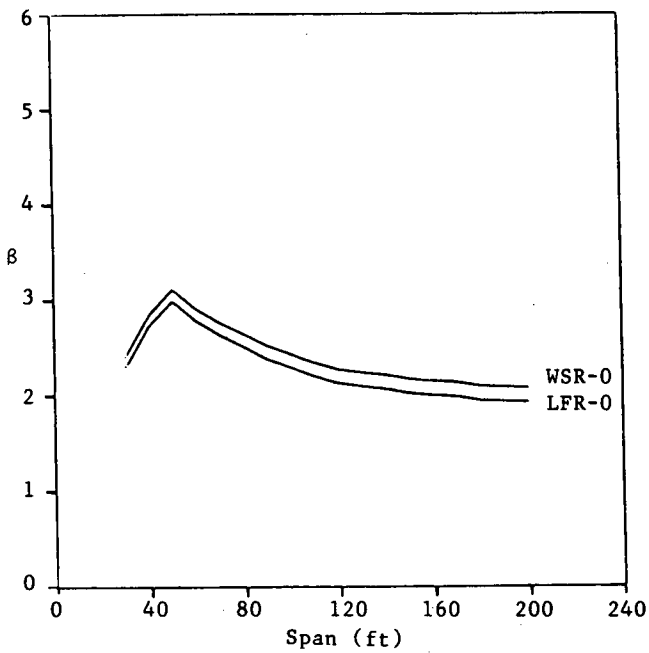


Figure 23(a). β for operating checks (unenforced, light volume traffic).

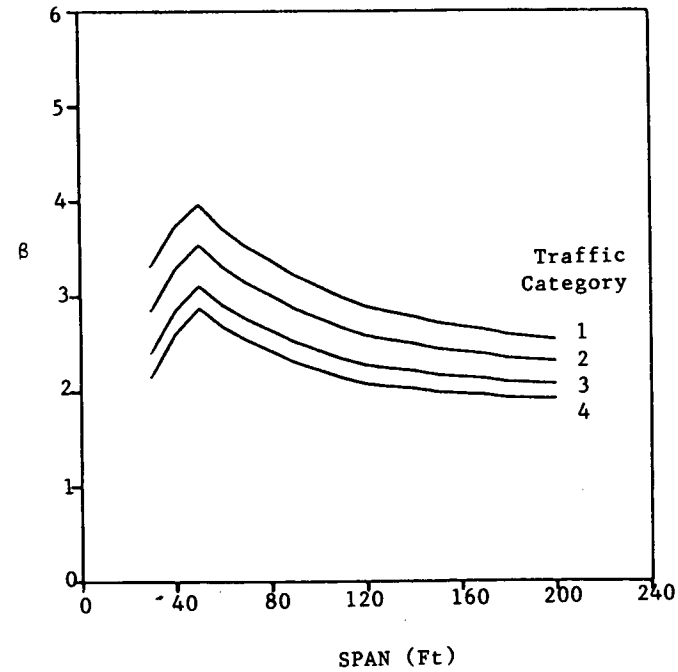


Figure 23(b). β for WSR-O for 4 different traffic categories.

account for the severest traffic possible (which corresponds to unenforced, heavy traffic), whereas calibration to the operating level should be for unenforced, light traffic ("reasonably controlled" according to the AASHTO Manual).

SELECTION OF RESISTANCE AND DEAD LOAD FACTORS

The factors will be selected using the foregoing data and a calibrated target beta for redundant components. A number of considerations went into the selection of the load and resistance factors for the proposed guidelines. An observation of Eq. 26 shows that ϕ, γ_D, and γ_L are not independent for a single checking case. If all factors are raised or lowered uniformly, the rating factor remains the same. However, for convenience, the load factors γ_D and γ_L should be the same for all steel, concrete, and other bridges. ϕ accounts for the bias and coefficient of variation of the particular material or member type (connection, girder, truss) being checked. A formula, developed by Lind and used in the American Institute of Steel Construction's Load and Resistance Factor Design Specification, gives an initial means of establishing ϕ, i.e. (*8*):

$$\phi = B_R \exp(-0.55\, \beta V_R) \tag{29}$$

where B_R is the bias in R or ratio of mean to nominal value, β is the target safety index, and V_R is the resistance coefficient of variation. For example, with V_R = 12 percent and B_R = 1.1, ϕ is determined as a function of β from Eq. 29 as:

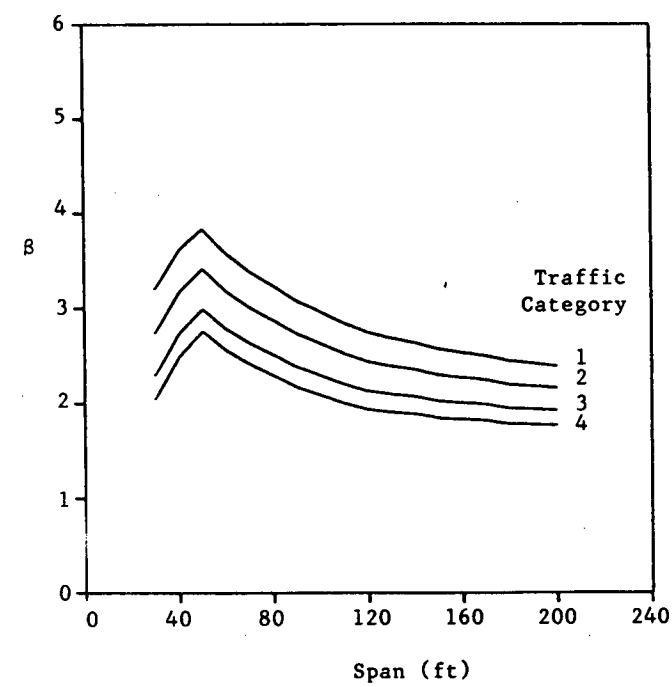


Figure 23(c). β for LFR-O for 4 different traffic categories.

Safety Index, β	Resistance Factor, ϕ
2.0	0.96
2.5	0.93
3.0	0.90
3.5	0.87
4.0	0.84

Based on this approximate table and results from a more exact level II reliability analysis, a value of $\phi = 0.95$ is selected as the reference case for a good condition element for both steel and prestressed concrete. The value of ϕ is subsequently adjusted to recognize the data available from inspection and the degree to which maintenance can be expected to detect any important defects. This means a change in bias and V_R. For example, a member that shows a high amount of corrosion will have larger uncertainty in the strength. By using Eq. 29 (and target β = 2.5 and 3.5 as examples) the variation of ϕ with V_R is found for the three categories discussed for resistance in Chapter Three, section under the heading "Resistance."

Resistance Factors, ϕ		Bias and COV of Resistance		Comment
$\beta = 3.5$	$\beta = 2.5$			
0.87	0.93	1.1	12%	good condition
0.78	0.85	1.05	16%	slightly corroded
0.68	0.76	1.0	20%	heavy corrosion

Thus, nonredundant components which require higher betas must be assigned a smaller ϕ to provide the higher reliability. Note, the foregoing table is only approximate and in this study all β's are checked by the more accurate AFOSM (Advanced First Order Second Moment) beta program described earlier.

After selecting $\phi = 0.95$ for the good condition steel girder case, γ_D was selected. This factor will also remain constant in the checking and only γ_L will vary based on information of the traffic and loading. Because ratios of D/L will vary, γ_D was selected to ensure a sufficient capacity for a situation in which dead loading effects are dominant, e.g., long span bridges. For this case, the failure margin, g, is found from Eq. 29 as:

$$g = R - D \tag{30}$$

$$R_n = \frac{\gamma_D D_n}{\phi} \tag{31}$$

where D_n is the nominal dead loading and equals its mean value. (V_D = 10 percent and V_R = 12 percent, $B_R = 1.1$ and $\phi = 0.95$.) For illustration, an approximate log normal distribution model gives the necessary value of γ_D as a function of β as (*11*):

$$\gamma_D = \frac{\phi}{B_R} \exp \beta \sqrt{V^2{}_D + V^2{}_R} \tag{32}$$

Safety Index, β	Dead Load Factor, γ_D
2.5	1.28
3.0	1.38
3.5	1.49

Inasmuch as it is an extreme case to assume no live load at all, a γ_D of 1.20 will be accepted as satisfactory. It will be checked on the example bridges below with the level II program. It should be noted, however, that there may be some validity in specifying a beta for a dead load check higher than the beta for the live loading. The bridge engineer wants to be confident that a bridge can safely support the dead weight. The live loading has an implicit assumption that the likely failure event will be caused by an overloaded vehicle. This removes some of the liability from the engineer. However, if it fails under dead weight with, say, only car traffic, the responsibility is solely on the evaluator. For this reason, it is possible to recommend that any check for dead weight alone should be more conservative. However, unlike building design, there is no precedent for performing an independent check with dead load only. Because an evaluation is being done, it is presumed that the structure is already safe for the dead load acting alone.

The ratio of γ_D/ϕ, which is 1.2/0.95, equals 1.26. This is only 3 percent below the corresponding LFD ratio which is 1.30. Because it applies only to the "best condition" case, it should be deemed acceptable in this direct comparison independent of the reliability level.

SELECTION OF LIVE LOAD FACTORS

A major aim of the reliability modeling is to characterize bridge loading using relevant load statistics based on recent data and develop live load factors in conjunction with the loading vehicles to produce a uniform reliability over all simple span structures. First a choice of the rating vehicles has to be made. The HS-20 model is already almost 50 years old and is based on truck combinations that are obsolete. This is recognized in the present AASHTO *Maintenance Inspection Manual* which uses the three rating vehicles (shown in Figure 24). Other states, such as Ohio, Michigan, and California, supplement or replace the AASHTO rating vehicles with additional vehicle characteristics based on their own traffic limitations. A characteristic of the AASHTO rating vehicles is that they just satisfy the Bridge Formula adopted by Congress in 1974.

In a sense, the AASHTO vehicles model three portions of the Bridge Formula which may control short, medium, and long spans. Because they are based on the present legal limits, it is presumed that AASHTO's vehicles model much of the configurations of present truck traffic. Because the AASHTO vehicle models provide in the analysis a generally uniform reliability and are also familiar to engineers, the models are used herein as the reference nominal loading. The use of the HS-20 load model was also considered, but it does not provide as uniform a reliability, for the reasons cited in Chapter Three, for the bridge design example. One way to compare the degree of uniformity is to calculate for different spans the required live load factor that realizes the target index using different load models. If the live load factors vary, the nominal load model (axle spacings and wheel load distributions) is not uniform with regard to reliabilities. Figure 25 shows a plot of the load factors required with both the HS-20 model and AASHTO rating vehicles needed to achieve a target beta of 2.5 for the heaviest traffic category. The resistance factor and dead load factor were fixed at 0.95 and 1.2, respectively, as discussed earlier.

The live load factors required to achieve the target safety index are determined by trial and error. The live load factors required are typically lower for HS-20 because the moment effects calculated are higher than for the AASHTO rating vehicles. The plots also show that the load factors for an HS-20 model are not uniform and would vary from about 1.4 to over 2.0 for different spans. It is seen from Figure 25 that the load factor for the AASHTO rating vehicle model is more uniform and that a live load factor of 1.95 will suffice for realizing the target safety index. The reason for this is that each of the three AASHTO vehicles models closely the effect of traffic for the span lengths they govern. This imparts a greater flexibility as compared to the HS-20 vehicle which is used for all possible

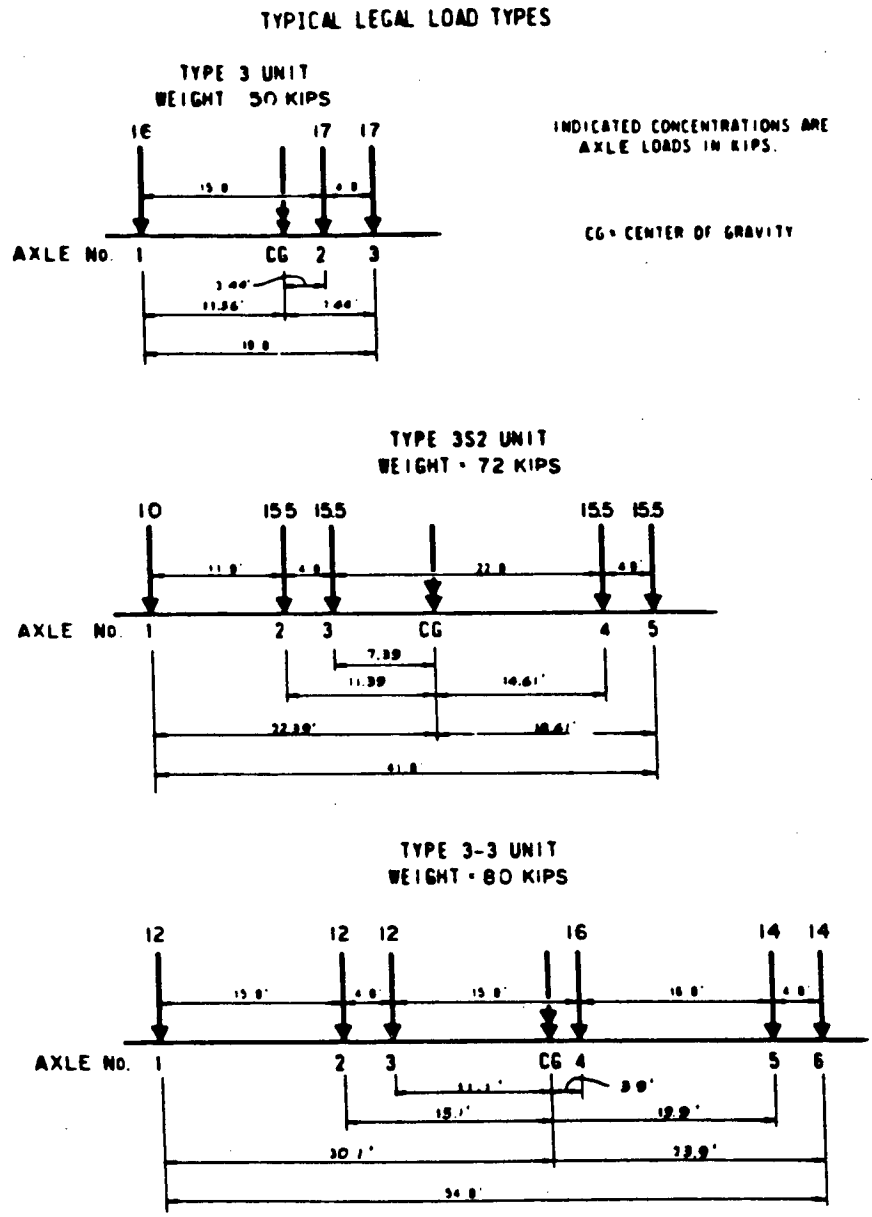


Figure 24. AASHTO rating vehicles.

ranges. The AASHTO vehicles are thus retained as the nominal loading because of the aforementioned factors.

Different load factors are used to try and realize constant reliability for each of the truck traffic cases presented in Table 4. For example, the selection of γ_L for each case is illustrated for safety indices of 2.5 to produce the same uniform beta value over all spans. Thus, sites with the heavier truck traffic will require a higher load factor. These different load factors are given in Table 14 for target safety indices of 2.5 and range between 1.4 and 1.95. Figure 26 shows a plot of beta for each of the four traffic cases for the 2.5 target and illustrates the relative uniformity of β for each category and over each span length.

The load factors in Table 14 have been derived based on target safety indices implicit in WSR-O ratings. Similarly target safety indices can be established using other criteria, i.e., WSR-I, LFR-I, and LFR-O, by using average beta values from Figures 22 and 23. Once the target safety index is established, the selection of the load factors follows essentially the same steps as above. Maintaining the same ϕ and γ_D values gives the results for both the target safety indices and the corresponding load factors, as shown in Table 15, for the four different traffic categories. The recommended load factors in the Appendix A guidelines are based on a WSR-O calibration because these represent the most prevalent state practices.

As an alternative, the target safety index using LFR-O prac-

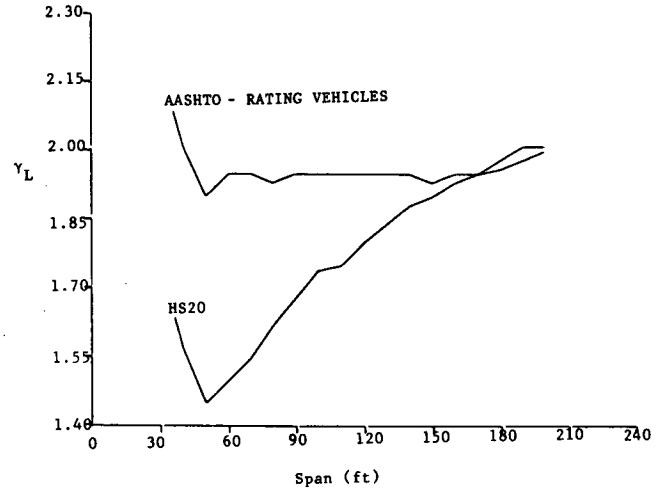


Figure 25. Live load factors required to achieve a target safety index of 2.5 (unenforced, heavy traffic), excluding proposed lane load.

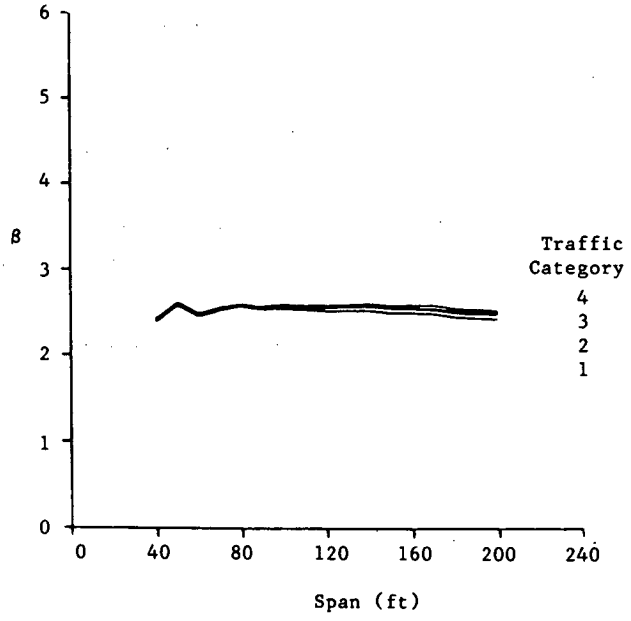


Figure 26. β for different spans with proposed load factors (average β = 2.5).

tices is obtained as 2.3 from Figure 23(a) with the corresponding load factors given in Table 15. These range from 1.35 to 1.90 or about 5 percent below the values derived from implicit WSR-O safety indices. A target safety index for inventory practices (WSR-I and LFR-I) can be extracted from Figure 22 for the nonredundant spans. Different target safety indices for WSR-I and LFR-I can be chosen. At this point, however, a single target safety index of 3.5 was adopted, which is also consistent with the value adopted by the *Ontario Highway Bridge Division Code* (OHBDC) (*3*). Using this target, the live load factors can again

Table 14. Proposed load factors for a target beta of 2.5.

ROADWAY CATEGORY	LIVE LOAD FACTORS γ_L TARGET BETA = 2.5
1) Low volume roadways, reasonable control of overloads at the source, reasonably vigorous enforcement	1.4
2) Moderate truck traffic, limited sources of overloads, occasional enforcement	1.6
3) Low volume roadway, significant sources of overweight trucks, no apparent enforcement	1.8
4) Weigh limits difficult to enforce, moderate to high ADTT, many likely sources of overloads	1.95

Table 15. Target safety indices and live load factors for different rating criteria (ϕ = 0.95, γ_D = 1.2).

RATING CRITERIA	TARGET SAFETY INDEX	LIVE LOAD FACTORS Category of Traffic			
		1	2	3	4
WSR - O	2.5	1.40	1.60	1.80	1.95
WSR - I	3.5	1.75	2.00	2.30	2.50
LFR - O	2.3	1.35	1.55	1.75	1.90
LFR - I	3.5	1.75	2.00	2.30	2.50

be determined and are shown in Table 15. In the actual format the effect of nonredundant spans will be included in the ϕ values rather than adjusting the live load factor.

STATE PRACTICES THAT MAY INFLUENCE RATING

In order to ensure the generality of these provisions, a number of state practices were uncovered which could affect the outcome of the rating and the appropriateness of the evaluation provisions. These factors include: (1) state criteria for posting, (2) distribution factors, and (3) load models.

Rather than initiate a new questionnaire, the project has relied on existing surveys—in particular, the work done by Imbsen (*6*). These surveys showed that state practices in posting showed marked differences. For example, states used, for apparently identical situations, rating stress criteria that ranged from WSR inventory levels to LFR operating levels. Similarly, most states used the existing AASHTO *S/D* factor, but some practices included equal distribution of load or the allowance of alternate "rational" analysis or use of measurements. Similarly, load models included AASHTO HS and the rating vehicles in the

Maintenance Inspection Manual but also other vehicles including overweight permits or special "grandfather" legal trucks. Rather than quantify statistically the results of a survey the intent here is to quantify the range of utilization and the implication of substituting the new provisions. In particular, if a state is "satisfied" with its present policies and operations, what steps must be followed for it to incorporate the new procedures and obtain consistent results. Similarly, if a state wants to reexamine the implications of its policies (e.g., is it more conservative than other states?) how does it carry out such comparisons.

Figure 27 compares existing deterministic evaluations using four possible criteria that encompass the most commonly used ratings, namely, WSR—Inventory, WSR—Operating, LFR—Inventory, and LFR—Operating. The parameters that must be prescribed for the comparison are the design vehicle model, distribution factor, and dead-to-live load ratio. Figure 27 assumes the HS-20 design model, AASHTO distribution (same for all cases so this is not a variable), and the assumed dead-to-live load ratios in Eq. 23. Rating is first done with the HS-20 model and the criteria shown including WSR-I, WSR-O, LFR-I, and LFR-O. For the LFR, only the elastic section modulus was used in the calculation, so the curves strictly apply to a noncompact steel section. It can be seen from Figure 27 that there is a large range between the four methods of rating. The reference value for introducing nominal strength in the figure is WSR-I, i.e., WSR-I = 1.0. Thus, for a 60-ft span the operating level rating is some 75 percent above the inventory. The major differences are between operating and inventory stress levels and *not* between WSR and LFR. Thus, the debate in some agencies about whether to adopt WSR or LFR is not as important as the stress or safety margin criteria.

Figure 28 shows the same comparison except that the three present AASHTO rating vehicles are used for evaluation rather than the HS-20 (with WSR-I (HS-20) = 1.0). Again, the rating factors, *R.F.*, are seen to be nonuniform among the four cases with different spans. It is important to keep in mind that these comparisons are totally deterministic in nature and do not reflect safety indices.

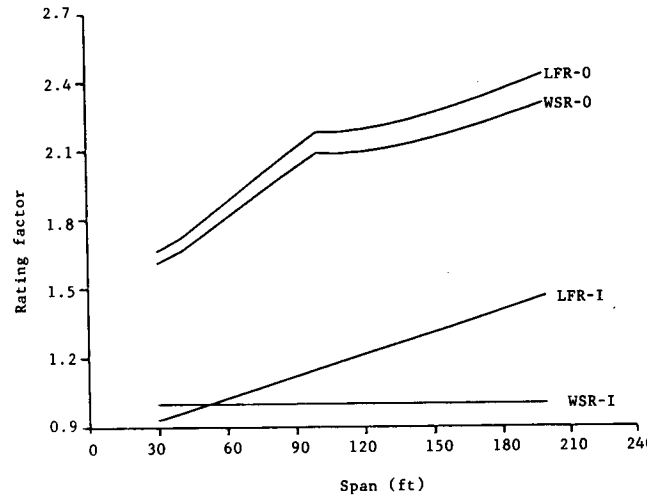


Figure 27. Rating factors with HS-20 (WSR-I = 1.0).

Figure 29 compares the influence of dead-to-live load ratio. Curve 1 is the base case (WSR-I, D/L from Eq. 23), while dead load is increased 25 percent in curve 2 and reduced 25 percent in curve 3. The same trends are observed as indicated above.

The influence of changing the distribution factor also depends on the dead-to-live load ratio. Figure 30 shows three cases including WSR-O (using an AASHTO distribution factor of 0.76) and two examples including reported measured distribution factors assumed as 0.50 (see Chapter Three) and an equal distribution factor taken as 0.33 (six-girder bridge and two loaded lanes). The influences are similar except that, again, uniform changes are not obtained in rating factors with these assumptions.

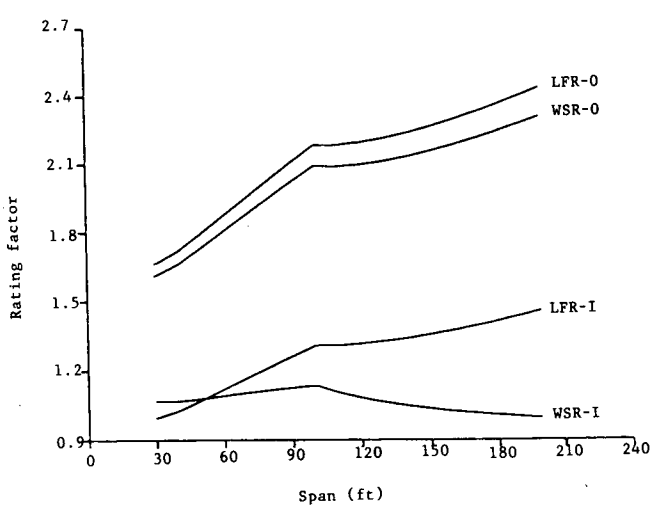


Figure 28. Rating factors with AASHTO vehicles.

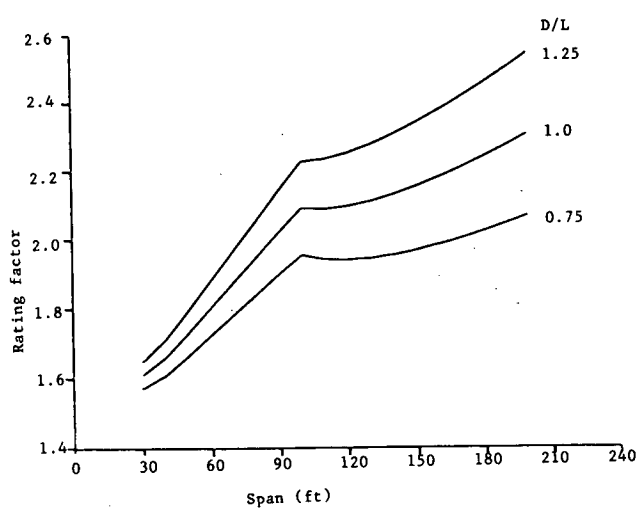


Figure 29. Rating factors with different D/L ratios.

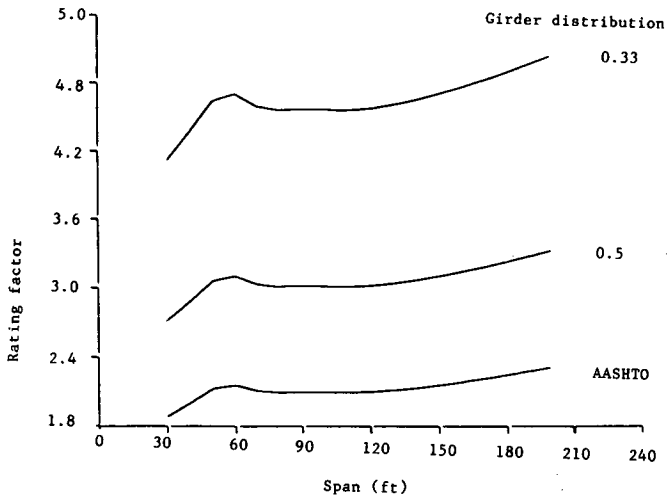


Figure 30. Rating factors for different girder distributions.

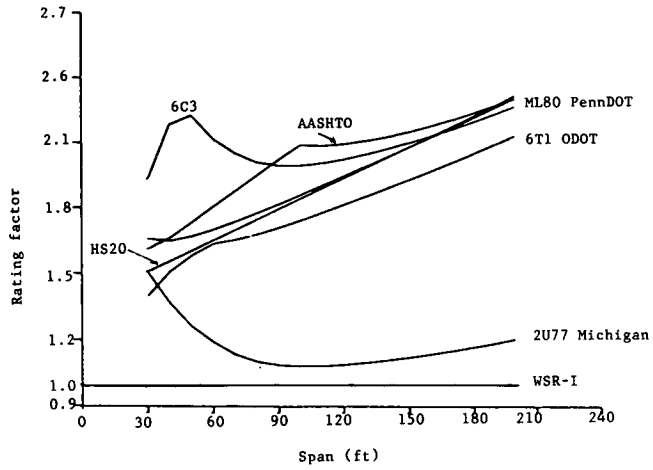


Figure 31. Rating factor for different rating vehicles.

Figure 31 compares rating factors (based on operating checks) obtained with HS-20, AASHTO rating, and a number of other vehicles including those from Ohio, Michigan, Pennsylvania, and California DOT's. Again, it is apparent that there are no consistent differences in the rating factors but that the range of *R.F.* values obtained varies a great deal from state to state. The rating factors for the vehicle 2U77 are much lower because of its considerably higher nominal load effect.

DEVELOPMENT OF LANE LOADS

It was seen in Chapter Three, section under the heading "Live Load Model" (Figure 11), that increases in the maximum load effect for spans greater than 200 ft can be considerable because of the possibility of following and preceding vehicles. The magnitude of this increase has also been determined previously. A similar conclusion can be obtained for one-lane bridges. The increase in projected moment for both one-lane and two-lane bridges is shown in Figures 32 and 33.

The final result is that for longer span bridges, there is a reduction in the safety indices if the proposed load factors are used without modification. Two options are available at this stage. One would be to have different load factors depending on the span of the bridge. This is not desirable because it would make the rating procedure too cumbersome. The other option is to develop a lane load that governs only for longer spans or where the increase in maximum load effect is significant. Thus, the aim is to maintain the same live load factor but to increase the nominal live load effect. Instead of introducing an AASHTO-type lane load, a uniformly distributed load plus some percent of a rating vehicle is preferred. The development of this lane load then is basically a trial and error procedure that is meant to produce uniform reliability over long spans (up to 300 ft). This arrangement of vehicle and lane loading model is similar to the Ontario design code except for the different values developed herein.

Figure 34 shows the safety indices for two lanes using the proposed procedure without the lane load. Figure 35 shows the

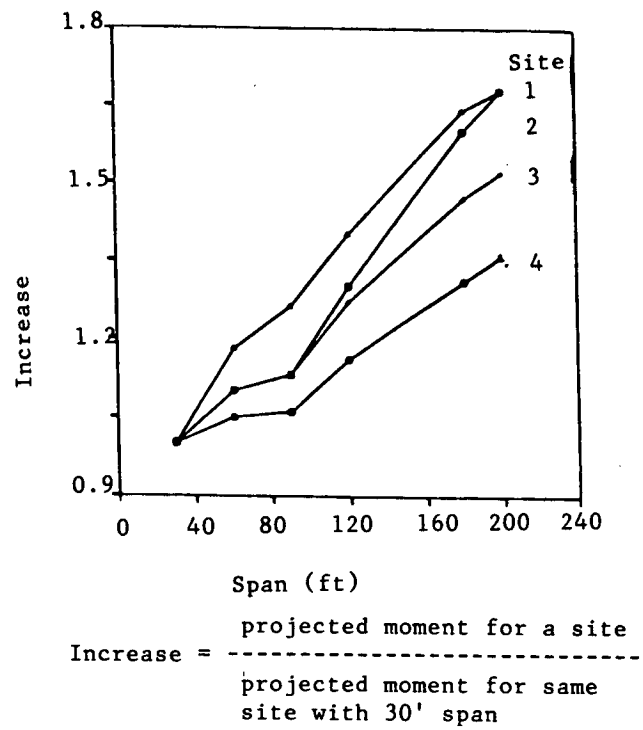
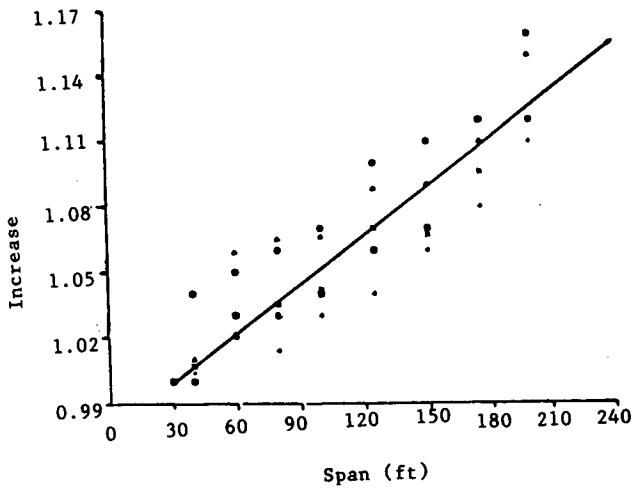


Figure 32. Increase in moment with span for one-lane bridges (sites from Ref. 4).

effect of a lane load consisting of a 0.20 kip/ft uniformly distributed plus 75 percent of the effect of the rating vehicle 3-3. This figure shows that the drop in safety indices for longer spans is compensated by the introduction of a lane load.

Figure 36 shows the safety indices using the proposed procedure (i.e., for those sections having a rating factor equal to 1.0) for one-lane traffic without using a lane load. Figure 37



$$\text{Increase} = \frac{\text{projected moment for a site}}{\text{projected moment for same site with 30' span}}$$

Figure 33. Increase in moment with span for two-lane traffic.

