NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **292**

STRENGTH EVALUATION OF EXISTING REINFORCED CONCRETE BRIDGES

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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STRENGTH EVALUATION OF EXISTING REINFORCED CONCRETE BRIDGES

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AREAS OF INTEREST:

Structures Design and Performance
Cement and Concrete
Maintenance
(Highway Transportation)

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study that was undertaken to provide guidance for the strength evaluation of existing concrete highway bridge superstructures including slab and girder, T-beam, and box beam bridges. The report includes recommendations for revisions to the present evaluation requirements that exist in the AASHTO *Manual for Maintenance Inspection of Bridges* along with a companion commentary. The contents of the report will be of immediate interest and use to bridge design and rating engineers, bridge maintenance engineers, researchers, specification writing bodies, and others concerned with the load capacity evaluation of existing concrete structures.

Reinforced 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 for any means, however, to reflect the actual condition of the structure or to quantify other important factors that might be considered in the rating process.

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, maintenance, and evaluation. 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.

This report presents the findings of a second phase of research on NCHRP Project 10-15, "Strength Evaluation of Existing Reinforced Concrete Bridges." The primary objective of this study was to develop an improved methodology for evaluating the structural capacity of existing reinforced concrete highway bridge superstructures and to present it in a specification format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures.

The first phase of research included findings and recommendations related to methods of predicting the structural capacity of reinforced concrete highway bridges for load rating purposes. The limit-state approach to bridge evaluation, combining probability theory and engineering judgment, was recommended at the conclusion of that phase. However, some of the load and resistance factors recommended in that research were not well documented and required further calibration.

The second phase of research had as its objective the further development of the limit-state approach to evaluate the structural capacity of reinforced concrete highway bridge superstructures. This report documents that work and provides recommendations for revisions to the AASHTO *Manual for Maintenance Inspection of Bridges*.

The recommendation specifications account for the superstructure condition, i.e., the degree of deterioration in the structure. Additionally, for deteriorated structures, the recommended specifications give consideration to field measurements on member sizes and material properties, frequency of bridge inspection, maintenance effort on the structure, and traffic mix and volume. The recommended procedure is also flexible enough to allow for the systematic incorporation of an engineer's subjective judgment during the bridge rating process.

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Special recognition is extended to Mr. Eldon Klein of California Department of Transportation for many patient, informal discussions on the bridge maintenance operations.

STRENGTH EVALUATION OF EXISTING REINFORCED CONCRETE BRIDGES

SUMMARY

This report describes the work conducted on the strength evaluation of existing reinforced concrete bridges including T-beams, box beams, girder and slab bridges. The methodology is developed in the load and resistance factor format that is very similar to the Load Factor Method in the current AASHTO *Standard Specifications for Highway Bridges.*

Currently, reinforced concrete bridges are evaluated according to the requirements presented in AASHTO's *Manual for Maintenance Inspection of Bridges.* Two levels of rating are specified: inventory rating and operating rating. Since existing bridges may have very different structural condition (deterioration) and live load environments, the current two-tier rating procedure is apparently too rigid. It is difficult to incorporate bridge engineers' judgments based on field experiences into the overall rating process. As a result, bridge ratings do not reflect the inherent safety level.

In the methodology developed herein, different categories of strength and live loading are defined. Based on the superstructure condition rating, r, which is conducted routinely by bridge bureaus, three broad categories are defined to reflect the degree of deterioration: (1) good or fair ($r \geq 7$), (2) deteriorated ($r = 4, 5, 6$), and (3) seriously deteriorated ($r \leq 3$).

For deteriorated bridges, additional considerations are given to the following factors: (1) field measurements, (2) inspection frequency, and (3) maintenance effort.

For each of these categories, coefficient of variation and bias coefficient are assigned based largely on subjective engineering judgment.

Gross vehicle weight (GVW) of trucks varies considerably from trucks that are empty to overloaded trucks. To estimate the maximum load effect, engineers are primarily interested in the upper tail of the GVW histogram. The characteristic weight of the 95th percentile level is a good indicator of load intensity of individual heavy trucks. The maximum load effect due to random truck traffic is affected, in addition to GVW variability, by the multiple presence of trucks either in the same lane or in adjacent lanes. Based on field measurements, realistic live-load parameters are defined according to truck traffic volume (ADTT), bridge span length and weight control enforcement.

For a family of existing T-beam and slab bridges built between 1911 and 1963, the safety indices are computed. By correlating the safety index β to the operating level rating, a target safety index of 2.8 was selected which, on the average, corresponds to an operating rating of 1.0. This target value is deemed adequate for existing bridges comparing to the values (β = 3.5 to 4+) for new bridges.

A limit-state calibration was then carried out to determine the resistance factors and load factors for various strength and live load categories. The flexure resistance factor may vary from a low of 0.54 for a seriously deteriorated bridge to a high of

0.95. The live load factor may vary from 1.48 for light truck traffic and good weight enforcement to 1.93 for heavy truck traffic and poor enforcement.

Based on the actual truck traffic volume and superstructure condition, a bridge rating may now range from a level below the current inventory rating to a level beyond the current operating rating. The new procedure is sufficiently flexible to allow for the systematic incorporation of engineers' subjective judgment. When a certain parameter is difficult to quantify, for example, the resistance factor, the bridge rater may select a lower and upper bound based on his best judgment and compute lower and upper bounds for the rating. This would facilitate more reasonable decision-making with respect to the overall bridge maintenance policy.

CHAPTER ONE

INTRODUCTION

This report describes the work conducted on the second phase of NCHRP Project 10-15(1), "Strength Evaluation of Existing Reinforced Concrete Bridges." The objective of this project is to employ recently developed reliability theory to develop a rational approach to the evaluation of strength of existing reinforced concrete bridges. The guidelines as developed herein will be submitted to the AASHTO Highway Subcommittee on Bridges and Structures for consideration as a guide specification to supplement the rating procedures described in the AASHTO *Manual for Maintenance Inspection of Bridges.* The proposed guidelines and commentary are included in Appendixes A and B, respectively. Appendix C contains a selected group of examples illustrating the use of the guidelines together with a comparison to rating factors obtained using the existing AASHTO specifications.

BACKGROUND

The bridge engineer responsible for operating and maintaining a network of bridges on a modern roadway system is continually faced with the task of evaluating the live load capacity of existing bridges. These bridges are typically of diverse vintage and have been constructed to meet a wide range of different design criteria, which can result in a wide variation in live load capacities. Additionally, several other factors, such as changing live load configurations, structural modifications, and deterioration, are continually altering the conditions at each bridge. These changes make it necessary to monitor the allowable load capacity of the bridges. The economic and political pressures to allow heavier loads on the roadway system, combined with the need to protect the safety and economic interests of the public, make it crucial that engineers strive to improve the processes for evaluating the live load capacity of bridges.

Since the collapse of the Silver Bridge over the Ohio River at Point Pleasant, West Virginia, in 1967, there has been an increased public awareness about the condition of existing highway bridges in the United States. This has resulted in a legislatively mandated program to inventory, inspect, and improve the nation's highway bridges. Initial legislation in 1968 required the development of National Bridge Inspection Standards (NBIS). This was followed by the Federal Aid Highway Act of 1970 which made funding available for the Special Bridge Replacement Program. This program was in effect through 1978 and resulted in the replacement of 2,100 bridges. In 1978, legislation entitled the Surface Transportation Assistance Act became effective. Under this Act, the Highway Bridge Replacement and Rehabilitation Program (HBRRP) made funding available through 1982 for bridge replacement and rehabilitation. Furthermore, this program extended funding to include bridges not on the present federal aid system. The HBRRP was extended through fiscal year 1986 by the Surface Transportation Assistance Act of 1982. Hence, about 50 percent of the highway bridges in the nation qualifies for federal funding under the current program.

However, available funds are limited, and administrators must allocate these resources effectively. Engineers performing bridge evaluations have the difficult responsibility of determining bridge load capacity for load limit posting, and administrators are faced with the task of making decisions regarding replacement and rehabilitation for existing bridges. At the present time recommendations for rehabilitation and replacement must rely on the evaluation procedures described in the AASHTO *Manual for Maintenance Inspection of Bridges (2).* These AASHTO procedures were actually developed as a secondary consideration from the design procedures for new structures. Because most existing bridges have satisfactory past performance records, many bridge evaluation engineers intuitively believe that a great number of bridges have more capacity than indicated by the AASHTO *Manual.* However, engineers lack the analytical justification and/or methodology to supersede the AASHTO requirements. As a result, a great need exists for a new specification developed solely for the purpose of evaluation that will accurately reflect observations about bridge performance.

NCHRP Project 10-15 was created because of a gap in recent research in the structural strength evaluation of existing reinforced concrete bridges and because of an apparently large discrepancy between the observed and calculated live load capacities for this class of bridges. Phase I of this project began in 1980 and was completed in September of 1982. Initially, the project emphasis was placed on improving the analytical procedures for determining live load effect and structural resistance. A methodology was considered that made use of generalized influence surfaces for determining wheel load distribution. In addition, guidelines for assessing the extent of deterioration by standard inspection procedures were developed. During the course of the project it became apparent that a restructuring of the rating process was in order, however, and the project emphasis shifted to this task.

In Phase I of this project, a limit states approach in load-and-resistance factor format was proposed for evaluating reinforced concrete bridges. The proposed evaluation procedure, which was different from that used in design, included factors for simulation (i.e., field inspection and evaluation effort), maintenance effort, load distribution, variations in dead load, dead load distribution, load limit control, and live load distribution. The proposed procedure was presented to introduce the concepts and the variables to be included in the evaluation procedure. Because of limited amounts of statistical data and the primary goal of introducing the concepts, many assumptions were made with regard to variables in the preliminary calibration process. Also, variables were introduced which may or may not have a significant effect on the rating factor.

The response of professional bridge engineers to the proposed evaluation procedure has, in general, been favorable. Further refinement of the procedure is necessary, however, before it can be presented to the profession in a form suitable for implementation into practice. Because of both the outcome of the first phase and the overall project objective to develop an implementable rating procedure for existing reinforced concrete bridges, a second phase of the projected was initiated.

OBJECTIVES AND RESEARCH APPROACH

Phase II of this project involves a more rigorous calibration of the proposed evaluation procedure for existing reinforced concrete bridge superstructures. This evaluation procedure is governed by the following equation:

$$\phi\, R > \sum_{i=1}^{m} \gamma_{Di}\, D_i + \sum_{j=1}^{n} \gamma_{Lj}\, L_j\, (1.0 + I) + \gamma_{LR}\, L_R\, (1.0+I) \qquad (1)$$

By imposing equality and introducing the rating factor, RF, Eq. 1 may be arranged as follows:

$$RF = \frac{\phi\, R - \sum \gamma_{Di}\, D_i - \sum \gamma_{Lj}\, L_j\, (1.0+I)}{\gamma_{LR}\, L_R\, (1.0+I)} \qquad (2)$$

where RF = rating factor (the portion of the rating vehicle loading allowed on the bridge), ϕ = capacity reduction factor, m = number of elements included in the dead load, R = nominal resistance, n = number of live loads other than the rating vehicle, γ_{Di} = dead load factor for element i, D_i = nominal dead load effect of element i, γ_{Lj} live load factor for live load j other than the rating vehicle(s), L_j = nominal traffic live load effects for load j other than the rating vehicle(s), γ_{LR} = live load factor for the rating vehicle(s), L_R = nominal live load effect for the rating vehicle(s), and I = live load impact factor.

In the general checking format, Eqs. 1 and 2, the rating vehicle effect has deliberately been isolated from other live load effects so that special permit trucks may be treated differently from the typical highway truck traffic. From the point of view of permit issuing application, this separation is very desirable. However, to accomplish this refinement, a large amount of live load data, particularly permit truck data, are required. One example of the analysis of this mixed live load data was given by Moses et al. (*41a*). A comprehensive analysis is, however, beyond the scope of this project. In the remaining portion of this report, only random trucks are considered, i.e., all trucks, permit legal or illegal, are lumped into one group, i.e.,

$$\phi\, R \geq \gamma_D D + \gamma_L L\, (1+I) \qquad (3)$$

or

$$RF = \frac{\phi\, R - \gamma_D D}{\gamma_L L\, (1+I)} \qquad (4)$$

The recommended procedures, which vary slightly from those proposed during Phase I, will be presented to AASHTO in specification form for their consideration.

Phase II of the research included the following tasks:

1. Analyze statistical information available from FHWA's computerized national bridge inventory system for the purpose of identifying types of reinforced concrete superstructures that warrant attention in this study.
2. Conduct a sensitivity analysis to determine the effects of modifications to the load and resistance factors in the limit state approach to evaluation.
3. Identify and evaluate the applicability of available test results for calibrating the factors to be used in the limit states approach.
4. Prepare an interim report presenting the findings of the first three tasks and proposing a detailed working plan for the remainder of the study.
5. Calibrate the proposed method using available test data and other appropriate information.
6. Compare the results of the proposed method with results from currently used methods.
7. Present the proposed method, its rationale, and the justification for its adoption at the regional meetings of the AASHTO Bridge Committee in the spring of 1986.
8. Prepare a final report documenting the findings of both phases of research and presenting the recommended method in a format suitable for adoption by AASHTO.

ORGANIZATION OF THE REPORT

Various aspects of the reliability bases necessary for developing a rational design/evaluation procedure are briefly reviewed and summarized in Chapter Two. The emphasis is on the first-order, second-moment reliability analysis method. Relationships between failure probability, reliability, hazard environment, and service lifetime are discussed. The calibration procedure to determine suitable load and resistance partial safety factors is also described.

The specific limit state equation used and the statistical descriptions of the random variables involved are presented in Chapter Three. One aspect of the bridge live load model, the girder distribution factor, is discussed and some preliminary results presented.

To observe the safety level of bridges across the country and for future calibration purposes, sample bridges along with their evaluation data were gathered from state government agencies. A description of this data base is given in Chapter Four. From a survey of the FHWA bridge inventory file, it was decided that the effort in this project will be focused on two types of bridges: T-beam and slab bridges.

A family of hypothetical bridges based on the current AASHTO load factor design was also generated. Results of the sensitivity analyses for this family of hypothetical bridges and the sample of real bridges are presented in Chapter Five.

Chapter Six describes the calibration procedure used to establish load and resistance factors. Included is a description of the procedure used to establish the target safety index. A method of considering maintenance, inspection, and field measurement in the establishment of resistance factors is presented as is a method for considering traffic volume and level of load limit enforcement in the establishment of live load factors.

The conclusions of the study are summarized in Chapter Seven along with recommendations for further research.

As mentioned earlier, Appendix A contains the proposed guidelines; Appendix B, the accompanying commentary; and Appendix C, example problems. Appendix D covers details of the bridges in the data base. Appendix E includes the field investigation procedure. Appendix F is the bibliography. Appendix G concludes the report with a glossary of terms and definitions of nomenclature.

CHAPTER TWO

PROBABILISTIC BASES OF STRUCTURE RELIABILITY

Because of the uncertainties inherent in the material properties, dimensions, loads, and the like, used in the engineering design or evaluation calculations for a structure, a procedure that includes probabilistics in the calculations of structure reliability is the most reasonable approach (*4,19,22*). As in other design or evaluation procedures, the ultimate objective of such a probability-based procedure is to ensure that an appropriate safety margin is provided. One advantage of using a probabilistic approach is that uncertain variables may be considered jointly and their effects may be separated systematically, which allows for a more systematic determination of structural reliability. However, the problem of determining structural reliability becomes more complex when evaluating the overall safety of a structural system because all the possible limiting states (or failure modes) and their combinations must be considered. Because of many apparent difficulties, methods for evaluating overall safety are still in the developmental stage (*42,46,58*). Therefore, most of the recent code development (*9,19,50*) with a probability-based approach has emphasized *component reliability* and addressed the question of overall safety through vague requirements for system redundancy (*42*).

Only limited statistical descriptions for variables, such as material, dimensions, and loads, are currently available. In addition, most of the available data for these variables fall close to the medium range of the variables, which makes it difficult to ascertain the most accurate statistical distribution models. For example, in the case of the *Ontario Highway Bridge Design Code* (*27*), it was necessary to intentionally exaggerate the measured probability density function of overweight vehicles to account for the unlawful drivers who avoid weight measurements. Even selecting an acceptable probability of failure is a subjective decision.

Uncertainties may be classified into three distinct types (*6*): (1) fundamental uncertainty which is the randomness in nature, (2) statistical uncertainty associated with the parameter estimates of a selected probabilistic model, and (3) uncertainty associated with the probabilistic model itself.

The first type is irreducible; while the last two types may, in principle, be eliminated if a large amount of data can be obtained.

Despite the fact that there are some difficulties in describing the uncertainties of the parameters, a probability-based approach presented in a load and resistance factor format still represents the most rational, practical method for considering these uncertainties and provides a framework for future improvements as additional data become available. This chapter will summarize and discuss some of the basic aspects of such a probability-based approach.

STATEMENT OF THE PROBLEM

An engineer faced with initiating a design or evaluation of a structure must answer the following questions: (1) What are the limiting states by which to measure structural performance? (2) How much safety margin should be provided or, in other words, what is the acceptable probability of failure for a specified period of time? (3) How can this safety margin be achieved?

The first question usually poses no problem for an experienced engineer. The second and third questions are usually dictated by the applicable design codes. In the case of a probability-based approach, code-writing committees usually define the acceptable probability of failure in terms of a target reliability reflected in load and resistance partial factors. These partial factors will result in designs with actual performance reliabilities which, when averaged, approach a selected or defined target reliability. On the other hand, when evaluating an existing structure, the evaluator may choose to determine with greater certainty the variables affecting structure performance. Therefore, when evaluating an existing structure, it is justifiable to use load and resistance partial factors that differ from those used for a new design. Some existing codes recognize this difference and have proposed special factors for evaluation (*10*). However, this is the exception rather than the rule, and more effort is needed to assure that full advantage is taken of the increased certainty in the parameters for existing structures.