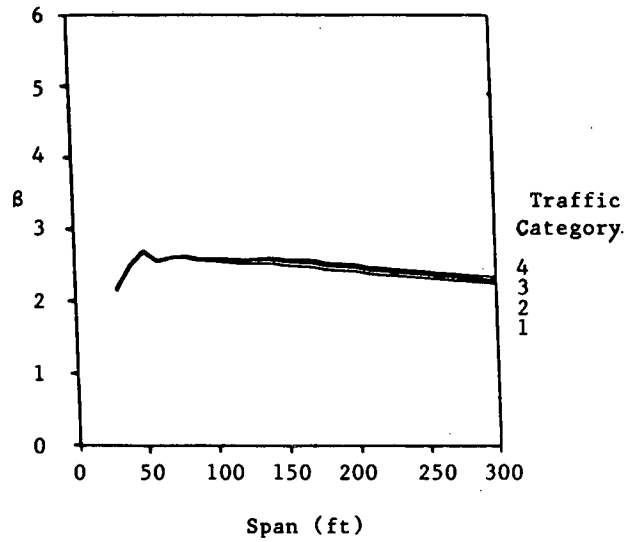


Figure 34. β for proposed live load factors without lane loads.

shows the effect of the same lane load of 0.20 kip/ft plus 75 percent of the rating vehicle 3-3. The lane load is again seen to compensate for the reduction in β's for longer spans. The live load factors are slightly higher in the shorter span range. Lower nominal loads or lower live load factors could have been chosen to bring the safety indices closer to the target for this span range, however, at the expense of complicating the rating strategy. So, such an alternative was not pursued. Thus, the above-derived lane load is seen to provide the required safety level reasonably uniformly and, hence, should be adopted.

Posted Bridges

For any bridge having a rating factor (R.F.) less than one (and hence posted), the live load effect will presumably be reduced. Because there are no data yet available on compliance with posting signs, a linear corresponding decrease of projected live load effect is assumed with decreasing values of R.F.

To note the effects on safety indices, first the safety indices are determined for generic bridges having different values of rating factors, using the proposed procedure (*assuming no de-*

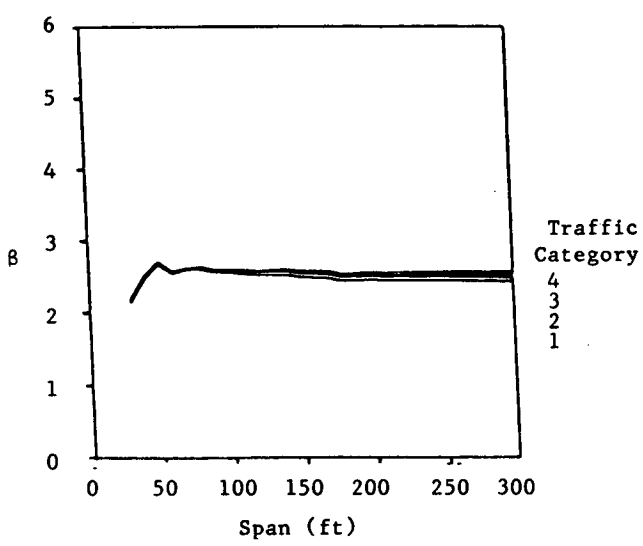


Figure 35. β for proposed live load factors with a lane load of 0.2 kip/ft and 75 percent of rating vehicle 3-3.

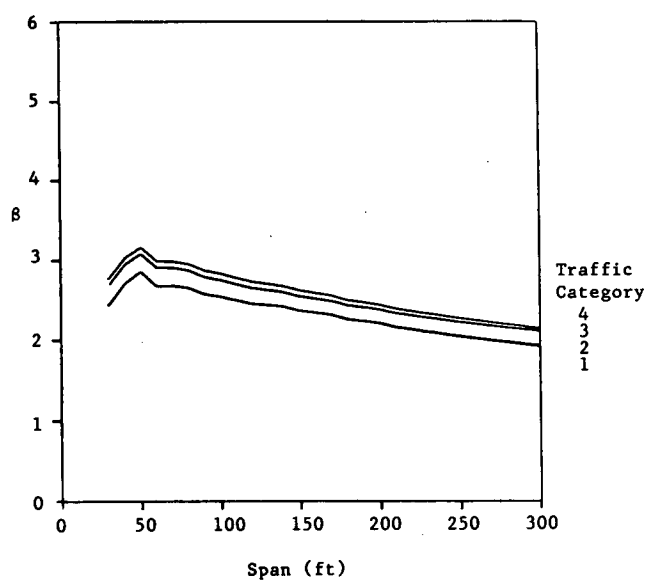


Figure 36. β using proposed live load factors for two lanes.

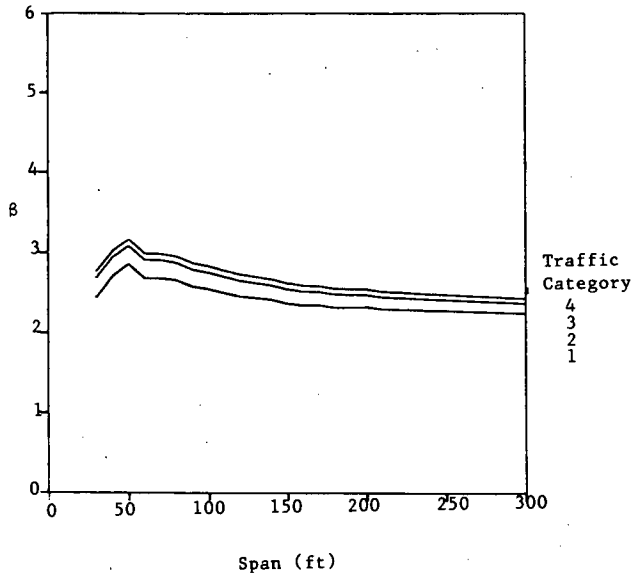


Figure 37. β for proposed live load factors with a lane load of 0.2 kip/ft and 75 percent of rating vehicle 3-3.

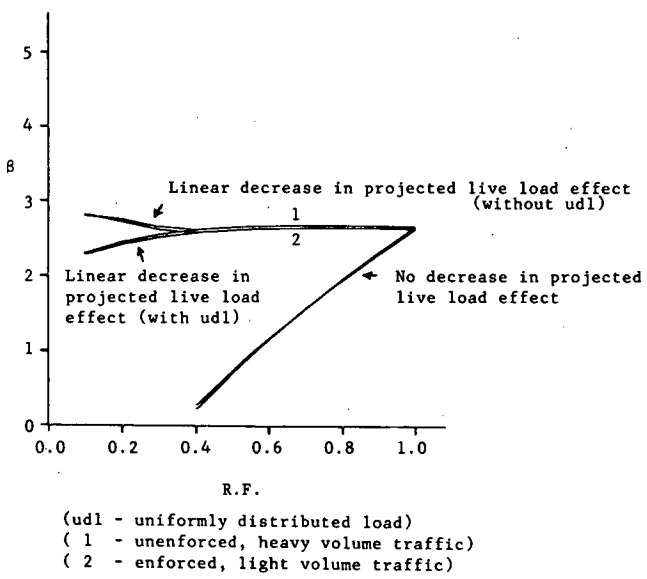


Figure 38. β for posted bridges.

crease in the projected live load effect). The results are plotted in Figure 38 for the two traffic categories (unenforced, heavy volume and enforced, light volume traffic) using a 100-ft span bridge as an illustration. The betas are seen to drop dramatically, which indicates the effect of unrestricted traffic on posted bridges. Now, using the above assumption of a linear decrease of projected live load effect with decreasing values of R.F., the safety indices can be calculated again for different values of R.F. These are also shown in Figure 38. This figure shows that if the live load effects decrease linearly, a safety index of 2.5 is still maintained for rating factors below 1.0. However, for low rating factors (below 0.4), the safety indices do rise above 2.5 because of the relatively higher ratio of dead to live loading. To provide a uniform safety index of 2.5, different load factors can be provided for bridges having low ratings and the previously developed factors can be used for all other cases. Another way of achieving the same result (for low *R.F.*) is to use the lane load together with the basic set of load factors developed before. Such a procedure avoids introducing too many new load factors and makes the rating strategy less cumbersome. Using the previously developed lane load of 0.20 kip/ft and 75 percent of the governing rating vehicle (3-3 for a 100-ft span), the safety indices are again determined for different rating factors and are shown in Figure 38. Use of such a lane load is then seen to provide the requisite uniformity in betas at the same time maintaining the simplicity and ease of application of the proposed specification. It should be noted, however, that this conclusion is dependent on the validity of the assumption that the projected live load effect decreases linearly with decreasing values of *R.F.*

CORRECTION FACTORS FOR IMPROVED ANALYSIS METHODS

Both analytical studies and field measurements have demonstrated that AASHTO distribution factors are conservative for steel members. This conservatism may have been deliberate because the existence of such a safety factor is not evident and obvious.

Therefore, bridge safety is adversely affected if any analysis method is used which reduces conservatism (or bias). Because options for conducting either a better analysis or doing field observations have been provided in the proposed rating strategy, certain correction factors must be developed to maintain the same safety level. Values of bias and COV for such methods have been determined in Chapter Three, under the section heading "Live Load Model." For each of these methods, the load factors required to reach the target safety level are determined. The correction factors are then equal to the ratio of these load factors to the proposed load factors. Because the bias present in AASHTO distribution factors for steel and prestressed concrete members is different, the correction factors for these types are not the same. The values of these correction factors are given in Table 16.

Table 16. Correction factors for analysis methods.

Method of analysis	Correction factor to 'g'	
	Steel	Prestressed concrete
(1) AASHTO Distribution	1.00	1.00
(2) Conventional analysis with simplifying assumptions	1.10	1.05
(3) Sophisticated analysis e.g finite elements, grillage analogy	1.07	1.03
(4) Field Measurements	1.03	1.01

REDUCTION FACTORS FOR BRIDGES WITH MORE THAN TWO LANES

Appendix D presents calculations for the projected load effect for three-lane and four-lane bridges. Using these values, a new set of load factors can be determined following exactly the same procedure as for two-lane bridges. These values are given in Table 17. The load factors required are lower due to the decrease in the maximum load effect divided by the number of lanes because of the reduced probability of three or four heavy side-by-side occurrences. Because the base case load factors prescribed are for two-lane loads, reduction factors for three and four lane loading have to be provided as shown in Table 18.

Also shown for comparison are reduction factors presented in current AASHTO and OHBDC codes. The reduction factors developed are quite comparable. However, it should be noted that such a comparison is not exactly correct because Ontario girder distributions are dependent on the number of lanes, whereas AASHTO (and the proposed) are independent of the number of lanes (for two or more lanes).

Table 17. Live load factors for three and four lanes.

Traffic Category	Live Load Factor	
	Three lane	Four lanes
1. Enforced, light volume	1.15	1.00
2. Enforced, heavy volume	1.30	1.00
3. Unenforced, light volume	1.40	1.15
4. Unenforced, heavy volume	1.55	1.45

Table 18. Reduction factors for multiple lane loadings.

Number of lanes	Reduction factors		
	Proposed	AASHTO	OHBDC
2	1.0	1.00	0.9
3	0.8	0.90	0.8
4	0.7	0.75	0.7

OPTIONS FOR DETERIORATED SECTIONS

Statistics for deteriorated steel sections have been established in Appendix H. Using those statistics and initial starting values from section under the heading "Selection of Resistance and Dead Load Factors," resistance factors for deteriorated steel sections are established below to achieve target indices:

Condition	Capacity Reduction Factor, ϕ (steel members)
1. Good condition	0.95
2. Slight corrosion, some section loss	0.85
3. Severe corrosion, considerable section loss	0.75

Repair and evaluation of damaged prestressed concrete girders has been the subject of an extensive study in NCHRP Project 12-21. However, it dealt mainly with accidental damage. The typical causes of damage are (*29*):

1. *Overheight vehicles*—Vertical clearance may be insufficient either because of illegal height of the truck, reduced clearance (e.g., an old bridge), or because of dynamic forces inducing vertical motion of the truck and cargo.
2. *Corrosion of strands*—The direct cause is insufficient cover either because of sloppy construction and/or a gradual reduction of the concrete wearing surface. The problem is particularly critical in the case of post-tensioned girders.
3. *Corrosion at joints*—Joints often leak causing serious problems, such as deterioration of concrete and corrosion of strands. Epoxy injections are used to repair the damaged concrete.
4. *Fire*—Fire under the bridge (by vandalism, explosion—due to accidents, etc.) causes deterioration of concrete, loss of concrete cover for strands, and corrosion of strands and may lead to replacement of the girder(s).
5. *Sloppy construction*—Prestressed girders are often exposed to critical stress conditions during construction or transportation. Inadequate understanding of structural behavior may lead to damage of the girder.

Overheight vehicles account for about 80 percent of the damage, and fire causes about 2.5 percent of the damage (*30*). Of the reported damage (*30*), minor and moderate damage account for 80 percent of the damages. Minor and moderate damage may involve loss of concrete up to the extent of exposing the prestressing strands and reinforcing bars. Severe and critical damage is due to major losses of concrete or damage to the reinforcing elements.

The above-cited study (*30*) indicates that load capacity is by far the most important rationale for selection of repair method (i.e., whether to repair-in-place or replace). A survey of the practices of various Departments of Transportation showed that inspection of damage ranges from visual means with varying degree of inspection detail, to structural analysis of the damaged girder compared to the original design. Serious damage is invariably inspected by licensed structural engineers. Most damage reports are in letter form and include damage description, severity of damage, repair recommendations, and estimated cost of repair.

The above discussion shows that damage assessment and repair of prestressed concrete girders is a reasonably well-documented procedure. However, for purposes of this study one has to determine the statistics of the strength of the damaged girders that are not repaired. Unfortunately, practically no such data are available.

Because inspection of prestressed concrete girders is invariably detailed in nature (specially for severely damaged members), it is postulated that the resistance factors for carefully inspected and deteriorated steel members will also apply in this case. Such a stipulation is necessary because of the uncertainties that may still be present even upon detailed inspection (refer to Appendix A). In the event of a repair, the resistance factor for good condition members should apply.

OPTION FOR NONREDUNDANCY

For reasons mentioned both earlier in this chapter under the

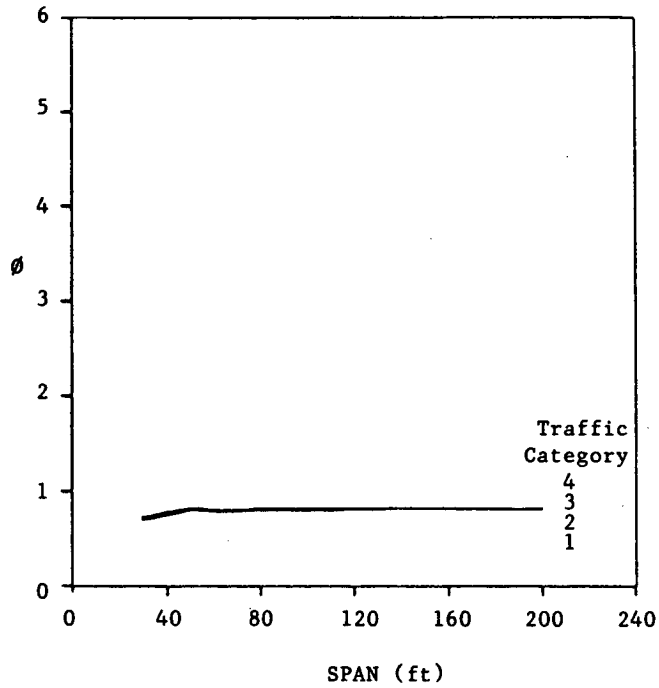


Figure 39. ϕ for nonredundant elements.

section heading, "Selection of Target Reliability," and in Appendix E, nonredundant elements are associated with a higher target safety of 3.5. To determine an appropriate capacity reduction factor a recalibration has to be done except that now the load factor is kept fixed for each traffic category (at the value determined under the section heading "Selection of Live Load Factors") and iterate on ϕ to achieve a uniform safety index of 3.5. Figure 39 shows the values of ϕ required to achieve a target safety index of 3.5 for all four traffic categories. They are seen to be quite uniform with an average value of 0.80, which will be chosen as the capacity reduction factor for nonredundant elements.

OPTIONS FOR IMPROVED INSPECTION AND MAINTENANCE

To make the evaluation strategy more flexible, options have to be provided for different maintenance and inspection. These options reward the rating engineer with higher ratings if more effort is put into the evaluation. At present, such a clear and explicit strategy is lacking in the AASHTO specifications (*6*). The advantage of such a procedure is that it allows the rating engineer to decide how much man-power, time, and other resources to expend to obtain a rating, at the same time maintaining an acceptable level of safety.

Because inspection and maintenance directly affect the component resistance, such options are provided by modifying the capacity reduction factor. The rationale is that increased inspection would reduce the uncertainties, thereby requiring a lower factor of safety which implies a higher capacity reduction factor. Therefore, levels of inspection and maintenance are assumed to affect only the COV of the resistance.

The effect of improved maintenance on the resistance of a component can be obtained by examining bridges under similar conditions, but different maintenance levels, over a period of time. Unfortunately, such data are just not available. The difficulty is compounded by the fact that the definition of level of maintenance is subjective to a great extent.

Similarly, it is assumed that under deteriorated condition, increased inspection effort would lead to a higher certainty in the amount of section loss, which implies that the penalties imposed for deteriorated sections can be reduced. This can be accomplished by providing correction factors to the capacity reduction factor. Again, backup data for such reductions are not available. Because of the absence of such data, the following values are suggested, similar to values contained in the NCHRP Project 10-15 study (*NCHRP Report 292*).

	Correction Factor to ϕ
Careful inspection	+0.05
Vigorous maintenance	+0.05
Intermittent maintenance	−0.05

Equation 29 is used to examine the implications of these values in terms of the coefficient of variation of the resistance. The results are given in Table 19. As expected, the values of COV increase with the amount of corrosion and decrease with increasing effort of inspection and maintenance. The tabulated coefficients of variation are seen to be reasonable and, hence, the correction factors to ϕ will be retained.

It should be noted here that maintenance and inspection are undoubtedly intertwined; however, they have been categorically separated for the purposes of this study. Maintenance will primarily refer to the efficacy of the preventive strategies employed, whereas inspection involves the accuracy and detail of the inspection procedure. For further elaboration and methods of choosing the appropriate categories, the reader is referred to Appendix I.

CALIBRATION FOR CONTINUOUS SPANS

Load effects from simulations for continuous two-span and three-span bridges have been established in Appendix D. For these cases, however, the safety indices have to be calculated for all the checkpoints (i.e., at the interior supports, at the center of the longer span for two-span bridges, and at 0.4 of the exterior span length for three-span bridges). Once the data base has been established, the methodology is essentially the same as for simple span structures.

Table 19. Effect of inspection and maintenance levels on COV of resistance.

Condition of Section	Base COV	COV for detailed inspection	COV for maintenance	
			Vigorous	Intermittent
Good Condition	12%	N.A.	N.A.	15%
Mild deterioration	16%	12%	12%	20%
Severe deterioration	20%	16%	16%	25%

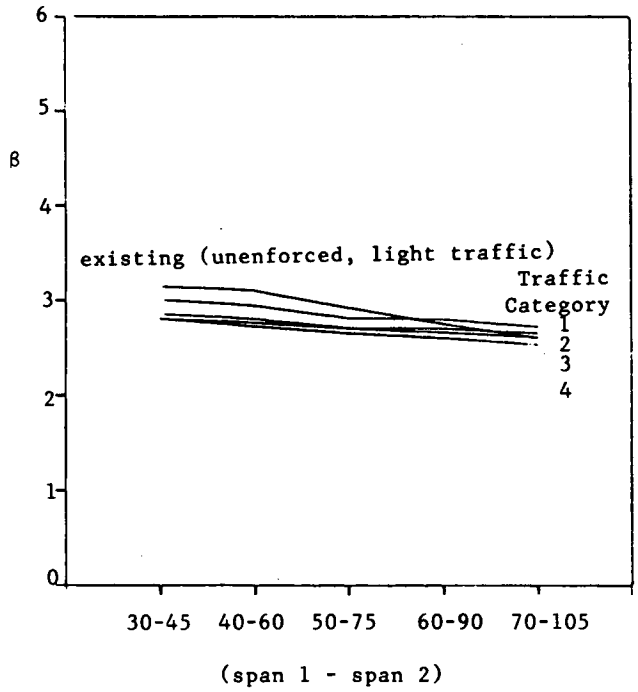


Figure 40. β for two-span continuous bridges (center of longer span).

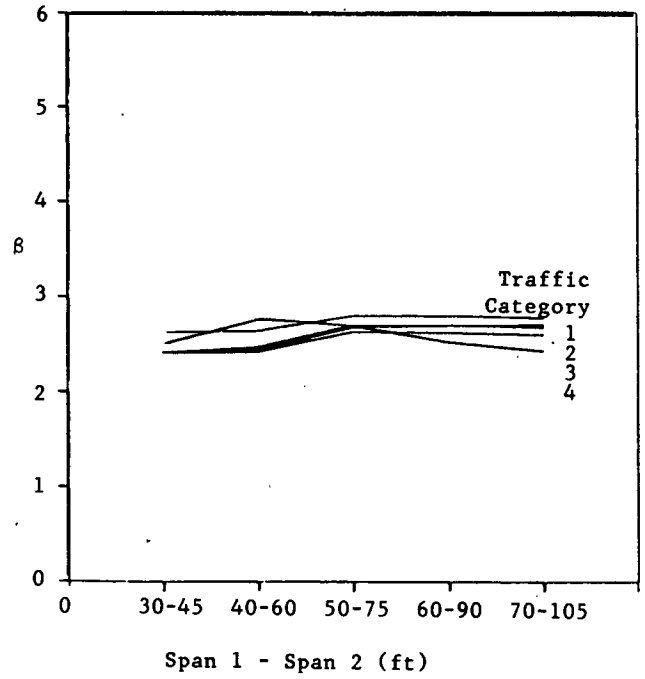


Figure 41. β for two-span continuous bridges (interior support).

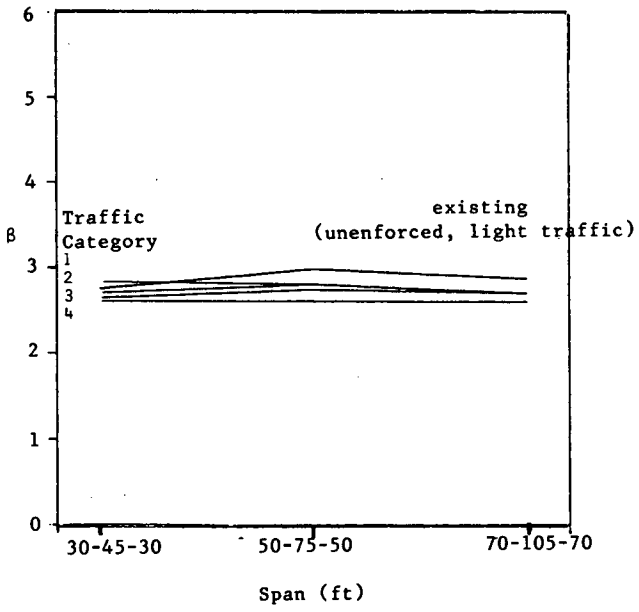


Figure 42. β for three-span continuous bridges (@ 0.4 of outer span).

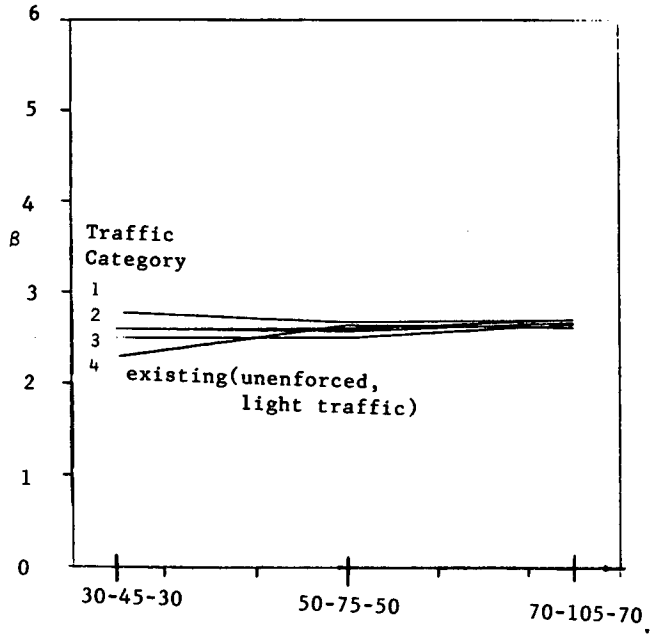


Figure 43. β for three-span continuous bridges (first interior support).

Figures 40 and 41 show the results of such a calibration for two-span bridges. It is seen that with the prescribed load factors, the safety indices are slightly higher and more uniform than existing ones. Figure 41, however, is not as completely uniform as Figure 40, but it is still more uniform than safety indices by existing (WSR-O) methods for unenforced, light traffic. Hence, the proposed load and resistance factors are still satisfactory. Similarly, Figures 42 through 44 show the safety indices for three-span continuous bridges. Again, the load and resistance factors are seen to be satisfactory.

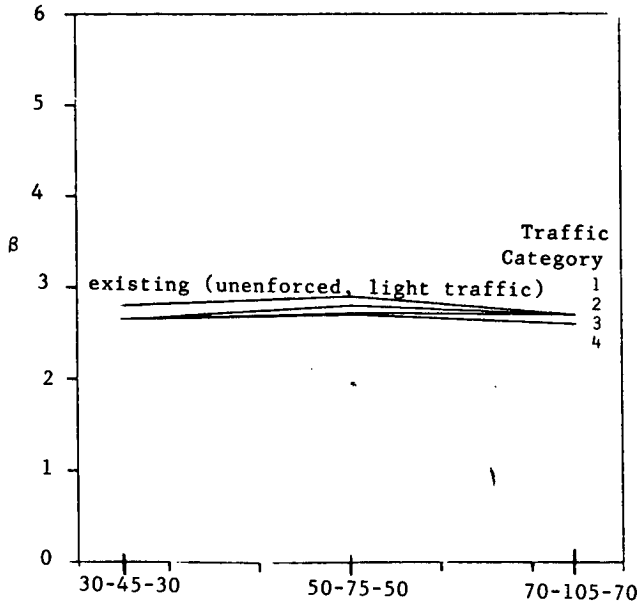


Figure 44. β for three-span continuous bridges (midpoint of center span).

BRIDGE DATA BASE

As part of this project the researchers examined a number of data bases that describe the inventory of existing bridges. There are several reasons for exploring such data bases and a variety of types of information that can be extracted. These are used for establishing the distribution of the sample bridge population that must be calibrated by "tuning" the load and resistance factors to achieve, on the average, the target reliabilities. Further, it provides specific examples that may be used to perform detailed investigation, to illustrate the proposed methods, and to study the range of actual rating factor values of existing structures. Among the features specifically considered in the data are the following points:

1. *Dead-to-live load ratios*—Load factor design and LRFD both provide for a higher live load and smaller dead load factor than the corresponding WSD. The overall balance is affected by the ratio of dead to live load effects (moment, shear, etc.). Equation 28 was used in the original LFR formulation and gives D/L as a function of span, but the actual range of D/L for a variety of cases is checked in the illustrative examples. In addition, if bridges have very high D/L, it is important that the dead load factor be controlled so the overall reliability for this case does not become too small.

2. *Span lengths and geometries*—In a level I LRFD format, there are inevitable compromises to reach the target betas. It would be preferred to "weight" each parameter so that any deviations from the target would occur for as few cases as possible. Inspecting the inventory provides specific examples of span ratios that should be investigated by comparing the effect of different influence diagrams on the distribution of maximum moment and also the moment produced by the proposed rating vehicles.

3. *Data review*—A data review provides information on rating and posting practices, especially with regard to how states use and interpret operating and inventory models.

4. *Specific bridge examples*—In addition to general data, specific examples have to be selected. This will determine the amount of data available and how the rating of these existing bridges will be affected by the proposed rating procedures. (Refer to Appendix C.)

EFFECT OF PROPOSED RATING

The previous section mentions that specific bridge examples must be used to check the validity of the proposed evaluation strategy and also compare it with existing ratings. This is done in Appendix C. However, a generic study can also be conducted to obtain such results.

To start, WSR-O will be compared with the proposed evaluation for good condition steel members. Using WSR-O, 0.75 $R = D + (R.F.)(L)$, where L includes impact also.

Rearranging gives $R = (1/0.75)/(D + (R.F.)(L))$; or

$$R = \frac{4}{3} D\,(1 + \alpha), \text{ if } R.F. = 1 \text{ and } \alpha = \frac{L}{D} \tag{33}$$

For the proposed procedure, $\phi\ R = \gamma_D D + \gamma_L(R.F.)\ L'$, where L′ is not the same as L because of differences in impact values and nominal load values (HS-20 compared to AASHTO vehicles). Also, R is now based on the plastic modulus which is typically 13 percent higher than the elastic section modulus. Therefore, $(R.F.)_{LRFD} = (\phi(1.13)R - 1.2D)/(\gamma_L(L'))$.

Substituting for R from WSR-O or Eq. 33, $(R.F.)_{LRFD} = (1.507\ \phi\ (1 + \alpha) - 1.2)/(\gamma_L(\alpha)\ (I_{LRFD}/I_{WSD}))$.

For a typical bridge having a span of, say, 75 ft, and having a smooth roadway surface, the proposed specifications give for the impact an I_{LRFD} of 1.1, whereas AASHTO specifications (*2*) give an I_{WSD} of 1.25.

Therefore, $I_{LRFD}/I_{WSD} \approx 1.10/1.25 \approx 0.88$. Thus, the above equation reduces to

$$R.F. = \frac{1.627\ (1 + \alpha) - 1.36}{\gamma_L \alpha} \tag{34}$$

Equation 34 compares ratings of spans having a rating factor of 1.0 by existing operating procedures using AASHTO legal vehicles to ratings by the proposed procedures. Results are given in Table 20 for different values of α (live load/dead load ratio). This table shows that for high values of α (short spans) and heavy traffic, the proposed ratings can fall below 1.0. On the other hand, for enforced traffic categories (light and heavy volume traffic), the proposed ratings are higher than existing ratings. This should be no surprise inasmuch as the AASHTO *Manual for Maintenance Inspection of Bridges* does suggest using higher load factors for heavy traffic.

A similar procedure for comparing LFR-O with the proposed procedure, denoted as $(R.F.)_{LRFD}$ gives

$$(R.F.)_{LRFD} = \frac{1.403(1 + \alpha) - 1.364}{(\gamma_L)\ (\alpha)} \tag{35}$$

However, now the extra 13 percent increase in plastic section

Table 20. Generic study of existing WSR (operating) ratings and proposed ratings.

Proposed R.F.

	Traffic Category				Approximate
	1	2	3	4	Span
α γ_L	1.4	1.6	1.8	1.95	(ft)
0.5	1.50	1.30	1.20	1.10	150
1.0	1.40	1.20	1.10	0.95	75
1.5	1.29	1.13	1.00	0.92	50
2	1.26	1.10	0.98	0.90	40
∞	1.16	1.02	0.90	0.83	-

$\alpha = \frac{L}{D}$

R.F. for WSR - 0 (AASHTO rating vehicles) = 1.0

Impact (by proposed specifications) = 1.1

Section: Plastic (by proposed specifications)

Condition of section: Good

Table 21. Generic study of existing LFR (operating) ratings and proposed ratings.

Proposed R.F.

	Traffic Category				Approximate
	1	2	3	4	Span
α γ_L	1.4	1.6	1.8	1.95	(ft)
0.5	1.06	0.93	0.82	0.76	150
1.0	1.03	0.90	0.80	0.74	75
1.5	1.02	0.89	0.79	0.73	50
∞	1.00	0.87	0.78	0.72	-

$\alpha = \frac{L}{D}$

R.F. for LFR - 0 (AASHTO rating vehicles) = 1.0

Impact (by proposed specifications) = 1.1

Section : Plastic

Condition of section: Good

cannot be used because it applies to both LFR and the proposed procedure. Table 21 shows the proposed rating factors as a function of load ratio α and hence span. The results indicate that the proposed rating factors are consistently lower for all but the lightest traffic category. This is again as expected because one has calibrated to WSR-O and not to LFR-O which normally gives higher ratings.

For comparing the proposed procedure with existing inventory ratings, the worst possible condition is chosen of a severely deteriorated redundant steel section with worst impact values (i.e., $\phi = 0.70$ and $I = 1.3$) to obtain for existing WSR-I ratings

$$(R.F.)_{LRFD} = \frac{1.38(1 + \alpha) - 1.15}{\gamma_L \alpha} \tag{36}$$

Results are given in Table 22. Once again, for shorter spans and heavier traffic, the proposed ratings are lower.

A similar procedure for comparing LFR-I with the proposed procedure gives $(R.F.)_{LRFD} = (0.875(1 + 1.6\,\alpha) - 1.15)/\gamma_L\,\alpha$.

Results are given in Table 23. In this case, a reversal of trend is observed (i.e., rating factors decrease with increasing span). This is because of the D/L load ratios. In fact, for very long and deteriorated spans, the proposed procedure is seen to give lower ratings than load factor inventory ratings. However, for a good condition section, proposed ratings are always greater than the existing inventory level.

Figure 45 compares the proposed rating (including the lane load) using the Appendix A guidelines and existing ratings for generic spans checked by WSR-O using the HS-20 vehicle. The WSR-O rating factors all equal 1.0 in Figure 45. This figure shows that the proposed rating can give higher rating factors for all traffic categories. For the most heavy traffic category, the rating factor will almost reach the WSR operating level.

Table 22. Generic study of WSR (inventory) and proposed procedure.

Proposed R.F.