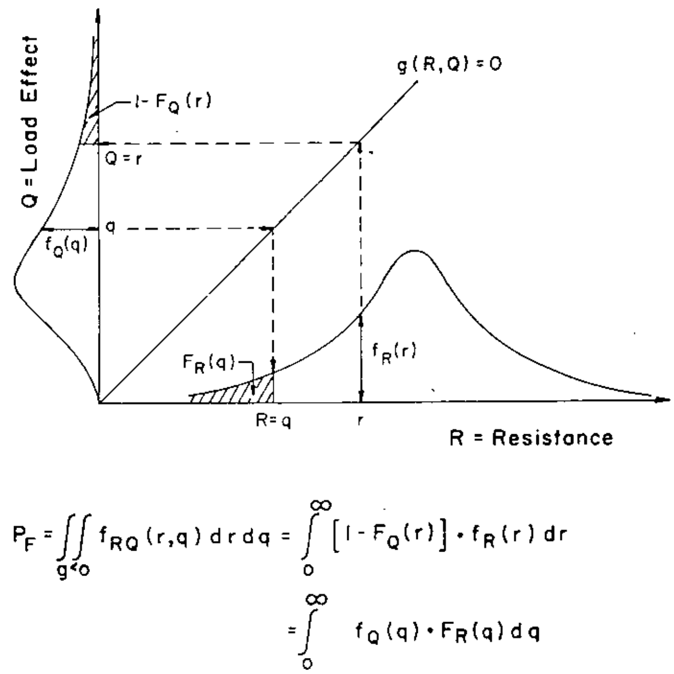

Figure 1. Probability of failure.

LIMIT STATE EQUATION

In the simplest form, a limit state equation may be written in terms of the safety margin, g,

$$g(R,Q) = R - Q \begin{array}{ll} < 0 & \text{failed} \\ = 0 & \text{limit state} \\ > 0 & \text{survived} \end{array} \tag{5}$$

where R is the resistance and Q is the load effect. However, R and Q are both random variables and usually functions of many other random variables, such as material strength, sectional dimensions, load intensity, etc. For bridge design or rating, the load effect is composed of different categories of dead load and live load effects. Expressed mathematically

$$\begin{aligned} R &= R(X_1,X_2,\ldots X_r) \\ Q &= Q(X_{r+1},\ldots X_{n-1},\, x_n) \\ g(\mathbf{X}) &= g(X_1,X_2,\ldots X_r,X_{r+1},\ldots X_n) \end{aligned} \tag{6}$$

Each of these random variables (i.e., $X_1,X_2,\ldots X_r,X_{r+1},\ldots X_n$), appearing in the vector, **X**, is described by some probabilistic distribution. The normal and log normal distributions are frequently used distribution types in structural reliability.

RELIABILITY MEASURES

A direct measure of reliability is the probability of failure. The probabilistic properties of the safety margin, g, defined in Eq. 5, may be described by the joint probability density function $f_{RQ}(r,q)$. By assuming statistical independence between resistance and load effect (i.e., $f_{RQ}(r,q) = f_R(r) \cdot f_Q(q)$), the probability of failure may be obtained from the following equation:

$$\begin{aligned} P_F = \iint_{g<0} f_{RQ}(r,q)drdq &= \int_o^\infty [1 - F_Q(r)]f_R(r)dr \\ &= \int_o^\infty f_Q(q)F_R(q)dq \end{aligned} \tag{7}$$

where F_Q and f_R are the cumulative probability distribution function and the probability density function of Q and R, respectively. Figure 1 shows the convolution process of Eq. 7.

Equation 7 is the formal statement for the probability of failure of the simple limit state condition, Eq. 5, involving two random variables only. For the more general limit state condition, Eq. 6, the probability of failure would have to be evaluated through a multitude of convolution integrals. Moreover, the probabilistic descriptions of the basic random variables are usually incomplete. At very low levels of probability, the probability of failure is very sensitive to the incomplete descriptions of the basic variables.

Alternatively, the reliability may be described by a dimensionless index, β, called safety index or reliability index. The most general definition of the safety index may be expressed as:

$$P_F = \Phi\,(-\beta);\ \beta = -\ \Phi^{-1}\,(P_F) \tag{8}$$

where $\Phi(\cdot)$ is the standardized normal distribution function. In other words, the original problem, regardless of the distribution type of the safety margin, is transformed into a "reference" problem with standardized, normally distributed safety margin. This definition is very general in that it preserves the same probability of failure; however, it is not very useful because of the difficulty in obtaining P_F.

As originally proposed by Cornell (*13*), the safety index is intended to be a distribution-free measure of reliability. In terms of only the first two moments of the safety margin, the safety index is defined as:

$$\beta = \frac{\bar{g}}{\sigma_g} \tag{9}$$

where $\bar{g}$ = the mean safety margin, and σ_g = the standard deviation of the safety margin.

Figure 2 shows the two measures of reliability. The structural safety may be measured by the failure probability, P_F, shown by the shaded area, or the reliability $(1 - P_F)$, shown by the unshaded area under the distribution curve. The safety index, β, defined in Eq. 9, measures the mean safety margin in terms of the number of standard deviations away from the critical state, as shown graphically in Figure 2. This safety index is a more convenient measure than the probability of failure because only the first two moments (i.e., the mean and the standard deviation) are needed for each of the random variables, hence avoiding the previously mentioned problems with calculating the probability of failure. With some additional assumptions, the safety index, β, may be related to the probability of failure.

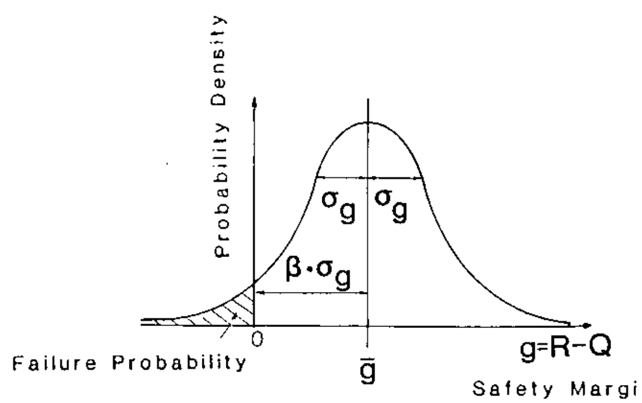

Figure 2. Measures of safety margin.

ADVANCED FIRST-ORDER SECOND-MOMENT METHOD

For the general limit state, Eq. 6, the safety index can be adequately described by the mean and the standard deviation of the safety margin Eq. 9. However, the safety index is sensitive to how $\bar{g}$ and σ_g are evaluated. Hasofer and Lind (*28*) showed that an *invariant* safety index can be obtained only if $\bar{g}$ and σ_g are evaluated at an expansion point which is on the limit state surface. Further, the expansion point is selected such that the calculated safety index is a minimum (*52*). For ease of analytical derivation, all the random variables affecting Q and R are transformed into their standardized forms:

$$x_i = \frac{X_i - \bar{X}_i}{\sigma_i} \tag{10}$$

where $\bar{X}_i$ and σ_i are the mean and the standard deviation of X_i, respectively. The standardized variables, x_i, have mean values of zero and standard deviations equal to unity. The limit state equation, Eq. 6, may be rewritten as:

$$g_1(\mathbf{x}) = g_1(x_1, x_2, \ldots x_n) \tag{11}$$

Some of these variables may be statistically dependent on one another. Approximate technique exists to transform these dependent variables into independent variables, e.g., Rosenblatt transformation (*17,29,52*). Without loss of generality, all the x_i's may be assumed to be statistically independent of each other.

Because of a scarcity of adequate field data, it is impossible to obtain full descriptions of random variables. However, successful descriptions have been obtained in many fields by using only the lower order moments of a random variable(*59*). The second-moment method utilizes only the mean and the variance of each random variable.

Figure 3 shows the limit state of two random variables, i.e., $g_1(x_1, x_2)$ in the standardized variable space. To facilitate the computation of β, it is necessary to linearize the limit state equation at a point on the limit state surface. This linearization procedure may be expressed mathematically as follows:

$$g_1(\mathbf{x}) = (\mathbf{x} - \mathbf{x}^*)^T \cdot \mathbf{G}^* \tag{12}$$

where superscript T indicates the transpose of a vector or matrix, and

$$G_i^* = \left(\frac{\partial g_1}{\partial x_i^*}\right) \tag{13}$$

are terms of the limit states surface gradient vector, $\mathbf{G}^*$, evaluated at the linearization point, $\mathbf{x}^*$, and $g_1(\mathbf{x}^*)$ equals 0. Therefore the mean and the standard deviation of the safety margin are, respectively

$$\bar{g}_1 = -(\mathbf{x}^*)^T \mathbf{G}^* \tag{14}$$

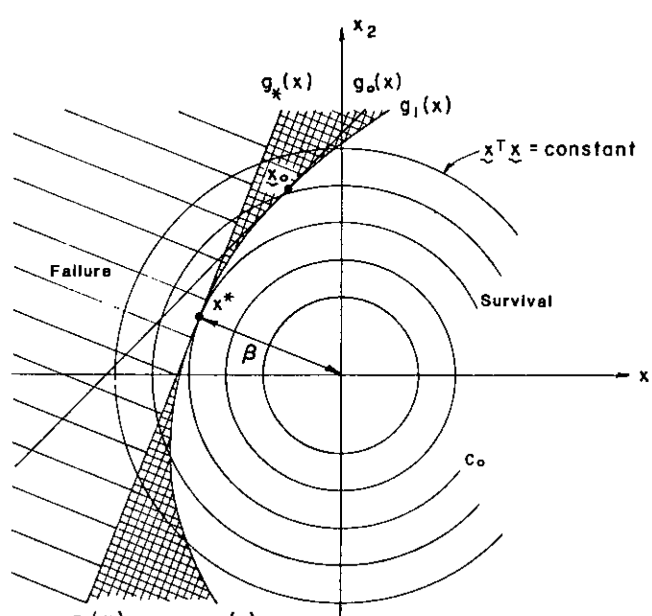

Figure 3. Advanced first-order second-moment reliability calculation.

$$\sigma_g = (\mathbf{G}^{*T}\mathbf{G}^*)^{1/2} \tag{15}$$

yielding for the safety index

$$\beta = \frac{-(\mathbf{x}^*)^T \mathbf{G}^*}{(\mathbf{G}^{*T}\mathbf{G}^*)^{1/2}} \tag{16}$$

From Eqs. 14, 15, 16 and Figure 3, it can be shown that the computed safety index depends on where the limit state equation is linearized. The Hasofer-Lind safety index is the minimum value obtained from Eqs. 14, 15, and 16. Rackwitz and Fiessler (*52*) suggested an iteration scheme to solve for the true safety index. Shinozuka (*56*) suggested using the optimization algorithm. The Phase I report (*31*) used the mean value method. As noted in the literature, this method may result in a different safety index if the problem is formulated differently (*19*). The computation of β, herein, is based on the iteration scheme suggested by Rackwitz and Fiessler (*52*). With some modifications, the computer algorithms for this computation were taken from *NBS Special Publication 577* (*19*).

RELIABILITY AND EXPECTED HAZARD ENVIRONMENT

Given a specified hazard environment, the reliability or the probability of failure is a function of the time period under consideration. If T is the random variable indicating the time to the first failure event, the probability of failure and the reliability during the time interval $[0,t]$ can be expressed as follows:

Probability of Failure within $[0,t]$ $= F_T(t) = \text{Prob. } [T \leq t]$ (17)

Reliability during $[0,t]$ $= L_T(t) = 1 - F_T(t) = \text{Prob. } [T > t]$ (18)

In fact, as indicated by this notation, $F_T(t)$ is the cumulative probability distribution function of T.

If the hazard environment is described by a hazard function, $h_T(t)$, such that $h_T(t)dt$ is the probability that failure first occurred during $[t, t+dt]$ under the condition that no failure occurred before t, (*7,22,42*), then

$$h_T(t)dt = \text{Prob. } [t < T < t+dt | T > t] \tag{19}$$

or

$$h_T(t) = \frac{f_T(t)}{1 - F_T(t)} = \frac{f_T(t)}{L_T(t)}$$

where f_T is the probability density function of T. The reliability may then be written in terms of $h_T(t)$:

$$L_T(t) = 1 - F_T(t) = \exp\left[-\int_o^t h_T(u)du\right] \tag{20}$$

For rating practice, Eq. 20 may be written in the following format (*35,42*):

$$L_T(t) = L_T(t_i) \exp\left[-\int_{t_i}^t h_{Ti}(u)du\right] \tag{21}$$

While t_i is the time that the i^{th} inspection and/or maintenance was performed, $L_T(t_i)$ is the structure reliability at that instance. For a new design, however, $t_o = 0$ and $L_T(0) = 1$, as indicated by Eq. 20.

When designing a new bridge, load growth and strength deterioration must be forecasted over the design lifetime. Considerable conservatism is required to ensure an adequate design. Once the bridge is built, information about the actual strength, load intensity, and response characteristics can be made available. Strength parameter and response mechanism used in the original design are usually conservative. Therefore, even the strength would deteriorate with time; the actual strength in existing bridges is often higher than expected. By gathering more information through tests and measurements, the hazard environment to which a bridge is exposed can be estimated more realistically. Equation 21 demonstrates conceptually how the changing estimate of the hazard environment and known satisfactory performance of the bridge can be used in the safety evaluation. In the context of this study, the more realistic hazard environment can be estimated based on inspection and maintenance results, truck weight, and traffic data, and so on.

PROBABILITY OF FAILURE VS. SAFETY INDEX

Several approaches may be used to quantify the probability of failure. If utility loss is considered to be "failure," about 44 percent of the nation's bridges have failed because 44 percent of the nation's bridges have been classified as structurally deficient or functionally obsolete (*23*). On the other hand, if only catastrophic events, such as collapse, are considered, only two reinforced concrete bridges are reported to have failed within the United States during the 10-year period (*30*).

Galambos (*23*) reported that of the 560,000 bridges in the United States, about 5,000 are built each year to replace deficient bridges. This figure corresponds to an annual probability of failure of 0.009. In another survey, Imbsen (*30*) reported that the number of bridges closed during the last 10 years was 5,950, corresponding to an annual probability of failure of 0.001. However, new bridges may be built and existing bridges may be closed for many reasons other than structural failure. In the same study, Imbsen reported that the total number of bridges failed due to live load in the 10 years period is 596, which corresponds to an annual probability of failure 10^{-4}.

Most observed bridge failures are caused by floods (approximately 150 annually according to Galambos (*23*)), fires, and earthquakes. These rare events have destroyed many bridges otherwise safe and sound. By excluding these rare events, the rate of annual bridge failure is in the range of 10^{-4} to 10^{-5}.

Augusti, Baratta and Casciati (*5*) suggested that the acceptable lifetime probability of failure in civil engineering works should be: 10^{-3} for limit states that do not endanger lives, and 10^{-5} to 10^{-6} for disastrous limit states.

Available data are rarely sufficient to establish a probability of failure for a certain class of structures. The acceptable lifetime probability of failure is also more or less a subjective decision. In code applications, it is generally accepted that the safety index β, as defined earlier, is a preferred safety measure. If the safety margin has a normal distribution, the safety index β may be translated to the probability of failure according to Eq. 8 (see Fig. 4).

CALIBRATION PROCEDURE

Several major differences exist between evaluating an existing bridge and designing a new bridge:

1. An existing structure already has a satisfactory or an unsatisfactory service record.
2. The expected remaining service lifetime of an existing bridge is usually much shorter than that of a new bridge.
3. Deterioration and hazard environment (such as overload trucks) may be constantly monitored and updated on an existing structure through efforts of inspection, maintenance, and load control.
4. The initial impact on the regional economy that results from the opening of a new bridge (or highway) will have already stabilized for an existing bridge, and the future growth of truck traffic volume in the shorter period of remaining lifetime may be predicted with greater certainty.
5. With the ongoing development of field measurement and nondestructive testing methods, the actual member strengths and load effects of an existing structure may be verified or revised from the original design values.

Ideally, these aspects should all be reflected in the code format for rating practice. The following factors are included in the proposed evaluation procedure: (1) quality of construction, (2) maintenance/inspection effort, (3) truck traffic volume and characteristic truck weight (including efforts of load control), and (4) efforts spent in determining the load effects (dead load and live load distribution factors).

In this project an ensemble of real bridges along with related data were gathered from across the country. Using representative statistical data from the literature for the variables considered, the safety indices were evaluated. These safety indices represent the level of safety in existing bridges. Their sensitivity to construction quality, maintenance effort, load control, etc. are identified. By using these observed safety indices as a guide, the target safety index and corresponding partial safety factors were determined.

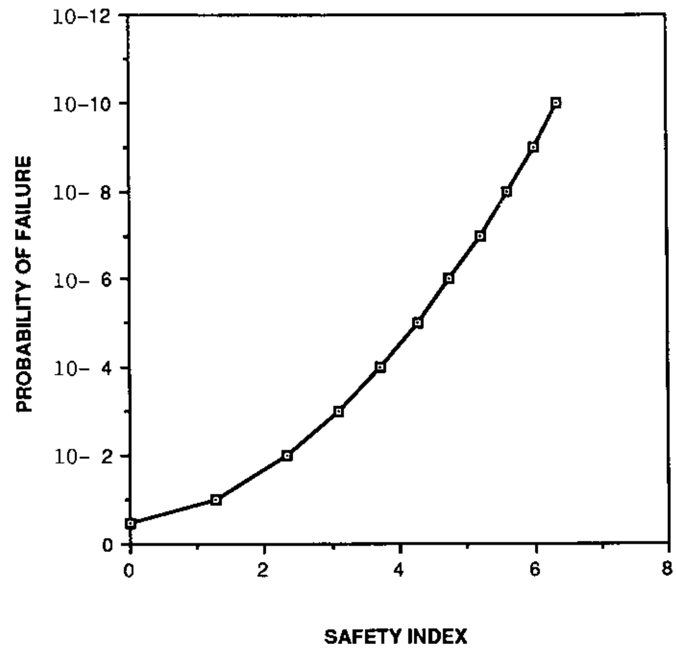

Figure 4. Probability of failure vs. safety index—the safety margin is assumed to be a normal distribution.