	Traffic Category				Approximate
	1	2	3	4	Span
α γ_L	1.4	1.6	1.8	1.95	(ft)
0.5	1.31	1.15	1.02	0.94	150
1.0	1.15	1.00	0.89	0.83	75
1.5	1.10	0.96	0.85	0.79	50
∞	0.99	0.96	0.85	0.79	-

$\alpha = \frac{L}{D}$

R.F. for WSR - I (AASHTO rating vehicles) = 1.0

Impact (by proposed specifications) = 1.3

Section: Plastic

Condition of section: Heavily deteriorated

Also shown are current LFR-I and WSR-I ratings, using AASHTO specifications and nominal live loads as the AASHTO legal vehicles. Figure 46 repeats the above procedure except that the generic spans now checked have WSR-I rating factor all equal 1.0. The same general conclusions can be drawn. Overall, the proposed rating factors are still very high compared to the WSR-I levels. However, part of this is due to the higher nominal load effect of the HS-20 vehicle.

The basis for the above calculations for Figures 45 and 46 has been the HS-20 vehicle (which has a higher nominal moment). To obtain a different comparison the AASHTO rating vehicles are used as the basis. Figures 47 and 48 show the effect of the proposed evaluation procedure (including the lane load)

Table 23. Generic study of LFR (inventory) and proposed procedure.

		Proposed R.F.				
		Traffic Category				Approximate
		1	2	3	4	Span
α	γ_L	1.4	1.6	1.8	1.95	(ft)
0.5		0.65	0.57	0.51	0.47	150
1.0		0.85	0.74	0.66	0.61	75
1.5		0.91	0.80	0.71	0.66	50
∞		1.09	0.91	0.81	0.75	-

$\alpha = \frac{L}{D}$

R.F. for LFR - I (AASHTO rating vehicles) = 1.0

Impact (by proposed specifications) =1.3

Section : Plastic

Condition of section: Heavily deteriorated

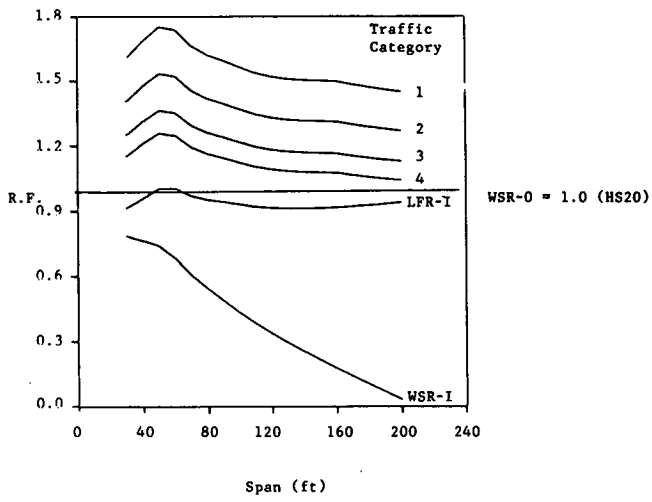


Figure 45. Comparison of proposed and existing ratings.

on good condition sections which rate at 1.0 by existing WSR-O and WSR-I methods. Both these figures show that rating factor decreases with increasing severity of traffic. However, the proposed ratings will fall below existing operating ratings for spans up to 100 ft and with a large volume of heavy traffic. These figures are then just graphical descriptions of Tables 20 and 22.

To facilitate visualizing the differences in the existing and proposed rating, Table 24 gives a comparison of the maximum simple span bending moments of HS-20, governing AASHTO rating vehicle (including the lane load) and the expected maximum moment (median) predicted by Eq. 19 for all four categories of traffic. Table 24 is illustrated graphically in Figure 49 and shows that the proposed nominal load effects (AASHTO legal loads and proposed lane loads) approximately envelop the moment effect predicted by Eq. 19.

SENSITIVITY STUDY

A major part of any reliability code calibration is a sensitivity study. Thus, changes in bias and coefficient of variation of any variable will change computed betas. Because the target beta is extracted from current performance levels, it too is affected by the changes in the data parameters. The aim of a sensitivity study is to make sure that uniform consistent betas are not affected by reasonable changes in the input statistical data. This is seen by changing the data and recomputing both the present performance beta with existing AASHTO ratings and then comparing them with the betas found using the same load and resistance factors selected with the base case data input. This sensitivity study is then essentially probabilistic in nature and is given at the end of this section. Another type of sensitivity study that is completely deterministic in nature can be performed by assuming a base case nominal resistance (by using, say, WSR-O (HS-20) = 1.0) and then varying the condition of section, impact, traffic category, etc., individually and examining the changes in *R.F.* using the proposed specifications. Both these approaches are followed in this section.

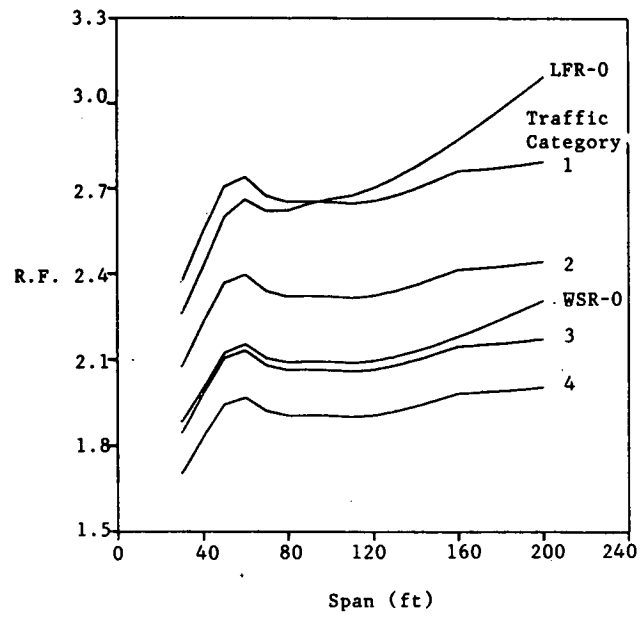


Figure 46. Comparison of proposed and existing ratings.

First, a deterministic sensitivity study on the proposed calibration was conducted to obtain an insight into the effect of varying different parameters. The base case considered is a good condition section (ϕ = 0.95), smooth road surface (I = 1.1), and AASHTO girder distribution. The nominal resistance is obtained by assuming a rating factor equal to 1.0 using WSR-O criteria with the HS-20 design vehicle.

The effect on the rating factor of varying the girder distribution analysis, impact (surface smoothness), and resistance (deterioration) is shown for traffic category (1) in Figure 50 and for traffic category (3) in Figure 51. These figures illustrate

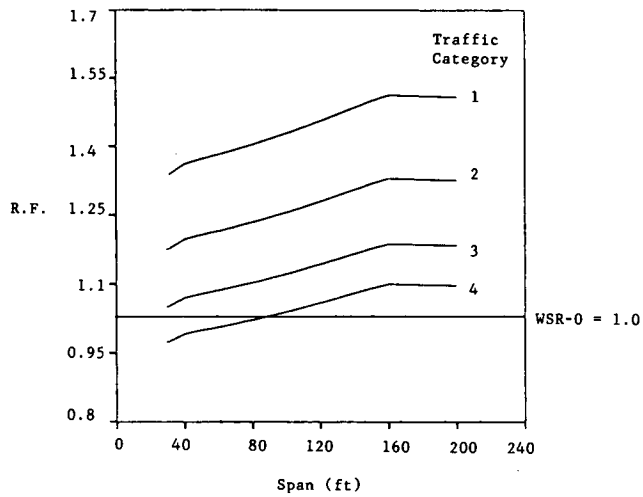


Figure 47. Comparison of existing and operating ratings.

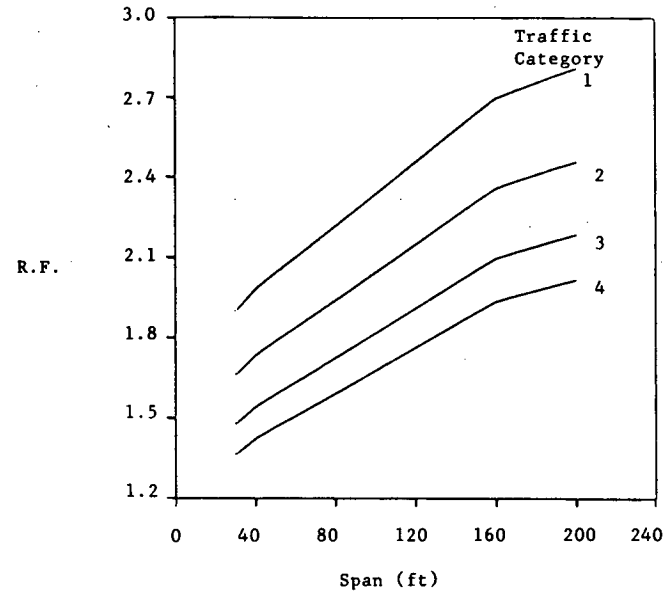


Figure 48. Comparison of existing and operating ratings.

Table 24. Median moment vs. AASHTO rating, HS-20 rating.

SPAN (FT)	HS 20 MOMENT (K-ft)	AASHTO RATING MOMENT (K-ft)	GOVERNING VEHICLE	MEDIAN MOMENT eqn. 3.3 CATEGORY OF TRAFFIC 1	2	3	4
30	282.1	223.2	1	146.0	159.0	191.0	199.0
40	449.8	346.0	1	228.2	248.1	297.6	310.1
50	627.9	471.0	1	296.9	322.7	387.2	403.5
60	806.5	612.0	2	482.2	510.6	595.5	638.4
70	985.6	784.8	2	608.4	644.0	751.5	805.8
90	1344.4	1142.0	2	915.3	969.1	1130.6	1211.9
120	1833.3	1740.0	3	1318.2	1395.7	1628.4	1744.4
140	2242.8	2140.0	3	1639.3	1735.8	2024.9	2169.7
160	2768.0*	2545.0	LANE	1987.9	2104.9	2455.6	2631.0
180	3402.1*	3015.0	LANE	2350.4	2488.6	2903.4	3110.7
200	4100.0*	3505.0	LANE	2699.1	2857.7	3333.9	3572.3

*includes HS20 lane load

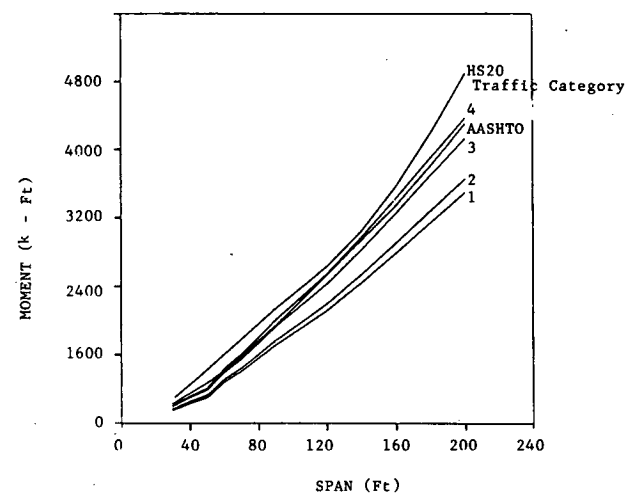


Figure 49. Comparison of moments of proposed traffic categories with HS-20 and AASHTO load models.

the importance and benefits associated with smooth roadway (low impact), improved girder analysis (e.g., finite element) and good maintenance (higher ϕ). (Note: In all the subsequent sensitivity studies, the proposed lane load is also considered.)

Figure 50 shows that with the proposed factors and traffic category (1) (enforced, light traffic) the base case *R.F.* will exceed the working stress (operating) *R.F.* for all spans. The comparison can be seen in the following example:

- Span = 60 ft.
- HS-20 design moment = 806.5 kip-ft.
- Rating vehicle moment = 612.0 kip-ft.
- AASHTO distribution factor = 0.72; Impact = 1.27.
- Girder moment, M = Design moment * g * I = 806.5 * 0.72 * 1.27 = 737.4 kip-ft.
- Nominal AASHTO vehicle moment = 612.0 * 0.72 * 1.1 = 484.7 kip-ft with a distribution factor same as AASHTO, Impact, Smooth = 1.1.
- A generic dead load effect is estimated from Eq. 28 and is given as: D = 0.0132 * 60.0 * 737.4 = 584.1 kip-ft.
- Nominal resistance, WSR-O with *R.F.* = 1.0 is:

$$R_n = \frac{D + R.F.\ (M)}{0.75} = \frac{584.1 + (1.0)\ 737.4}{0.75} = 1761.9 \text{ kip-ft}$$

- Proposed rating using good condition section, ϕ = 0.95; smooth road surface, I = 1.1; AASHTO distribution, g = 0.72; traffic category (1), γ_L = 1.4; and dead load factor, γ_D = 1.2.
- Substitution gives:

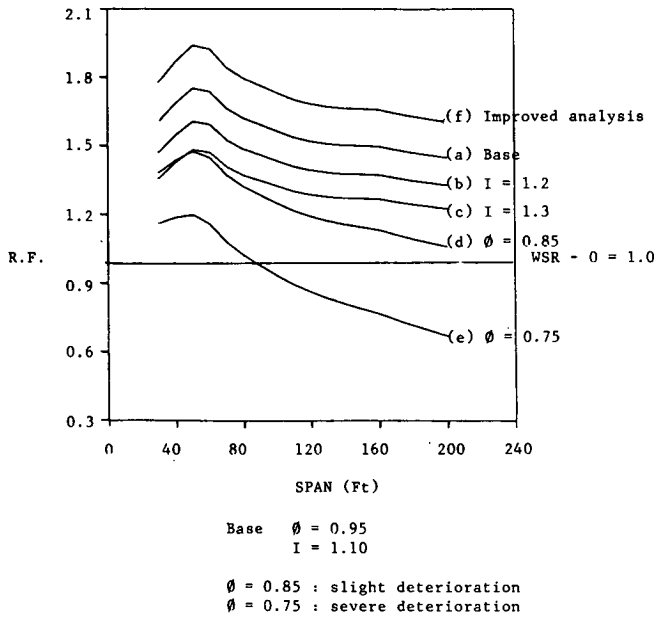


Figure 50. Sensitivity study for traffic category 1.

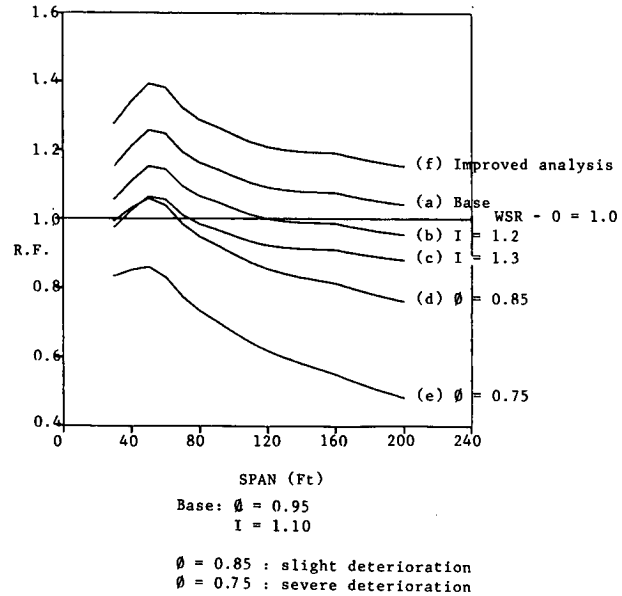


Figure 51. Sensitivity study for traffic category 3.

$$R.F. = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n}$$

$$R.F. = \frac{0.95 * 1761.9 - (1.2) * 584.1}{1.4 * 484.7} = 1.43$$

In Figure 50, only the base case uses the values in the example. It also recognizes compact sections. For the different pavement roughnesses, the mean impacts are changed, but γ_L is maintained the same. Curves (b) and (c) show that increasing pavement roughness leads to a decrease in the *R.F.* for all spans. So, a bridge that does not rate sufficiently might be made to do so by resurfacing. Further, this serves as an incentive to maintain the wearing surface.

The effect of a deteriorated section on the rating factor is more radical, as can be seen from the relatively low values of *R.F.* obtained in curves (d) and (e). The ϕ values for these deteriorated cases are given in Appendix A. They were calibrated to give a beta equal to 2.5 for the resistance data given in Chapter Three. To avoid the reduced ratings for deteriorated components based on the proposed specifications it is of advantage to have vigorous maintenance of the bridge. The effect of conducting a sophisticated analysis for load distribution is shown in (f). This will on the average lead to a lower, less conservative value of girder distribution equal to the bias in *g* for steel members. The γ_L to be used with these analyses is, however, higher. This is because of the reasons cited in Chapter Three. (Refer to the guidelines presented in Appendix A for the increase in γ_L for each type of analysis used.) This decrease in the distribution factor (which removes the present analysis bias in AASHTO) is offset to some extent by the increased load factor, but there is still an overall increase in the rating factor. Instead of increasing the load factor, the guidelines recommend increasing the girder distribution by specified percentages depending on the type of analysis. This is done so as to keep the number of live load factors to a minimum. The increases shown in the guidelines (Appendix A) were calibrated to maintain a beta of 2.5 for the data shown in Chapter Three.

Figure 51 repeats the same procedure for traffic category (3) (unenforced, light volume traffic). Comparison of Figures 50 and 51 shows that for unenforced traffic (typically having higher weights), there is a considerable reduction in the rating factors, with most of the rating factors falling below 1.0. This can be regarded as an extra incentive for effective enforcement and traffic control.

Figures 52 and 53 repeat the foregoing procedure, but now the nominal resistance is obtained by WSR-I. This enables a comparison of the proposed procedure for bridges that just satisfy a rating factor equal to 1.0 obtained with inventory stress level and HS-20 loading model.

Comparison of Figures 50 and 52 shows much higher rating factor for bridges checked with WSR-I (which is as expected). Comparing Figures 52 and 53 shows the same trends as for WSR-O, i.e., rating decreases with deterioration and increased impact and increases with a better analysis. These figures also illustrate the various options open to an evaluator for making an insufficient bridge rate better.

The basis for all of the foregoing calculations has been the HS-20 vehicle (which has a higher nominal moment). To obtain a different comparison the AASHTO rating vehicles are used as the basis. A sensitivity study for traffic categories 1 and 3 is conducted. It is assumed that all sections rate at 1.0 by existing working stress inventory methods. The base case is taken to be a bridge with smooth surface, good condition sections and light and low volume traffic. Each of these parameters is varied individually from the base level. The results of such a study are shown in Figures 54 and 55. Both figures show a considerable drop in ratings with increasing deterioration. Increasing deck roughness also reduces the ratings but not to that great an extent.

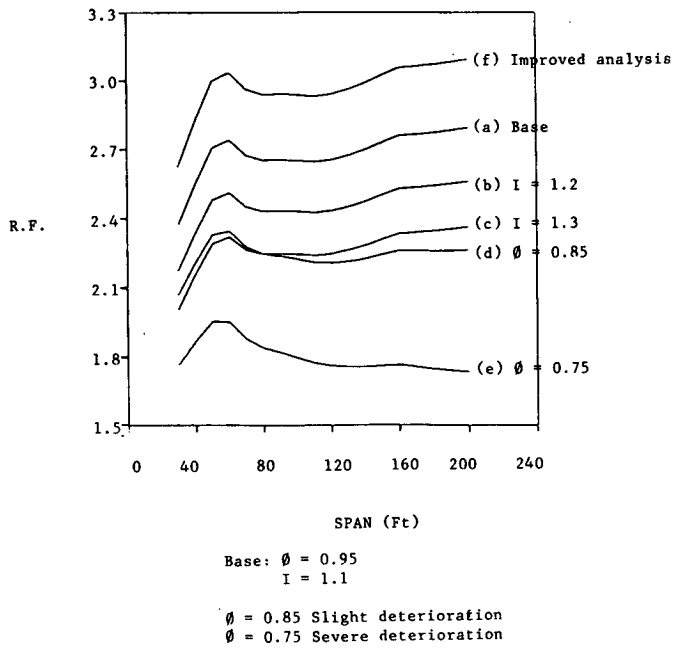


Figure 52. Sensitivity study for traffic category 1.

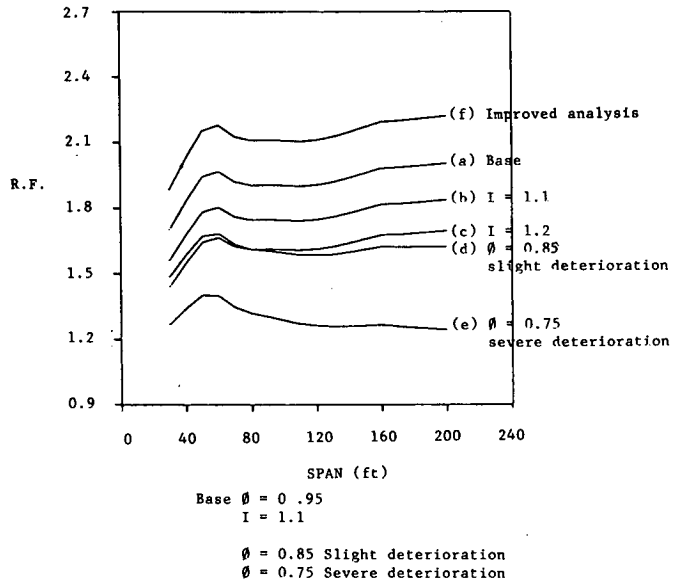


Figure 53. Sensitivity study for traffic category 3.

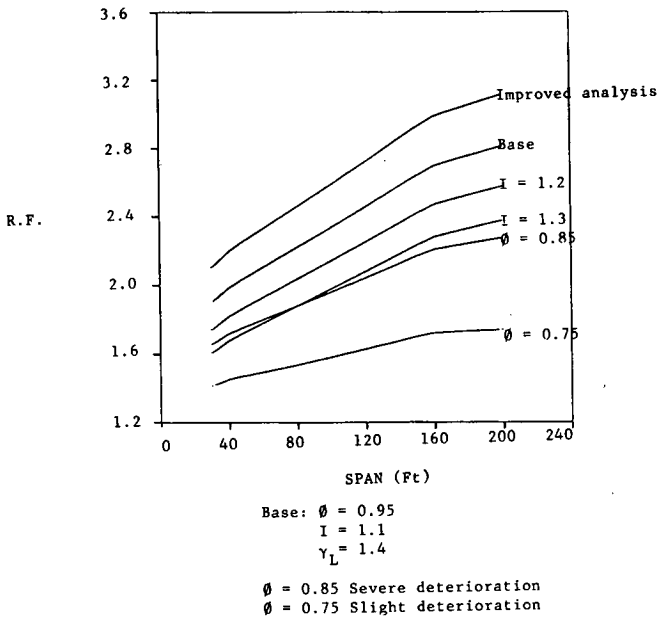


Figure 54. Sensitivity study for traffic category 1.

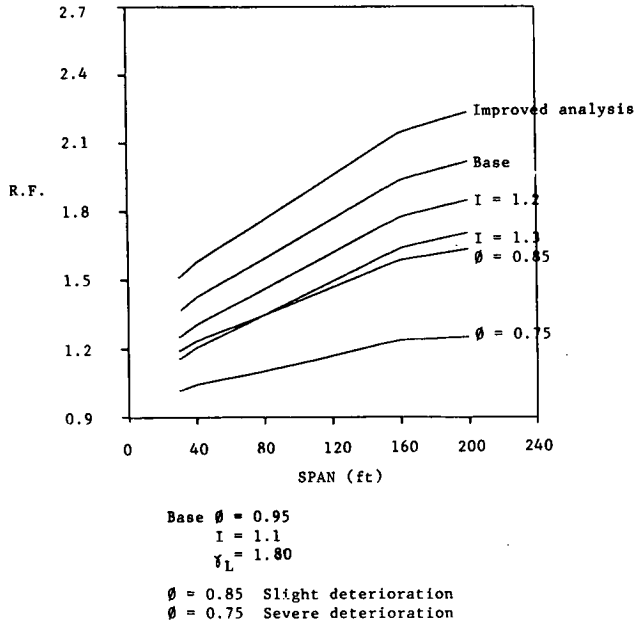


Figure 55. Sensitivity study for traffic category 3.

Figures 56 and 57 show an identical sensitivity study except that now it is assumed that all sections rate at 1.0 by existing working stress operating methods. Figure 56 shows that ratings by the proposed procedure can drop considerably below one for severe deterioration. For unenforced traffic, however, even slight deterioration can lead to ratings below one, as shown in Figure 57. A very rough surface can also lead to the same result. These two figures show the detrimental effects of heavy traffic (especially when coupled with other load capacity reduction factors like deterioration and higher impact). In this way, the proposed strategy directly embodies the spirit of the AASHTO *Manual for Maintenance Inspection of Bridges,* which suggests the use of higher load factors for heavy traffic.

The study above concludes the deterministic sensitivity analysis, which shows the changes in the rating factors upon varying individual terms of the rating equation. At the beginning of this section it was mentioned that a probabilistic type of sensitivity study (i.e., determining changes in the safety indices and their

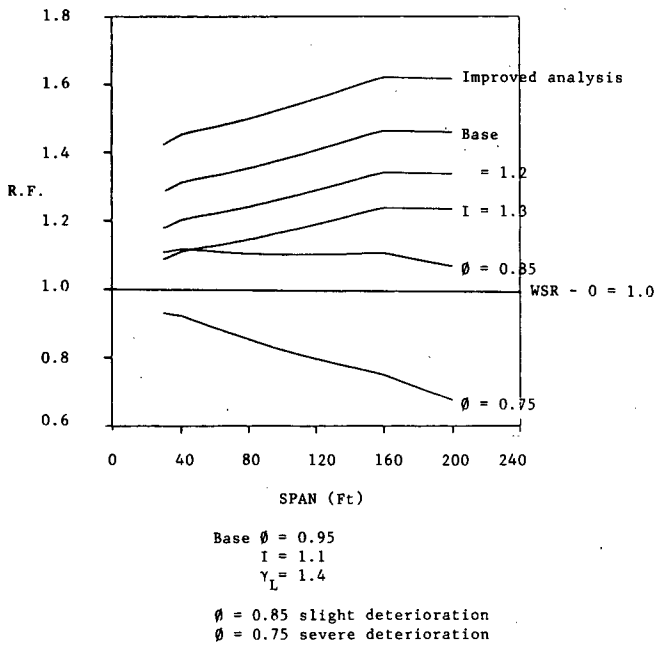


Figure 56. Sensitivity study for traffic category 1.

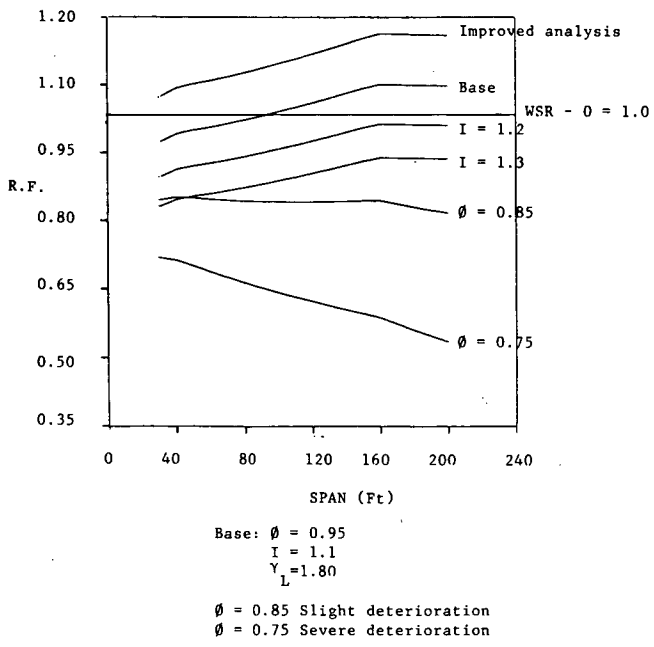


Figure 57. Sensitivity study for traffic category 3.

consistency with changes in the input statistical data) would also be conducted. To do such a study, the bias and COV of the relevant variables (resistance, dead load, live load) are varied and the changes noted in the implied target safety index; one also notes if the uniformity of the betas is still achieved (using the proposed specification and previously calibrated factors).

First, the statistics of the resistance are varied. The bias of the resistance was taken as 1.1 for the calibration; therefore, for a sensitivity study, a bias of 1.2 and 1.0 is examined. Following the same steps as in the section under the heading "Selection of Target Variability," safety indices implicit in current practices (WSR-O) were established for unenforced, light traffic and are shown in Figure 58 along with the base case of a resistance bias of 1.1. This figure shows that changing the bias of the resistance does have a considerable effect on the safety indices, with higher values of bias giving higher safety indices. Similarly, the COV of the resistance is varied from a base case of 12 percent to higher and lower values of 15 percent and 9 percent respectively. The results are shown in Figure 59, which shows that COV also plays a major role in determining the target safety index. From these figures, a target safety index of 3.0 is appropriate for either a resistance bias of 1.2 and COV of 9 percent. Similarly, a target safety index of 2.0 is chosen for either a resistance bias of 1.0 or a COV of 15 percent. Using this changed statistical data, safety indices by the proposed specification (redundant, good condition member with smooth roadway surface) were calculated and are shown in Figures 60 through 63. All of these figures show that the target safety indices mentioned above are still uniformly attained for the four traffic categories. Therefore, changes in the statistics of the resistance undoubtedly influence the target beta, but still do not influence the uniformity of the betas by the proposed specification (though now a different safety index is achieved). This uniformity highlights the calibration process which is related to performance history and the best estimates of the statistical parameters of the data base.

The next factor to be examined is the dead load. First, the bias is varied from a base of 1.0 to higher and lower values of

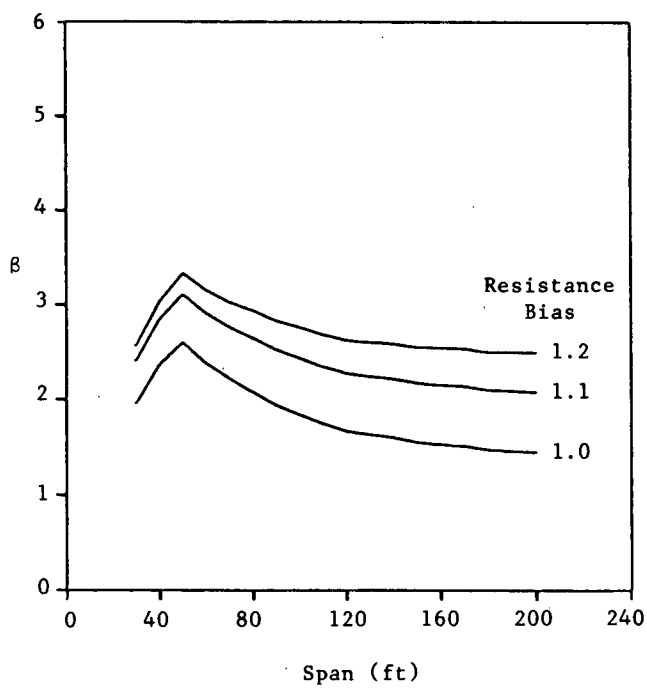


Figure 58. β for different resistance bias (unenforced, light traffic).

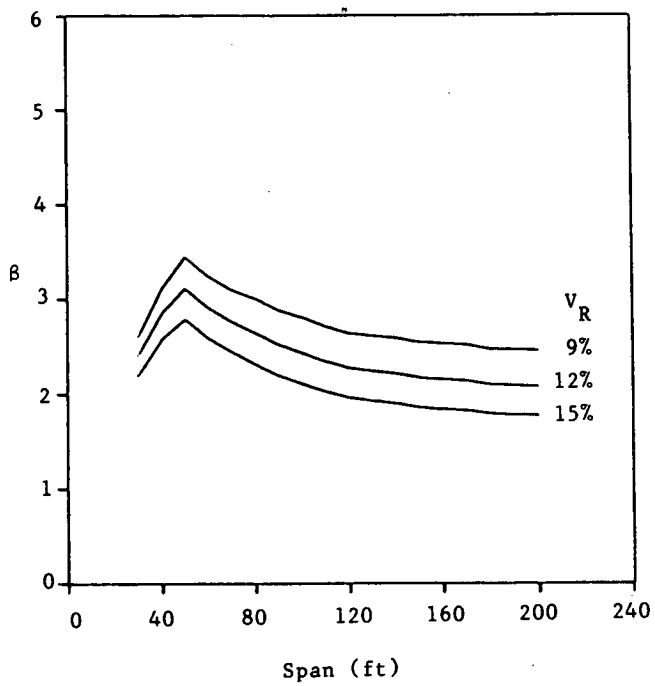


Figure 59. β for different COV's of resistance (unenforced, light traffic).

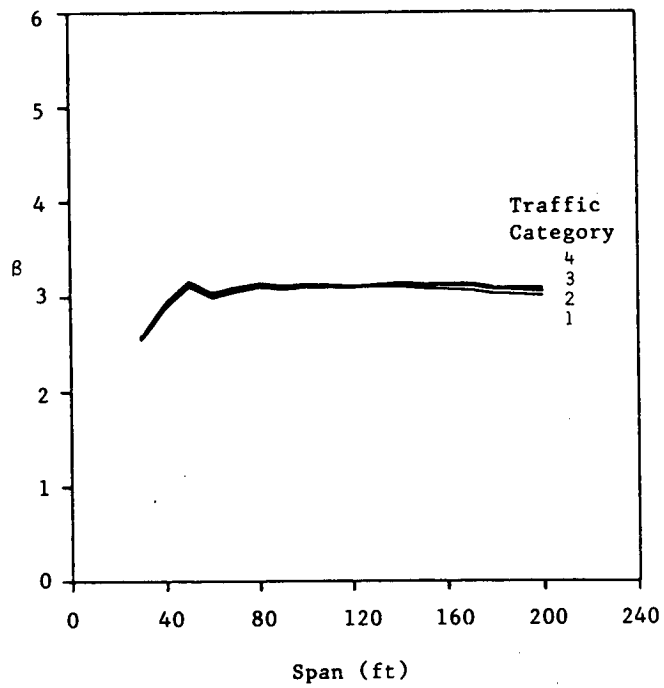


Figure 60. Calibration assuming a resistance bias of 1.2.