CHAPTER THREE

STATISTICAL DESCRIPTION OF DESIGN PARAMETERS

The main objective of the second phase of this project was to pursue the calibration procedure more rigorously. Statistical parameters for the random variables discussed in this chapter were obtained from the literature whenever available. If the available information was limited, the formulation of the design variables was kept in its basic form so that calibrations can be updated as more information becomes available. In addition, the calibration may be repeated when necessary to account for unique environmental or traffic conditions of a particular state or geographic location.

The basic form of the limit state equation is

$$g = R - DL - LL \tag{22}$$

where g is the safety margin as described in Chapter Two, and R, DL, and LL are resistance, dead load effect, and live load effect, respectively. Note also that random variables R, DL, and LL are functions of other random variables.

RESISTANCE

The basic random variables affecting the resistance, R, of reinforced concrete members are the actual concrete strength in tension and compression, the yield strength of steel reinforcement, and the dimensions of the given member. For example, the nominal, ultimate flexural strength of an under-reinforced concrete beam may be expressed as

$$R = A_s f_y \cdot \left(d - \frac{c}{2}\right) \tag{23}$$

where A_s = area of steel reinforcement, f_y = yield stress of steel reinforcement, c = depth of Whitney's compressive stress block, and d = distance from extreme fiber in compression to the centroid of steel reinforcement.

Each of these basic variables is a random variable. In addition, the form of Eq. 23 itself is only a theoretical approximation with inherent random error. To account for this random error, a correction random variable, N_R, may be introduced as follows (*4*):

$$R = N_R \cdot A_s f_y \cdot \left(d - \frac{c}{2}\right) \tag{24}$$

All of the basic variables appearing in Eq. 24 may be considered explicitly in the limit state equation by substituting Eq. 24 for resistance, R, into Eq. 22. Alternatively, however, a statistical analysis for the resistance, R, may be performed, thus allowing R to remain as a single random variable, as shown in Eq. 22. This second approach is the one most commonly used in probability based design codes.

MacGregor and others (*37,38,39*) have recently conducted an extensive review of the available test data and field measurements for reinforced concrete members. They performed Monte Carlo simulation analyses to determine the statistics of the resistance of reinforced concrete members. The purpose of their review was to establish a national standard for building structures. Three major assumptions were made in carrying out the statistical analyses: (1) The variabilities of material strengths and dimensions correspond to average quality construction. (2) Material strengths are representative of relatively slow loading rates (i.e., "static" loading rate for steel yield strength and one hour loading-to-failure for crushing and tensile strengths of concrete). (3) Long-term strength changes caused by increased maturity of concrete, deterioration of concrete, and corrosion of reinforcement were ignored.

The variabilities of the random variables are reproduced in Table 1 for material strength and Table 2 for member dimensions. The error in the modeling, i.e., factor N_R in Eq. 24, was determined by comparing analytical predictions with test results. The bias (mean to nominal ratio) and the coefficient of variation for N_R were determined to be 1.01 and 0.046, respectively.

A few representative member cross sections were selected for the analyses. Comprehensive Monte Carlo analyses were conducted to generate an ensemble of R/R_n for each selected cross section. R_n is the nominal resistance value calculated by the ACI code procedure and R is the resistance obtained from the Monte Carlo analysis. Because the values of R/R_n below the fifth percentile are most important in the reliability study, a normal distribution curve was selected to fit this portion of the data. The statistical data for this normal distribution curve are reproduced in Table 3. Typical values for the bias and the coefficient of variation are 1.10 and 0.12, respectively (*24,37*).

When evaluating an existing structure, assumptions (1) and (3) above need further examination and should be included in the evaluation process. Although the results presented in the studies by MacGregor and others are considered state-of-the-art knowledge about resistance of reinforced concrete members, one possible flaw in applying their hypothesis to bridge structures is that the quality control in bridge construction is normally better than average quality construction.

Table 1. Material properties of concrete and steel reinforcement. [After MacGregor et al. (*29*)]

	Mean	Bias	V	Standard Deviation
Concrete Normal Control				
Compressive strength in structure loaded to failure in one hour.				
f'c = 3000 psi	2760 psi	0.920	0.18	---
= 4000 psi	3390 psi	0.848	0.18	---
= 5000 psi	4028 psi	0.806	0.15	---
Tensile strength in structure loaded to failure in one hour				
f'c = 3000 psi	306 psi	---	0.18	---
= 4000 psi	339 psi	---	0.18	---
= 5000 psi	366 psi	---	0.18	---
Reinforcement				
Grade 40, static yield	45.3 ksi	1.133	0.116	5.3 ksi
Grade 60, static yield	67.5 ksi	1.125	0.098	6.6 ksi

Table 2. Dimensions of reinforced concrete sections. [After MacGregor et al. (*29*)]

	Mean Error	Standard Deviation
(Overall depth--nominal		
Slab (1696 Swedish slabs)	+0.03 in.	0.47 in.
(99 slabs)	+0.21 in.	0.26 in.
Beam (108 beams)	-0.12 in.	0.25 in.
(24 beams)	+0.81 in.	0.55 in.
(Effective depth--nominal)		
One-way slab; top bars		
(1696 Swedish slabs)	-0.75 in.	0.63 in.
(99 slabs)	-0.04 in.	0.37 in.
Values used	-0.40 in.	0.50 in.
One-way slab; bottom bars		
(2805 Swedish slabs)	-0.13 in.	0.34 in.
(96 slabs)	-0.16 in.	0.35 in.
Values used	-0.40 in.	0.35 in.
Beam, top bars	-0.22 in.	0.53 in.
(Beam stem width--nominal width)	+0.10 in.	0.15 in.
(Column width or breadth--nominal)	+0.06 in.	0.25 in.
Cover, bottom steel in beams	+0.06 in. -0.35 in.	0.45 in. 0.28 in.

DEAD LOAD EFFECTS

Two basic variables used in predicting the dead load effect, *DL*, are: (1) analysis variables to account for the uncertainties and bias of the analytical idealization, which transforms loads to load effects; and (2) load intensity and load placement on the bridge.

In the Phase I final report for this project, the total dead load effect was defined as

$$DL = C_D \sum_i A_i D_i \tag{25}$$

where D_i is the load intensity of i^{th} dead load category; A_i is the associated analysis uncertainty variable; and C_D is a deterministic constant converting load intensity to load effect.

With respect to analysis uncertainty, researchers generally agree (*4,37*) that there is an equal tendency to overestimate or to underestimate the dead load effect; hence, the mean value of

Table 3. Resistance statistics. [After MacGregor et al. (29)]

Action	Type of Member	Details	$\bar{R}/R_n$	V_R
Flexure reinforced concrete	Continuous one-way slabs	5 in. thick, Grade 40	1.22	0.16
		5 in. thick, Grade 60	1.21	0.15
	Two-way slabs	5 in. thick, Grade 60	1.16	0.15
		7 in. thick, Grade 60	1.12	0.14
	One-way pan joists	13 in. overall depth, Grade 60	1.13	0.135
	Beams, Grade 40, f'c = 5 ksi	ρ = 0.005 = 0.09 ρ_b	1.18	0.14
		ρ = 0.019 = 0.35 ρ_b	1.14	0.14
	Beams, Grade 60, f'c = 5 ksi	ρ = 0.006 = 0.14 ρ_b	1.04	0.08
		ρ = 0.015 = 0.31 ρ_b	1.09	0.11
		ρ = 0.027 = 0.57 ρ_b	1.05	0.11
		ρ = 0.034 = 0.73 ρ_b	1.01	0.12
Axial load and flexure	Short columns, compression failures	f'c = 3 ksi	1.05	0.16
		f'c = 5 ksi	0.95	0.14
	Short columns, tension failures	f'c = 3 and 5 ksi	1.05	0.12
	Slender columns, kl/h = 20, compression failures	f'c = 5 ksi	1.10	0.17
	Slender columns, kl/h = 20, tension failures	f'c = 5 ksi	0.95	0.12
Shear	Beams with a/d $\geq$ 2.5, ρ_w = 0.008	No stirrups	0.93	0.21
		Minimum stirrups	1.00	0.19
		$\rho_v f_y$ = 150 psi	1.09	0.17
	Flat plate slabs	No shear reinforcement	1.34	0.27

A_i was set at 1.0. The coefficients of variation of A_i should decrease as the levels of sophistication and rigor in the chosen analytical procedure increase (*4,31*).

Rosenblueth (*55*) cited five examples where dead load intensities and their effects were significantly underestimated. Four major sources of error were reported: (1) Actual thickness of structural elements almost systematically exceed their nominal values (*33*). Typical examples are those listed in Table 2. (2) Evaluation failed to consider portions of the structural or nonstructural elements. (3) The unit weight provided by the material manufacturer tends to be less than the actual average. (4) Major discrepancies may have been caused by architectural changes or structure modifications not shown on the plans.

These errors point to the likelihood that the dead load effects may not have been accurately predicted. Bias coefficients of the total dead load effects of 1.10 to 1.25 had been suggested in the literature for buildings (*55*). However, in reinforced concrete bridge structures, a large portion of the dead load is contributed by the weight of the structural concrete section. The only nonstructural elements generally contributing to the dead load of a bridge are the railing and the A.C. overlay. As demonstrated in Chapter Five, the most significant contribution to the dead load effect is due to the self-weight of the structural section. The bias coefficient and coefficient of variation assigned to this dead load category dominate all the dead load categories and thus will be considered as the basic random variable for dead load in the calibration. Galambos et al. (*24*) suggested using 1.05 for the bias and 0.08 for the coefficient of variation for the total dead load. These values have also been adopted in the Canadian Standard Association's specifications for highway bridge design (*10*) for field cast concrete with good control. Based on this recommendation, the values suggested in the Phase I final report (*31*) seem appropriate, i.e., $\delta_D = 1.05$ and $V_D =$ 0.05 to 0.10.

LIVE LOAD EFFECTS

Four basic variables used in predicting the live load effect, *LL*, are: (1) analysis variables to account for the uncertainties and bias of the analytical idealization, (2) load intensity and load placement on the bridge, (3) the dynamic (impact) effect, and (4) load combinations to account for the multiple presence of trucks on the bridge.

Live Load Model

Live load modeling, which includes these variables, is currently the most complicated aspect in the limit state equation for highway bridges. Extensive analytical treatment (*26,27,32*) and limited field measurements (*25,44*) have contributed to identify the most important parameters in the live load model, which include: (1) truck configuration (truck type, axle spacing, and axle percentage of weight distribution); (2) gross truck weight; (3) multiple presence of trucks on the bridge (including lane occupancy effect, which is dependent on the headway distance between trucks, truck traffic volume, span length, and side-by-side effect that depends on the maximum number of traffic lanes); and (4) impact, which is dependent on the roadway roughness, vibration characteristics of the superstructure, and dynamic characteristics of the vehicle.

Although the important parameters have been identified, the greatest uncertainty comes from probable future trends to increase the weight of vehicles (*44*).

In the Phase I final report, the equation for the live load effect model was defined as

$$LL = C_L\, BL\, (1 + I) \tag{26}$$

Table 4. Characteristic truck weight, W, (95 percentile value) based on the truck traffic volume for the two vehicles suggested by Moses.

Truck Traffic Volume	W (kips) Single	Semi
Light (200 Trucks/day)	38	68
Moderate (2,000 Trucks/day)	47	75
Heavy (10,000 Trucks/day)	52	82

Table 5. Mean multiple presence factor, H, based on the truck traffic volume and span length.

Truck Traffic Volume	Span Length	$\bar{H}$		
		< 40 ft	40 ft-100 ft	>100 ft
Light (200 Trucks/day)		2.2	2.42	2.64
Moderate (2,000 Trucks/day)		2.4	2.64	2.88
Heavy (10,000 Trucks/day)		2.5	2.75	3.00

where L is the equivalent load intensity, B is the analysis uncertainty variable, I is the dynamic impact effect, and C_L is a deterministic constant converting load intensity to load effect. Moses (*25,44*) recently proposed a more pragmatic live load model in which the formulation for the maximum moment, LL, in superstructures is expressed as:

$$LL = a m W H g\,(1+I) \tag{27}$$

where a = deterministic constant which converts loads to load effects, m = random variables accounting for variations in the axle spacing and the axle percentage of weight distribution, W = truck weight random variable, H = random variable accounting for the multiple presence of vehicles, g = girder distribution variable accounting for the analysis uncertainties, and I = impact.

It should be noted that Eqs. 26 and 27 can be made equivalent in the Second-Moment Method used herein. However, Eq. 27 provides more flexibility to incorporate the contribution of each basic random variable. The uncertainty in the live load effect is believed to be larger than that of the resistance and dead load effects.

In this study, Eq. 27 is simplified to

$$LL = a W H g\,(1+I) \tag{28}$$

where W is the characteristic truck weight (95th percentile value), and the uncertainty in W and the random variable m, as defined in Eq. 27, are lumped into the random variable H. Based on weigh-in-motion studies, Moses (*41*) suggested tentative values for the characteristic truck weight, W, mean multiple presence factor, $\bar{H}$, and the coefficients of variation for the multiple presence factor, V_H, which are given in Tables 4, 5 and 6, respectively. The characteristic trucks used in Table 4 are explained in detail in the next section.

Converting Live Load Moments Obtained in Field Measurements to the HS20 Design Truck

As previously mentioned, one of the parameters in the live load model is the truck configuration which includes truck type, axle spacings, and percentage of axle weight distribution. Based on field measurements, Moses suggested two vehicle configurations, a single body truck (designated "single") and a tractor semi-trailer (designated "semi"), as representative trucks on the highway that are shown in Figure 5 (a,b). The maximum live load effect on a bridge structure will be the larger load

Table 6. Coefficients of variation for the multiple presence factor. V_h, based on the weight control and span length.

Weight Control	Span Length	V_H		
		< 40 ft	40 ft-100 ft	>100 ft
Enforced		0.20	0.17	0.15
Unenforced		0.23	0.20	0.18

effect due to either one of the two trucks, depending on the span length.

However, in design and rating practice, the AASHTO HS standard truck is used to standardize the maximum live load

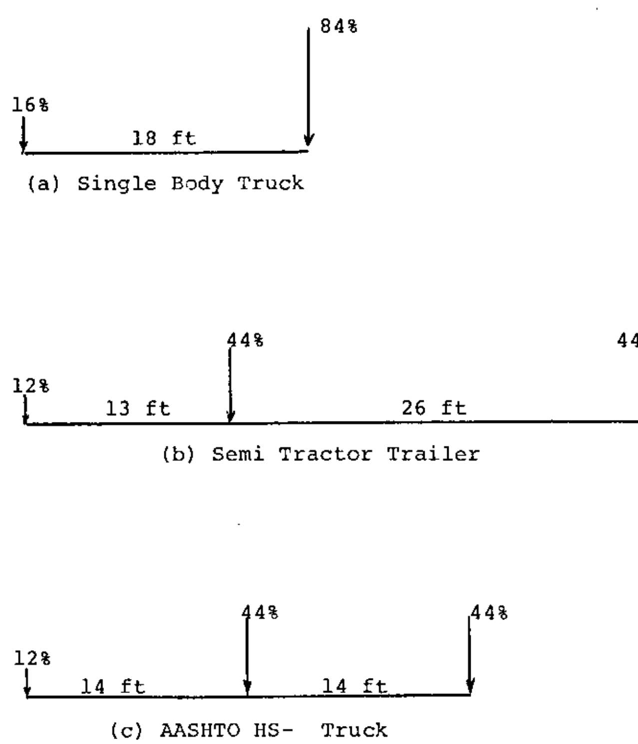

Figure 5. Truck configurations for a single body truck and a tractor semi-trailer truck suggested by Moses, and an AASHTO HS20 truck.

effect. To make use of the field data obtained from weigh-in-motion measurements (*43,45*) such that the actual truck loading on the highway may be estimated, it is necessary to convert this HS20 live load effect to the two load configurations suggested by Moses. A sketch of the HS20 design truck showing the axle spacing and percentage axle weight distribution used for the conversion is included in Figure 5(c).

Parameter studies were conducted to compute the moment envelopes for the HS20 design truck and the two trucks proposed by Moses for simple-span bridges with span lengths ranging from 5 ft to 100 ft. The longitudinal moment envelopes were computed using a plane frame analysis program (BDS: Bridge Design System (*20*)). The maximum values of the moment envelopes due to unitized gross vehicle weights are given in Table 7.

The characteristic truck weights (95 percentile value) for the two representative configurations and different truck traffic volumes reported by Moses are given in Table 4. Using the gross vehicle weight of 72 kip for an HS20 truck, the moment ratio, r_M, is defined as

$$r_M = \frac{M_{Vehicle}}{M_{HS20}} = \frac{a^*W}{a_{HS20}{}^*72} \qquad (29)$$

where a and a_{HS20} are the generalized influence coefficients for moment listed in Table 7. Using the load intensities, W, for the three designated categories of truck traffic volume, moment ratios, r_M, for the two trucks were computed for span lengths up to 100 ft. Plots of these moment ratios versus span length for heavy, medium, and light truck traffic are included in Figures 6(a), 6(b), and 6(c), respectively.

Table 7. Maximum longitudinal moment influence coefficients, *a*, due to HS20 design truck and trucks suggested by Moses for unitized gross vehicle weight.