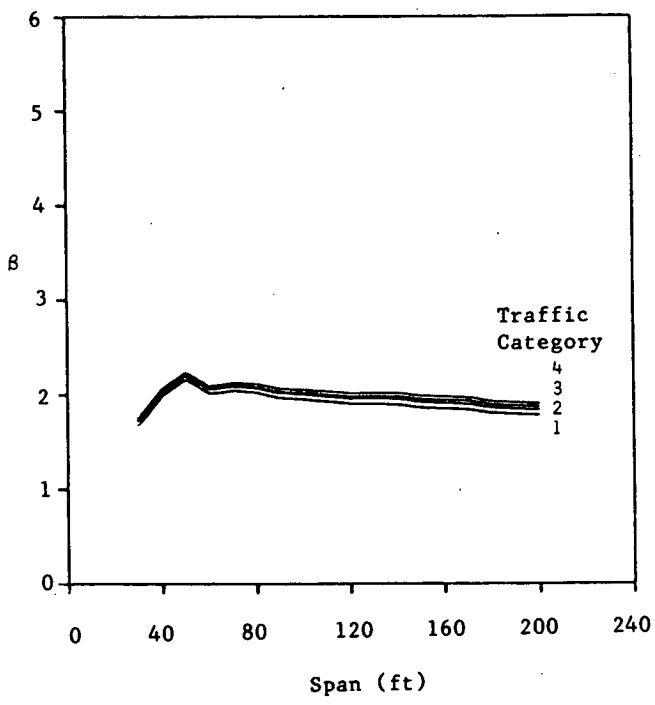


Figure 61. Calibration assuming a resistance bias of 1.0.

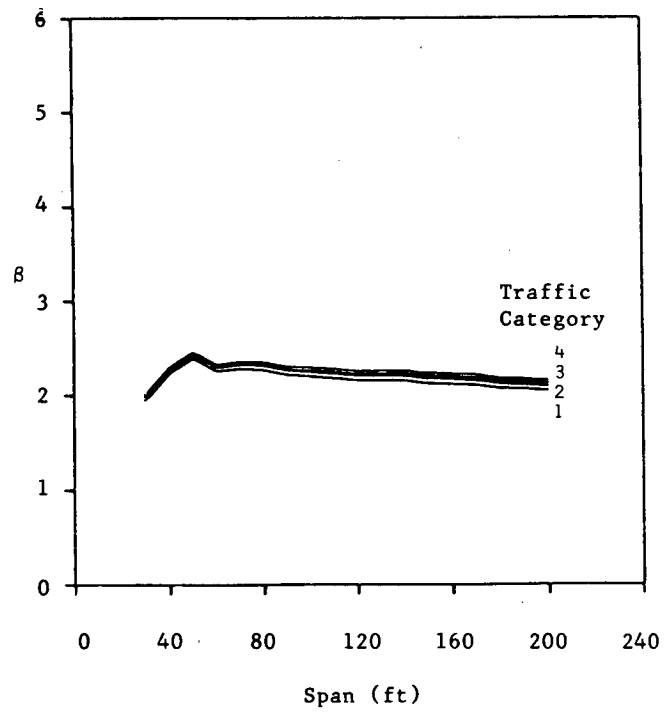


Figure 62. Calibration assuming a resistance COV of 15 percent.

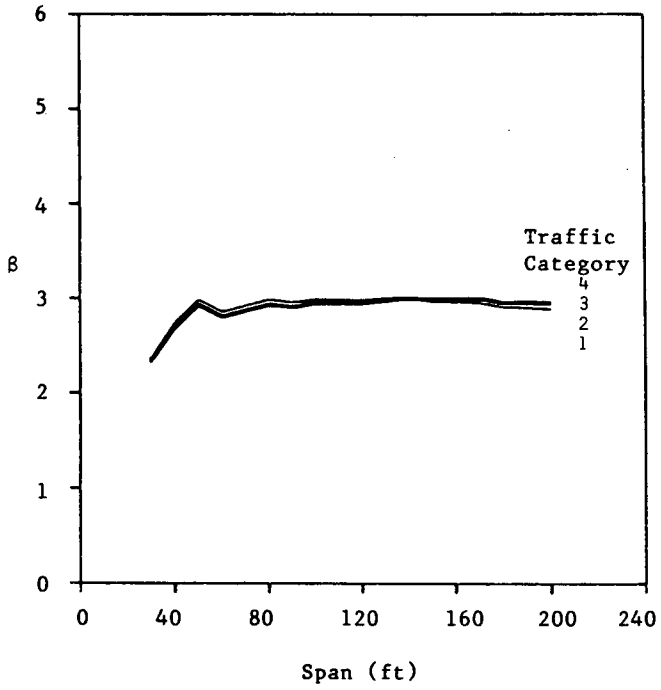


Figure 63. Calibration assuming a resistance COV of 9 percent.

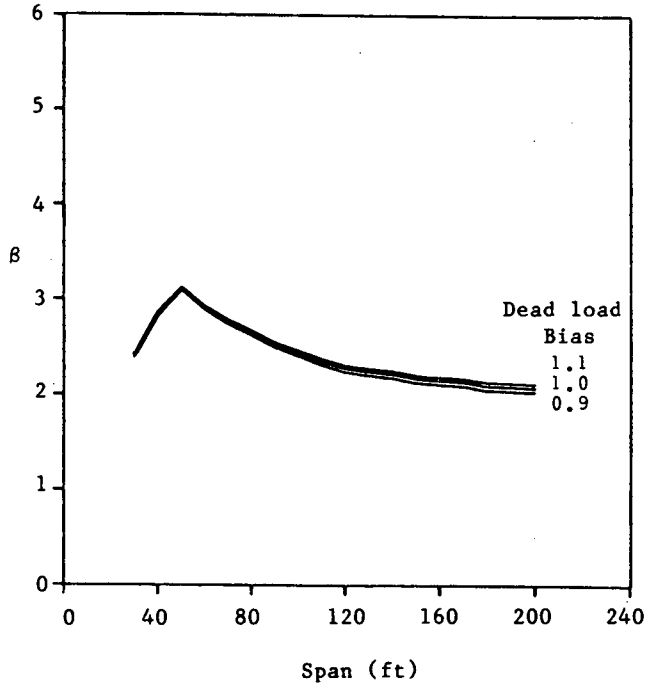


Figure 64. β for different dead load bias (unenforced, light traffic).

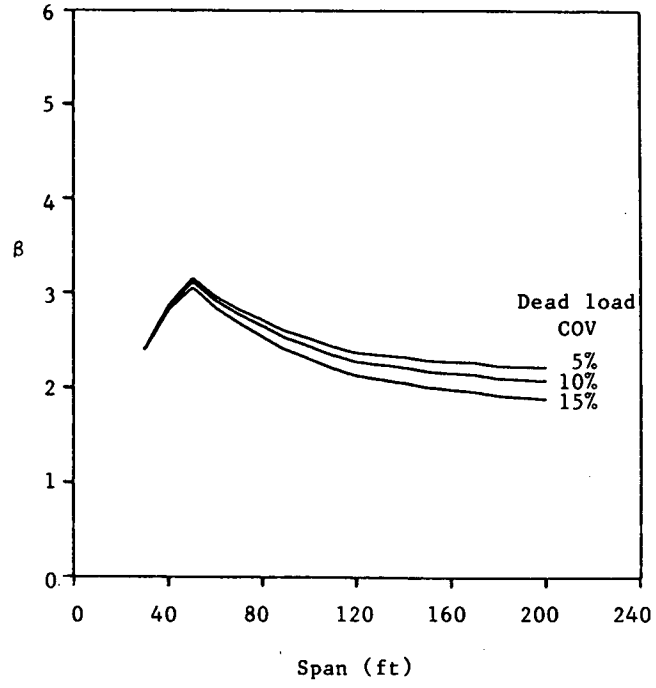


Figure 65. β for different dead load COV's (unenforced, light traffic).

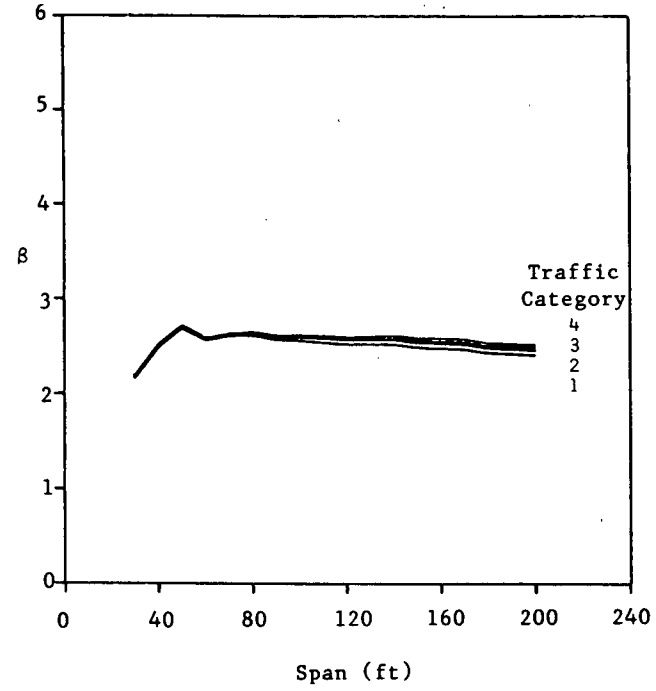


Figure 66. Calibration assuming dead load bias of 1.1.

1.1 and 0.9. Changes in safety indices using current methods (WSR-O) are seen to be minimal as can be seen from Figure 64. Similarly, the COV of the dead load is varied from a base case of 10 percent to higher and lower values of 5 percent and 15 percent respectively. Again, as can be seen from Figure 65, the variations in the safety indices using WSR-O are not very significant. Because the betas are not sensitive to the statistics of the dead load, one would expect safety indices by the proposed

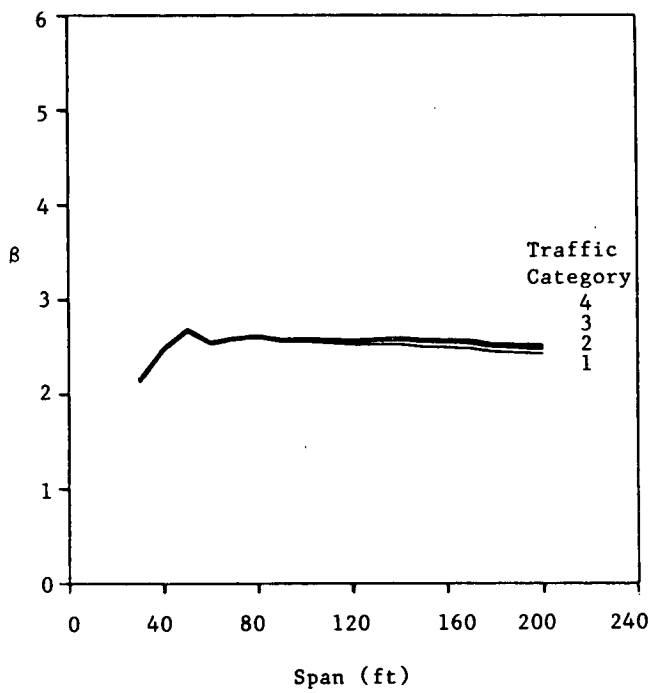


Figure 67. Calibration assuming dead load bias of 0.9.

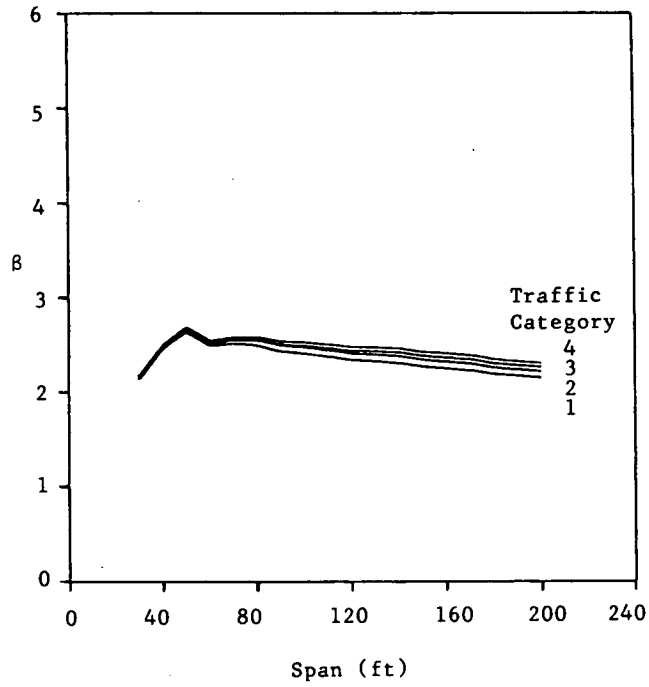


Figure 68. Calibration assuming a dead load COV of 15 percent.

specification (assuming redundant, good condition steel member with smooth roadway surface) to approach 2.5. This indeed is the case as can be seen from Figures 66 through 69, which show the safety indices by the proposed specifications using different assumed values of bias and COV of dead load. Again, the uniformity of betas is maintained even upon "reasonable" changes in the data base.

Finally, the effect of varying the statistics of the girder distribution (for steel members) is examined. Similar results will then be applicable to prestressed concrete members. First, the bias of the girder distribution is varied from a base case of 0.9 to a high value of 1.0 and a low value of 0.8. Safety indices implicit in WSR-O ratings (for unenforced, light traffic) are determined and are shown in Figure 70. Similar to the resistance, a target safety index of 3.0 is chosen corresponding to a bias of 0.8 and a target safety index of 2.0 is selected for a bias of 1.0.

Using these new values of bias, safety indices by the proposed specification are evaluated and are shown in Figures 71 and 72. They are seen to be uniform and also attain the appropriate target safety index (3.0 for a bias of 0.8 and 2.0 for a bias of 1.0).

The foregoing discussion shows that even if some subjective errors are present in the data base, the main aim of achieving consistent target safety indices is still attained because these errors affect the target beta also.

As a final check on the provisions of the proposed evaluation specification, the project research team obtained some actual examples from the consultants to this project to compare the proposed and existing ratings. The results of this effort are presented in Appendix C.

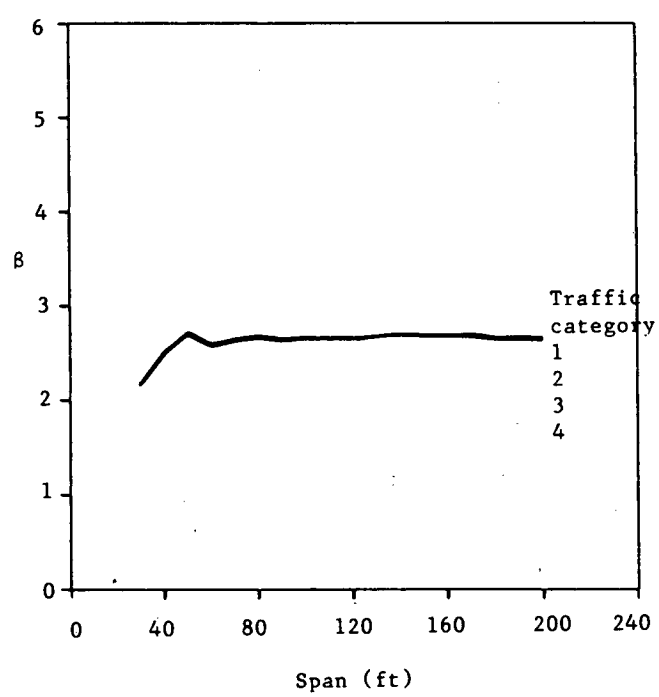


Figure 69. Calibration assuming a dead load COV of 5 percent.

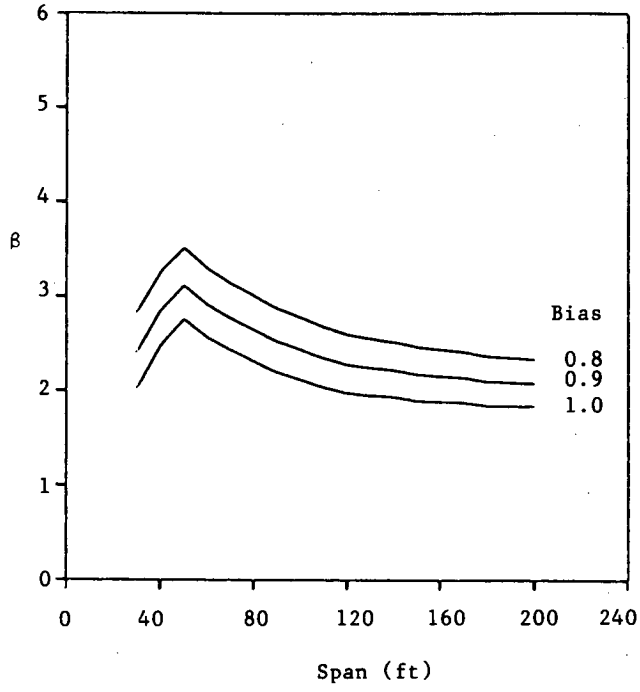


Figure 70. β (WSR-O) for different values of bias for girder distribution (unenforced, light traffic).

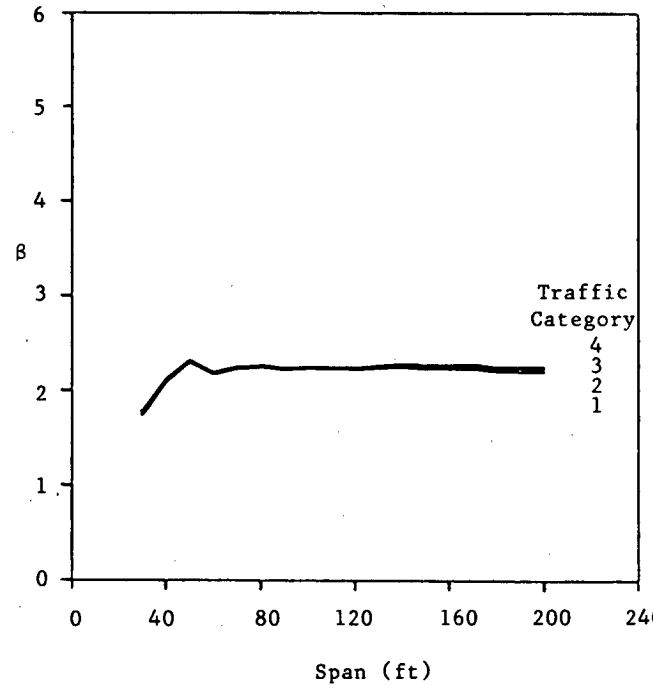


Figure 71. Calibration assuming a girder distribution bias of 1.0.

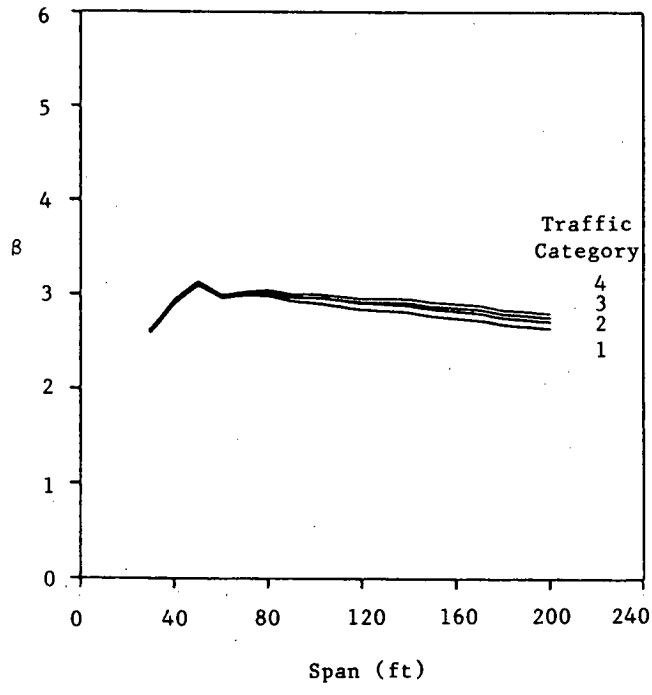


Figure 72. Calibration assuming a girder distribution bias of 0.8.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the accomplishments in this project the following goals have been met.

1. A reliability-based strategy for evaluating bridge components has been formulated.
2. Data for the loading model have been assembled from a variety of sites throughout the United States using weigh-in-motion and bridge testing data.
3. Load and resistance factors have been recommended that lead to uniform and consistent reliability levels.
4. Explicit rating factors for a bridge are given which will depend on the site traffic, method of girder analysis, roadway roughness (impact), and bridge condition and maintenance. These satisfy the implied goals in the present AASHTO *Manual for Maintenance Inspection of Bridges* (henceforth referred to as the AASHTO *Maintenance Inspection Manual*) which suggests the needs for flexible rating stresses and safety margins.
5. Engineers not satisfied with an initially computed rating have an option, through additional effort, to possibly improve the rating. These efforts may include the acquisition of truck traffic information, improved distribution analysis, resurfacing, or recommending additional maintenance.
6. Guidelines encompassing these efforts are contained herein in a format suitable for inclusion in the AASHTO *Maintenance Inspection Manual.* A commentary to the aforementioned guidelines is also provided.
7. A target reliability level has focused on levels implicitly contained in present operating stress levels assuming "reasonably controlled" traffic, AASHTO distribution factors, reasonable enforcement of weight laws, good deck surface, and high level of maintenance. In such instances the proposed guidelines will be similar or even exceed the present AASHTO operating-based rating factors. For cases where traffic is heavy and weight enforcement not present, deck surface is rough, sections are deteriorated, maintenance is poor, and no redundancies are present, the rating factors can fall even below the present inventory-based rating factors.
8. For convenience, the proposed loading model uses the present AASHTO rating vehicles and lateral distribution values.
9. Numerous comparisons are contained herein to illustrate the effects on rating for different factors and options contained in the proposed rating guidelines. Examples obtained from the consultants working on this project have been used to test the guidelines using actual bridges.
10. Lower (more conservative) resistance factors are proposed for components that are defined as nonredundant in the AASHTO *Maintenance Inspection Manual* guidelines.

Advantages of Proposed Evaluation Guidelines

The proposed evaluation guidelines have a number of advantages that include:

1. It is reasonably similar to present procedures in that AASHTO rating vehicles and distribution factors may continue to be used. This will significantly reduce any programming changes for states using large-scale data-based rating systems.
2. The rating approach is consistent with present recommendations in the AASHTO *Maintenance Inspection Manual* which consider site-specific traffic including the likelihood of overweight vehicles and the multiple presence of more than one truck on the bridge.
3. The proposal removes the confusing distinction between inventory and operating rating levels. Rather, it substitutes a full range of possible ratings that lead in the best situations to ratings above the operating level (such as low traffic, good maintenance, rigorous analysis), while in poor situations the ratings may fall even below inventory levels (heavy volume, no enforcement, poor maintenance, rough deck, and so on). For nonredundant components, ratings fall to inventory levels and lower depending on the traffic, condition of section, and roadway roughness.
4. Evaluators not satisfied with an initial level screening may improve the rating value by obtaining more data or recommending better maintenance.
5. Nonredundant components are assigned more conservative ratings in the proposed procedures than redundant components.
6. The proposed rating strategy is based on reliability principles and risk analysis. These should help promote more consistent agency policies with regard to posting, repair, replacement, and enforcement.

Implementation

Implementation of the proposed load and resistance factor procedure for bridge rating would involve several steps. As part of the project, implementation was discussed with CALTRANS and the following observations were noted:

1. Acceptance of the concept by the management is the first step. The decision-making management level of a bridge bureau must first be convinced that the probability-based LRFD approach is more rational in philosophy. For agencies where the current AASHTO load factor method is already in use, such as at CALTRANS, this is an easy task because the LRFD is a logical extension of LFD.

For others, however, it would take convincing that: (a) verification of the proposed values is consistent with field input; (b) the new procedure, which deviates from the traditional working stress method, would not impose additional professional risk to the individual engineer; and (c) there is potential economical benefit in their capital investment.

No estimate of effort was available for this stage.

2. Modification of existing rating programs can be done either in-house or through a consultant. The effort involved is conceivably small and can be accomplished within the usual program maintenance budget. An estimated one man-week of effort is suggested.

3. Teaching and training of rating engineers would be most helpful. Even though the probabilistic aspect of the new procedure is transparent to the rating engineer, it is helpful for the engineer to have sufficient background knowledge about the probability concept involved. In CALTRANS, key personnel have followed the development of NCHRP Projects 10-15 and 12-28 (1) for some time and are familiar with the implications. In other agencies, an introductory short course may prove to be helpful. A two-day seminar is estimated to be adequate.

4. Currently under federal regulation, state bridge bureaus are required to report their bridge ratings to the NBIF at operating and inventory levels. Unless the federal government lifts the weight limit to a higher level such as HS-25, it is unlikely that a state bureau will revise all their bridge inventory using the new criteria. As indicated by CALTRANS, they will use the new criteria only when capital investment for bridge repairment is involved. Very likely, state bureaus will use the current AASHTO criteria as a screening device to identify the candidate bridges and use the new procedure to justify the bridge's load-carrying capacity.

5. Once a bridge is rated by the new procedure, the state agency would have to decide whether to report to the NBIF and at what level (operating or inventory). Unless there is a change in NBIF to allow for the recording of this new rating, the state agency using the new procedure and report at inventory level may be penalized in the FHWA cost allocation calculation.

RECOMMENDATIONS FOR FURTHER BASIC RESEARCH

One major difficulty faced during the course of this effort was the lack of definite statistical data in certain areas. The calibration was based on data available to date; however, certain judgmental decisions were made at this stage for those variables that were backed by an insufficient data base. Specific examples include the following:

1. Effect of atmospheric conditions and corrosion on the strength of prestressed concrete beams.
2. Impact of maintenance in preventing the further deterioration of member cross section.
3. The benefits of detailed inspection in determining the properties of a structural member, specially in conditions of high corrosion.
4. Effect of roadway conditions on the impact of vehicles of different weights.
5. Live load data for posted bridges, one-lane bridges, and three-lane and four-lane bridges.
6. Live load models and corresponding load statistics applicable to describing permit activities.

It is anticipated that further experimental and analytical research will provide more of such data, the results of which can be incorporated into the proposed specification.

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APPENDIX A

GUIDELINES FOR STRENGTH EVALUATION OF EXISTING BRIDGES

This appendix contains guidelines for a strength evaluation specification based on study in NCHRP Project 12-28(1). This section is self-contained and intended for possible inclusion in the AASHTO *Manual for Maintenance Inspection of Bridges* as a reliability-based alternative checking format. A commentary is available (Appendix B) to assist the reader.

SECTION 1
INTRODUCTION

1.1 PURPOSE

These proposed guidelines establish a methodology for rating existing bridges.

1.2 SCOPE

The methodology is presented in a general format utilizing load and resistance factors. This procedure allows for combining probability theory, statistical data, and engineering judgment into a rational decision-making tool. In particular, the procedure allows the engineer to use site-specific information in a consistent manner to improve, if necessary, his judgment on the safe rating level for a particular bridge. In addition, the format incorporates existing methodology for considering local laws and regulations and methods of calculation.

1.3 APPLICABILITY

This methodology is intended for evaluating almost all existing bridges. Steel spans include simple and continuous girder bridges and trusses and floor systems. Concrete spans recognized include slab, girder, T-beam and box beam bridges with short to medium span length. Provisions for evaluation of reinforced concrete sections will be adopted in the future from the NCHRP Project 10-15 report *NCHRP Report 292.* Prestressed beams, although of recent vintage, are included herein.

SECTION 2
SYMBOLS AND DEFINITIONS

ADT	Average daily traffic
ADTT	Average daily truck traffic
F_y	Nominal or specified yield stress
g	Girder distribution, which denotes the distribution of load effect to individual girder components
I	Impact factor to magnify static loading due to dynamic amplification
L_n	Nominal live load effect
$R.F.$	Rating factor
R_n	Nominal strength or resistance
S_x	Elastic section modulus
Z	Plastic section modulus
γ_D	Dead load factor
γ_L	Live load factor
ϕ	Resistance factor

SECTION 3
STRENGTH EVALUATION OF BRIDGES

3.1 GENERAL

The procedure for rating of existing bridges requires knowledge of the physical conditions of the bridge and the applied loadings. A safe level of rating presupposes that nominal strengths should be estimated from a detailed investigation of the structure's physical condition and any continuing attempts to alleviate any signs of deterioration. Further, knowledge of

traffic conditions including signs of overweight vehicle combinations combined with accurate methods of structural analysis should be used when necessary to estimate load effects. The safety factors that must be applied should rationally recognize the corresponding uncertainties in making these judgments on strength, analysis, and loading. The concepts of structural reliability are a means for consistently representing these uncertainties and allowing bridge engineers to select proper safety factors for rating specific bridges.

3.2 SAFE EVALUATION

The strength evaluation procedures presented herein are intended to recognize a balance between safety and economics. Detailed presentations of the theory and the calibration of the factors contained herein are given in *NCHRP Report 301.* The previously existing distinction of operating and inventory stress levels for rating are no longer maintained. Rather, a single load rating will be produced by these guidelines. The rating engineer will find that, with the factors specified herein, bridges may reach or even exceed their previous operating rating for those bridges which receive frequent qualified inspection and have adequate maintenance programs and loads corresponding to reasonable levels of traffic and enforcement. Conversely, sites that do not maintain these conditions and have nonredundant critical components will find their ratings falling to inventory levels or even lower. Evaluators will find options in these guidelines by which ratings can be improved by recommendations for more frequent and detailed inspection and maintenance, improved structural analysis, and especially control of heavy overweight vehicles.

These guidelines are intended to produce rating factors for routine evaluation and posting considerations. Evaluation of live load for issuance of permits may require factors different from rating and shall also utilize the actual vehicle size, weight, and configuration.

3.3 THE RATING EQUATION

The evaluation is carried out with a comparison of the factored live load effects and the factored strength or resistance. The load factors are used to account for uncertainties in load effects due to uncertainties in analysis as well as load magnitudes. The dead load factor includes normal variations in material dimensions and densities. The live load factor accounts for uncertainties in expected maximum vehicle loading effect, impact, and distribution of loads during a time period between inspections. The resistance factor accounts for uncertainties in strength prediction theories, material properties and deterioration influences over time periods between inspection. Furthermore, the factors are adjusted to produce an overall safety margin which leads to an adequate level of safety considering all uncertainties described above.

The rating procedure is carried out for all strength checks (moment, shear, etc.) at all potentially critical sections with the lowest value determining the rating factor for the entire span. The rating equation to be used throughout the application of these guidelines is:

$$\phi R_n = \gamma_D D + \gamma_L (R.F.) L (1 + I) \qquad (1)$$

$$\text{or } R.F. = \frac{\phi R_n - \gamma_D D}{\gamma_L L (1 + I)} \qquad (2)$$

where the terms are defined in Section 2. The rating factor is the ratio of the safe level of loading to the load produced by the nominal or standard vehicle. It may be used in the consideration of posting levels and/or the consideration and justifications for future repairs or replacement. In determining factors for the rating equation, the following steps shall be carried out in evaluating a bridge span:

1. Collection of information.
2. Selection of nominal loadings and resistances.
3. Distribution of loads.
4. Selection of load and resistance factors.
5. Calculation of rating factors.

A flowchart for the rating procedure is also provided in Figure 1. The evaluator should note that potential improvement in the rating factor may come from selecting options in each step. These generally provide a less conservative factor provided additional evaluation effort is performed and no unsatisfactory information is uncovered.

3.3.1 Collection of Information. This task shall be the same as the provisions in the existing AASHTO *Manual for Maintenance Inspection of Bridges* except that the following items should be noted because they have an influence on the selection of load and resistance factors:

1. Deck Condition—The impact factors in AASHTO design specification are deliberately selected to be conservative with respect to most conditions. Field tests have shown that the single most important factor affecting impact is roadway roughness and any bumps, sags, or other discontinuities which may initiate or amplify dynamic response to truck passages. Any of these surface factors should be noted during a bridge inspection.
2. Structural Condition—Signs of recent deterioration in structural members that may go unchecked and increase the likelihood of further section capacity loss before the next cycle of inspections and rating should be noted. Conversely, maintenance efforts to mitigate such deterioration should also be noted. An allowance for structural deterioration should note when this is either an expected or conservative estimation because further deterioration may increase the uncertainty regarding reliable section properties and strength during the next inspection interval.
3. Traffic Conditions—The expected loading during the inspection interval is affected by the truck traffic at the site. In the best instance, data will be available from traffic surveys including objective truck weight operations. Alternatively, advice should be sought from the traffic division regarding truck traffic volume, composition, permit activities, overload possibilities, and degree of enforcement.