Span Length (ft)	Type of Truck: HS 20	Single	Semi
5	0.56	1.05	0.56
10	1.11	2.10	1.11
20	2.22	4.20	2.22
30	3.92	6.30	3.42
40	6.25	8.56	4.82
50	8.72	11.06	6.48
60	11.20	13.55	8.89
70	13.69	16.06	11.28
80	16.18	18.56	13.69
90	18.67	21.06	16.09
100	21.17	23.56	18.50

Gross Vehicle Weight = 1

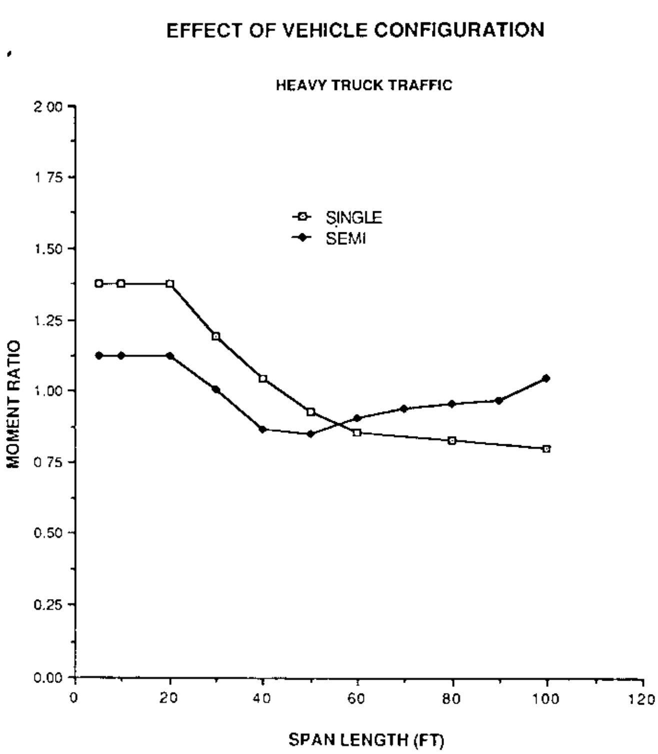

Figure 6(a). Moment ratios for converting simple span longitudinal moments for the AASHTO HS20 truck to representative trucks suggested by Moses—heavy truck traffic.

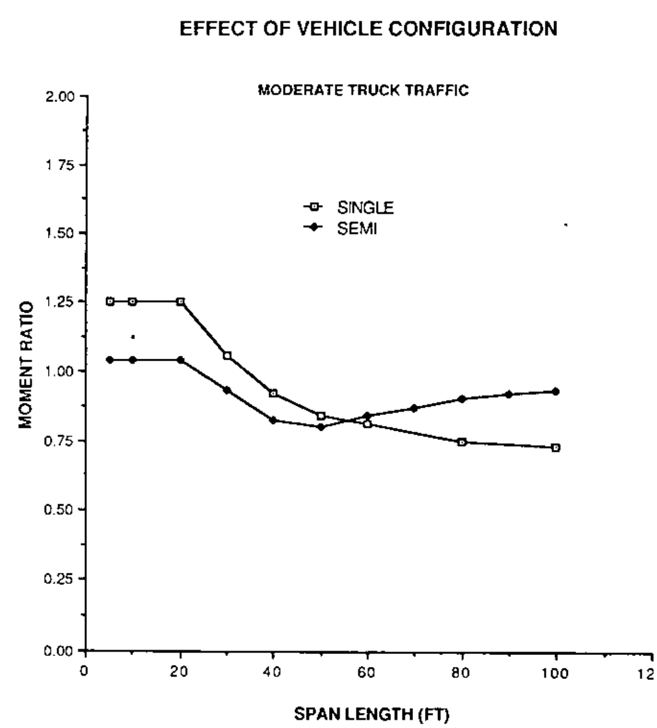

Figure 6(b). Moment ratios for converting simple span longitudinal moments for the AASHTO HS20 truck to representative trucks suggested by Moses—moderate truck traffic.

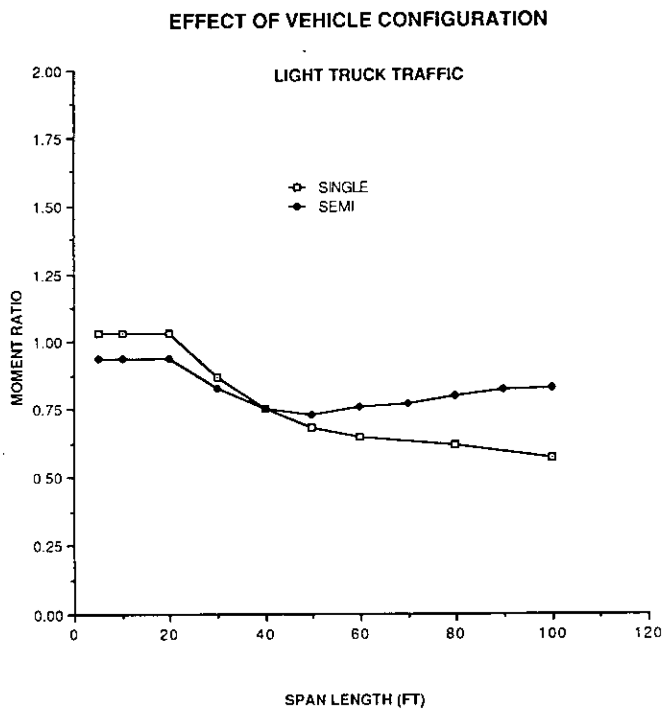

Figure 6(c). Moment ratios for converting simple span longitudinal moments for the AASHTO HS20 truck to representative trucks suggested by Moses—light truck traffic.

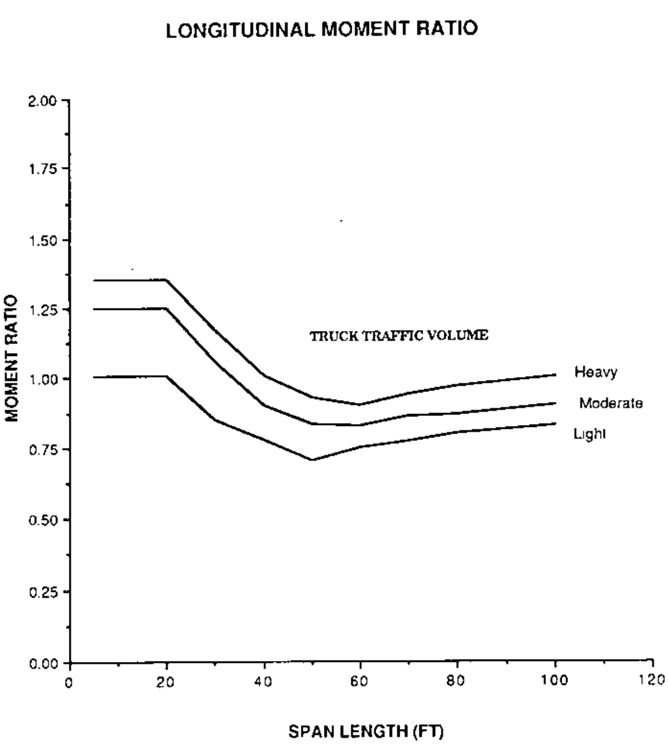

Figure 7. Moment ratio based on the controlling single or semi-trucks for varying degrees of truck traffic volume.

For span lengths shorter than about 60 ft, the maximum longitudinal moment is generally governed by the single body truck; while for span length longer than 60 ft, the tractor semi-trailer governs.

To estimate the maximum moment corresponding to those measured by Moses, the HS20 moment should be multiplied by a conversion factor, which is simply the larger of the two moment ratios plotted in Figures 6(a), 6(b), and 6(c). The controlling moment ratios for the three truck traffic volumes are plotted in Figure 7 and are given in Table 8 for span lengths considered.

Load Distribution of Concrete T-Beam Bridges

In the current AASHTO code, the girder distribution factor, *g*, i.e., number of wheel lines per girder, is defined in terms of girder spacing *S*,

$$g = \begin{cases} S/6.5 & \text{for a bridge designed with one lane loaded} \\ S/6 & \text{for a bridge designed for two or more lanes} \end{cases}$$

To study the lateral load distribution among girders and to examine the accuracy of the current AASHTO "S-over" formula, a series of simple-span, five-girder T-beam bridges were studied herein. The girder depth, slab thickness and overhang width was based on typical values as defined in Appendix C of the Phase I report (*31*). Span lengths of 30, 45, 60, and 80 ft and girder spacings of 4, 5, 6, 7, 8, and 9 ft are considered. This comprised a family of 24 bridges.

Vehicle loadings in both one and two lanes were considered. Trucks were placed on the bridge to maximize the moments in both the interior and exterior girders.

Table 8. Live load moment correction factor, r_M, to be applied to AASHTO truck for correlation with that actually measured by Moses.

Span Length (ft)	Traffic Volume		
	Light	Moderate	Heavy
20	1.01	1.25	1.38
30	0.86	1.06	1.18
40	0.74	0.91	1.01
50	0.71	0.84	0.93
60	0.76	0.84	0.91
70	0.78	0.87	0.95
80	0.80	0.89	0.97
90	0.82	0.90	0.99
100	0.83	0.91	1.00

Note: Above the dash line, Single Truck controls. Below the dash line, Tractor Semi-Trailor controls.

Two methods of analysis were employed. In the first method (*40*), the bridge superstructures were modeled with a grillage system composed of interconnected longitudinal and transverse beam elements. The longitudinal beams, which modeled the girder, were placed eccentric to the middle surface of the deck slab using a rigid link; the transverse beams, modeling the transverse flexural effect, were placed at the middle surface of the deck slab.

The second method (*34*) used was a semi-analytical procedure. The flexural response was decomposed into a longitudinal variation and a transverse variation, $w(x,y) = u(x)*v(y)$.

The longitudinal variation, $u(x)$, is represented by Fourier series; while the transverse variation, $v(y)$, is modeled by discrete elements. The structure is considered as an assemblage of interconnected rectangular elements, simply supported at two ends. The applied forces are resolved into Fourier series components. A direct stiffness analysis based on classical thin plate bending theory and plane stress elasticity theory is carried out for each Fourier component. The final results are obtained by summing the solutions of all Fourier components. This type of analysis is efficient and accurate; but is limited to bridges without skew or curve. Furthermore, cross-sectional properties must be constant for the entire length of the structure.

Both methods gave essentially the same results (the differences were within 4 percent). The second method is simpler to apply, but with some restrictions as noted above. The first method is more versatile for any kind of nonprismatic variation, skewed support, etc.

A cross section of the representative five-girder T-beam bridge is shown in Figure 8. Also, shown in the figure are the positions of the wheel lines to produce the maximum girder moments.

The wheel load distribution between the girders for the 45-ft span bridges are shown in Figures 9 and 10 for one lane loaded case and in Figures 11 and 12 for two lanes loaded case. Due primarily to the torsional rigidity of the superstructure cross section as a whole, exterior girders carry much higher truck load than the interior girders. In comparing Figures 10 and 12, the contribution due to the second (adjacent) loaded lane, which increases with larger girder spacing, may be determined. At 9-ft girder spacing, the contribution from the second loaded lane is about 34 percent of the first loaded lane. Contribution due to additional truck loads (if possible) will be much less because they will have to be placed further away from the exterior girder under consideration. For example, the contribution to the exterior girder load from the third loaded lane (for 9-ft girder spacing) is about 7 percent (estimated from Fig. 9) of the first loaded lane.

Figures 13 and 14 show the wheel load distribution factor *g*,

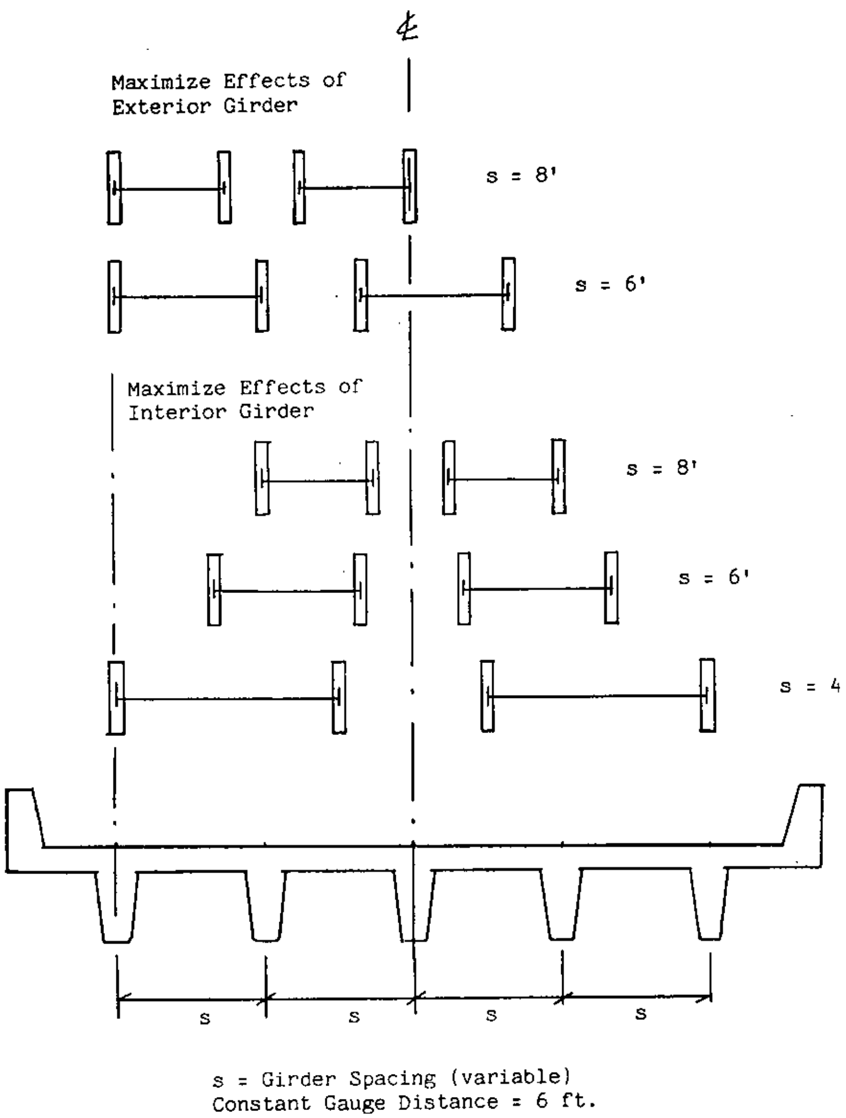

Figure 8. Schematic of a typical five-girder T-beam bridge and the placements of two trucks with wheel gages of 6 ft.

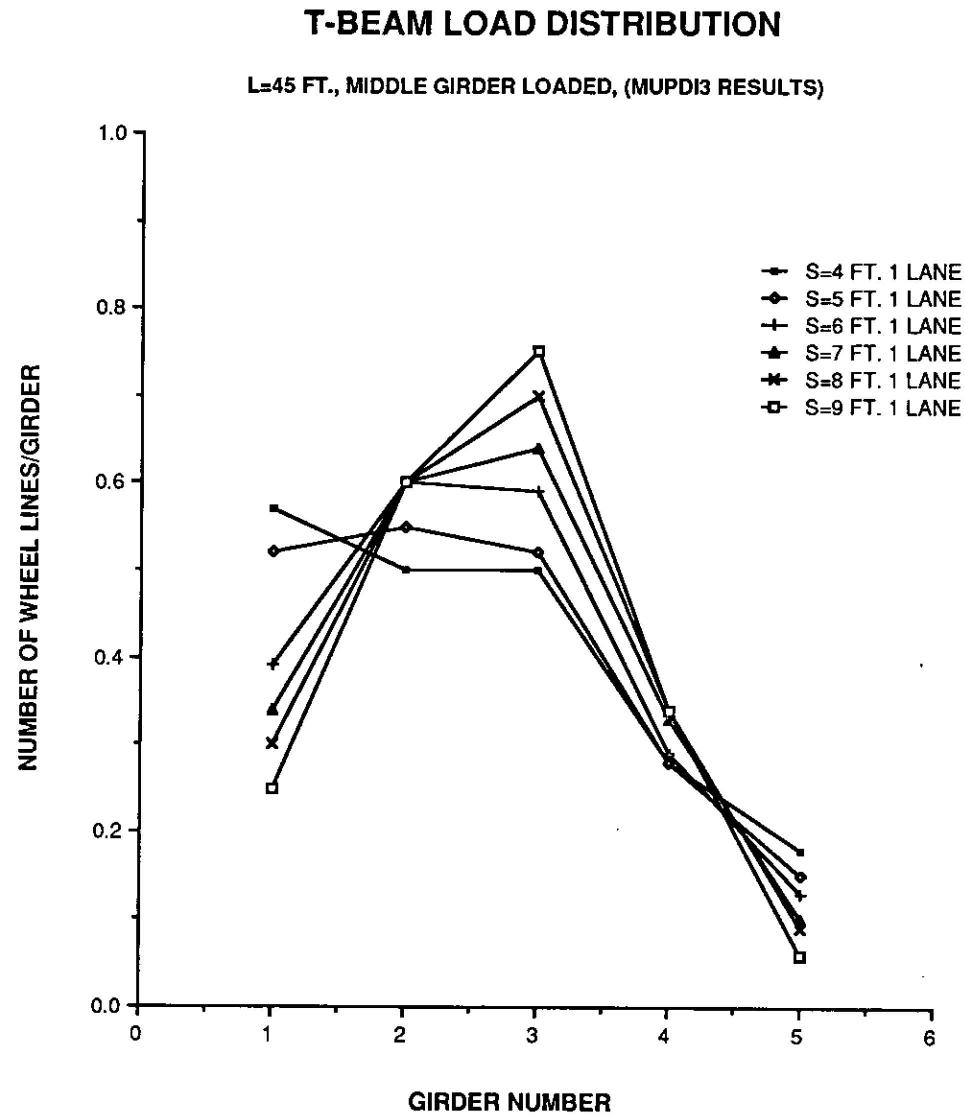

Figure 9. Wheel load distribution for a typical T-beam bridge ($L = 45$ ft) with varying girder spacings with one lane loaded to maximize the central girder load.

with respect to girder spacing for interior and exterior girders, respectively. Results for four different span lengths are based on the case with two lanes loaded. For interior girders, g decreases with increasing span length; whereas the contrary is true for the exterior girder. In any case, the variation of g with respect to span length is much smaller than its dependence on the girder spacing. Also shown in the figures are the empirical AASHTO relationships for girder distribution. Even for the simple structural configuration (straight, simply supported, no skew) considered here, AASHTO values tend to be conservative for interior girders with girder spacings greater than 8 ft, and unconservative for exterior girders. Further examination of the wheel load distribution problem is beyond the scope of this study. An ongoing study (NCHRP Project 12-26) is devoted to evaluating the adequacy of existing empirical formulas for different structural configurations and the analytical procedures available. At this stage, it is assumed that the analytical solutions obtained may apply to all reinforced concrete girder bridges.