3.3.2 Selection of Nominal Loadings and Resistances.

3.3.2.1 Dead Loads—The dead load shall be estimated from

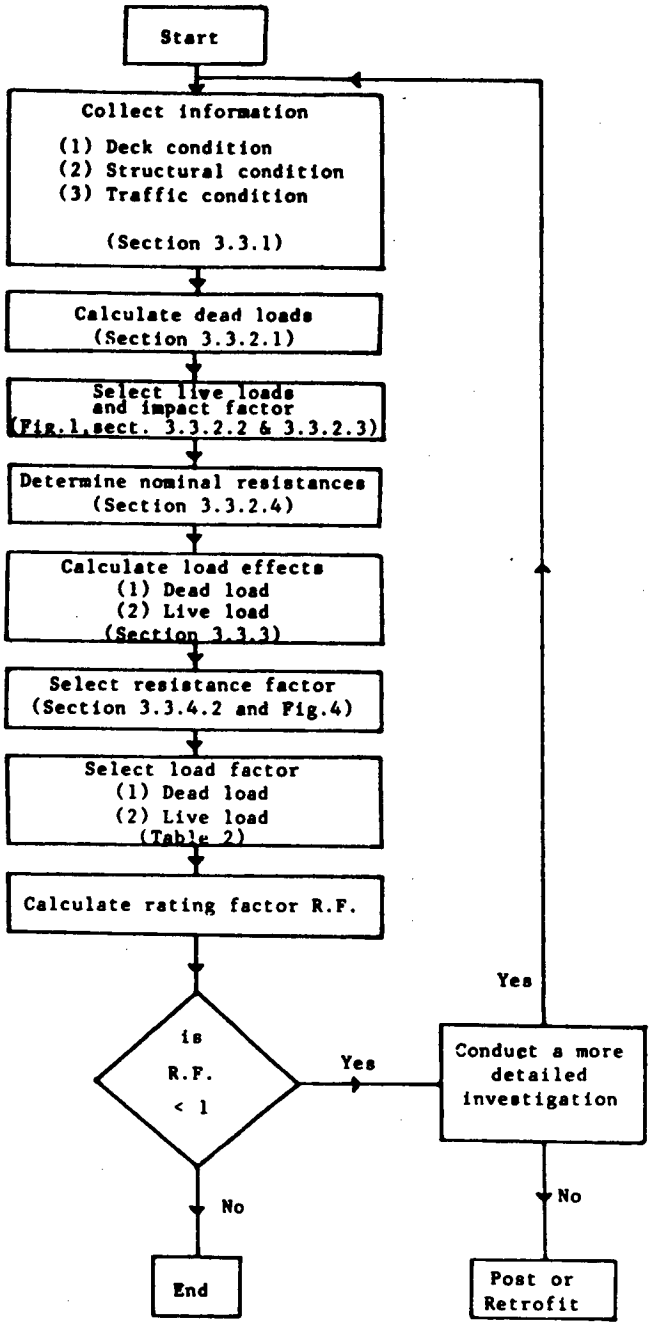


Figure 1. Flowchart for the evaluation process.

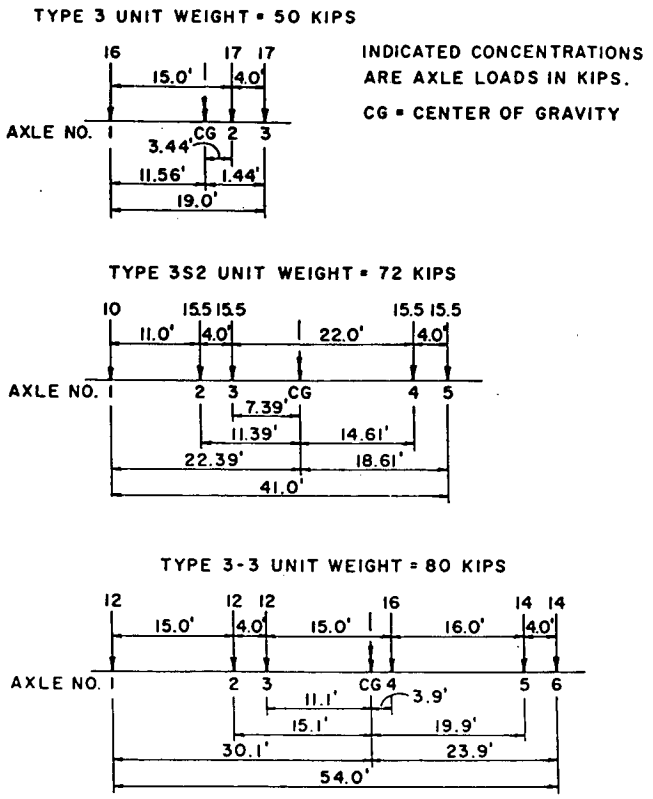


Figure 2. AASHTO rating vehicles.

data available from the inspection at the time of analysis. The dead load factor accounts for normal variations of material densities and dimensions. Nominal dimensions and densities shall be used for calculating dead load effects. For overlays, either cores shall be used to establish the true thickness or an additional allowance of 20 percent should be placed on the nominal overlay thickness indicated at the time of analysis.

3.3.2.2 Live Loads—The moving loads to be applied on the deck for calculating maximum nominal live loading effects shall be the three AASHTO legal vehicles shown in Figure 2. The spacings and axle weights chosen for these vehicle types were selected from actual truck weight surveys conducted throughout the United States. They also correspond to actual maximum legal loads conforming with regulations of most states. It is believed that these typical vehicles correspond better to existing traffic and will provide more uniform reliability than the standard AASHTO H or HS design loading. Hence, the latter are not recommended for bridge evaluation purposes. Additional vehicles should be added to those shown in Figure 2 to conform with maximum legal weights and lengths in specific jurisdictions. Adjustments of axle weights and spacings of the three AASHTO legal vehicles are not recommended.

In computing load effects, one vehicle shall be considered present in each lane. It is unnecessary to place more than one vehicle in a lane because the load factors shown below have been modeled for this possibility. These factors shall be considered applicable for spans up to 200 ft. For longer spans the lane-type loading given in Figure 3 will govern the evaluation. It is a combination of a vehicle load and a uniformly distributed load. When the rating factor is less than one, it may be necessary to place more than one vehicle in each lane. In lieu of this, the evaluator should check the lane loading for all span lengths together with the rating vehicle as shown in Figure 3. Where maximum load effects in any member are produced by loading a number of traffic lanes simultaneously, reduction factors as given in Table 4 should be applied.

In checking special permits, the actual vehicle weights and dimensions shall be used. If the number of such permits in one year are frequent, it shall be assumed that two lanes are occupied by such a vehicle. Otherwise, standard vehicles may be placed in the other lanes. Upon special investigation, the load factor

for a controlled permit use is reduced below the value taken for ordinary traffic conditions.

3.3.2.3 Impact—An impact allowance shall be added to the static loads used for rating as shown in Eq. 1. Impact values in the AASHTO design specifications reflect pessimistic conditions that may possibly prevail under certain circumstances. Under an enforced speed restriction, impacts may be reduced. For smooth approach and deck conditions, the impact may be taken as 0.10.

For a rough surface with bumps, a value of 0.20 should be used. Under extreme adverse conditions of high speed, spans less than 40 ft and highly distressed pavement and approach conditions, a value of 0.30 should be taken.

If such a judgment cannot be made, refer to the bridge inspection report and relate impact to the condition of the wearing surface.

Condition of Wearing Surface	Impact
1. Good condition—No repair required	0.1
2. Fair condition—Minor deficiency, item still functioning as designed	0.1
3. Poor condition—Major deficiency, item in need of repair to continue functioning as designed	0.2
4. Critical condition—Item no longer functioning as designed	0.3

3.3.2.4 Resistances—Nominal component strengths shall be the same quantities now contained in the load factor sections of the AASHTO design specifications and the AASHTO *Manual for Maintenance Inspection of Bridges.* Effective areas shall take due account of decay or corrosion. Nominal unit stresses must depend on the type of steel used in the structural member. When tests are performed to assess yield stress, the mean values shall be reduced by 10 percent to produce nominal values for strength calculations. Provisions for calculating component strength from the existing manuals can be inserted herein for completeness. However, these values shall be nominal strength without any safety factor applied.

3.3.3 Distribution of Loads. The fraction of vehicle load effect transferred to a single member may be selected in accordance with current AASHTO design specifications. It is generally believed that these values are conservative and represent a possible combination of adverse circumstances. The option exists to substitute field-measured values, analytically calculated values, or those determined from advanced structural analysis methods using the properties of the existing span(s). Loadings shall be placed in positions causing the maximum response. Further, if such a measurement or analysis is made and the expected distribution value is obtained, this shall be increased by the factors given in Table 1 to reflect possible uncertainties in the measurement or analytical model.

3.3.4 Selection of Load and Resistance Factors.

3.3.4.1 Load Factors—The load factors shall be taken from Table 2. These are intended to represent conditions existing at the time this specification is written based on field data obtained from a variety of locations using weigh-in-motion and other data gathering methods. The live load factor accounts for the likelihood of extreme loads side-by-side and following in the same lane and the possibility of overloaded vehicles. Because one aim of these specifications is to protect the investment in the bridge structure, the live load factors do recognize the frequent presence of overweight trucks on many highways. An option to reflect

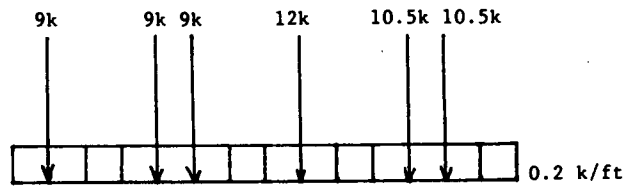


Figure 3. Lane loading.

Table 1. Correction factors for analysis.

DISTRIBUTION OF LOADS	CORRECTION FACTOR Steel	P/S
1) AASHTO Distribution	1.00	1.00
2) Tabulated analysis with simplifying assumptions	1.10	1.05
3) Sophisticated analysis finite elements, orthotropic plate, grillage analogy	1.07	1.03
4) Field measurements	1.03	1.01

Actual girder distribution shall be multiplied by the appropriate correction factors to obtain the girder distribution for rating.

*Correction factors are needed only if average or expected values are used for D.F. from analysis or measurements. The correction factor shall be used to increase the load factor taken from Table 2.

Table 2. Load factors.

Loading	Factor
Dead Load	$\gamma_D = 1.2$

Allow an additional allowance of 20% on overlay thickness if nominal thicknesses are used. No allowance is needed when measurements are made for thickness.

Live Load Category	
1) Low volume roadways (ADTT less than 1000), reasonable enforcement and apparent control of overloads	$\gamma_L = 1.4$
2) Heavy volume roadways (ADTT greater than 1000), reasonable enforcement and apparent control of overloads	$\gamma_L = 1.6$
3) Low volume roadways (ADTT less than 1000), significant sources of overloads without effective enforcement	$\gamma_L = 1.8$
4) Heavy volume roadways (ADTT greater than 1000), significant sources of overloads without effective enforcement	$\gamma_L = 1.95$

If unavailable from traffic data, Estimates for ADTT may be made from ADT as follows: urban areas, ADTT = 15% of ADT; rural areas, ADTT = 25% of ADT

Table 3. Resistance factors.

I. Resistance Factors - Good Condition

Nominal resistance equations are to be those indicated in the AASHTO Standard Specifications for Highway Bridges under the sections on load factor design. Resistance (capacity reduction) factors are to be applied to the following for the case where members are in good condition.

Redundant* Steel Members:	Ø = 0.95
NonRedundant Steel Members:	Ø = 0.80
Prestressed concrete beams:	Ø = 0.95

II. Influence of Deterioration

1. Where field inspection and condition survey reports indicate no deterioration, the provisions of this section should not be used.
2. Where field inspection and condition survey reports indicate slight deterioration with some possible loss of section, the resistance values above shall be decreased by 0.1.
3. Where field inspection and condition surveys report significant deterioration and heavy section loss, the resistance values shall be reduced by 0.2.
4. If such information is not available than bridge records shall be used. Reduce the resistance values by 0.1 for superstructure condition of 5 or 6. Reduce the resistance values by 0.2 for a superstructure condition of 4 or less. If these reductions are made then the next two sections should be ommitted.

III. Inspection**

1. Where field inspection and condition survey reports indicate no deterioration, the provisions of this section should not be used.
2. Where section losses have been carefully estimated in the calculation of remaining section areas the resistance factors may be increased by 0.05.
3. Where material yield stress has been estimated by physical testing, a mean value x 0.90 may be used for calculating strength together with the capacity reduction factor contained in the design rules.

IV. Maintenance**

1. Where maintenance activity is vigorous and likely to correct deficiencies which may lead to further section loss, increase Ø by 0.05.
2. Where maintenance activity is intermittant and may not correct defects that have lead to section loss, decrease Ø by 0.05.

*Examples of redundant members include parallel stringers (three or more), parallel eye bars (four or more). Example of nonredundant component include two girder system(s) and trusses with single members.

**In no instance shall Ø be taken to exceed 0.95.

effective overload enforcement is contained herein with a reduced live load factor. The presence of illegal loads has also been noted, and if such vehicles are present in large numbers at the site, this may lead to unacceptable ratings and enforcement efforts should be instituted.

When the rating factor is less than 1.0, the loads are to be restricted. In instances where the rating factor is less than 1.0, consideration should be given to truck weight surveys and vigorous enforcement programs. If there is a reason to believe that truck posting signs are being ignored, consideration should be given to raising the live load factor.

3.3.4.2 Resistance Factors—The resistance factors or capacity reduction factors in the AASHTO *Standard Specifications for Highway Bridges* are intended for new components with current methods of high quality control. The nominal (unfactored) strengths to be used for evaluation represent an estimate of strength using data pertaining to member properties and conditions at the time of inspection. The resistance factor shall consider both the uncertainties in estimating these member properties and also any bias or conservativeness deliberately introduced into these estimates. Because further changes may occur to the section during the inspection interval, there is some dependence of these properties on the quality of maintenance. Also, the level and detail of inspection is important because it may reveal actual properties to be used in section calculations. The resistance factors for members in good condition are given in Table 3, section I. The influence of deterioration, inspection, and maintenance are given in Table 3, sections II, III, and IV. A flow chart for obtaining the resistance is presented in Figure 4.

3.3.5 Calculation of Rating Factors. The rating factor is to be calculated from Eq. 1. If it exceeds 1.0, the span is satisfactory for the legal loads in that jurisdiction. (In the present specifications, there is only a single rating value, eliminating the operating and inventory levels, which determines the allowable loads.) The safety factors (load and resistance) have been calibrated to provide adequate safety under the inspection, maintenance, analysis, redundancy, and loading conditions cited. These provisions have the capability for evaluations to be improved by using options related to more intensive inspection and maintenance or control of heavy overloads.

The rating factors obtained herein may also safely be applied to permit loadings. In some instances where a permit might otherwise be rejected, the live load factors contained herein may be reduced to reflect known weight conditions associated with the permit vehicle. This reduction in load factor may depend on the degree of control of the permit and the number of permits that may be issued. Fatigue life should be a consideration in the issuance of overload permits and this factor is taken up in NCHRP Project 12-28(3) (*NCHRP Report 299*).

An example of the use of this rating procedure is given in the following.

3.3.5.1 Example—

- Span = 60 ft, simply supported steel girders.
- Section W 30 × 132, 8-ft spacings.
- No shear connectors.
- Properties I_{xx} = 5,770 in.^{4} (Noncomposite)
  S_x = 380 in.^{3}
  Z_x = 437 in.^{3}
- Steel—A36.
- Use AASHTO girder distribution ($S/11$) = 0.727 (based on truck moment).
- Use AASHTO legal trucks as rating vehicles.
- Condition of wearing surface—good, take impact as I = 0.1.

Vehicle	Rating Vehicle Moment (kip-ft)	Girder Rating Moment (including impact) (kip-ft)
3	596	472
3-S2	612	485
3-3	564	451

3.3.5.2 Data Required for Rating—

- Inspection: once every two years (*normal*).
- Maintenance: biannual.
- Traffic: high ADTT with no apparent enforcement, category 4: γ_L = 1.95.

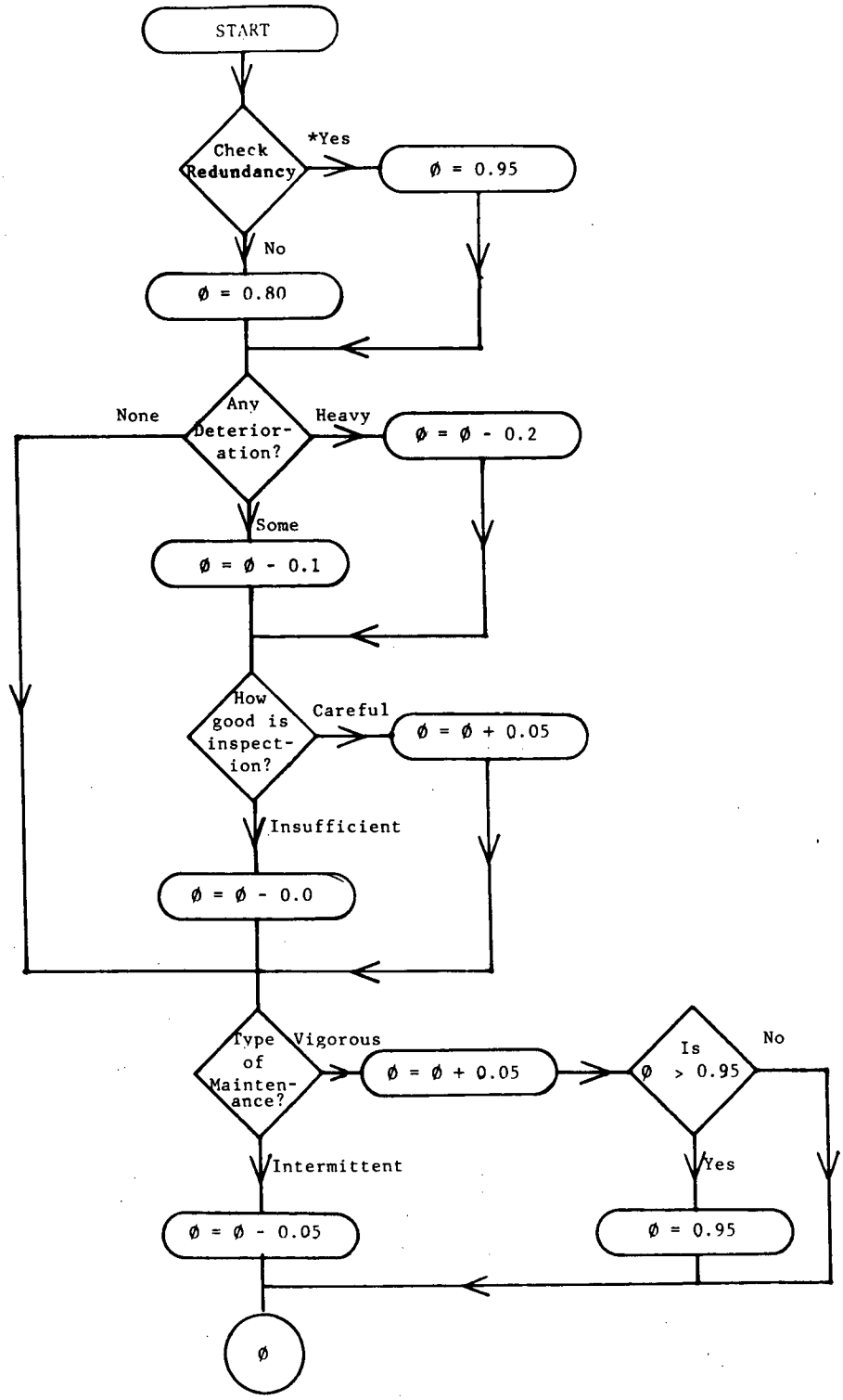


Figure 4. Flowchart for selecting resistance factors.

- Type of member: redundant steel member.
- Condition: no deterioration. Use $\phi = 0.95$.
- Superstructure summary: 8.
- From Table 3:

Section I, $\phi = 0.95$
Section II, no changes
Section III, no changes
Section IV, no changes

- Estimated dead load = 306 kip-ft (including additional 20 percent allowance for overlay.
- $R.F. = \dfrac{\phi F_y Z_x - 1.2 * D}{1.95 \times \text{Moment}}$

$$= \frac{0.95 * 36 * 437/12. - 1.2 * 306}{1.95 \times \text{Moment}}$$

PROPOSED RATING FACTORS	
Rating Vehicle	R.F.
3	0.95
3-S2	0.93
3-3	1.00

No Posting Necessary. Using prevalent practices, rating factors are rounded to nearest 0.05 value. No posting necessary for resulting values of 0.95 or higher.

COMPARISON WITH EXISTING RATING PROCEDURES		
	AASHTO	
Rating Vehicle	Operating	Inventory
a. HS-20	0.74	0.44
b. AASHTO legal: 3	1.00	0.59
3-S2	.98	0.57
3-3	1.05	0.62

Table 4. Reduction factors for live load.

NUMBER OF LANES	REDUCTION FACTOR
One or two lanes	1.0
Three lanes	0.8
Four lanes	0.7

APPENDIX B

COMMENTARY ON PROPOSED BRIDGE STRENGTH EVALUATION PROCEDURES

BACKGROUND

Need

The present specifications of the AASHTO *Manual for Maintenance Inspection of Bridges* (henceforth referred to as the AASHTO *Maintenance Inspection Manual*) are used by most states for their bridge evaluations. A considerable variation in interpretation of these specifications leads to a wide range of ratings for similar bridges. Also, the large safety margins present in some cases are not always recognized. This presents difficulties for highway agencies in dealing with the large number of structurally deficient bridges in their jurisdiction. Moreover, specific methods are lacking for incorporating site traffic and structural behavior data into the evaluation. There is a need for a comprehensive, yet flexible, methodology for evaluating existing bridges which is still consistent with today's high standards of safety.

Advantages of New Procedures

1. They provide uniformly consistent procedures for evaluating existing bridges.
2. They permit suitable flexibility in making evaluations.
3. They provide uniform levels of reliability developed from performance histories.
4. They are based on extensive truck traffic and bridge response data.
5. They permit introduction of site-specific data into the evaluation in a rational and consistent format.
6. They permit different levels of effort that involve progressively more work, with correspondingly greater rewards in terms of more beneficial ratings.
7. They include the same nominal dead and live load calculations and resistances as in the present AASHTO specifications.
8. They allow distinction between evaluation of redundant and nonredundant components.

Reliability Concepts

Conservative assumptions are made in each step of a strength design or checking procedure to safeguard against the worst possible conditions expected to occur during the lifetime of a structure. In other words, the probability of failure is made

exceedingly small by providing large safety margins to cover the uncertainties in predicting load effects and resistance of a bridge.

Reliability principles utilizing site data have been used to evaluate the uncertainties and the safety levels or indices implicit in current designs. The rating methodology and safety factors have then been subsequently developed to maintain consistent safety levels for the aforementioned uncertainties. Options for incorporating site-specific traffic and loading data and higher levels of effort by the engineer are introduced inasmuch as these lead to a reduction in the overall uncertainty. The lower safety margin required to maintain the same safety level means more beneficial ratings. At no stage is it necessary for the evaluation engineer to use probabilistic methods. The necessary reliability-based load and resistance factors have been tabulated for the evaluation.

EVALUATION PROCEDURES

Scope

These procedures apply to concrete slab, girder, T-beam and box beam bridges of short to medium span length, simple and continuous steel girder bridges, steel trusses, and prestressed beams (The enclosed material does not yet cover concrete slab or beam bridges. They will be integrated into the proposed specifications using results from NCHRP Project 10-15 (*NCHRP Report 292*) before a final presentation.)

General Procedure

A load factor procedure is prescribed. The rating check is done by comparing the factored load effects (both dead and live) with the factored resistance at all critical sections. The output is a rating factor which determines the suitability of the given bridge for the loads under consideration. If the bridge rating is not acceptable, several options for a more detailed analysis are given. Each of these options is associated with an increasing level of effort and may be done if the rating engineer warrants their use. An initial screening level, however, is provided for routine investigations.

LOADINGS

Dead Loads

The dead load of the structure is computed in accordance with the conditions existing at the time of the analysis.

The prescribed dead load factor recognizes the uncertainties in the nominal dimensions and analysis of dead load effects. Overlay thicknesses are a source of greater uncertainty in the dead load, so they are assigned a 20 percent higher load factor unless cores or more detailed measurements are made.

Live Loads

The three AASHTO legal trucks are recommended as evaluation vehicles. States may substitute their own legal vehicles at this stage. These vehicles, together with the prescribed live load factors, give a realistic estimate of the maximum live load effects of a variety of heavy trucks in actual traffic.

The load factors have been derived so as to maintain the AASHTO recommendations for both the number of loaded lanes and using a single truck loading per lane. For longer spans, a lane loading is specified in the evaluation. Reduction factors for live loading of more than two traffic lanes are provided. These rationally account for the lower possibility of such occurrences.

Impact Loads

Impact loads are taken to be primarily caused by the roughness or unevenness of the road surface, especially the approach spans. Three values of impact factors are provided by correlating the roughness of the surface to the deck condition survey values. The information is more likely known during evaluation than in the original design.

Lateral Distribution

Lateral distribution refers to the fraction of the live load carried by the member under consideration. Methods in the AASHTO design specifications are followed. However, their conservative nature (for steel members) has been recognized by calibrating the load factors accordingly. Options exist for using tabulated values (*NCHRP Report 301*), more refined analysis (e.g., finite elements), and field measurements. Each of these options involves a greater level of effort and more accuracy, so adjustments to the basic live load factors are provided for both steel and prestressed concrete members. These adjustments implicitly recognize that more refined analysis may in some instances remove the implicit conservativeness present in some simplified distribution formulas and are therefore treated accordingly.

Resistances

Nominal resistances for members are based on AASHTO's design specification contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength. Specifications for both these factors have been provided. Options exist for incorporating data on structural condition obtained from the site. Careful estimation of losses and deterioration are awarded a higher resistance factor. Similar gains are also given for vigorous maintenance and inspection schedules. Options also exist for obtaining more precise material strength through tests.

Live Load Factors

Live load factors have been provided to account for the large uncertainty of the maximum live load effects on a structure over a period of time. A large amount of field data has been modeled to estimate the maximum live load effect together with its uncertainty. Based on these data, degree of enforcement, volume and type of traffic are isolated as the major factors influencing the live load effect. The live load factors have been derived from these data for bridges with a single lane, two lanes, and three

and four lanes. Instead of providing different sets of load factors for different number of lanes, only one set of load factors is provided with corresponding reduction factors for other cases.

Four categories of live load are provided with varying volumes and degrees of enforcement, each with its corresponding live load factor. Site truck traffic data recorded by the engineer may also be included by reference to NCHRP Project 12-28(1) report (*NCHRP Report 301*).

Resistance Factors

Resistance factors account for uncertainties in nominal dimensions and material strength. A basic set of resistance factors is provided. The reliability levels are calibrated to produce different resistance factors for redundant and nonredundant spans with the latter having lower (more conservative) factors. The redundancy definitions are the same as given in the present AASHTO fatigue design provisions. The resistance factors can be further modified depending on the amount of deterioration and type of inspection and maintenance. Options exist for conducting detailed measurements of strength losses. Also included are benefits for vigorous maintenance schedules. This allows the evaluation to be flexible enough and also covers a large range of types and conditions of members that may be encountered.

APPENDIX C

ILLUSTRATIVE EXAMPLES

This appendix contains a set of 13 example bridges which illustrate the methodology of the proposed procedure. Included are the current ratings as obtained from the consultants working with this project. This highlights the differences and the increased flexibility of the proposed procedure as compared to current practice. It also serves as a check for the proposed strategy, thereby ensuring that no vast differences in rating are encountered.

Because the assembled bridges cover a large variety of spans, geometries, types, and conditions, it is hoped all the provisions of this specification have been tested. Also provided are a data sheet and rating sheet which incorporate all the provisions of the proposed specifications. Use of these sheets serves the following purpose: (1) standardizes the procedure, (2) ensures that no provisions have been omitted, (3) summarizes all the information necessary to do a rating calculation, and (4) eases the application of the proposed methodology.

Illustrations of the uses of these sheets have also been provided. All the examples have been classified according to type and are given as: (1) Steel Pony Truss, (2) Thru-Truss, (3) Steel Thru-Truss, (4) Steel Deck Truss, (5) Pin-Connected Truss, (6) Steel Stringer, (7) Steel Girder, (8) Thru-Plate Girder, (9) P/S Concrete Stringer, (10) P/S Concrete I-Beam, (11) P/S Concrete Box Beam, (12) P/S Concrete Box Girder, and (13) P/S Concrete T-Beam.

A review of the following examples shows that, in general, the proposed ratings will reach or even exceed AASHTO operating ratings for (1) good condition section, (2) redundant element, (3) smooth roadway, (4) low to moderate traffic, (5) vigorous maintenance, (6) detailed inspection, and (7) high dead-to-live load ratios.

Conversely, the proposed ratings can fall even below current inventory levels for (1) nonredundant elements, (2) deteriorated condition, (3) rough roadway, (4) heavy traffic with numerous overloads, (5) intermittent maintenance, and (6) low dead-to-live load ratios.

The proposed ratings may be improved by the engineer by obtaining more traffic information, performing a rigorous distribution analysis, recommending resurfacing to reduce impact, detailed inspection, vigorous maintenance, and control overweights (reduce traffic categories).

To summarize the results of the examples, the rating loads by rating vehicle 3S2 (i.e., rating factor*72 kip) vs. existing ratings have been plotted for the different elements of the bridges examined and are shown in Figure C-1 (for nonredundant elements) and Figure C-2 (for redundant elements). For nonredundant elements, the proposed ratings are compared to existing inventory values (because the calibration was to inventory levels), whereas for redundant elements, the proposed ratings are compared to existing operating ratings. Also shown in Figures C-1 and C-2 is a 45-deg line. Examples which give the same rating factor by proposed and existing methods fall on this line. All the points falling above this line imply higher ratings by the proposed specifications. All points falling below this line imply ratings lower than existing values. Figure C-1 shows that a majority of points fall above the 45-deg line, indicating that the proposed ratings are generally higher than the existing ratings. Figure C-2, however, shows similar number of points falling above and below the 45-deg line, which indicates that the proposed ratings are higher than the existing values for some cases and lower for other cases. The comparisons depend on the traffic conditions, condition of section, and so on. Therefore, each bridge should be examined individually to obtain a clearer understanding of the underlying assumptions (resistance factor, live load factor, etc.) and their implications.

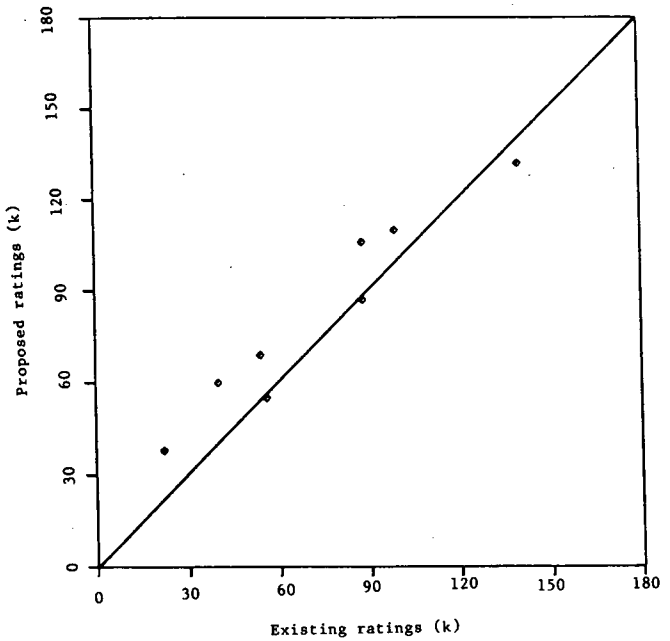


Figure C-1. Comparison of proposed and existing (inventory) ratings (non-redundant elements).

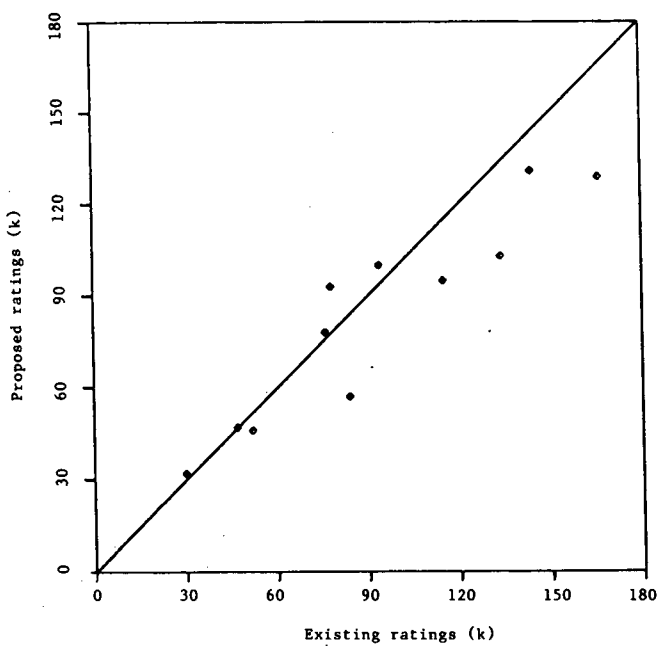
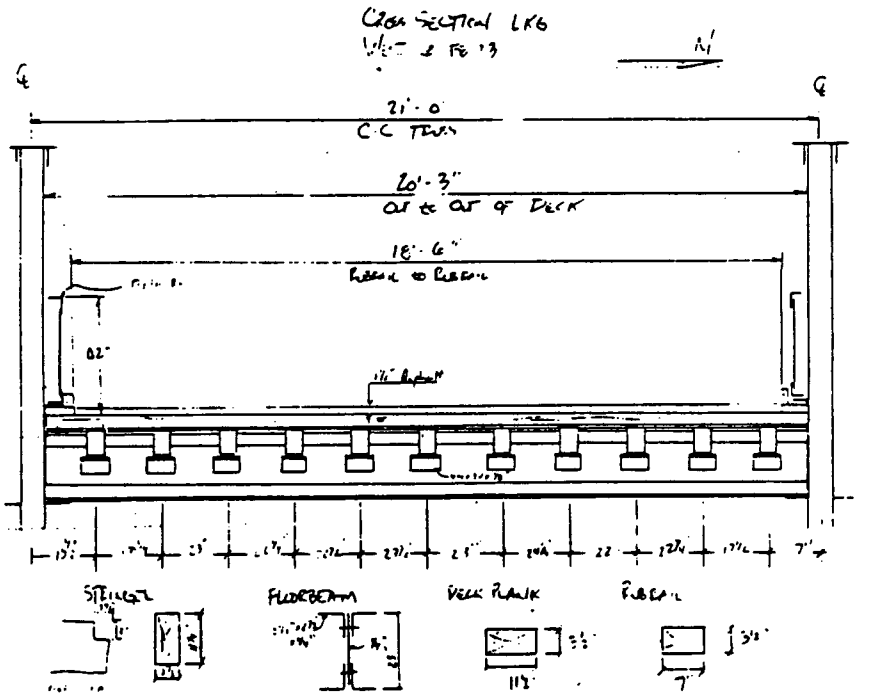
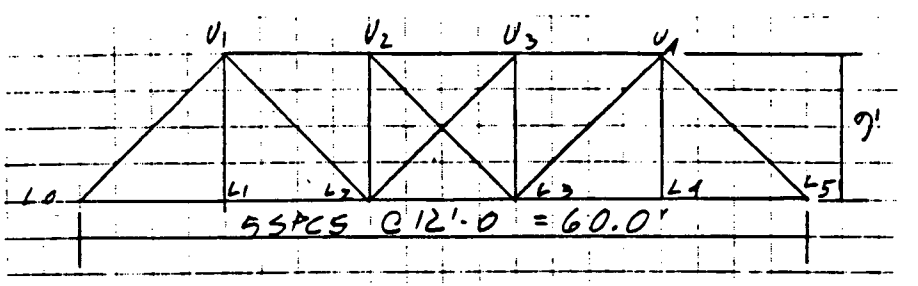


Figure C-2. Comparison of proposed and existing (operating) ratings (redundant elements).