The numbers of T-beam bridges at each span length range were obtained from a survey of the National Bridge Inventory File (NBIF) for the maximum span lengths of each bridge type, as shown in the histogram included in Figure 15. Assuming that for a given girder spacing, the distribution of span length is the same as shown in Figure 15, the weighting factor based on the percentage of bridges in the span length range may be assigned to each span length accordingly. The weighting factors corresponding to the four ranges of span length are given in Table 9.

Thus for interior girders, the mean girder distribution, $\bar{g}$, for the given spacing may be obtained as a weighted average of the girder distribution corresponding to the four span lengths for girder spacing ranging from 4 to 9 ft, as shown in Table 10. Also shown in the table is the computed bias, δ_g, of the AASHTO to the mean girder distribution $\bar{g}$ determined analytically. A coefficient of variation of 5 percent for the girder distribution was used in the calibration.

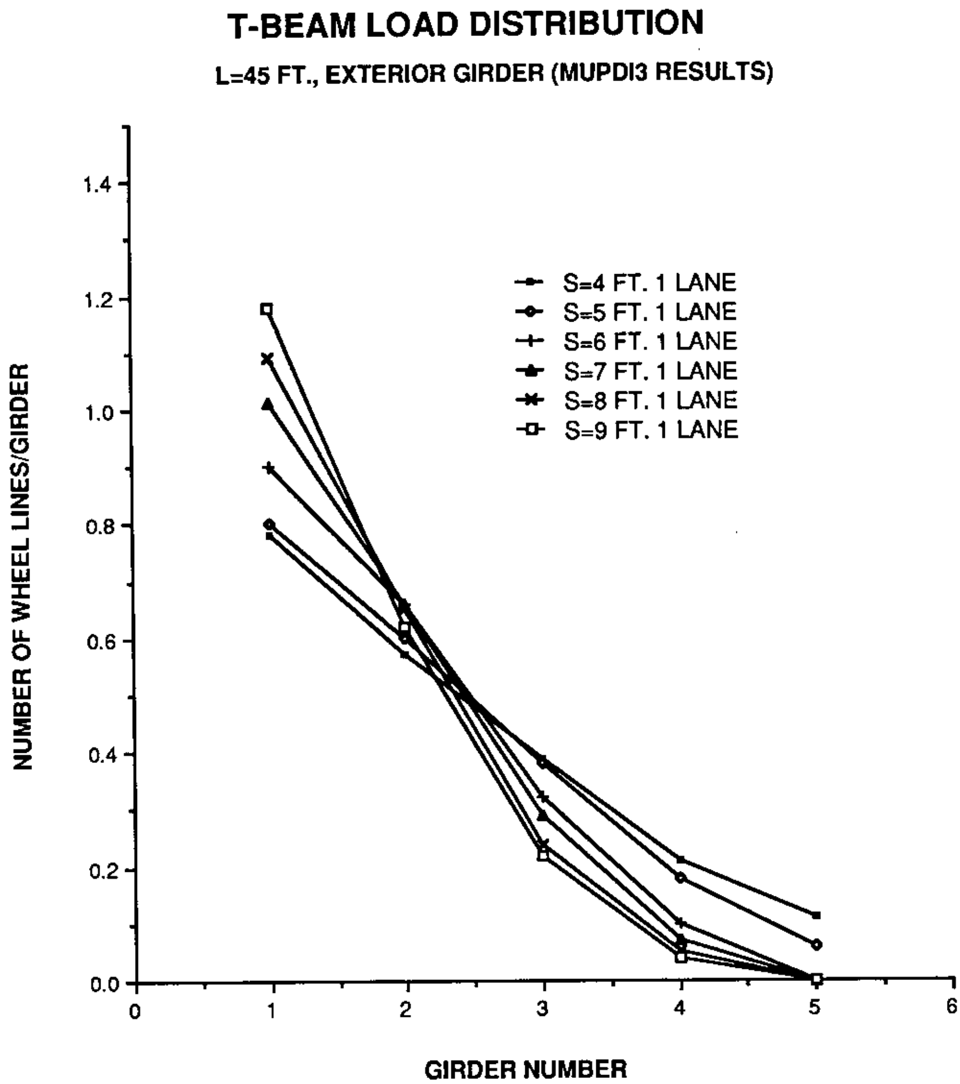

Figure 10. Wheel load distribution for a typical T-beam bridge (L = 45 ft) with varying girder spacings with one lane loaded to maximize the exterior girder load.

Table 9. Weighting factors for the span length of T-beam bridges based on the NBIF (1985).

Span Length L (ft)	Weighting Factors
30	0.63
45	0.18
60	0.14
80	0.05

Table 10. Mean values and bias coefficients for wheel load distribution in T-beam bridges for a given girder spacing.

Girder Spacing S (ft)	4	5	6	7	8	9
$\bar{g}$	0.777	0.897	1.076	1.198	1.305	1.423
δ_g	1.166	1.076	1.076	1.027	0.979	0.949

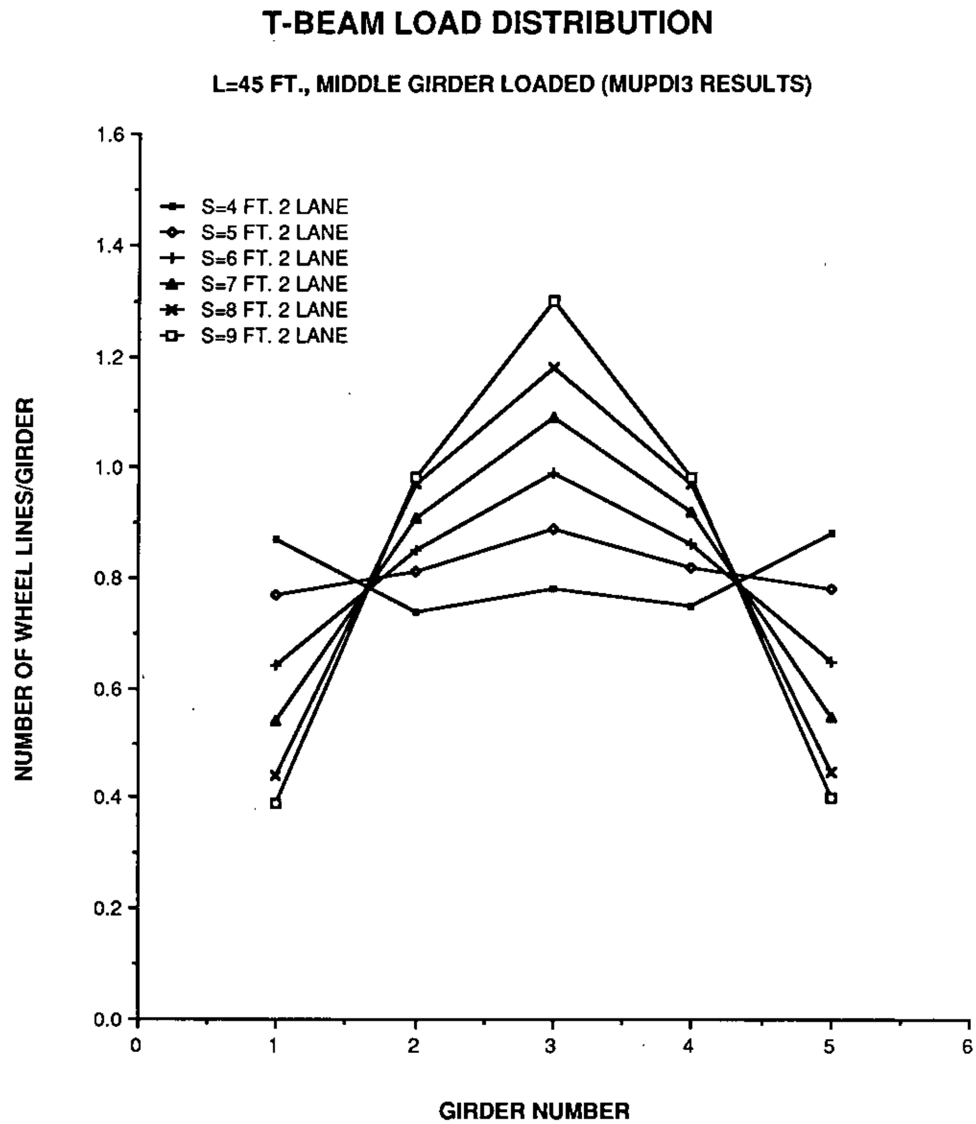

Figure 11. Wheel load distribution for a typical T-beam bridge ($L = 45$ ft) with varying girder spacings with two lanes loaded to maximize the central girder load.

Load Distribution of Concrete Slab Bridges

In the AASHTO code,the load distribution of slab bridges is specified as a distribution width, E, for each wheel line loading, expressed as $E = 0.06\ L + 4 \leq 7$ ft. The girder distribution factor g, i.e., number of wheel lines per unit slab width, is $g = (1)/(E)$.

To evaluate the accuracy of the AASHTO formula, a series of slab bridges with span length ranging from 5 ft to 100 ft (i.e., spans 5, 15, 25, 40, 50, 60, and 100 ft) and width of 25 ft and 50 ft were studied using the two methods described previously. Two trucks and four trucks are placed close to the edge of 25 ft and 50 ft wide bridges, respectively. Both methods gave essentially the same results.

Figure 16 shows the lateral distribution of moment for bridges with 25-ft span length.

Figure 17 shows the distribution width E obtained from these analyses and also the AASHTO E values. Except for span length less than about 20 ft, the AASHTO formula seems to overestimate the wheel line distribution width considerably, resulting in an unconservative design.

The histogram included in Figure 18 shows the number of slab bridges within each span length range. Weighting factors corresponding to each span length range are given in Table 11.

The mean values of lateral distribution, $\bar{g}$, are 0.193 and 0.206 for two-lane and four-lane bridges, respectively. The bias coefficients, δ_g, reflect the differences between g (i.e., $1/E$) given by the AASHTO formula and those obtained analytically. The bias coefficients for two and four lanes for span lengths 15 to 60 ft are given in Table 12.

As for the T-beam bridges, a coefficient of variation of 5 percent will be used for the slabs in the subsequent calibration.

Impact

The impact effect is the additional dynamic response caused by the vehicle-induced bridge vibration. The vehicle-bridge in-

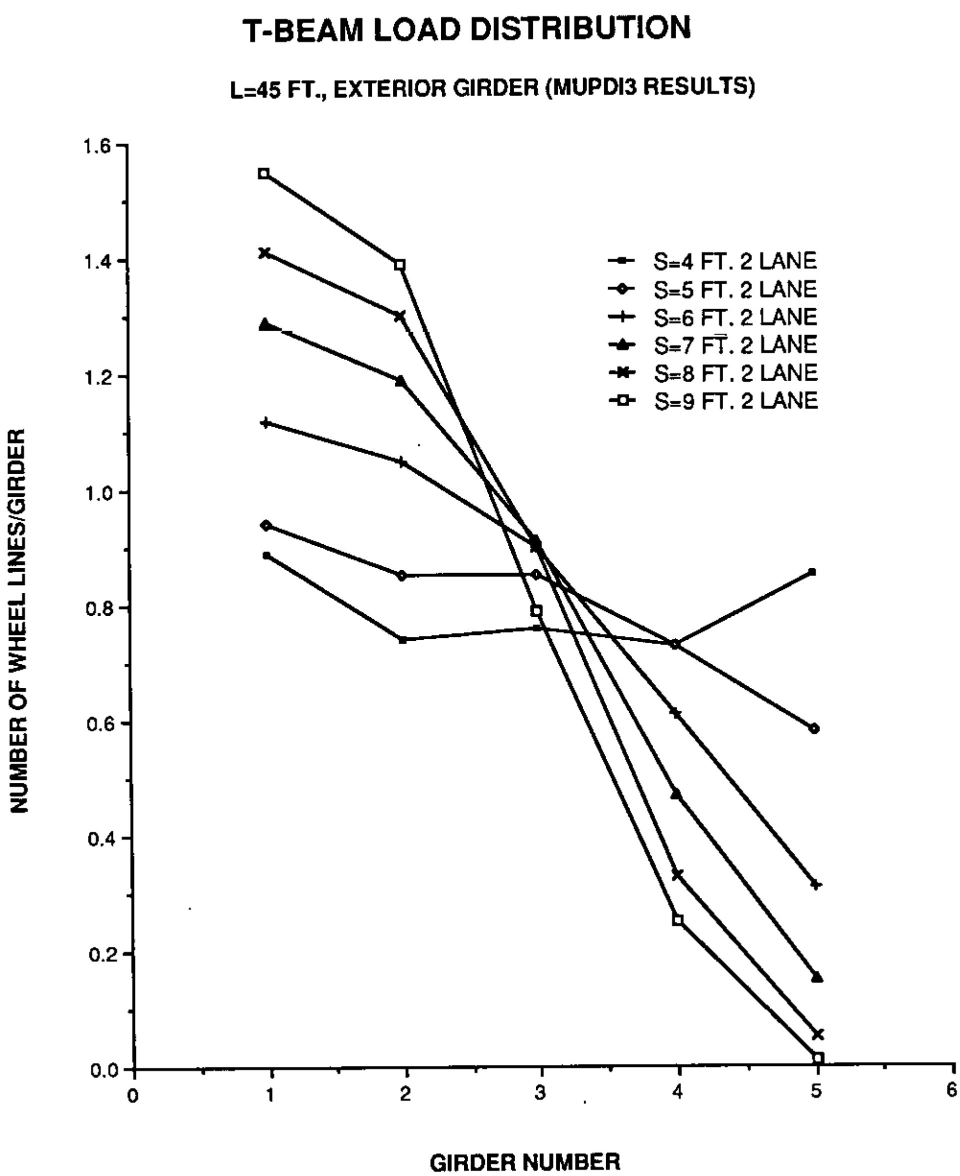

Figure 12. Wheel load distribution for a typical T-beam bridge (L = 45 ft) with varying girder spacings with two lanes loaded to maximize the exterior girder load.

teraction is governed by the following factors: (1) vehicle vibrations (bouncing, pitching, and wheel hopping modes), (2) natural frequency of bridges, and (3) roadway roughness and vehicle speed.

The majority of existing bridge design and rating codes relates the impact effect to the maximum span length. The recent *Ontario Highway Bridge Design Code* (*50*) assigns the impact factor directly as a function of the bridge's fundamental frequency of vibration. Implicitly, this assumes that the excitations imparted to the bridge, due to combined vehicle vibration and roadway roughness, are within two specific frequency ranges. These are the body bounce frequency in the range of 2 to 5 Hz and the "wheel hop frequency" in the 10- to 15-Hz range. Whenever the fundamental frequency of a bridge is within these ranges, the impact effect is amplified due to the presumed resonance.

The physical phenomenon is, however, much more complicated. The vehicle suspension is a nonlinear mechanical system and its frequency will decrease with increasing amplitude of excitation. The roadway roughness may be characterized by the amplitude A and the wavelength λ (*57*). For vehicles moving with speed V, the excitation frequency is $f = V/\lambda$. Depending on the vehicle speed, V, and roadway roughness, (λ, A), different vehicle vibration modes will be excited.

Theoretically, resonance may occur if these frequencies are tuned to each other. Although resonance may occur under a single vehicle loading; it is, however, unlikely to occur under multiple random traffic loading conditions. There is only a small likelihood that vehicles undergoing "impact" motion will be completely in-phase with one another.

For beam and slab bridges with span lengths less than about

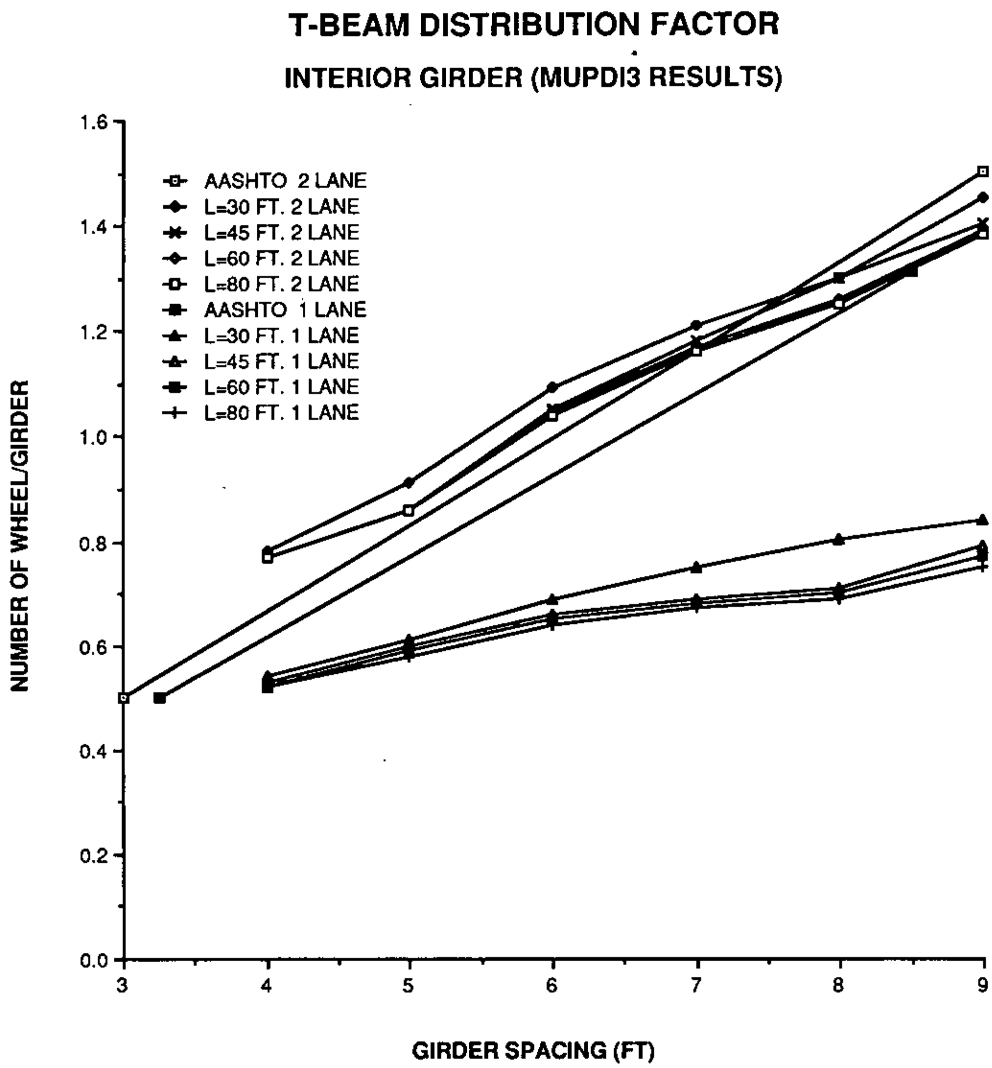

Figure 13. Wheel load distribution factor, g, determined by using MUPDI3 for the interior girders of typical T-beam bridges.

82 ft, the fundamental flexural frequency is usually higher than 5 Hz (*51*). The vehicle bouncing frequency is typically 2 to 4 Hz or lower. Therefore, for the class of bridge structures under consideration, the resonance amplification is very unlikely. Rather, the bridge structure will undergo forced vibration with excitation frequencies depending mainly on vehicle speed and roadway roughness.

Although higher impact factors have been reported in isolated physical tests (*12*) they have been caused by test vehicles that are lighter than the fully loaded design vehicle.

Based on weigh-in-motion measurements of bridges under normal traffic, Moses (*41*) suggested that impact may be assigned based on the roadway roughness. Accordingly, the following mean impact factors were used in the calibration and the recommended rating guidelines.

For smooth roadway $\overline{(1+I)} = 1.10$

and for rough roadway $\overline{(1+I)} = 1.20$

A coefficient of variation of 8 percent was used for the impact factors, $(1+I)$, in the calibration.

Multiple Occurrences of Trucks

The maximum live load effect is usually caused by the multiple occurrences of trucks on the bridge, i.e., side-by-side or same lane occupancy effects. To account for this effect, a multiple presence factor, H, is introduced into the load model in Eq. 28: $LL = aWHg\,(1+I)$.

In deriving the statistics for the random variable H in Eq. 28, all trucks in the site are considered as random trucks. Ideally, it would be desirable to formulate load models to account for the mixed occurrences of random trucks and the much heavier permit trucks. As suggested by Moses et al. for a two-lane bridge (*41a*), the load model may be revised for the occurrence of a permit truck alongside with a random truck,

$$LL = (aW\alpha + P)\,C_h\,g(1+I) \tag{30}$$

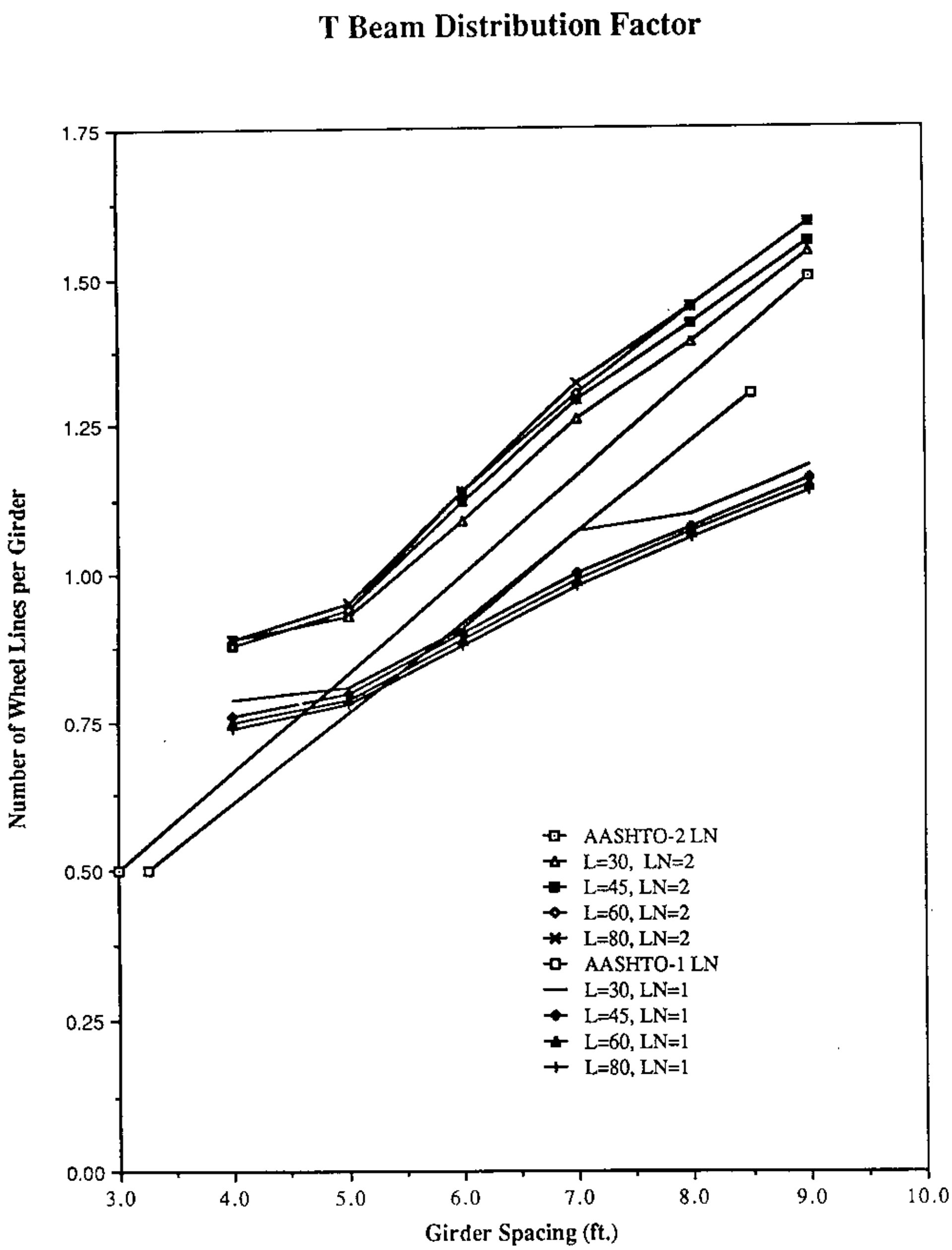

Figure 14. Wheel load distribution factor, g, determined by using MUPDI3 for the exterior girders for typical T-beam bridges.

where special permit trucks are separated from remaining random trucks. In Eq. 30, α is the overload variable for a single-lane random truck, P is the maximum moment (random variable) due to a permit truck alongside, and C_h is a correction variable to account for the same lane occupancy effect. Furthermore, for the rare case of the side-by-side occurrence of two permit trucks, the live load model may be written as:

$$LL = 2P\ C_h\ g(1 + I) \tag{31}$$

For a two-lane bridge, the foregoing equations cover all possible truck occurrence combinations. If the load data are sufficient to provide all necessary statistics, such as percentage of permit trucks and the probability of the mixed truck occurrences in the two-year rating period, the permit trucks may be isolated from the remaining random trucks and a general checking format such as presented in Eqs. 1 and 2 may be derived. However, the data available are limited, and it is out of the scope of this study to address this issue. In the forthcoming NCHRP 12-28(11) project entitled "Development of Site-Specific Load Models for Bridge Rating," this important issue undoubtedly will be addressed more carefully.

As presented by Moses et al. (*41a*) the multiple presence factor, *H*, is calculated by the following four steps:

1. Calculate the histogram of the total truck weight of side-by-side occurrences.

2. Project the expected maximum weight for a rating period of two years based on the total (side-by-side) weight histogram.

3. Determine the ratio of this expected maximum weight to that of the 95 percentile characteristic weight of a single truck.

4. H is obtained by multiplying the ratio by a correction factor C_h to account for the same lane occupancy effect.

Note that most of the live load data available to date are based on measurements of two-lane bridges (41a). As is evident from section under "Load Distribution of Concrete T-Beam Bridges," the third loaded lane has only a minimal effect on the maximum moment in a bridge component. In addition, the probability of three lanes simultaneously loaded is much less than two. Based on these considerations, it is concluded that the maximum load effect in a bridge component is governed by the two-lane loaded case. Thus, the data gathered in determining the multiple presence factor, H, is consistent for use in bridges with two or more lanes. An example is included in the following section to demonstrate how the HS20 moment effect can be converted to the "average" random trucks as shown in Figure 5.

Example—Woodbridge Bridge, California

- Single span, simply supported, two lane, T-beam bridge.
- Span length = 26 ft.
- Girder spacing = 6.52 ft.
- For one HS20 truck, the maximum moment at midspan is $M_{HS20} = [(32)(26)]/4 = 208$ kip-ft.
- AASHTO girder distribution: $g_{AASHTO} = 6.52/6 = 1.0867$ wheel lines.
- AASHTO impact: $1+I = 1.33$ (use 1.3).
- Therefore, the design maximum girder moment is: $M^G_{max} = (208/2)(1.0867)(1.3) = 147$ kip-ft.

For calibration purpose, the mean maximum girder moment based on limited field measurements and an improved response model is expressed in the format of Eq. 28. The longitudinal moment ratio is applied to obtain maximum girder moment caused by the two trucks that have been suggested by Moses as being representative of vehicles on the highway.

From Eq. 29, the moment effect caused by one of the measured trucks is: $aW = r_M * M_{HS20} = r_M * a_{HS20} * (72)$.

From Figure 7, the longitudinal moment ratio for span length of 26 ft and light traffic is: $r_M = 0.92$.

At a particular site, the characteristic truck weight, W, depends on the controlling truck configuration and traffic volume (Table 4). Since these two factors have been taken into account in determining r_M, aW as appeared in Eq. 28 can be replaced by the product of r_M and the HS20 moment, M_{HS20}, which is readily available.

Other parameters needed in calibration are:

- $\overline{H} = 2.2$; $V_H = 0.23$ (Tables 5 and 6).
- $V_g = 0.05$ (assumed).
- $\delta_g = 1.051$ (Table 10).
- $\overline{g} = \delta_g * g_{AASHTO} = 1.051\ (1.0867) = 1.14$.
- $\overline{(1+I)} = 1.10$; $V_{1+I} = 8$ percent (smooth roadway).
- Therefore, the expected maximum moment due to a single random truck is: $M^G = a * W = r_M * M_{HS20} = (0.92)(208) = 191$ kip-ft.

Accounting for the effect of multiple truck occurrence, the expected maximum girder moment is: $M^G_{max} = aW\overline{H}\overline{g}\ \overline{(1+I)} = (191)(2.2)(1.14/2)(1.10) = 264$ kip-ft.

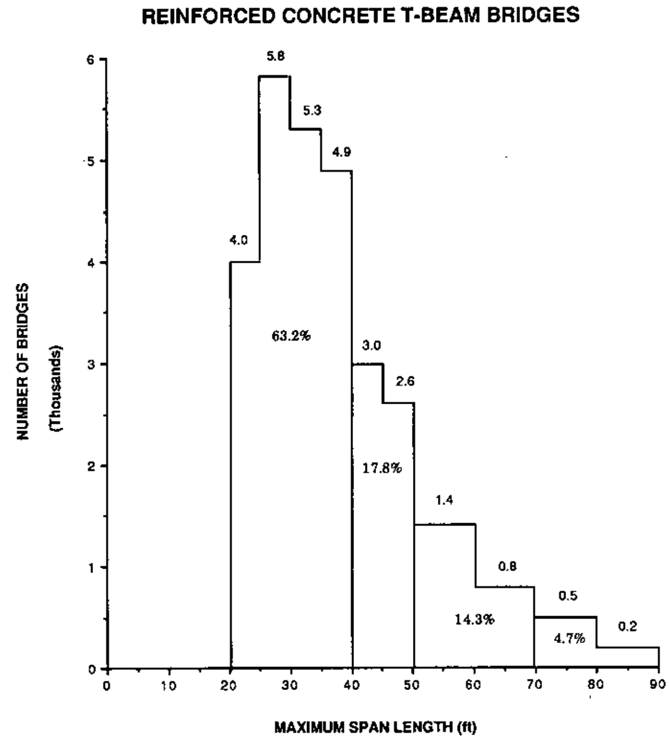

Figure 15. Histogram showing the distribution of maximum span length for reinforced concrete T-beam bridges in the United States (NBIF 1985).

Live Load Effect for Routing Rating

To conduct routing bridge rating, AASHTO has three legal trucks (Type 3, Type 3S2, and Type 3-3) in addition to the HS-design trucks. In previous discussions, the AASHTO HS20 truck was used. However, it is not difficult to convert the moment effect from one truck to another type of truck, as illustrated by the longitudinal moment ratio.

An essential part of this study is to calibrate load factors based on measured truck load data. Strictly speaking, the live load effect, L, should be computed for the controlling single or semi-trucks as suggested by Moses, and the rating factor corresponding to the actual truck traffic may be determined as follows: $RF = \phi R - \gamma_D D/\gamma_L L$

For HS20 truck ratings using the same load factor, the rating factor is:

$$RF_{HS20} = \frac{\phi R - \gamma_D D}{\gamma_L M_{HS20}} = \frac{L}{M_{HS20}} * \frac{\phi R - \gamma_D D}{\gamma_L L} = r_M * RF \quad (32)$$

The rating factor for the actual truck traffic can be obtained by a simple conversion

$$RF = \frac{1}{r_M} RF_{HS20} \quad (33)$$

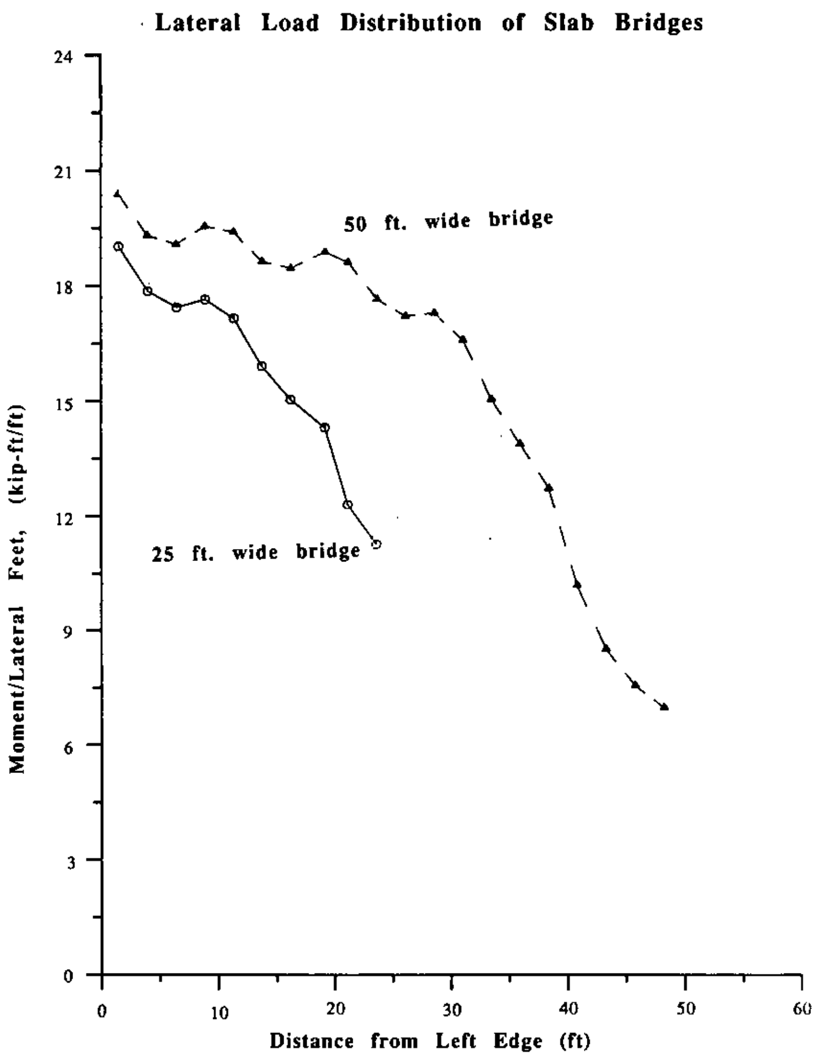

Figure 16. Transverse distribution of longitudinal moment in a typical slab bridge (L = 25 ft), determined by using MUPDI3.