1. STEEL PONY TRUSS





DATA SHEET

BRIDGE: Steel Pony Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss members BY:	REF.
I TRAFFIC		
1 ADT	Low	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS Non-redundant diagonal of steel pony truss

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Pony Truss			ELEMENT: Truss member U_2L_3			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	48 K	34 K	76 K	54 K	94 K	46 K
PROPOSED	44 K		69 K		85 K	

Comments: Proposed ratings are lower than operating because of non-redundancy.

RATING SHEET

BRIDGE: Steel Pony Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss member U_2L_3	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 0.56 K FROM CORES ___		1.03 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	0.80		1.24		
TOTAL	2.04 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	20.79	18.95	17.12		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	22.87	20.85	18.83		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT	32.02	29.19	26.36		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	37.44 K				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.8				Data Sheet
3 FACTORED RESISTANCE 1 x 2	29.95 K				= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	44 K	69 K	85 K		

DATA SHEET

BRIDGE: STEEL PONY TRUSS	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beams BY:	REF.
I TRAFFIC		
1 ADT	LOW	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS Floor beams assumed non-redundant.

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Pony Truss			ELEMENT: Floor Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	24 K	14 K	36 K	22 K	46 K	28 K
PROPOSED	25 K		38 K		49 K	

Comments: Proposed ratings reach operating levels because of low traffic even though they are assumed non-redundant.

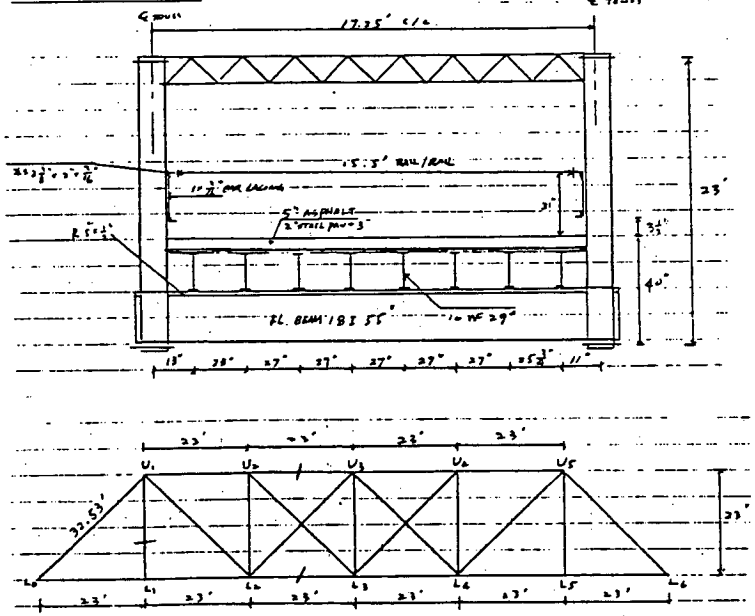
RATING SHEET

BRIDGE: STEEL PONY TRUSS	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beams	FORCE: Bending/Span = 21'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 12.5K-ft FROM CORES ___		16.7 K-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	18.0 K-ft		20.0 K-ft		
TOTAL	38.04 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	163.3	154.0	134.6		3.3.3.2
2 DISTRIBUTION FACTOR	Simply	Supported	Analysis		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	179.6	169.5	148.1		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT	251.4	237.3	207.3		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			204.8 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.80		Data Sheet
3 FACTORED RESISTANCE 1 x 2			163.8 K-ft		= B

$$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$$

IV POSTING LOAD	VEHICLE 1	VEHICLE 2	VEHICLE 3
	$R.F._1 \times 50$	$R.F._2 \times 72$	$R.F._3 \times 80$
	25 K	38 K	49 K

2. THRU TRUSS BRIDGE



DATA SHEET

BRIDGE: Thru Truss Bridge	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Stringers BY:	REF.
I TRAFFIC		
1 ADT	Low	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant X - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE ___ SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS X INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	$\phi = 0.95 + 0.05 \leq 0.95$ $\phi = 0.95$	FIGURE 3 TABLE 3

COMMENTS Very low dead load

RATING SHEET

BRIDGE: Thru Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Stringers	FORCE: Bending/Span = 23'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 7.14 K-ft FROM CORES ___		3.70 K-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS X 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	10.28 K-ft		4.44 K-ft		
TOTAL	14.72 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	81.5	74.44	67.10		3.3.3.2
2 DISTRIBUTION FACTOR	0.5	0.5	0.5		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	44.83	40.94	36.90		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT	62.76	57.32	51.66		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			93.23 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.95		Data Sheet
3 FACTORED RESISTANCE 1 x 2			88.56 K-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	59 K	93 K	114 K		

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Thru Truss			ELEMENT: Stringers			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	50 K	34 K	78 K	52 K	96 K	64 K
PROPOSED	59 K		93 K		114 K	

Comments: Proposed ratings exceed operating levels because of very low dead load, stringers assumed compact and low traffic.

DATA SHEET

BRIDGE: Thru Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor beams BY:	REF.
I TRAFFIC		
1 ADT	≈ 500	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS X INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.85	FIGURE 3 TABLE 3

COMMENTS

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Thru Truss			ELEMENT: Floor Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	32 K	18 K	50 K	28 K	60 K	34 K
PROPOSED	41 K		65 K		80 K	

Comments: The proposed ratings exceed operating because of low traffic, good maintenance, low impact and compact section even though the floorbeams are assumed to be non-redundant.

RATING SHEET

BRIDGE: Thru Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beams	FORCE: Bending/Span - 17.25'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 41 K-ft FROM CORES ___		25.95 K-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	59.04 K-ft		31.14 K-ft		
TOTAL	90.18 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	107.5	98.73	89.36		3.3.3.2
2 DISTRIBUTION FACTOR	FROM AASHTO TABLES				3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K-ft)	118.30	108.60	98.30		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K-ft)	165.60	152.00	137.60		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			267.50 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.85		Data Sheet
3 FACTORED RESISTANCE 1 x 2			227.38 K-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	41 K	65 K	80 K		

DATA SHEET

BRIDGE: Thru Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss member U_2U_3 BY:	REF.
I TRAFFIC		
1 ADT	≈ 500	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS X INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.85	FIGURE 3 TABLE 3

COMMENTS: Concentrically loaded compression member

$P_u = 0.85\ A_s F_{cr}$

$F_{cr} = F_y[1 - F_y/4\pi^2 E\ (K\ell_c/r)^2]$

$K\ell_c/r = (77.7)(0.875)$

$F_y = 26$ ksi

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Thru Truss			ELEMENT: Truss Member U_2U_3			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	64 K	38 K	70 K	40 K	74 K	42 K
PROPOSED	62 K		60 K		70 K	

Comments: The proposed ratings are slightly lower than operating because of non-redundancy and the fact that this is a compression member. The ratings are helped by smooth surface and good maintenance.

RATING SHEET

BRIDGE: Thru Truss	FILE:		DATE:		
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss member U_2U_3	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 45 K FROM CORES ___		41.9 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS ≈ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	64.80 K		50.28 K		
TOTAL	115.08 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	50.00	74.50	71.18		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K)	55.00	82.00	78.30		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K)	77.00	114.80	109.62		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			248.00 K		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.85		Data Sheet
3 FACTORED RESISTANCE 1 x 2			210.80 K		= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	62 K	60 K	70 K		

DATA SHEET

BRIDGE: Thru Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss Member L_2L_3 BY:	REF.
I TRAFFIC		
1 ADT	≈500	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant X - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS X INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS : Bottom chord assumed redundant due to floor system

$A_s = 6.0 \text{ in}^2$

$F_y = 26$ ksi

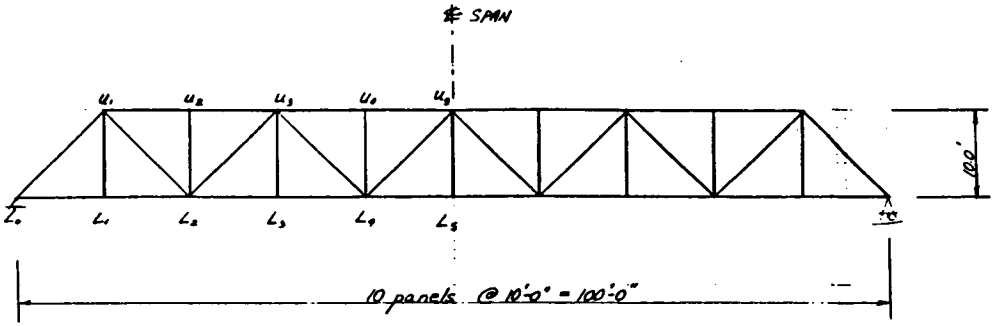
COMPARISON OF PROPOSED AND EXISTING RATINGS

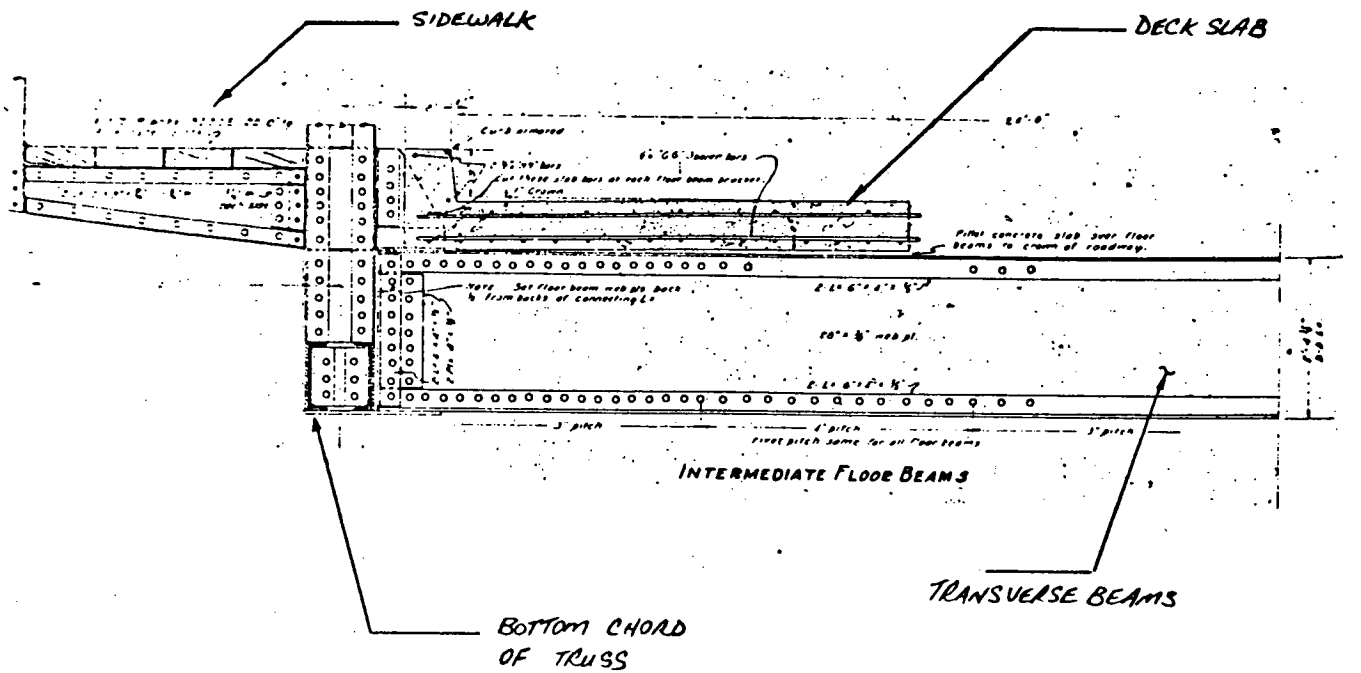
BRIDGE: Thru Truss			ELEMENT: Truss Member L_2L_3			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	48 K	12 K	52 K	12 K	54 K	14 K
PROPOSED	43 K		46 K		49 K	

RATING SHEET

BRIDGE: Thru Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss Member L_2L_3	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 40.0 K FROM CORES ___		37.2 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS ≈ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	57.6 K		44.7 K		
TOTAL	102.3 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	35.0	47.1	48.9		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	38.5	51.8	53.7		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K)	53.9	72.5	75.2		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			156 K		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.95		Data Sheet
3 FACTORED RESISTANCE 1 x 2			148.2 K		= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	43 K	46 K	49 K		

3. STEEL THRU TRUSS





DATA SHEET

BRIDGE: Steel Thru Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss Member U_2U_3 BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH ___ ROUGH X	
3 IMPACT FACTOR	0.1 ___ 0.2 X 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS

$F_{cr} = F_y[1 - F_y/4\pi E\ (KL_c/r)^2]$

$KL_c/r = 19.15$

$A_s = 33.6\ in^2$

$P_u = 0.85\ F_{cr}A_s$

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Thru Truss			ELEMENT: Truss Member U_3U_4			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	125 K	81.5 K	133 K	88 K	144 K	98 K
PROPOSED	81 K		87 K		97 K	

Comments: Ratings by the proposed specifications drop to inventory levels because of non-redundancy, 0.2 impact, frequent heavy trucks and compression member.

RATING SHEET

BRIDGE: Steel Thru Truss	FILE:		DATE:		
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss Member U_3U_4	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ None FROM CORES ___		264 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	-		316.8 K		
TOTAL	361.8 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	103.7	139.6	138.4		3.3.3.2
2 DISTRIBUTION FACTOR	-	-	-		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	124.4	167.5	166.1		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K)	223.9	301.5	298.9		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	849 K				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.8				Data Sheet
3 FACTORED RESISTANCE 1 x 2	679.2 K				= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	81 K	87 K	97 K		

DATA SHEET

BRIDGE: Steel Thru Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beams BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH ___ ROUGH X	
3 IMPACT FACTOR	0.1 ___ 0.2 X 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS $F_y = 30$ ksi
$Z = 380.45$ in^3
$M_u = 951.13$ K-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Thru Truss			ELEMENT: Floor Beams			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	96 K	63 K	151 K	99 K	186 K	122 K
PROPOSED	70 K		110 K		136 K	

Comments: Even though the member is assumed non-redundant, the ratings are higher than inventory because of the good condition of the member. The proposed rating is reduced because of the observation that there are frequent heavy (over loaded) trucks at the site. Ratings can be improved by better enforcement.

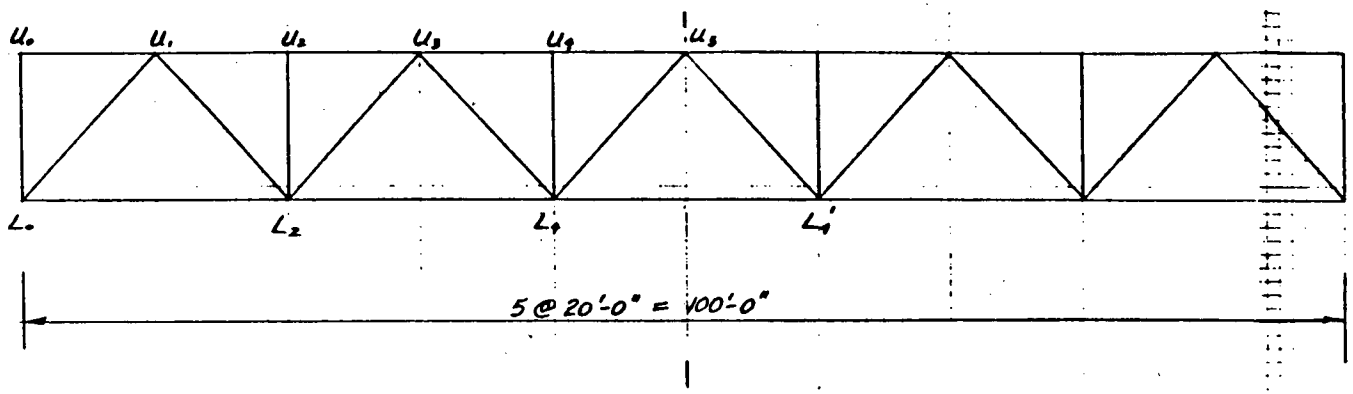
RATING SHEET

BRIDGE: Steel Thru Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beams	FORCE: Bending/Span = 25.5'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ None FROM CORES ___		98.4 K-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	-		118.1 K-ft		
TOTAL	118.1 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	213.0	194.2	175.4		3.3.3.2
2 DISTRIBUTION FACTOR	SIMPLY	SUPPORTED	ANALYSIS		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K-ft)	255.6	233.0	210.5		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K-ft)	460.1	419.4	378.9		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			951.13 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.80		Data Sheet
3 FACTORED RESISTANCE 1 x 2			760.90 K-ft		= B

$$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$$

IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$
	70 K	110 K	136 K

4. STEEL DECK TRUSS



DATA SHEET

BRIDGE: Steel Deck Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss Member L_4U_5 BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH X	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 X 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS $KL_c/r = 66.20$

$F_{cr} = F_y\,[1 - F_y/4\pi^2 E\,(K\ell_c/r)^2] = 26.55$ ksi

$A_s = 9.13$ in^2

$P_u = 0.85\,F_{cr}A_s = 206$ K

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Deck Truss			ELEMENT: Member L_4U_5			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	183 K	131 K	199 K	140 K	216 K	153 K
PROPOSED	124 K		132 K		150 K	

Comments: Existing inventory ratings are comparable to the proposed ratings. Operating ratings are higher because of 0.2 impact and also because it is a non-redundant compression member.

RATING SHEET

BRIDGE: Steel Deck Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss Member L_4U_5	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ None FROM CORES ___		15.53 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	-		18.64 K		
TOTAL	18.64 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	27.2	36.8	36.1		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	32.6	44.2	43.3		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT	58.7	79.6	78.0		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	206.0 K				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.80				Data Sheet
3 FACTORED RESISTANCE 1 x 2	164.8 K				= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	124 K	132 K	150 K		

DATA SHEET

BRIDGE: Steel Deck Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beams BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH X	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 X 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS

COMPARISON OF PROPOSED AND EXISTING RATINGS

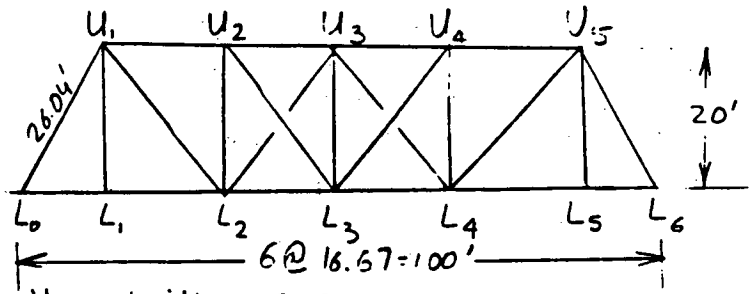
BRIDGE: Steel Deck Truss			ELEMENT: Floor Beams			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	56 K	36 K	89 K	56 K	109 K	69 K
PROPOSED	35 K		55 K		68 K	

Comments: As before, the effect of non-redundancy is to lower the ratings to existing inventory levels. However, the advantage with the proposed specs is that they allow posting loads to be increased if the traffic is controlled.

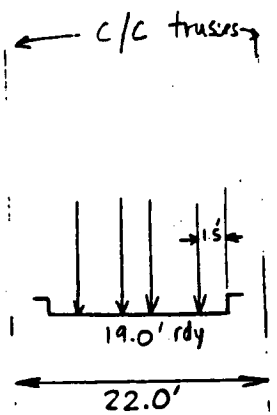
RATING SHEET

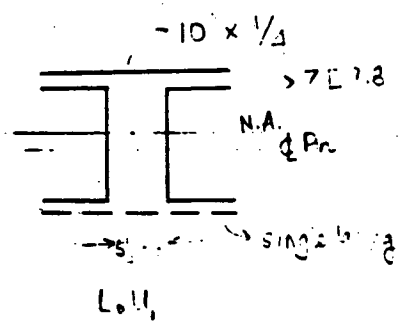
BRIDGE: Steel Deck Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beams	FORCE: Bending/Span = 22'				
I DEAD LOAD FORCES	ASPHALT			REMAINING	
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ None FROM CORES ___			52.7 K-ft	
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20			1.2	3.3.2.1
3 FACTORED D.L. ON ELEMENT	-			63.2 K-ft	
TOTAL	63.2 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	138.7	126.5	114.2		3.3.3.2
2 DISTRIBUTION FACTOR	SIMPLY SUPPORTED ANALYSIS				3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K-ft)	166.4	151.8	137.0		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K-ft)	299.5	273.2	246.6		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			340 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.8		Data Sheet
3 FACTORED RESISTANCE 1 x 2			272 K-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	35 K	55 K	68 K		

5. PIN-CONNECTED TRUSS



Year built 1909





DATA SHEET

BRIDGE: Pin-Connected Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Truss Member L_0U_1 BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant ___ - non-redundant X	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER	N.A.	TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.80	FIGURE 3 TABLE 3

COMMENTS

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Pin-Connected Truss			ELEMENT: Truss Member L_0U_1 (compression)			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	27 K*	not available	32 K*	not available	34 K*	not available
PROPOSED	24 K		28 K		31 K	

Comments: *Based on ratings given in AASHTO Manual for Maintenance Inspection of Bridges, 1970.

RATING SHEET

BRIDGE: Pin-Connected Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Truss Member L_0U_1	FORCE: Axial				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ FROM CORES ___ None		29.3 K		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	-		35.2 K		
TOTAL	35.2 K				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K)	51.6	63.4	64.7		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR					TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K)	56.8	69.7	71.2		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K)	102.2	125.5	128.1		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			106 K		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.80		Data Sheet
3 FACTORED RESISTANCE 1 x 2			84.8 K		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	24 K	28 K	31 K		

DATA SHEET

BRIDGE: Pin-connected Truss	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beams BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 X Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks X Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 X 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH X ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 X 0.2 ___ 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant X - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE X SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS

$F_y = 30$ ksi

$Z_x = 125$ in^3

$R_n = F_y Z_x = 312.5$ K-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Pin-Connected Truss			ELEMENT: Floor Beams			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	31 K	*	47 K	*	58 K	*
PROPOSED	31 K		47 K		61 K	

Comments:

* none available

RATING SHEET

BRIDGE: Pin-Connected Truss	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beams	FORCE: Bending/Span =21'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 8.9 K-ft FROM CORES ___		35.9 K-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS $\bar{x}$ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	12.9 K-ft		43.1 K-ft		
TOTAL	56 K-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	FROM AASHTO PLATE 3				3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	-	-	-		TABLE 1
4 IMPACT FACTOR	0.1	0.1	0.1		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (K-ft)	215.0	204.0	174.0		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (K-ft)	387	367	313		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			312.5 K-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.95		Data Sheet
3 FACTORED RESISTANCE 1 x 2			296.8 K-ft		= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	31 K	47 K	61 K		

6. STEEL STRINGER

DATA SHEET

BRIDGE: Steel Stringer	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Stringer BY:	REF.
I TRAFFIC		
1 ADT	400	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 ___ Estimated from ADT X	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal X	
4 LIVE LOAD FACTOR	1.4 X 1.6 ___ 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH X	
2 CONDITION OF WEARING SURFACE	SMOOTH X ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 X 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL X - redundant X - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE ___ SOME X HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED X $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.85	FIGURE 3 TABLE 3

COMMENTS

$Z_x = 37.82\ in^3$ (10% losses; 1/16" from both flanges)

$F_y = 33$ ksi

$R_n = 104$ k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

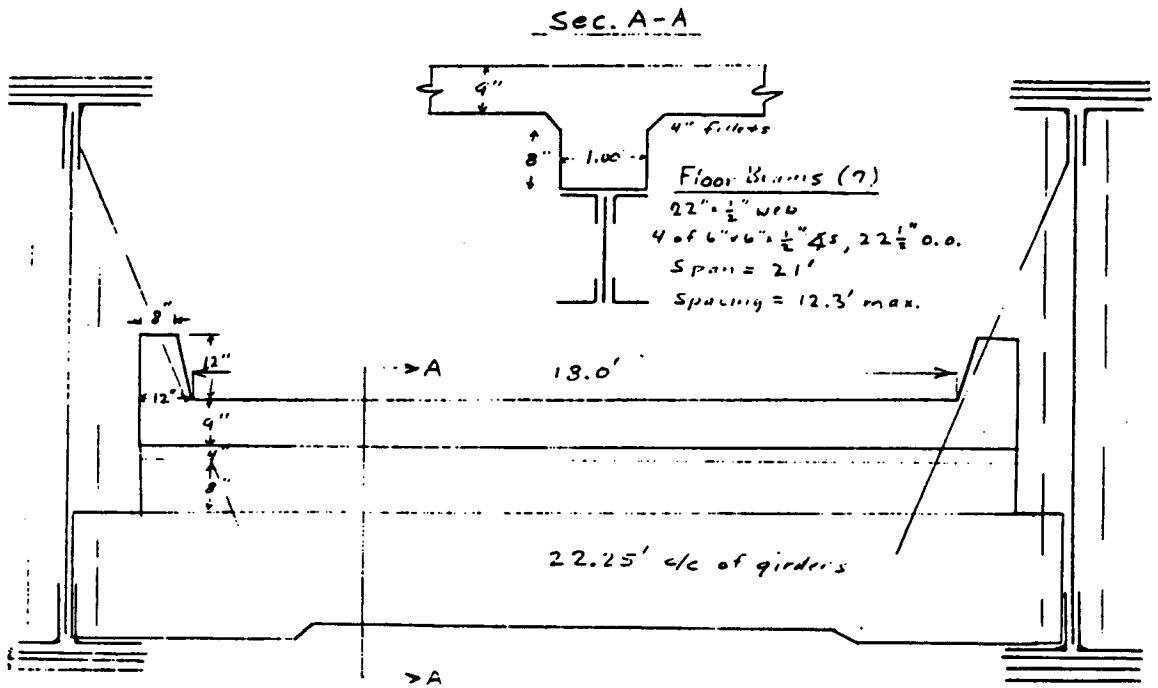
BRIDGE: Steel Stringer			ELEMENT: Stringer			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	48k	34k	76k	54k	92k	64k
PROPOSED	49k		78k		95k	

COMMENTS: Even though the section is corroded, ratings reach existings operating levels because of low traffic

RATING SHEET

BRIDGE: Steel Stringer	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Stringer	FORCE: Bending/Span = 21.9′				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 1.8 k-ft FROM CORES ___		5.04 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS ≤ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	2.6 k-ft		6.05 k-ft		
TOTAL	8.65 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (k-ft)	38.6	34.9	31.8		3.3.3.2
2 DISTRIBUTION FACTOR	1.26	1.26	1.26		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	___	___	___		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	58.4	52.9	48.1		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.4	1.4	1.4		TABLE 2
7 FACTORED LIVE LOAD EFFECT K-ft	81.8	74.1	67.3		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			104 k-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.85		Data Sheet
3 FACTORED RESISTANCE 1 x 2			88.4 k-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	49k	78k	95k		

7. STEEL GIRDER



96" × 7/16" web

3 of 16 1/2" × 11/16" plates top and bottom.

4 of 8" × 8" × 3/4" ∠s, 47" out to out

Other items are OK by computer [illegible].

DATA SHEET

BRIDGE: Steel Girder	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Main Girders BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH x ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH ___ ROUGH x	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL x - redundant ___ - non-redundant x	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT x $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.75	FIGURE 3 TABLE 3

COMMENTS

F_y = 33 ksi

Z_x = 6497.6 in^3

R_n = 17868.4 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Girder			ELEMENT: Main Girder			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	454k	308k	568k	386k	642k	436k
PROPOSED	320k		401k		453k	

COMMENTS: Even with non-redundancy and poor maintenance, the ratings are very high indicating the conservatism of the original design. However, existing inventory ratings are comparable to the proposed ratings.

RATING SHEET

BRIDGE: Steel Girder	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Main Girders	FORCE: Bending/Span = 82.67'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 70.7 k-ft FROM CORES ___		1774.5 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	101.9 k ft		2129.4 k-ft		
TOTAL	2231.3 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	908.3	1043.7	1026.5		3.3.3.2
2 DISTRIBUTION FACTOR	AASHTO Maintenance Manual				3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	Plate 3				TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT	1090.0	1252.4	1231.8		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT	1744.0	2003.8	1970.9		= C_i
III RESISTANCE			17868.4 k-ft		
1 CALCULATED ELEMENT RESISTANCE R_n			0.75		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			13401.3 k-ft		Data Sheet
3 FACTORED RESISTANCE 1 x 2					= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	320k	401k	453k		

DATA SHEET

BRIDGE: Steel Girder	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beam BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH x ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH ___ ROUGH x	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL x - redundant ___ - non-redundant x	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT x $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.75	FIGURE 3 TABLE 3

COMMENTS

F_y = 33 ksi

Z_x = 280.6

R_n = 771.7 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Steel Girder			ELEMENT: Floor Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	88k	50k	132k	88k	170k	113k
PROPOSED	70k		106k		136k	

COMMENTS: Proposed ratings are midway between operating and inventory ratings because of non-redundancy and 0.2 impact.

RATING SHEET

BRIDGE: Steel Girder	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beam	FORCE: Bending/Span = 21'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 4.1 k-ft FROM CORES ___		93.1 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS & 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	5.9 k-ft		111.7 k-ft		
TOTAL	117.6 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT k-ft	171.5	163.0	141.3		3.3.3.2
2 DISTRIBUTION FACTOR	AASHTO Maintenance Manual				3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	Plate 3				TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	205.8	195.6	169.6		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft	329.3	312.9	271.4		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			771.7 k-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.75		Data Sheet
3 FACTORED RESISTANCE 1 x 2			578.8 k-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	70k	106k	136 k		

8. THRU-PLATE GIRDER

DATA SHEET

BRIDGE: Thru-Plate Girder	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Floor Beams BY:	REF.
I TRAFFIC		
1 ADT	≈ 1300	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL x - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT x $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.90	FIGURE 3 TABLE 3

COMMENTS

$F_y = 30$ ksi

$Z_x = 136.9$ in^3

$R_n = 342.3$ k-ft

RATING SHEET

BRIDGE: Thru-Plate Girder	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Floor Beam	FORCE: Bending/Span = 23.5'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 34.5 k-ft FROM CORES ___		87.7 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS ≅ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	49.7 k-ft		105.2 k-ft		
TOTAL	154.9 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (k-ft)	204	183.7	168		3.3.3.2
2 DISTRIBUTION FACTOR	simply	supported	analysis		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR					TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	244.8	220.4	201.6		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft)	391.7	352.6	322.6		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			342.3 k-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.90		Data Sheet
3 FACTORED RESISTANCE 1 x 2			308.1		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	20k	32k	38k		

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: Thru-Plate Girder			ELEMENT: Floor Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	18k	8k	30k	12k	36k	14k
PROPOSED	20k		32k		38k	

COMMENTS: Proposed ratings are higher than operating because traffic is enforced at this site.