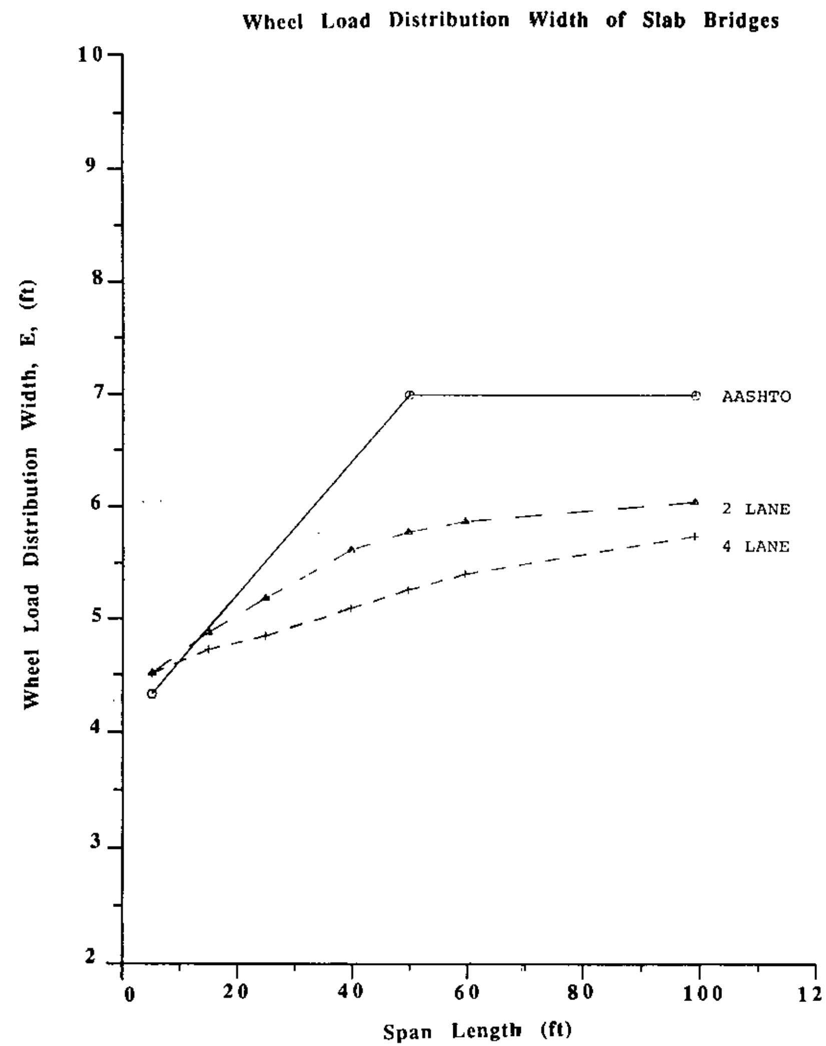

Figure 17. Wheel load distribution width, E, determined by using MUPDI3 for typical slab bridges.

Table 11. Weighting factors for the span length of slab bridges based on the NBIF (1985).

Span Length L (ft)	Weighting Factors
15	0.26
25	0.52
40	0.15
50	0.05
60	0.02

Table 12. Bias coefficients for wheel load distribution in slab bridges for a given span length.

L (ft)	15	25	40	50	60
Two lane	0.946	1.062	1.236	1.352	1.352
Four lane	1.008	1.132	1.317	1.441	1.441

From Table 8, it can be seen that for bridge spans greater than 30 ft under heavy truck traffic, the HS20 rating factor is very close to the actual truck rating (less than 9 percent difference). However, for light traffic conditions, the HS20 rating could underestimate the actual truck rating and the controlling single or semi-truck rating is shown in Figure 19 for different truck traffic.

Similar steps can be taken to convert AASHTO legal vehicle ratings to actual truck ratings. The longitudinal moment ratios, r_A, to convert AASHTO legal truck moments to HS20 moments may be computed as

$$r_A = \frac{a * W_{AASHTO}}{a_{HS20} * (72)} = \frac{M_{AASHTO}}{M_{HS20}} \tag{34}$$

From this relation and the ratio r_M, M_{AASHTO} can be converted to the corresponding actual truck moment

$$M_{AASHTO} = \frac{r_A}{r_M} * L \tag{35}$$

Therefore, the AASHTO legal truck rating can be converted from HS20 rating or actual truck rating:

$$RF_{AASHTO} = \frac{1}{r_A} RF_{HS20} = \frac{r_M}{r_A} RF \tag{36}$$

The conversion factors between the AASHTO legal truck ratings and the controlling single or semi-truck ratings are shown in Figures 20(a), 20(b), and 20(c) for different truck traffic.

Note that the live load parameters reported in this chapter are based on limited field measurements and should be regarded as tentative values. However, the subsequently determined live load factors can also be used to rate HS20 or AASHTO legal trucks with the recognition that the rating factor corresponding to the controlling single or semi-trucks on the highway can be obtained by conversion factors, as shown in Eqs. 32 and 36.

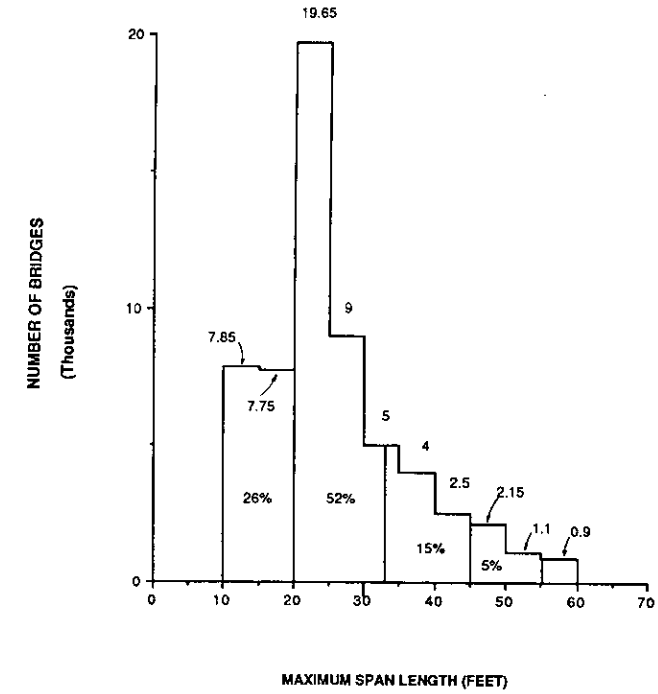

Figure 18. Histogram showing the distribution of maximum span length for reinforced concrete slab bridges in the United States (NBIF) 1985.

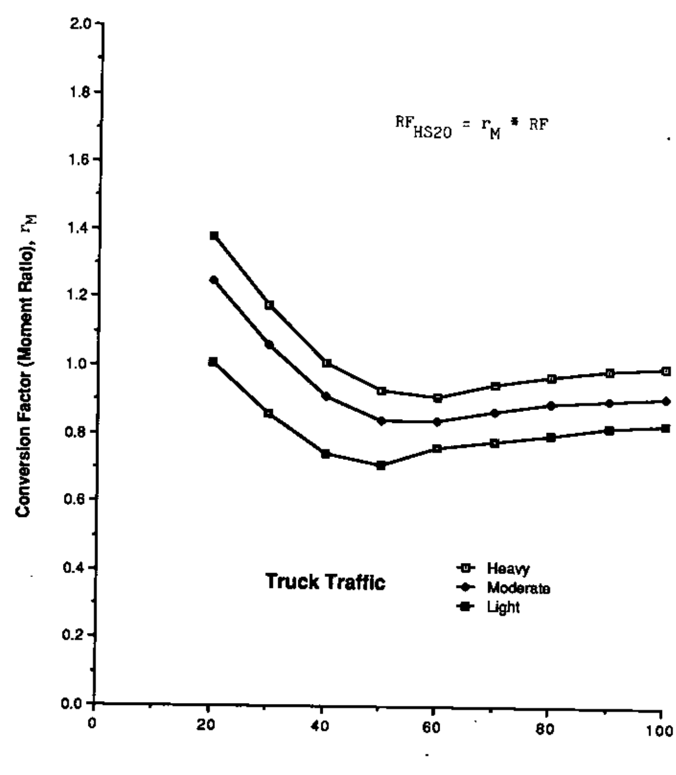

Figure 19. Conversion factors between HS20 truck rating and the rating for the controlling single or semi-truck as measured by Moses.

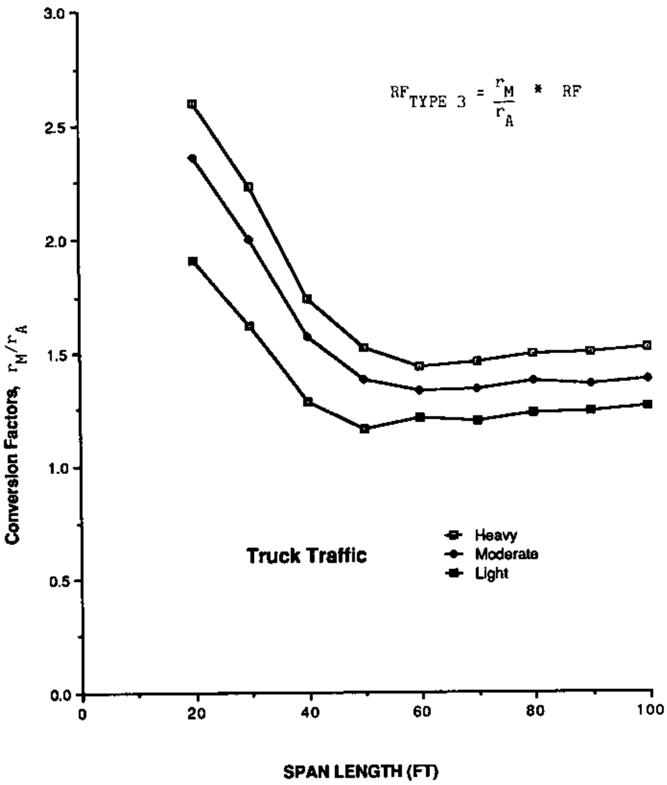

Figure 20(a). Conversion factor between the Type 3 truck rating and the rating for the controlling single or semi-truck as measured by Moses.

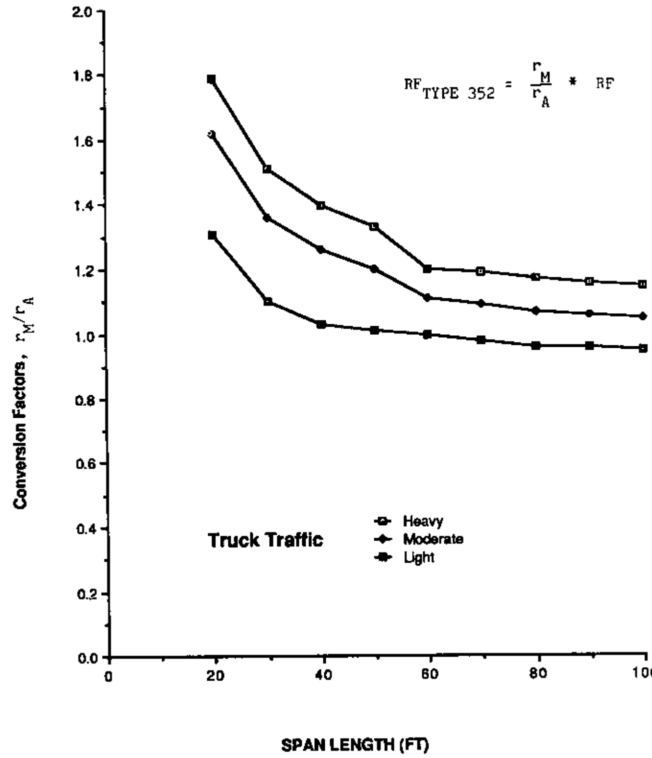

Figure 20(b). Conversion factor between the Type 3S2 truck rating and the rating for the controlling single or semi-truck as measured by Moses.

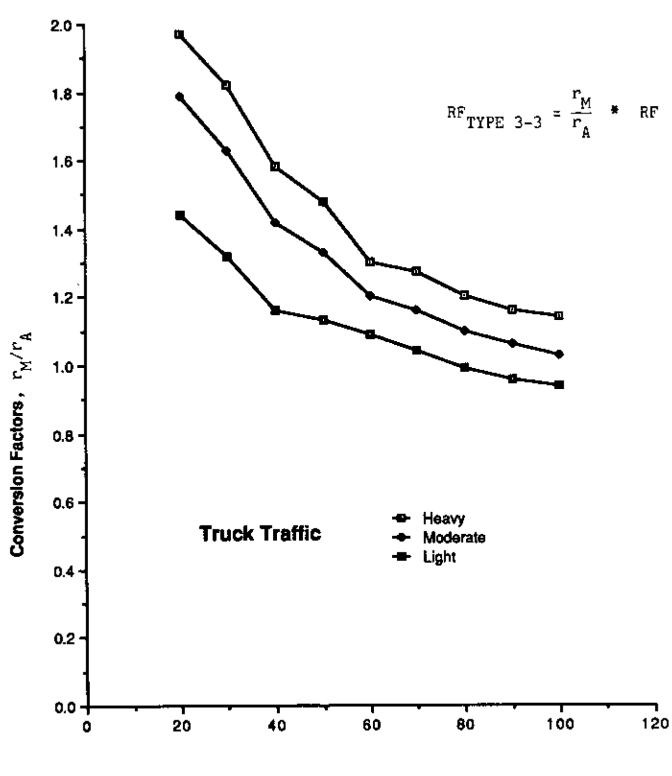

Figure 20(c). Conversion factor between the Type 3-3 truck rating and the rating for the controlling single or semi-truck as measured by Moses.

CHAPTER FOUR

AN ENSEMBLE OF EXISTING BRIDGES

An evaluation of selected portions of the FHWA National Bridge Inventory File (NBIF) (*60,61*) was conducted as part of this study. Table 13 includes a listing of an inquiry into the file to determine the various structural types and performance histories of reinforced concrete bridges. The listing contains a breakdown by bridge types and by sufficiency ratings of the number of bridges built within a given time period. Based on national totals shown in this table, attention will be focused on the reinforced concrete slab and T-beam bridge types. Bridges having sufficiency ratings less than 50 are eligible for replacement, those equal to or greater than 50 but less than 80 are eligible for rehabilitation, and those equal to or greater than 80 are not eligible. Based on the percentage of bridges needing rehabilitation (i.e., sufficiency rating 50–80), from each of these year groups, as shown in the first column of Table 13, this project will focus on bridges built in the 1920 to 1950 period.

A summary of the vehicle types used to rate the bridges within each of the bridge-type categories is also included in this tabulation. The HS loading is used more than either the H loading or the other loadings (e.g., Type 3, Type 3 S2, etc.)

The tabulation in the last five rows gives the gross weight of the rating vehicle at the operating level. It should be noted that a significant number of slab and T-beam bridges have operating ratings below the HS20 loading.

Table 14 gives the total number of slab and T-beam bridges by state. The table also shows the distribution of sufficiency ratings for all reinforced concrete bridges within each state. This listing was compiled from the NBIF listings for each state similar to those in Table 13. Five States (California, Pennsylvania, New York, Ohio, and Illinois) were contacted to assist in gathering sample bridges for the sensitivity studies. Both slab and T-beam bridges were included from the five States contacted in the data base being assembled for this project. A total of 21 T-beam and 15 slab bridges were considered.

The physical characteristics of the bridges and load capacity ratings, and selected items from the NBIF are assembled in Tables 15 through 20. Additional pertinent information from the inspection reports obtained from individual states is assembled in these tables. In order to determine the truck traffic volume, the average daily traffic (ADT) from items 29 and 30 of the NBIF was utilized. The deck condition (item 58) provided a basis for determining the surface roughness that was used for determining impact effects. The superstructure condition (item 59) was used to indicate level of maintenance effort expanded on each of the bridges.

The rating factors, as calculated by the states for an HS20 AASHTO truck loading at inventory and operating levels, are included in Tables 16 and 19. As indicated, these rating factors are based on AASHTO load factor and/or working stress criteria.

DESCRIPTION OF THE EXISTING T-BEAM BRIDGES

Schematics of the T-beam bridges are included in Appendix D. These bridges have construction dates ranging from 1911 to 1963. As indicated in Table 15, the number of T-beams, span lengths, slab thicknesses, girder depths, depth-to-span ratios, concrete strengths, and the yield strengths of reinforcing steel for the bridges vary over a wide range. The load capacity ratings and the selected items from the NBIF and inspection reports are presented in Table 16 and Table 17, respectively.

DESCRIPTION OF THE EXISTING SLAB BRIDGES

Schematics of the slab bridges are also presented in Appendix D. The construction dates range from 1920 to 1952. The span length, slab thickness, depth-to-span ratio, concrete strength, and yield strengths of reinforcing steel for the bridges are given in Table 18. The load capacity ratings and the selected items from the NBIF and inspection reports are given in Table 19 and Table 20, respectively.

Table 13. Inquiry into the FHWA National Bridge Inventory File (NBIF).