DATA SHEET

BRIDGE: Thru-Plate Girder	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Girder BY:	REF.
I TRAFFIC		
1 ADT	≈ 1300	
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED ___ STEEL x - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE ___ SOME x HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED x $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT x $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.85	FIGURE 3 TABLE 3

COMMENTS

F_y = 30 ksi

Z_x = 665 in^3

R_n = 1662.5 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

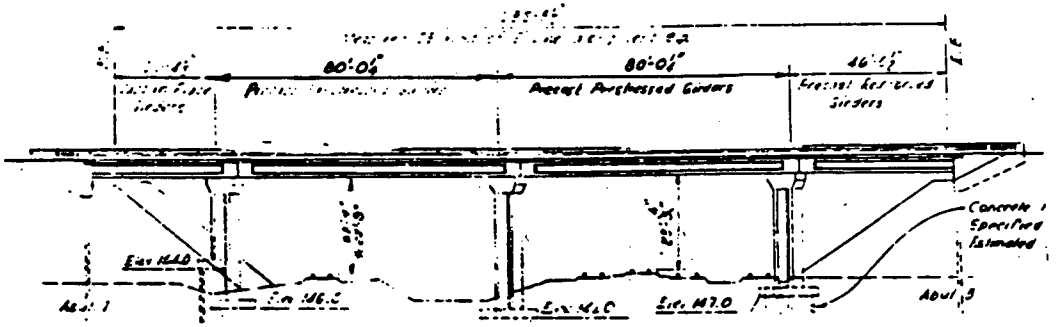
BRIDGE: Thru-Plate Girder			ELEMENT: Girder			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	58k	26k	84k	38k	106k	46k
PROPOSED	37k		57k		72k	

COMMENTS: Proposed rating is below operating due to deteriorated girder, intermittent maintenance and uneven deck approach

RATING SHEET

BRIDGE: Thru-Plate Girder	FILE:		DATE:		
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Girder	FORCE: Bending				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 161.8 k-ft FROM CORES ___		488.4 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS & 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	233.0 k-ft		586.1 k-ft		
TOTAL	819.1 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	417.4	389.1	345.8		3.3.3.2
2 DISTRIBUTION FACTOR					3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR					TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	500.9	466.9	415.0		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT	801.4	747.0	664.0		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			1662.5 k-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.85		Data Sheet
3 FACTORED RESISTANCE 1 x 2			1413.1 k-ft		= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	37k	57k	72k		

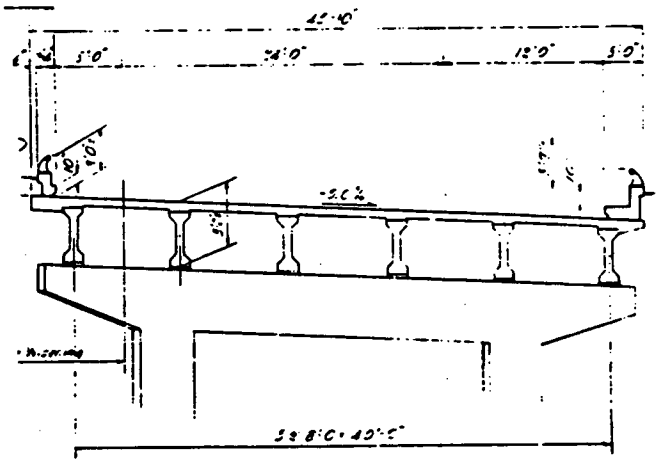
9. PRESTRESSED CONCRETE STRINGER



PRECAST
PRESTRESSED
GIRDERS

SPAN LENGTH ℄ to ℄
BRG. BRG.

= 79'-4" − 8" = 78'-8"
= 78.67'



TYPICAL SECTION

DATA SHEET

BRIDGE: P/S Concrete Stringer	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Stringer BY:	REF.
I TRAFFIC		
1 ADT		
2 ADT (if available)	Less than 2000 x Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks x Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 x 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED x STEEL ___ - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS M_u = 3251 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: P/S Concrete Stringer			ELEMENT: Stringer			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	91k	93k	115k	118k	132k	134k
PROPOSED	75k		95k		108k	

COMMENTS: The following formulas were used by the rating agency for interpreting rating for existing AASHTO; Consequently inventory may exceed operating by these equations.

$$R.F.\ [WSD\ (Operating)] = \frac{0.75\ M_u - D}{M_{LL+I}}$$

$$R.F.\ [WSD\ (Inventory)] = \frac{f_{all} - f_{DL} \pm f_{prestress}}{f_{LL+I}}$$

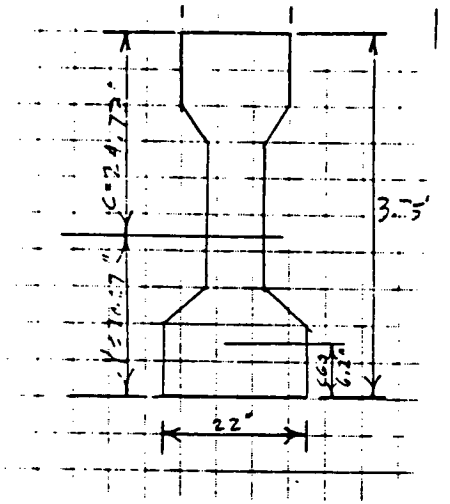
Minimum value at different levels of section chosen

Proposed ratings are lower because of the 0.2 impact and unenforced traffic. Any type of weight enforcement and smoothness of roadway would lead to higher ratings.

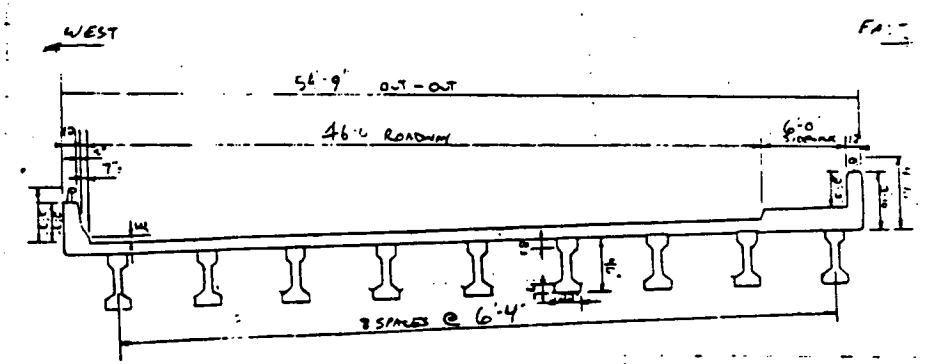
RATING SHEET

BRIDGE: P/S Concrete Stringer	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Stringer	FORCE: Bending/Span = 78.67'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ FROM CORES ___]None		956 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT			1147.2 k-ft		
TOTAL	1147.2 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	415.9	470.3	458.6		3.3.3.2
2 DISTRIBUTION FACTOR	1.45	1.45	1.45		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	—	—	—		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	723.6	818.4	798.0		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft)	1302.5	1473.1	1436.4		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	3251 k-ft				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.95				Data Sheet
3 FACTORED RESISTANCE 1 x 2	3088 k-ft				= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	75k	95k	108k		

10. PRESTRESSED CONCRETE I-BEAM



45" x 22" (TYPE III)



SECTION
NTS

DATA SHEET

BRIDGE: P/S Concrete I Beam	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: I-Beam BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED x STEEL ___ - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS

M_u = 2610 k-ft (AASHTO 9.17.3)

COMPARISON OF PROPOSED AND EXISTING RATINGS

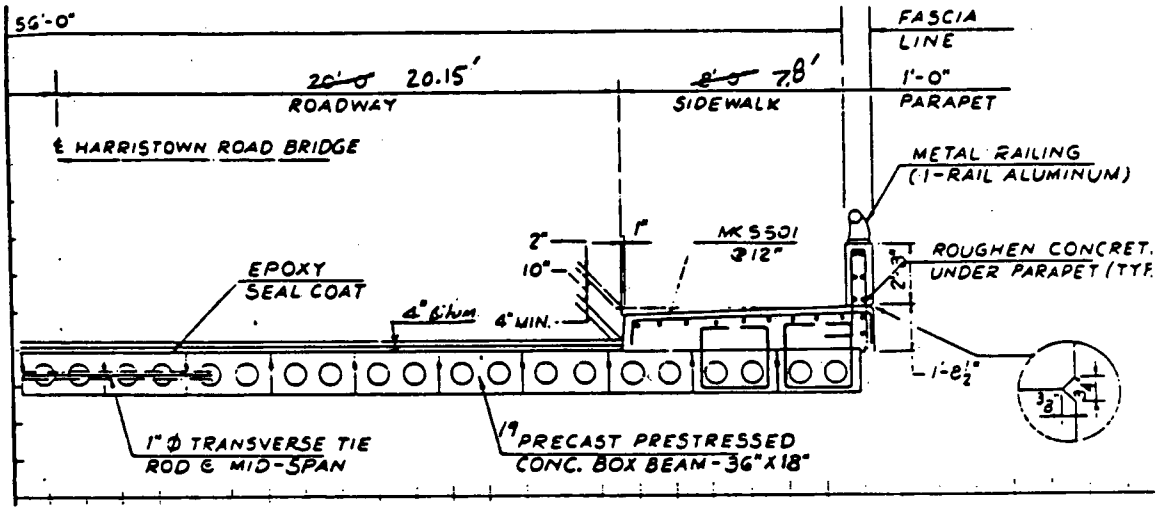
BRIDGE: P/S Concrete I Beam			ELEMENT: I-Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	68k	90k	94k	126k	100k	134k
PROPOSED	78k		100k		116k	

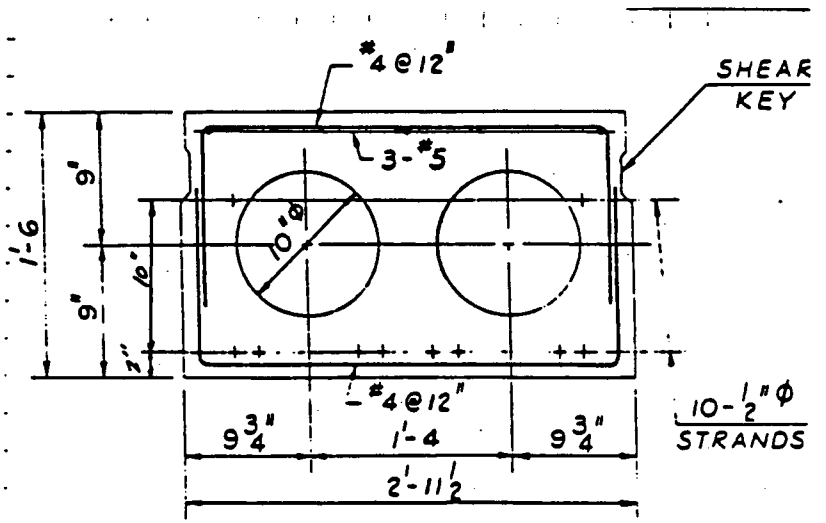
COMMENTS: Proposed rating may be improved with better deck approach. Inventory ratings greater than operating because of different formulae used.

RATING SHEET

BRIDGE: P/S Concrete I Beam	FILE:		DATE:		
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: I-Beam	FORCE: Bending/Span = 75.25'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ FROM CORES ___]None		932 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT			1118.4 k-ft		
TOTAL	1118.4 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (K-ft)	396.9	448.1	430.6		3.3.3.2
2 DISTRIBUTION FACTOR	1.14	1.14	1.14		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR					TABLE 1
4 IMPACT FACTOR	___	___	___		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	0.2	0.2	0.2		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft)	868.8	980.8	942.4		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	2610 k-ft				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.95				Data Sheet
3 FACTORED RESISTANCE 1 x 2	2479.5 k-ft				= B
$R.F._i = \dfrac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$		
	78k	100k	116k		

11. P/S CONCRETE BOX BEAM





SECTION B-B
SCALE: 1" = 1' - 0

DATA SHEET

BRIDGE: P/S Box Beam	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Box Beam BY:	REF.
I TRAFFIC		
1 ADT		
2 ADIT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED x STEEL ___ - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE ___ SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED x $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.90	FIGURE 3 TABLE 3

COMMENTS

M_u = 267.6 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

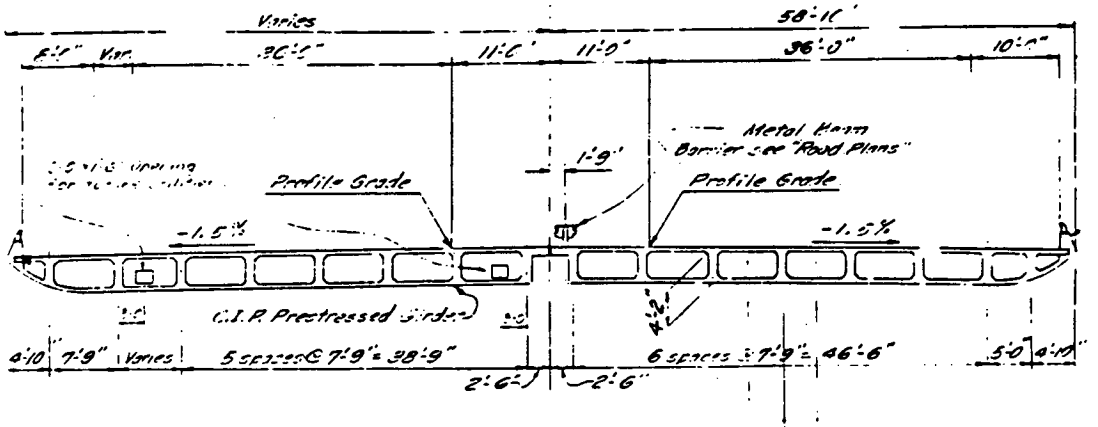
BRIDGE: P/S Concrete Box Beam			ELEMENT: Box Beam			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	90	50	134	74	178	98
PROPOSED	65		103		130	

COMMENTS: Proposed rating would increase if either approach condition was improved and/or maintenance level was higher

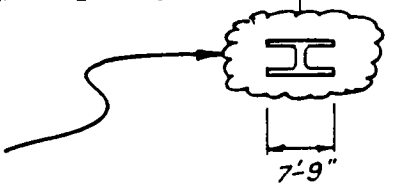
RATING SHEET

BRIDGE: P/S Box Beam	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Box Beam	FORCE: Bending/Span = 32.14'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 18.1 k-ft FROM CORES ___		94.3 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS $\cong$ 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT	26.1 k-ft		113.2 k-ft		
TOTAL	139.3 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT (k-ft)	68.3	61.5	54.3		3.3.3.2
2 DISTRIBUTION FACTOR	0.6	0.6	0.6		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	—	—	—		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	49.2	44.3	39.1		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft)	78.7	70.9	62.6		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	267.6 k-ft				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.9				Data Sheet
3 FACTORED RESISTANCE 1 x 2	240.8 k-ft				= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	65k	103k	130k		

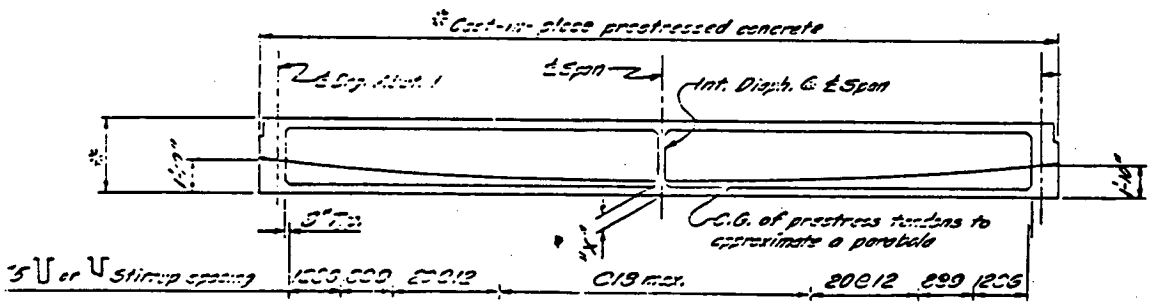
12. PRESTRESSED CONCRETE BOX GIRDER



TYPICAL SECTION
SCALE: 1/8" = 1' - 0"



TYPICAL GIRDER



LONGITUDINAL SECTION
NO SCALE

DATA SHEET

BRIDGE: P/S Concrete Box Girder	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: Box Girder BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 x Greater than 2000 ___ Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks x Normal ___	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 ___ 1.8 x 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH x ROUGH ___	
2 CONDITION OF WEARING SURFACE	SMOOTH ___ ROUGH x	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED x STEEL ___ - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS M_u = 6909 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: P/S Concrete Box Girder			ELEMENT: Box Girder			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	133k	76k	166k	88k	166k	95k
PROPOSED	111k		129k		139k	

COMMENTS:

$$\text{R.F. (Operating)} = \frac{0.75\, M_u - D}{M_{LL+I}}$$

$$\text{R.F. (Inventory)} = \frac{f_{allowable} - f_{DL} + f_{Prestress}}{f_{LL+I}}$$

Ratings can be improved by enforcement and by a smooth roadway surface.

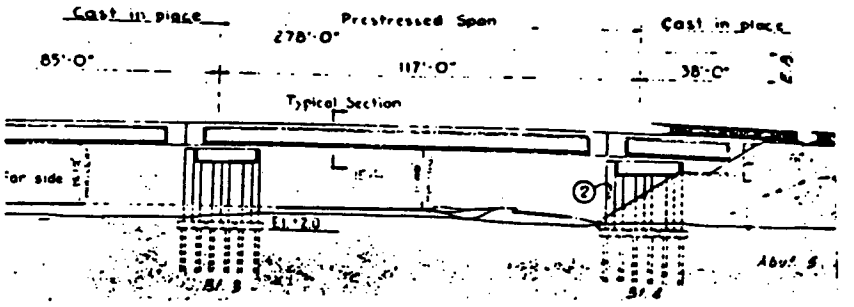
RATING SHEET

BRIDGE: P/S Concrete Box Girder	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: Box Girder	FORCE: Bending/Span = 113.8'				
I DEAD LOAD FORCES	ASPHALT		REMAINING		
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS ___ None FROM CORES ___		2671 k-ft		
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS - 1.44 FROM CORES - 1.20		1.2		3.3.2.1
3 FACTORED D.L. ON ELEMENT			3205.2 k-ft		
TOTAL	3205.2 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	633.3	783.8	807.2		3.3.3.2
2 DISTRIBUTION FACTOR	1.11	1.11	1.11		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	___	___	___		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	843.6	1044.0	1075.2		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.8	1.8	1.8		TABLE 2
7 FACTORED LIVE LOAD EFFECT	1518.5	1879.2	1935.4		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n			6909 k-ft		3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ			0.95		Data Sheet
3 FACTORED RESISTANCE 1 x 2			6564 k-ft		= B

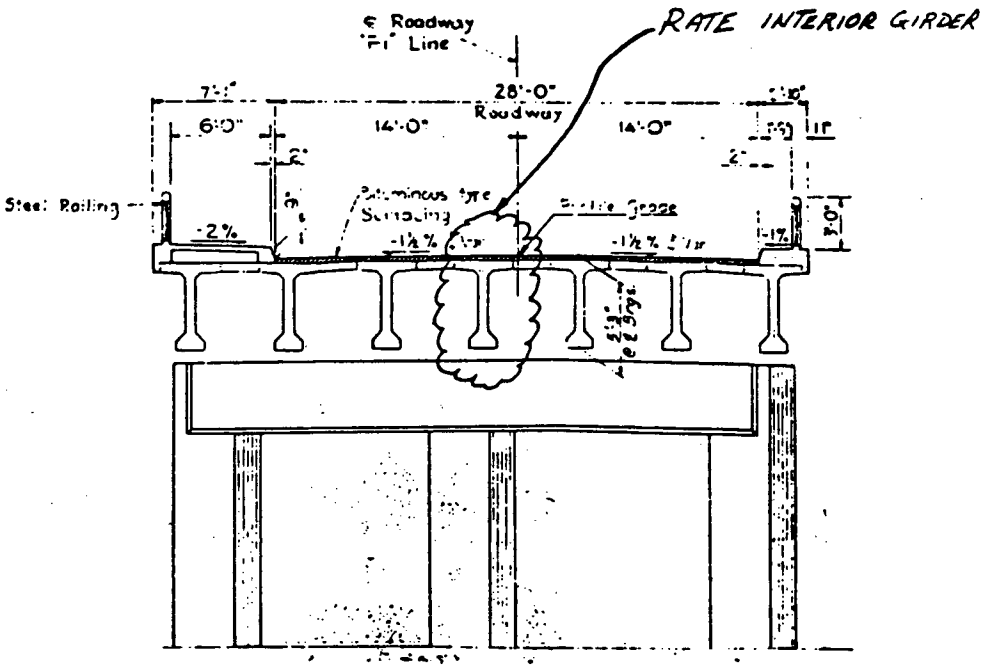
$$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$$

IV POSTING LOAD	VEHICLE 1 $R.F._1 \times 50$	VEHICLE 2 $R.F._2 \times 72$	VEHICLE 3 $R.F._3 \times 80$
	111	129	139

13. PRESTRESSED CONCRETE T-BEAM



SPAN LENGTH ℄ BRG to ℄ BRG = 115' - 6"



DATA SHEET

BRIDGE: P/S Concrete T-Beam	FILE: DATE:	
ELEMENT DATA SHEET 1 OF 1	ELEMENT: T-Beam BY:	REF.
I TRAFFIC		
1 ADT		
2 ADTT (if available)	Less than 2000 ___ Greater than 2000 x Estimated from ADT ___	
3 OVERLOADS	Frequent Heavy Trucks ___ Normal x	
4 LIVE LOAD FACTOR	1.4 ___ 1.6 x 1.8 ___ 1.95 ___	TABLE 2
II IMPACT		
1 CONDITION OF APPROACHES	SMOOTH ___ ROUGH x	
2 CONDITION OF WEARING SURFACE	SMOOTH x ROUGH ___	
3 IMPACT FACTOR	0.1 ___ 0.2 x 0.3 ___	3.3.2.3
III ELEMENT CONDITION		
1 TYPE OF MEMBER	PRESTRESSED x STEEL ___ - redundant x - non-redundant ___	TABLE 3 I
2 SUPERSTRUCTURE CONDITION NUMBER		TABLE 3 II
3 DETERIORATION	NONE x SOME ___ HEAVY ___ $\phi - 0.1$ $\phi - 0.2$	TABLE 3 II
4 LOSS OF SECTION	ESTIMATED ___ MEASURED ___ $\phi = \phi + 0.0$ $\phi = \phi + 0.05$	TABLE 3 III
5 QUALITY OF MAINTENANCE	VIGOROUS ___ INTERMITTENT ___ $\phi = \phi + 0.05$ $\phi = \phi - 0.05$	TABLE 3 IV
6 RESISTANCE FACTOR ϕ	0.95	FIGURE 3 TABLE 3

COMMENTS

M_u = 5125 k-ft

COMPARISON OF PROPOSED AND EXISTING RATINGS

BRIDGE: P/S CONCRETE BEAM			ELEMENT: T-BEAM			
METHOD	3 (50K)		3S2 (72K)		3-3 (80K)	
	OPERATING	INVENTORY	OPERATING	INVENTORY	OPERATING	INVENTORY
EXISTING	125k	73k	144k	84k	155k	91k
PROPOSED	113k		131k		141k	

COMMENTS: Proposed ratings can be improved by having a smooth roadway surface.

RATING SHEET

BRIDGE: P/S Concrete T-Beam	FILE:			DATE:	
ELEMENT RATING SHEET 1 OF 1	BY:				REF:
ELEMENT: T-Beam	FORCE: Bending/Span = 115.5'				
I DEAD LOAD FORCES	ASPHALT			REMAINING	
1 CALCULATED D.L. FORCE ON ELEMENT	FROM DRAWINGS 286.8 k-ft FROM CORES ___			1584.2 k-ft	
2 DEAD LOAD FACTOR γ_D	FROM DRAWINGS X 1.44 FROM CORES - 1.20			1.2	3.3.2.1
3 FACTORED D.L. ON ELEMENT	413 k-ft			1901 k-ft	
TOTAL	2314 k-ft				= A
II LIVE LOAD FORCES	VEHICLE 1 (3)	VEHICLE 2 (3S2)	VEHICLE 3 (3-3)	LANE LOAD (if applicable)	
1 CALCULATED LIVE LOAD EFFECT	645.2	806.8	825.3		3.3.3.2
2 DISTRIBUTION FACTOR	0.916	0.916	0.916		3.3.3
3 CORRECTION TO DISTRIBUTION FACTOR	___	___	___		TABLE 1
4 IMPACT FACTOR	0.2	0.2	0.2		3.3.2 Data Sheet
5 LIVE LOAD EFFECT + IMPACT (k-ft)	709.2	880.8	907.2		1x2x3x (4 + 1.0)
6 LIVE LOAD FACTOR γ_L	1.6	1.6	1.6		TABLE 2
7 FACTORED LIVE LOAD EFFECT (k-ft)	1134.7	1409.3	1451.5		= C_i
III RESISTANCE					
1 CALCULATED ELEMENT RESISTANCE R_n	5125 k-ft				3.3.2.4
2 CAPACITY REDUCTION FACTOR ϕ	0.95				Data Sheet
3 FACTORED RESISTANCE 1 x 2	4868.8 k-ft				= B
$R.F._i = \frac{\phi R_n - \gamma_D D}{\gamma_L L_n (1 + I)} = (B-A)/C_i$					
IV POSTING LOAD	VEHICLE 1 $R.F._1$ x 50	VEHICLE 2 $R.F._2$ x 72	VEHICLE 3 $R.F._3$ x 80		
	113	131	141		

As part of the discussion on redundancy, the project sought to identify cases of nonredundant concrete spans. One such example is trough-shaped railroad bridges introduced by the California Division of Highways in 1965. Design of the Cordelia underpass substituted a trough-shaped section characterized by somewhat massive, 3-ft thick slabs and webs for the deep steel trusses and heavy, riveted steel plate girders used previously.

Advantages of the superstructure configuration are evident. Its smooth profile is aesthetically superior to the clutter of truss members or the multitude of crimped stiffeners, angles, rivets, and splice plates characterizing steel structures. The concrete trough is readily adaptable to curved alignment. It is not vulnerable to the extended closures that may result when derailment with the confinements of steel structures destroy the inner knee braces; moreover, it is more likely to confine a derailed railroad car than is a deck structure. Initial cost of such a structure is less than comparable to riveted or welded steel structures, and, lacking painted steel components, it is certainly more maintenance free. Additionally, this structure type allows crossings to be provided with a minimum of embankment fill costs. California, at the time of this writing, has eight trough-shaped concrete railroad structures open to traffic. Listed in order of time of completion, they are:

1. Cordelia Underpass, Cordelia
2. R Street Underpass, Sacramento
3. Antler Underpass, Lakehead
4. F Street Underpass, Chula Vista
5. Reed Avenue Underpass, Broderick
6. Land Park Underpass, Sacramento
7. North Auburn Underpass, Auburn
8. South Los Angeles Underpass, Los Angeles

Sketches of five of these are shown in Figures C-3(a) through C-3(e).

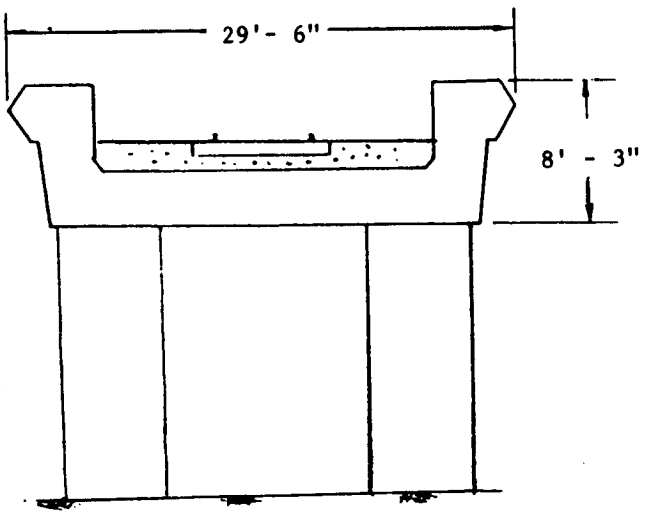


Figure C-3(a). "F" Street underpass.

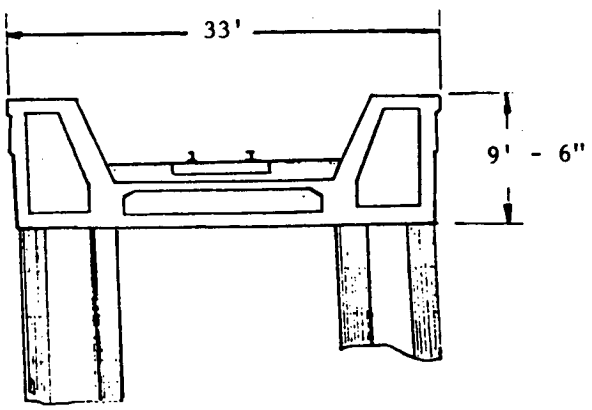


Figure C-3(b). South Los Angeles underpass.

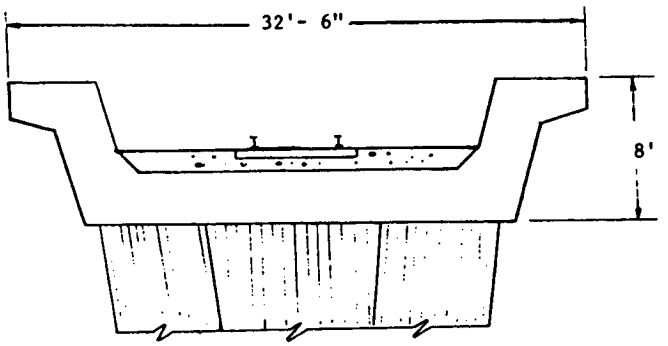


Figure C-3(c). Antler underpass.

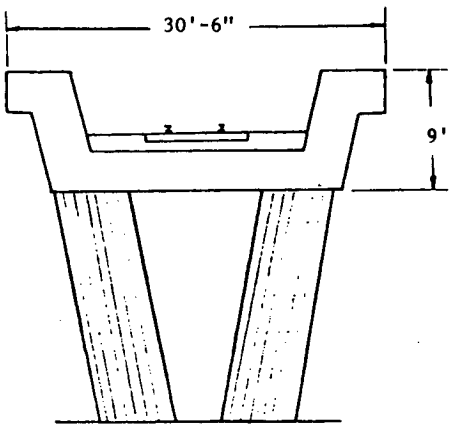


Figure C-3(d). Cordelia underpass.

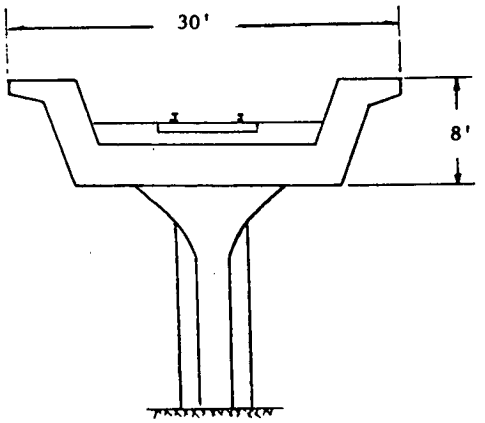


Figure 3-C(e). "R" Street underpass.

APPENDIXES D, E, AND F

UNPUBLISHED MATERIAL

Appendixes D, E, and F contained in the report as submitted by the research agency are not published herein. Their titles are listed here for the convenience of those interested in the subject area. Qualified researchers may obtain loan copies of the agency report by written request to the NCHRP, Transportation Research Board Publications Office, 2101 Constitution Avenue, NW, Washington, D.C. 20418.

The titles are:

Appendix D—Review of WIM Data
Appendix E—Mathematical Aspects of Reliability Theory
Appendix F—Detailed Illustration of Betas

APPENDIX G

SIMPLIFIED METHODS FOR LIVE LOAD DISTRIBUTION

The proposed evaluation specifications contain options for conducting more accurate live load distribution analyses, e.g., using charts or tabulated values or using more accurate formulas. This appendix gives a summary of recently developed methods that are seen to give more accurate results than the present AASHTO specifications with a minimal increase in the amount of time and effort. Most of these methods have been developed during the course of NCHRP Project 12-26, which also identifies some more complicated methods that can be used. Another method as proposed by Bakht and Moses is also reviewed.

METHODS REVIEWED IN NCHRP PROJECT 12-26

A large number of formulas have been reviewed and tested on a data base of bridges assembled in NCHRP Project 12-26, "Distribution of Wheel Loads on Highway Bridges." An excerpt of the simple and more accurate formulas is presented below. For further details, the reader is directed to Ref. *36*.

The first formula was developed at the University of Illinois and is given as $g = S/Q$, where g = wheel load distribution factor, S = girder spacing, in ft; $Q = (0.01538 + S/150)(L/H^{0.5}) + 4.26 + S/30$; L = span length, in ft; $H = E_g I_{cg}/LD$; E_g = modulus of elasticity of girder; I_{cg} = moment of inertia of composite girder (transformed to girder properties); $D = E_s t^3_s/12(1 - \nu^2_s$; E_s = modulus of elasticity of the slab; t_s = slab thickness; and ν_s = Poisson's ratio of slab.

As explained in Ref. *36*, the only drawback of this method is that it gives errors on the unconservative side for a few isolated cases.

The other method was developed during the course of NCHRP Project 12-28 (6) and is identified as IAI-4. The girder distribution (applied to a wheel line) is given by

$$g = 0.1 + (S/3)^{0.6} (S/L)^{0.2} [(1 + Ae^2)/Lt^3_s)]^{0.1}$$

where I = moment of inertia of girder only (transformed to slab material); A = area of girder (transformed to slab material); and e = distance from neutral axis of girder to mid-depth of slab (eccentricity).

Other methods identified are those developed by Sanders, Henry and at Lehigh. However, as shown in Ref. *36*, results from these are not as accurate as those given above.

Another method has been recently developed by Walker (*51*) by AISI Project 332 entitled, "Lateral Load Distribution for Multi-Girder Bridges." Three analytical models have been implemented in this study and the applicability and limitations of the AASHTO S/D rule have been explored. The use of a grid model to represent the essential predictions of the more exact analytical models has been explored. The grid model is seen as a useful option for a lateral load distribution provision in the specification. A simple microcomputer implementation of a grid-model-based provision is seen to be a workable possibility.

BAKHT AND MOSES METHOD

This method (*41*) is applicable to slab on girder bridges with little or no skew. It has been derived from further simplifications of the Ontario methods. The formula retains the essential simplicity of the AASHTO distribution method; however, it gives better results because it accounts for the major parameters affecting bridge response. It is given as $g = S/D_d$, where $D_d = D[1 + (\mu C_f + C_e)/100]$, in which:

$\mu = (W_e - 11)/2 < 1.0$;
W_e = width of design lane, ft;
C_f = correction factor for design lane width;

C_e = correction factor for edge stiffening and vehicle edge distance;
S = spacing of girders, ft;
D = basic value.

This approximation was developed based on the same principles as used in the *Ontario Highway Bridge Design Code,* OHBDC, i.e., orthotropic plate theory. The application of this method requires determining the value of D depending on the span of the bridge, the type of girder (external or internal), and the number of lanes from Figure G-1. Equivalent spans for continuous bridges are shown in Figure G-2. Correction factors are then applied to this basic value of D to obtain the design value D_d. These correction factors also depend on the span and the vehicle edge distance and are obtained from Figures G-1 and G-3.

The validity of this method was established by comparing the results to those obtained by the rigorous finite-element analyses of several actual bridges. For example, consider a right single span bridge with steel girders (*41*), having a span, L, of 110 ft and a lane width of, W_d, of 14 ft. The center-to-center spacing of the girders is 8.5 ft. It is required to determine the design value of D, i.e., D_d, for moments in internal girders corresponding to the strength design. From Figure G-1, the values of D for internal girders and C_f are found to be approximately 5.8 ft and 8 percent respectively. The value of μ is found to be 1.5. Substituting the values of D, C_f, and μ in the equation above gives D_d to be equal to 6.6 ft. It is noted that this design value of D is 20 percent smaller than the corresponding AASHTO value which is equal to 5.5 ft. A rigorous finite element study for the same bridge (*36*) gives a D value of 7.0 ft, which is a deviation

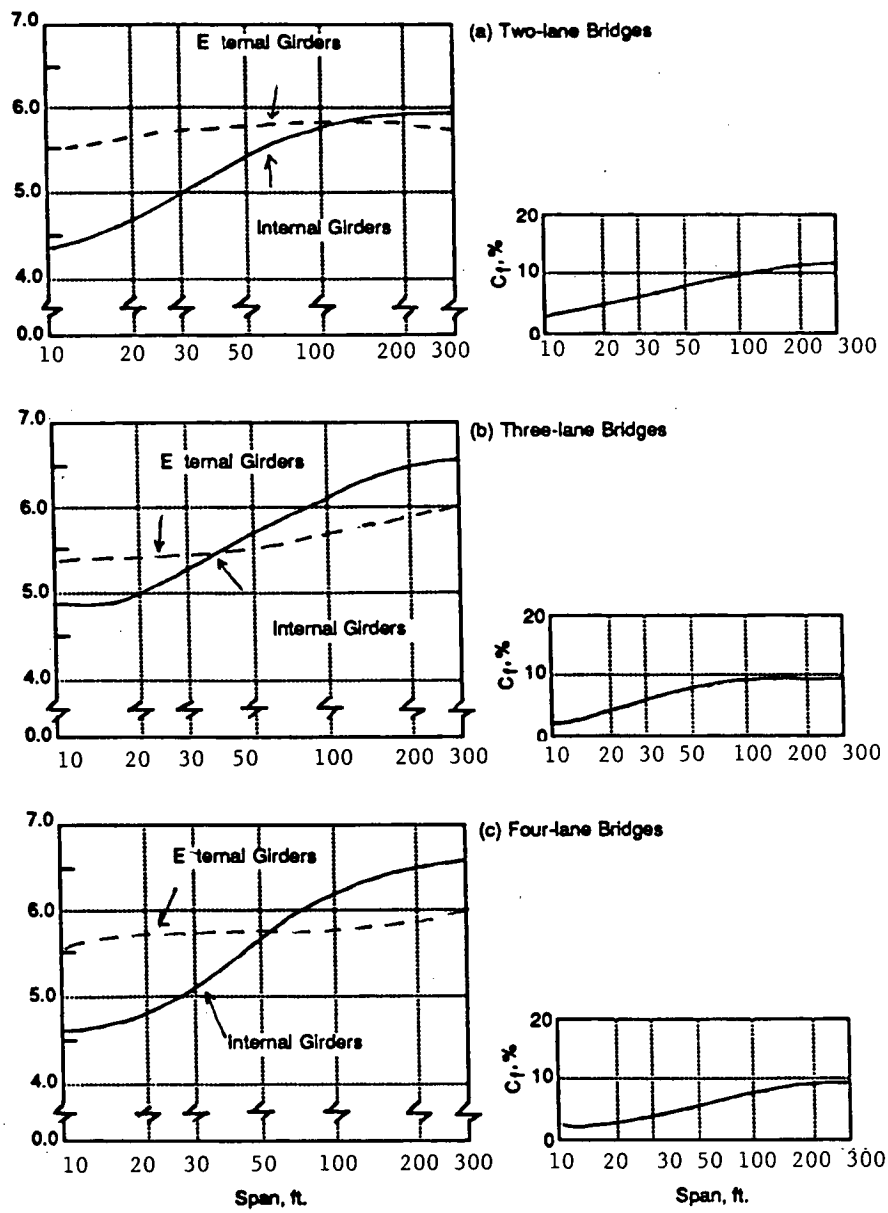


Figure G-1. D and C_f values for slab on girder bridges.

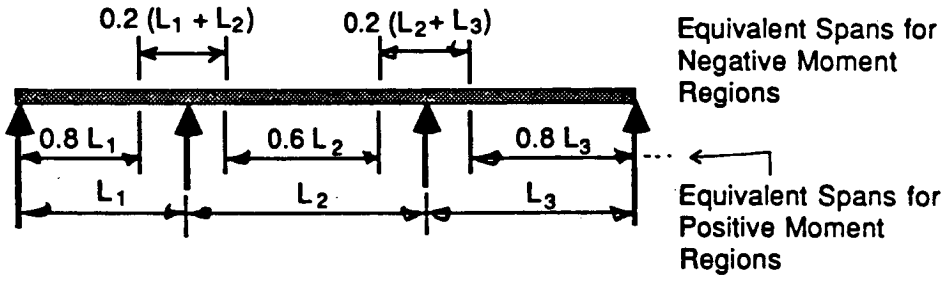
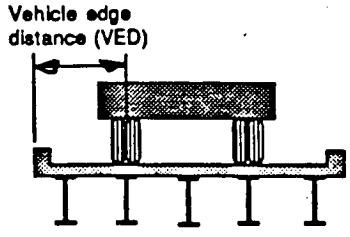


Figure G-2. Equivalent spans for continuous spans.



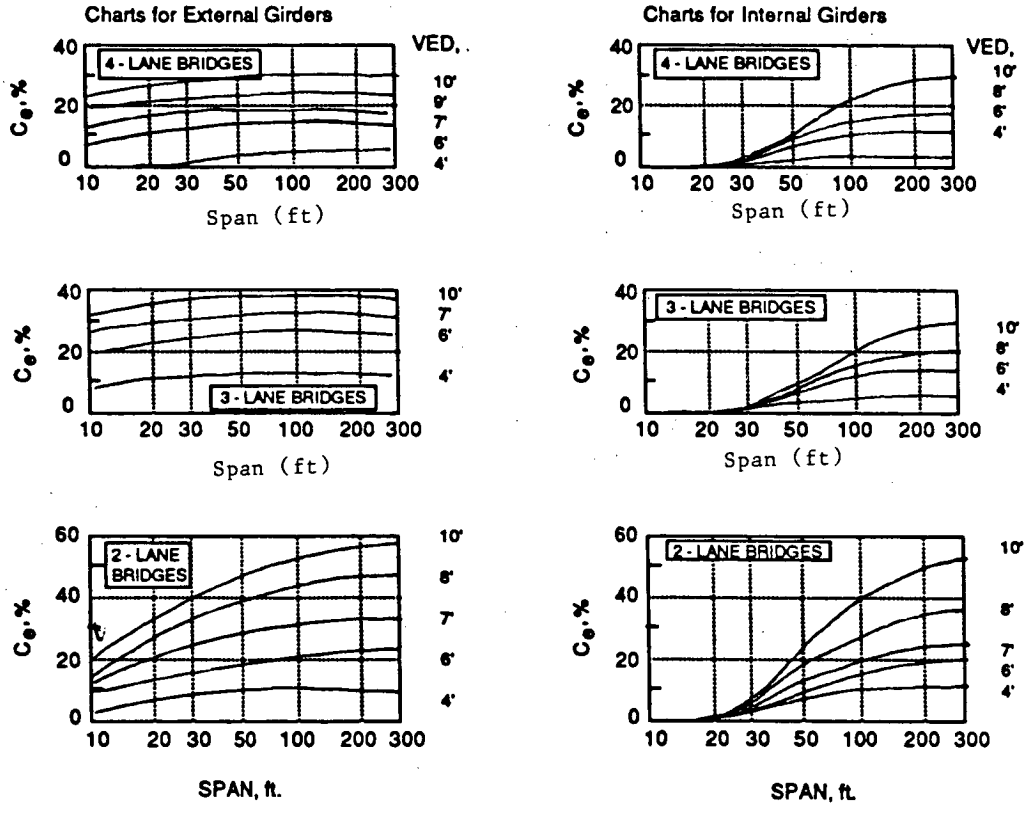


Figure G-3. Values of C_e for governing load cases in slab-on-girder bridges.

of 6 percent from that obtained using the proposed procedure. Further examples and comparisons are also given in Ref. *41.*

CONCLUSIONS

The above two sections show that a variety of reasonably simple and more accurate methods (as compared to AASHTO) exist. An option for the use of such methods is provided in the proposed specifications, however, with certain documented correction factors that reinstate the bias implicit in current practices. Thus, with a small degree of effort, the evaluator may be able to obtain improved ratings, at the same time maintaining an adequate level of safety.

APPENDIX H

DETERIORATION OF STRUCTURAL MEMBERS

This appendix examines the effects of deterioration of members due to natural causes. Damage caused by accidents is specifically not considered (a methodology for evaluation and repair due to accidents is provided in *NCHRP Reports 226, 271,* and *280.*

ATMOSPHERIC CORROSION OF STEEL MEMBERS

To obtain some measure of the amount of degradation that can occur over a period of time it is necessary to collect data on the atmospheric corrosion of metals used in bridges (typically carbon or weathered steel). Fortunately, a large body of data is available in the form of tests conducted by ASTM over a large period of time and at sites all over the world. Unfortunately, deriving generalizations using these data is difficult.

At the outset of this discussion it should be realized that this is by no means an exhaustive account of the effects of the atmosphere on steel bridge members. Moreover, the aim is to examine trends in light of assumptions rather than to specify absolute numerical values because doing this would require an extensive amount of data.

Atmospheric corrosion of bridge members depends both on the macroenvironment, which refers to the general atmosphere at and around the bridge; and on the microenvironment, which refers to localized conditions, like leaky expansion joints, which might alter considerably the deterioration due to just the macroenvironment. This is also referred to as service corrosion.

Both the macroenvironment and the microenvironment play an important role in determining the deterioration of a bridge element, and it is the latter which compounds the difficulty of obtaining any generalization or conclusions from the experimental data collected.

Study of atmospheric corrosion of metals has been an important activity of ASTM and of many other private and public agencies. Information has been obtained on corrosion loss of numerous metal samples exposed at sites throughout the world.

ASTM Standard Practice G1 (*53*) establishes the methodology for establishing the corrosion effects of the atmosphere on test specimens. Results are established in terms of average corrosion penetration per side, which is calculated from the mass lost during the time of exposure.

These test specimens have been exposed to a variety of macroenvironments, broadly classified into rural, urban/industrial, and marine depending upon the aggressiveness of the environment to steel.

Rural environments are not very aggressive towards steels because of the absence of corrosive agents like carbon dioxide, sulfur oxides, and so on.

On the other hand, urban and industrial environments contain all of the foregoing which, along with moisture, promote the corrosion of steel.

The losses from corrosion are greatest in a marine environment because of salt spray, humidity, winds, and daily temperature fluctuations.

A measure of the magnitude of these losses in different environments can be obtained from the calibration of atmospheric test sites conducted by ASTM committee G-1 which annually exposed carbon steel specimens at a number of sites around the world. A compilation of their results is given in Table 19 in Ref. *27,* which summarizes the average corrosion penetration per side (in μm) for carbon steel specimens after 2-year exposure periods. Also given is the rating of each site relative to the corrosion penetration measured at State College, Pennsylvania, a rural site where the data were most nearly reproducible. It is apparent that a wide diversity in atmospheric corrosiveness exists between the various test sites. This can also be seen from Figure H-1 (*28*) which shows the range of corrosion (with a low of 1.1 mils/side/year for rural environments to a high of 40 mils/side/year for marine environments).

This ASTM study suggests that rural atmospheres are less corrosive than industrial atmospheres and much less corrosive than marine atmospheres. But as mentioned before, the microclimate may play a very important role in modifying this behavior. For our purposes corrosiveness will be classified into (1) severe (for marine atmosphere), (2) slight or medium (for urban/industrial atmospheres, and (3) light or normal (for rural atmospheres)

Such an explicit relation between the macroenvironment and

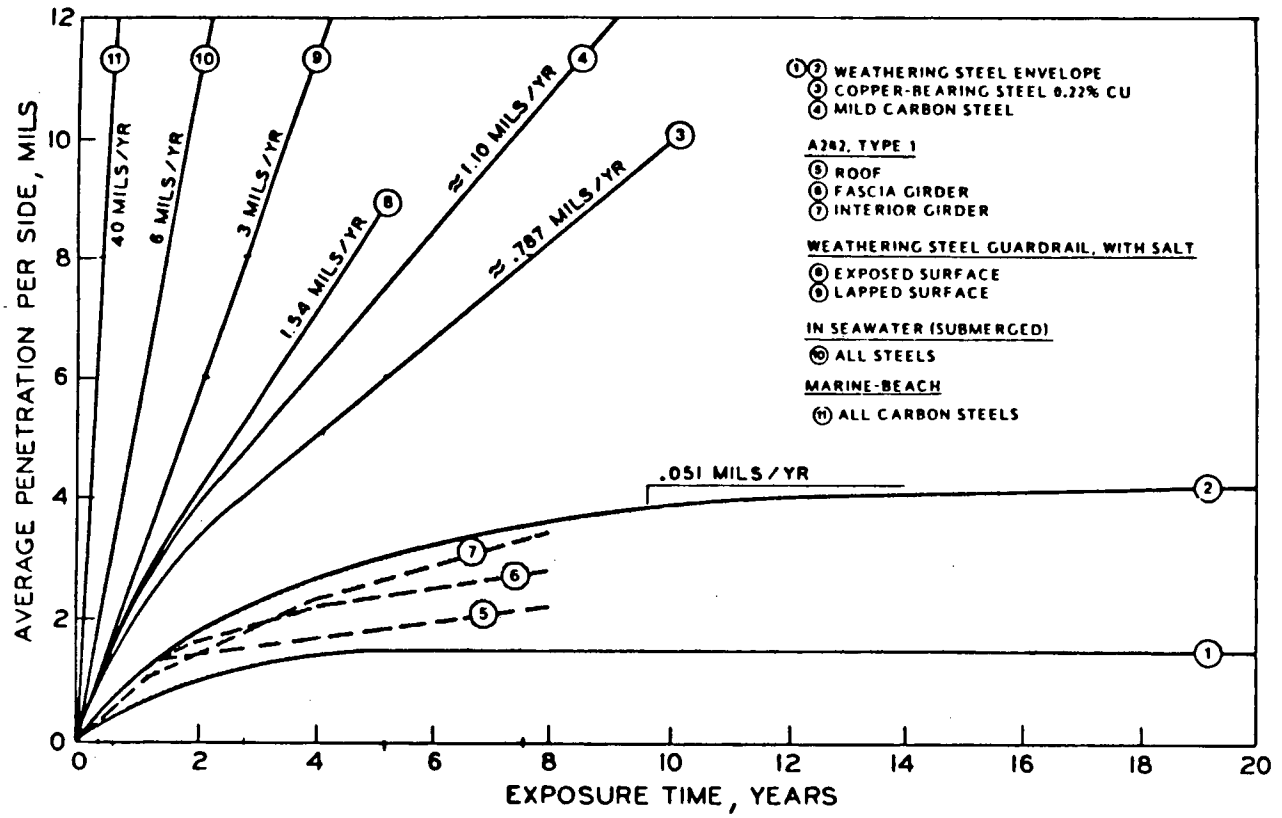


Figure H-1. Corrosion rate of different types of steel. (Ref. 28)

the amount of corrosion is also given by the West German specifications (Ref. *54*).

Albrecht (*27*) has conducted a regression analysis of the corrosion data from various sites and has fitted it to a power function given by

$$C = A\,t^{B} \qquad \text{(H-1)}$$

where C = average corrosion penetration in μm determined from weight loss, t = exposure time in years, A = regression coefficient numerically equal to the penetration after 1-year exposure, and B = regression coefficient numerically equal to the slope of Eq. H-1 in a log-log plot. Such an approach has also been followed by Bohnenkamp and Townsend.

Based on the ASTM data, the values for the constants in Eq. H-1 have been determined (*27*). Average values for carbon steel for different environments are:

Corrosion of Section	Type of Environment	Eq. H-1
Normal, good condition	Rural	$C=34t^{0.65}$
Medium, slight corrosion	Urban/industrial	$C=65t^{0.5}$
Severe corrosion	Marine	$C=80t^{0.8}$

These equations imply that, on the average, corrosion penetration per side will be of the order of 2, 3.5 and 5.5 mils for 2 years in rural, industrial, and marine environments respectively.

Using these averages as base values, the cumulative effect of the microclimate has to be determined as applicable to bridges for each of the above environments. These, in turn, will determine the statistical parameters for the resistance of deteriorated members.

The corrosion behavior of steels under localized or microenvironmental conditions is more typical of the service conditions encountered at bridge sites. The main effects are due to initial climate, sheltering, orientation, angle of exposure, time of wetness, atmospheric pollutants, and salt. Extensive data and subsequent analysis are given in Ref. *27*. Only a brief summary as relevant to the discussion will be provided here.

1. *Initial climatic condition*—This refers to the climatic condition at the beginning of the exposure period. Enough evidence exists to show that up to a twofold difference in corrosion penetration can exist, depending on when the steel was initially exposed.

2. *Shelter and orientation*—Tests conducted by Larrabee have shown that exposed (skyward) surfaces corrode less than sheltered (groundward) surfaces because of washing of the skyward surface by rain and also because of more rapid drying.

Tests done by Cosaboom and Zoccola confirm that shelter is detrimental to corrosion with the corrosion rates varying from 1.65 to 6 times that of boldly exposed specimens in industrial environments.

Other tests by McKenzie show that for rural environments, there is not much difference between open and bridge shelter environments. At marine sites, however, the opposite is true, indicating the adverse effect of chlorides under sheltered conditions.

As a rough estimate, the following values are adopted:

Environment	Corrosion for sheltered conditions / Corrosion for exposed conditions
Rural	1.0
Industrial	1.7
Marine	2.0

3. *Angle of exposure*—It is observed that under bold exposure, vertical and horizontal specimens corrode more than specimens exposed at an angle close to the latitude of the site.

Vertical specimens should corrode more than horizontal specimens in bold exposures, but not in sheltered exposure. Some data confirm these expectations; other data do not. Hence, this variable will not be taken into consideration.

4. *Continuously moist conditions*—Continuously moist conditions are seen to be very detrimental to steel elements. Tests by Larrabee have shown that under sheltered conditions, the corrosion under continuously wet conditions is about 3 times that in dry atmosphere which, in turn, is twice that of corrosion under bold exposures.

Continuously moist conditions (as opposed to wet conditions) are obtainable when the relative humidity exceeds a certain critical level. As developed by Sereda, the percentage of time this critical humidity is exceeded, called the "time of wetness", is a very important factor promoting the atmospheric corrosion of metals.

However, the time of wetness of bridges at different locations can be greatly influenced by local atmospheric and service conditions. There is at present no reliable method of estimating the time of wetness for various surface conditions from meteorological records.

5. *Deicing Salts*—Deicing salts spread on highways during the winter can create a more corrosive environment for the bridge than that encountered at marine sites. On heavily salted highways, the salt contaminates the steel structure in the following ways: (a) Salt water leaks through the bridge deck, mainly at expansion and construction joints, and drains longitudinally along sloped bottom flanges. (b) A mist of salt-water runoff is kicked up in the wake of trucks passing beneath the bridge and settles on the steel structure. (c) Dust containing salt particles is blown against the steel structure.

As noted by Albrecht and Naeemi (*27*), the corrosion due to salt is approximately 2.75 times the corrosion in the absence of salt. Because this factor is independent of the type of environment, the increase in corrosion due to salt will be taken as 2.75 for both rural and industrial environments. The effect of salt is also included for marine environments even though it may be implicitly present.

The foregoing discussion has established the base values for amount of deterioration and has also identified the major quantifiable factors that affect these base values. Using these estimates establishes the following comprehensive values for deterioration over a 2-year period as a function of environment (or condition of section)

Condition of Section	Amount of Thickness Loss per Side, mils
Good condition	6
Slight/mild corrosion	17
Heavy corrosion	30

Using these values one calculates the mean amount of reduction in section modulus for a typical beam, say a W 27×94 (assuming constant losses on all surfaces).

Condition of Section	% Reduction in Section Modulus (mean, 2-year period)
Good condition	1.8
Slight/mild corrosion	5.0
Heavy corrosion	9.0

Even for the severest corrosion, local buckling of either the flange or the web does not occur. Hence, the strength is governed by the reduction in section. Because the bias for a good condition section is taken to be 1.1, the bias for slight corrosion and severe corrosion equals the remaining percentage of section multiplied by the original bias, which is calculated as $0.95 * 1.1 = 1.05$ and $0.91 * 1.1 = 1.0$ respectively.

Another important point to note here is that for evaluation, one already has a certain record of the corrosion rate for a site, so extrapolating for a short period of 2 years is not as difficult a task as for a new bridge.

Establishing the coefficient of variation, COV, for deteriorated steel members is more difficult. As noted by McCrum et al. (*28*), the statistical analysis of any detailed measurement regarding corrosion losses in bridges is quite difficult because of the wide tolerances in initial starting sizes. Hence, relying on site data alone would lead to overly penalizing situations. Therefore, some subjective estimates are made at this point, the implications of which are examined in Chapter Four. Summarizing the values for bias and COV one gets:

	Bias	COV
New condition, steel members	1.10	12%
Partially corroded with some section loss	1.05	16%
Severe corrosion with considerable loss of section	1.00	20%

The increase in COV with increasing corrosion recognizes the added uncertainty of the section available to resist the loads in increasingly severe environments.

Careful inspection implies using more detailed measurements/refined techniques to obtain a better picture of the condition of the bridge which has undergone some kind of deterioration. In turn, this means that one is more certain of the remaining section left for resisting loads. Therefore, a correction factor to the value of ϕ can be applied to avoid the penalizing situation for the case where such measurements are not available. Similarly, efficient maintenance strategies would help in reducing further section losses that would take place if unchecked. Thus, the values of COV as derived above can be reduced. The converse is true for intermittent maintenance schedules. Correction factors for either case have then to be applied to the basic capacity reduction factor. However, there are no data available to guide us regarding these issues. Hence, some values were arbitrarily selected and the implications of their use are examined in Chapter Four.

PRESTRESSED CONCRETE MEMBERS

Repair and evaluation of damaged prestressed concrete girders has been the subject of an extensive study in NCHRP Project 12-21. However, it dealt mainly with accidental damage. The typical causes of damage are (*29*):

1. *Overheight vehicles*—Vertical clearance may be insufficient either because of illegal height of the truck, reduced clearance

(e.g., an old bridge), or because of dynamic forces inducing vertical motion of the truck and cargo.

2. *Corrosion of strands*—The direct cause is insufficient cover either because of sloppy construction and/or gradual reduction of the concrete wearing surface. The problem is particularly critical in the case of post-tensioned girders.

3. *Corrosion of joints*—Joints often leak, causing serious problems like deterioration of concrete and corrosion of strands. Epoxy injections are used to repair the damaged concrete.

4. *Fire*—Fire under the bridge (vandalism) causes deterioration of concrete, loss of concrete cover for strands, and corrosion of strands and may lead to replacement of the girder(s).

5. *Sloppy construction*—Prestressed girders are often exposed to critical stress conditions during construction or transportation. Inadequate understanding of structural behavior may lead to damage of the girder.

Overheight vehicles account for about 80 percent of the damage and fire causes about 2.5 percent of the damage. Of the reported damage (*30*), minor and moderate damage account for 80 percent of the damages. Minor and moderate damage may involve loss of concrete up to the extent of exposing the prestressing stands and reinforcing bars. Severe and critical damage is due to major losses of concrete or damage to the reinforcing elements.

The above-cited study (*30*) indicates that load capacity is by far the most important rationale for selection of repair method (i.e., whether to repair-in-place or replace). A survey of the practices of various departments of transportation showed that inspection of damage ranges from visual means with varying degree of inspection detail, to structural analysis of the damaged girder compared to the original design. Serious damage is invariably inspected by licensed structural engineers. Most damage reports are in letter form and include damage description, severity of damage, repair recommendations, and estimated cost of repair.

Up until now, the discussion has been limited to damage due to accidental causes. The effect of atmosphere on the strength of prestressed concrete members is a largely unexplored area primarily because they are of recent vintage and not many problems have been reported. Studies (*42,43,44,45,46*) do exist which discuss the durability of prestressed concrete beams and also the corrosion of prestressing tendons; however, none of them provide data which could be used by the research team directly.

For purposes of this study, the statistics of the strength of damaged and deteriorated prestressed concrete members have to be determined. Unfortunately, there are practically no such data available. Because inspection of prestressed concrete girders is invariably detailed in nature (specially for severely damaged members), it is postulated that the resistance factors for carefully inspected and deteriorated steel members will also apply in this case. Such a stipulation is necessary because of the uncertainties that may still be present even upon a detailed inspection. These factors have been derived in Chapter Four of the main text.

APPENDIX I

MAINTENANCE AND INSPECTION

A modern highway bridge is a very costly and complex structure. The elements of the structure, from the foundation to parapets and railings, must interact with each other in a unique, efficient way. The malfunction of one element can affect the overall operational efficiency of the structure.

As soon as a bridge is constructed and put into service, deterioration may begin. The changes that develop are gradual and usually slow and there is a tendency to give them little attention. Experience in highway operations shows that continuous and systematic maintenance of a bridge will extend its service life and reduce its operating expense. Nevertheless, maintenance of bridges and their approaches is often the most neglected phase of highway operation. This appendix attempts to identify the major types of preventive maintenance and inspection that can be associated with the main elements of a bridge (*37*). Existence of such procedures can provide a basis for choosing the various options on maintenance and inspection in the proposed evaluation specification. The main elements with which one will be concerned are the roadway and approaches, superstructure and, to a certain extent, substructure.

ROADWAY AND ROADWAY APPROACHES

The bridge roadway includes the deck, wearing surface, joints, railings, parapets, curbs, sidewalks, and drainage systems.

Periodic maintenance should include cleaning and flushing of concrete decks, lower chords, expansion joints. This prevents ponding of water and accumulation of debris and salt-laden dirt which may cause scaling, corrosion of reinforcing steel, deterioration of paint systems, and corrosion of supporting members and deterioration near cracks and joints.

Sealing of cracks and treatment of decks for protection against moisture and effects of deicing chemicals are necessary to prevent corrosion of reinforcement and spall development.

Cleaning expansion joints and expansion devices of any incompressible material is necessary for them to function as designed. Failure to do so could lead to crushing of the deck or girder ends, undue pressure on the superstructure bearings, and cracking or spalling of superstructure caps.

Deck drains should be periodically cleaned to prevent ponding of water which might promote rapid deck deterioration.

Bridge approaches should be leveled to prevent excessive live load impact.

Build-up of approach roadway shoulders should be checked regularly to avoid any ponding of water.

SUPERSTRUCTURE

The superstructure component includes main members, floor system, secondary members, and bearing elements. Main members are those whose failure would result in collapse of the structure, including girders, truss chords, diagonals, and verticals. Floor systems include members which transmit loads from the roadway to the main members. Failure of the floor system members would usually have only local effects. Secondary members add stiffness to the main members. Bearings transfer the loads from main members to the substructure and also allow for longitudinal and rotational movements of the main members.

Preventive maintenance for most steel superstructures involves keeping the integrity of the protective coating intact. Areas where the prime coat has failed and corrosion has begun must be spot cleaned, primed, and top coated promptly to prevent further corrosion.

Areas like open deck expansion joints, lower flanges of outer girders, and lower chord and floor beam connections on truss spans should be properly cleaned and maintained on a regular basis to prevent debris accumulation which may lead to corrosion loss.

Preventive maintenance of concrete members involves sealing of any minor crack or spall with an appropriate grout or patching compound.

Bearings should be regularly cleaned of debris so as to prevent any chance of "freezing" which could cause undue stresses in the girder or substructure cap leading to some sort of damage.

SUBSTRUCTURES

Substructures transmit loads from the superstructure down to the soil and include two types: abutments and interior supports. Routine maintenance involves cleaning of dirt and debris accumulated on caps under open expansion joints that may cause deterioration and spalling of concrete and corrosion of reinforcement.

The foregoing list of desirable maintenance procedures, although not exhaustive, should serve as a guideline for what can be regarded as vigorous maintenance schedules. For a more detailed discussion of preventive maintenance strategies, the reader is directed to Refs. *1* and *38*.

The foregoing discussion was for preventive maintenance strategies. The proposed specifications also provide options for corrective maintenance inspections which involve determining the extent of any deficiency if present. Such a procedure involves recognizing various kinds of bridge deterioration, pinpointing its location, and categorizing and describing its severity.

The type of materials used in the bridge will establish initially the particular kinds of deterioration the inspector might be looking for during his inspection. Hence, this description will be categorized on the basis of materials and will give an overview of what could be considered good inspection practice.

CONCRETE

The major types of deterioration associated with concrete include the following:

1. *Scaling*—The inspector should describe the character of scaling, the approximate area involved, the location and also the depth if possible.
2. *Cracks*—Cracks can be classified into 8 major types (transverse, horizontal, longitudinal, vertical, diagonal, pattern, D, and random cracks). Irrespective of their type, inspection reports should include size, length, direction, and location of the crack.
3. *Spalling*—The location of the spall along with its size, area, and depth should be measured. Existence of any hollow areas should be documented and their location determined if possible.
4. *Joint spalls, pop-outs, mudballs*—In recording these defects, their type, depth, dimension, and location should be described preferably with a sketch, drawing, or photograph as appropriate.
5. *Deterioration*—Any adverse change of normal mechanical, physical, and chemical properties (disintegration, distortion, pitting, erosion, leaching, wear) should be recorded with sketches and measurements wherever possible.

STEEL

The major types of deterioration that may be encountered are as follows:

1. *Rust*—This might vary from light rust to severe scaling or pitting. The inspector should note the location, the characteristics, and the extent of the rusted areas. The depth of heavy pitting should be measured, and the size of any perforation caused by rusting should be recorded as accurately as possible.
2. *Cracks*—Any type of crack is serious and should be reported immediately along with a description of the location, length, and size of the crack.
3. *Buckles and kinks*—Members damaged in such a fashion should be noted along with the type, location, extent of damage, and amount of deformation if possible.
4. *Stress concentrations*—Fine cracks in the paint around the connections should be noted. Also, sheared or deformed bolts and rivets should be marked.

As before, the foregoing list is certainly not complete in nature. However, it may be taken as one of the possible means of determining if the actual inspection can be categorized as detailed, or not according to the proposed evaluation specification.

APPENDIX J

GLOSSARY OF SYMBOLS

a	Constant which relates load effect to a reference loading model
A	Area of girder, regression coefficient (Appendix H)
ADT	Average daily traffic
ADTT	Average daily truck traffic
B	Regression coefficient
B_i	Bias of variable i (bias equals ratio of mean to nominal value)
C	Average corrosion penetration in μm
C_e	Correction factor for edge stiffening and vehicle edge distance
C_f	Correction factor for design lane width
COV	Coefficient of variation
D	Dead load effect
e	Distance from neutral axis of girder to mid-depth of slab
E_g	Modulus of elasticity of girder
E_s	Modulus of elasticity of slab
$E[R_s]$	Expected value of system resistance
$f_X(x)$	Probability density function of X
$F_X(x)$	Probability distribution function of X
g	Safety margin or girder distribution factor
$\bar{g}$	Mean safety margin
G_m	Maximum lifetime response
H	Influence of multiple presence
H_{moment}	Influence of multiple presence on moment
H_{shear}	Influence of multiple presence on shear
$HW._{95}$	Projected load effect
I	Impact
I_{cg}	Moment of inertia of composite girder
I_n	AASHTO impact
L	Nominal live load effect due to traffic, span (Appendix G)
LFD	Load factor design
LFR-I	Load factor resistance (inventory)
LFR-O	Load factor resistance (operating)
LRFD	Load and resistance factor design
LSD	Limit state design
m	Influence of dominant vehicle type and configuration at site
M	Predicted maximum load effect
M_n	Nominal moment
P_f	Probability of failure
R	Resistance
$\bar{R}$	Mean resistance
R_m	True mean resistance
R_n	Nominal resistance
$R.F.$	Rating factor
S	Loads, spacing of girders
$\bar{S}$	Mean loads
S_i	Force in member i
$S.F.$	Safety factor
t	Exposure time in years
t_s	Thickness of slab
udl	Uniformly distributed load
USD	Ultimate strength design
V	COV (coefficient of variation = sigma/mean)
V_A	COV of analysis
V_D	COV of dead load
V_F	COV due to fabrication
$V_{HW.95}$	COV of projected load effect
V_M	COV of material properties
V_P	COV due to scatter in prediction theory
V_R	COV of resistance
ν_S	Poisson's ratio
$Var\ [R]$	Variance of system resistance
$Var\ [S]$	Variance of loads
W	Environmental load effect
W_e	Width of design lane
$W._{95}$	Characteristic truck weight at a site
WSD	Working stress design
WSR-I	Working stress resistance (inventory)
WSR-O	Working stress resistance (operating)
α	Ratio of live load effect to dead load effect
β	Safety index
β_e	Element safety index
β_s	System safety index
γ_D	Dead load factor
γ_L	Live load factor
ϕ	Capacity reduction factor
Φ	Normal probability distribution function
σ_{Dm}	Standard deviation of member dead load
σ_i	Standard deviation of variable i
σ_{Lm}	Standard deviation of member live load
σ_R	Standard deviation of member resistance
$\sigma_{RSYS}, \sigma_{R_S}$	Standard deviation of system resistance

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