MATRIX

11/19/84

U.S. DEPARTMENT OF TRANSPORTATION - FEDERAL HIGHWAY ADMINISTRATION
NATIONAL BRIDGE INVENTORY DATA AS OF OCTOBER 11, 1984
HIGHWAY BRIDGES WITH CONCRETE STRUCTURE MATERIAL ITEM 43A = 1 OR 2
AND TYPE OF DESIGN ITEM 43B = 01 THROUGH 06

		Slab I43B= 01	Stringer = 02	Girder and Floorbeam = 03	Tee Beam = 04	Box Beam Multiple = 05	Box Beam Single = 06	COUNT	Sufficiency Rating SR < 50.	50.- 80.	> 80.
	COUNT	64,406	19,542	2,963	34,371	7,307	822	129,411	15,519	57,331	57,274
Year Built	I27A<=20	4,633	1,253	777	1,799	241	15	8,718	2,985	3,910	1,850
	=21-30	6,463	2,286	877	6,992	280	14	16,912	4,330	9,154	3,468
	=31-40	7,776	2,004	384	6,161	267	13	16,605	3,133	9,354	4,148
	=41-50	5,524	1,257	153	2,793	171	12	9,910	1,503	5,609	2,830
	=51-60	13,309	4,499	318	7,042	1,289	157	26,614	1,955	14,545	10,306
	> 60	26,046	8,089	454	9,389	5,846	610	49,634	1,420	14,388	34,216
Operating Rating: Type of Loading	I64A = 1	18,855	8,443	917	9,942	978	76	39,211	5,549	22,681	11,045
	= 2	24,017	8,032	974	13,605	5,313	218	52,159	3,584	18,001	31,102
	= 3-9	19,112	2,176	1,003	10,197	892	513	33,893	5,863	14,904	13,235
Operating Rating: Gross Loading in Tons	I64B <=5	935	274	84	263	34	3	1,593	1,158	128	310
	=06-15	6,507	1,529	336	1,170	149	7	9,698	3,919	5,307	476
	=15-19	3,120	932	222	1,318	236	12	5,840	1,827	3,310	708
	=20-35	18,776	7,855	1,177	8,947	858	122	37,735	4,425	20,683	12,688
	> 35	34,724	8,814	1,245	22,695	5,977	669	74,124	4,660	28,145	41,954

Type of loading:
1 = H loading   2 = HS loading   3-9 = Other loads

Table 14. National Bridge Inventory totals.

STATE	CONCRETE SLAB	CONCRETE T-BEAM	SUFFICIENCY RATING SR< 50	SUFFICIENCY RATING 50<SR>80	SUFFICIENCY RATING SR>80
Alabama	681	2074	616	2523	811
Alaska	9	3	1	10	12
Arizona	773	151	49	314	717
Arkansas	1925	1994	309	2702	973
California	4654	3337	597	2954	8395
Colorado	167	929	71	435	705
Connecticut	207	131	49	284	243
Delaware	61	1	1	14	52
D.C.	3	3	2	12	9
Florida	1762	141	332	1373	778
Georgia	1153	1947	120	1940	1114
Hawaii	179	242	118	286	134
Idaho	64	324	67	192	404
Illinois	3622	1787	794	2040	3466
Indiana	2041	367	739	1373	2087
Iowa	2852	428	196	1226	2526
Kansas	3501	1288	802	3112	2381
Kentucky	317	3138	443	2148	1045
Louisiana	3640	471	404	2001	2182
Maine	378	310	42	382	271
Maryland	309	54	132	233	159
Massachusetts	278	177	140	134	367
Michigan	263	778	149	451	737
Minnesota	341	0	215	474	472
Mississippi	1174	250	658	2301	1258
Missouri	2256	1012	1324	1839	756
Montana	209	234	55	167	274
Nebraska	1162	210	57	747	723
Nevada	150	14	4	43	326
New Hampshire	178	91	32	165	73
New Jersey	274	67	79	156	173
New Mexico	267	34	23	129	326
New York	871	222	482	635	218
North Carolina	180	955	271	749	149
North Dakota	184	267	57	230	524
Ohio	5826	1271	981	2814	3782
Oklahoma	1385	237	597	859	648
Oregon	512	1043	61	790	1320
Pennsylvania	1515	2964	775	2100	2410
Rhode Island	49	21	11	24	47
South Carolina	3438	1040	130	3387	977
South Dakota	1313	27	87	535	948
Tennessee	2963	14	1061	3219	1654
Texas	5472	1376	958	5033	5219
Utah	118	206	59	102	205
Vermont	197	338	85	348	110
Virginia	1324	1078	203	1318	1181
Washington	1027	631	83	1121	1557
West Virginia	719	140	318	570	152
Wisconsin	1573	8	609	873	1166
Wyoming	472	350	5	128	766
Puerto Rico	418	196	66	336	292
TOTALS	64406	34371	15519	57331	57274

Table 15. Physical characteristics of existing T-beam bridges used in sensitivity studies.

Seq. No.	State	Bridge No.	Name (Year Built)	No. Spans	No. T-Beams	Span Length L(ft.)	Girder Spacing S(ft.)	Slab Thickness t(in.)	Girder Depth h(in.)	Depth to Span Ratio	f'c (ksi)	fy (ksi)	Modifications to Structure	A.C. Overlay (in.)
1	CA	29-149	East Pine St. Overcrossing (1963)	4	5	33.75 53.50*	7	6.25	42	0.104 0.0654	3.25	60		---
2	CA	29C-190	Woodbridge Irrigation Canal (1925)	1	5	26.00	6.52	6.00	30	0.0962	3.00	33		5
3	IL	058-0041	(1931)	(replaced)										
4	IL	058-0037	(1928)	-	6	61.50	6.50	6.00	47.25	0.0640	3.00	33	Originally 4 T-Beams, 2 Fascia T-Beams Added 1956	---
5	NY	02240	Hannacroix Ck. Bridge (1931)	(replaced)										
6	PA	LR 69 (5+02)	Armstrong Co. (1931)	1	9	21.50	5.08	11.00	23.06	0.0894	2.50	33		2
7	PA	LR 10070 (212+60)	Butler County (1940)	1	6	38.00	5.08	8.50	26	0.0570	3.00	30		2
8	PA	LR 436 (7+85)	Jefferson Co. (1926)	2	6	31.75	5.08	11.00	29.56	0.0776	3.00	33	2 Additional P/S Box Beam	2
9	CA	52-50	Branch of Calleguas Ck. (1911)	1	5	27.75	5.084	6.00	28	0.0841	2.50	33		1
10	CA	2-11	Shasta River (1928)	5	3	47.50* 60.00	9.333 9.167	9.50 9.50	54 66	0.0947 0.0917	2.50	33		3
11	CA	51-91	El Jaro Creek (1929)	2	3	30.00* 60.00	9.125 8.834	9.50 9.50	47 62	0.131 0.0861	2.50	33		1
12	PA	LR 237 (283+06)	Clarion Co. (1931)	1	6	41.50	5.080	11.00	38.56	0.0774	2.50	40		1
13	NY	332803	Cazenovia Ck. (1929)	2	6	40.48	5.900	8.00	46	0.0947	3.30	30		7
14	NY	332758	Hampton Brook (1931)	1	10	32.00	4.500	7.00	30	0.0781	3.30	30		6
15	NY	331257	Butternut Ck. (1923)	1	8	28.00	3.600	7.00	31	0.0923	3.30	30	1971 deck widened on both sides with 3 steel I-beams	12
16	NY	331309	Skaneatles Ck. (1924)	1	8	28.00	3.700	7.00	33	0.0982	3.30	30	1959 deck widened on one side with 3 steel I-beams	3
17	NY	305051	Lishakill (1911)	1	5	36.00	5.160	7.00	55	0.1273	3.30	30		6
18	OH	30-0876	Sandy & Beaver Ck. (1932)	1	9	31.70	4.960	6.50	27	0.0710	3.00	33		6
19	OH	250-0104	Ashland County (1928)	1	7	29.00	5.190	6.50	24	0.0690	3.00	33		9
20	OH	61-0481	Branch of Black Fork (1940)	1	7	37.70	4.960	6.50	31	0.0685	3.00	33		3.75
21	OH	20-0217	East Fork Vermillion River (1936)	1	6	37.00	4.960	6.50	31	0.0698	3.00	33		8.5
22	IL	001-0005	Adams Co. Rte. 36 (1924)	1	4	51.50	6.580	6.00	41.5	0.0672	3.00	33		1.5
23	IL	001-0022	Adams Co. Rte. 96 (1926)	1	5	31.50	6.810	6.00	26.5	0.0701	3.00	33		0

*Bridge rated only for this span length

Table 16. Load capacity ratings of existing T-beam bridges used in sensitivity studies.

Seq. No.	State	Span Length L(ft.)	Method of Calculation	LOAD FACTOR Nominal Moment Resistance (k-ft)	WORKING STRESS DESIGN MOMENT RESISTANCE (k-ft) Inventory	Operating	(LL+I) Moment(HS20) (k-ft)	Dead Load Moment (k-ft)	LOAD FACTOR DESIGN (HS20) RF IN	RF OP	WORKING STRESS DESIGN (HS20) RF IN	RF OP
1	CA	53.50	BDS Program	1147	NA	NA	282	146	1.38	2.30	NA	NA
2	CA	26.00	Hand Calculation	493	NA	NA	147	124.2	0.89	1.48	NA	NA
3	IL	(replaced)										
4	IL	61.50	BARS Program	2729	1382.4	1824.8	567.9	636.8	1.32	2.21	1.31	2.09
5	(Replaced)											
6	PA	21.50	Hand Calculation	341.9	NA	NA	94.8	59.3	1.12	1.87	NA	NA
7	PA	38.00	Hand Calculation	560.3	NA	NA	297	178.1	0.42	0.71	NA	NA
8	PA	31.75	Hand Calculation	670.6	NA	NA	169.5	143	1.14	1.90	NA	NA
9	CA	27.75	Plane Frame Program	399.9	NA	NA	133.1	81.9	0.88	1.46	NA	NA
10	CA	47.50	Plane Frame Program	6887.8	NA	NA	1681.4	2044.2	0.92	1.54	NA	NA
11	CA	30.00	Plane Frame Program	1621.7	NA	NA	661.4	523.3	0.54	0.89	NA	NA
12	PA	41.50	Hand Calculation	1379.4	NA	NA	285.8	283.0	1.41	2.35	NA	NA
13	NY	40.48	Bridge Load Rating Edit Program	1050.0*	600.3	833.8	292.0	406.5	NA	NA	0.66	1.46
14	NY	32.00	Bridge Load Rating Edit Program	663.0*	362.8	503.9	151.8	152.9	NA	NA	1.38	2.31
15	NY	28.00	Bridge Load Rating Edit Program	265.0*	151.2	210.0	97.8	114.4	NA	NA	0.38	0.98
16	NY	28.00	Bridge Load Rating Edit Program	285.0*	162.7	226.0	100.8	82.4	NA	NA	0.80	1.42
17	NY	36.00	Bridge Load Rating Edit Program	600.0*	348.3	483.7	211.0	303.3	NA	NA	0.21	0.86
18	OH	31.70	BARS Program	603.3	283.4	433.8	163.1	159.0	0.95	1.59	0.76	1.68
19	OH	29.00	BARS Program	498.5	236.1	346.9	135.9	147.3	0.87	1.46	0.65	1.47
20	OH	37.70	BARS Program	908.1	431.2	646.8	214.2	184.2	1.25	2.08	1.15	2.16
21	OH	37.00	BARS Program	908.0	431.1	646.6	207.3	241.3	1.12	1.87	0.92	1.96
22	IL	51.50	BARS Program	1678.8	831.1	1133.1	455.4	529.4	0.83	1.39	0.66	1.33
23	IL	31.50	BARS Program	582.9	288.6	393.5	211.7	175.7	0.65	1.08	0.53	1.03

*Not supplied by the State - calculated using section properties

Table 17. Selected items from the NBIF and inspection reports of existing T-beam bridges used in sensitivity studies.

							Inspection Reports		
Seq. No.	State	Item 29 ADT	Item 30 Year of ADT	Item 58 Deck	Item 59 Superstructure	Item 67 Struc. Cond.	Wearing Surface Description	Deck Description	Beams/ Girder Description
1	CA	---	---	8	6	6	---	---	---
2	CA	2000	1982	6	7	3	---	---	
3		(Replaced)							
4	IL	7700	1983	6	5	4	---	---	---
5		(Replaced)							
6	PA	14600	1982	5	5	3	Bit.-overlay Few Cracks	Conc. Few Hairline Cracks	Transverse Hairline Cracks
7	PA	1400	1973	4	4	4	---	Concrete Spalls	---
8	PA	3750	1973	4	4	4	Bit.-cracked Bit.-Patched	Small spalls hairline cracks-Eff. rust stains	Disintegrated Rebars exposed
9	CA	5894	1974	6	6	4	---	---	---
10	CA	1000	1975	5	6	4	---	---	---
11	CA	2600	1976	6	6	3	---	---	---
12	PA	2800	1973	5	5	4	---	---	---
13	NY	943	1965	3	4	2	New Condition	Ser. det. to pot hazard	Serious deterioration
14	NY	2314	1974	-	-	8	Minor Deterioration	Serious det.	Serious deterioration
15	NY	500	1977	4	4	4	Minor det. to new condition	Ser. det. to minor det.	Serious deterioration
16	NY	1200	1977	5	4	4	Minor det. to new condition	Minor det.	Serious deterioration
17	NY	2100	1974	5	3	3	Minor det. to new condition	Minor det.	Potentially hazardous
18	OH	3560	1983	5	5	4	Good condition	Fair condition	Fair condition
19	OH	2320	1980	5	4	4	Fair condition	Fair condition	Major deficiency
20	OH	2700	1980	4	5	4	Fair condition	Major deficiency	Good condition
21	OH	3970	1980	6	4	4	Fair condition	Fair condition	Major deficiency
22	IL	1000	1983	6	5	5	---	---	---
23	IL	2700	1983	5	6	4	---	---	---

Table 18. Physical characteristics of existing slab bridges used in sensitivity studies.

Seq. No.	State	Bridge No.	Name (Year Built)	No. Spans	Span Length L(ft.)	Slab Thickness t(in.)	Depth to Span Ratio	f'c (ksi)	fy (ksi)	Modifications to Structure	A.C. Overlay (in.)
1	PA	LR 73 (1102+00)	Butler Co. (1941)	1	8	10.5	0.109	2.5	40		2
2	PA	LR 66 (23+75)	Armstrong Co. (1925)	1	12	10.0	0.0694	3.0	33	Bridge widened in 1961	1
3	PA	LR 203 (330+18)	Armstrong Co. (1926)	1	19	12.04	0.0528	3.0	33	Bridge widened in 1961	2
4	CA	20-0149	Pocket Creek (1922)	1	17	12.00	0.0588	2.5	33	Bridge widened in 1955	2
5	CA	57-0014	Chihuahua Ck. (1941)	6	30.5* 8	19.50 (ave. thickness of haunched section	0.0533	2.5	33		1
6	IL	001-0039	Adams County Route 31 (1920)	1	25	18.50	0.0617	3.0	33		3
7	IL	001-0041	Adams County Route 102 (1927)	1	21	12.25	0.0486	3.0	33		1
8	OH	252-0325	Small Creek (1930)	1	25	17.75	0.0592	3.0	33		9
9	OH	38-2231	Spring Fork. Ck. (1955)	3	38 47.06	19.25		3.0	33		3
10	OH	39-0187	Honey Creek (1952)	3	20 25	12.25		3.0	33		4
11	OH	235-1081	Baker Ditch (1933)	1	11.5	11.00	0.0797	3.0	33		4
12	NY	330158	Basic Reservoir (1935)	1	26.5	25.00	0.0786	3.3	30		14
13	NY	226330	Saquoit Ck. (1929)	1	26	26.00	0.0833	3.3	30		1.5
14	NY	335756	Munger Hollow Creek (1935)	1	23	22.00	0.0797	3.3	30		5
15	NY	332643	Got Creek (1928)	1	24	23.50	0.0816	3.3	30		7

*Bridge rated only for this span length

Table 19. Load capacity ratings of existing slab bridges used in sensitivity studies.

Seq. No.	State	Span Length L(ft.)	Method of Calculation	LOAD FACTOR DESIGN Nominal Mom. Resistance (k-ft)	WORKING STRESS DESIGN Moment Resist. (k-ft) Inventory	Operating	(LL+I) Moment (HS20) (k-ft)	DL Moment (k-ft)	LOAD FACTOR DESIGN (HS20) RF IN	RF OP	WORKING STRESS DESIGN (HS20) RF IN	RF OP
1	PA	8	Hand Calculation	24.76	NA	NA	9.28	1.53	1.01	1.68	NA	NA
2	PA	12	Hand Calculation	24.88	NA	NA	13.23	2.60	0.66	1.11	NA	NA
3	PA	19	Hand Calculation	40.41	NA	NA	19.27	8.90	0.60	0.99	NA	NA
4	CA	17	Plane Frame Program	41.4	NA	NA	16.5	9.35	0.71	1.17	NA	NA
5	CA	30.5	Plane Frame Program	44.7	NA	NA	15.7	10.35	0.79	1.31	NA	NA
6	IL	25	BARS Program	116.71	57.8	78.8	23.6	23.8	1.45	2.41	1.41	2.33
7	IL	21	BARS Program	56.0	27.7	37.7	20.8	13.1	0.74	1.24	0.70	1.19
8	OH	25	BARS Program	84.5	41.3	64.6	23.6	27.0	0.80	1.33	0.61	1.59
9	OH	47.06	BARS Program	275.3	130.6	--	82.5	61.1	0.94	1.57	0.84	---
10	OH	20	BARS Program	38.5	17.5	23.9	16.6	5.5	0.77	1.28	0.72	1.11
11	OH	11.5	BARS Program	25.4*	13.2	20.2	12.7	3.1	---	---	0.80	1.38
12	NY	26.5	Bridge Load Rating Edit Program	130.0*	73.0	101.3	26.7	41.2	NA	NA	1.19	2.26
13	NY	26	Bridge Load Rating Edit Program	120.0*	68.3	94.8	26.0	30.3	NA	NA	1.46	2.48
14	NY	23	Bridge Load Rating Edit Program	74.6*	42.8	59.5	22.2	23.4	NA	NA	0.87	1.62
15	NY	24	Bridge Load Rating Edit Program	105.0*	60.8	84.5	22.9	28.5	NA	NA	1.41	2.44

* Not supplied by the states - values calculated based on section properties.

Table 20. Selected items from the NBIF and inspection reports of existing slab bridges used in sensitivity studies.

Seq. No.	State	Item 29 ADT	Item 30 Year of ADT	Item 58 Deck	Item 59 Super-structure	Item 67 Struc. Cond.	Inspection Reports: Wearing Surface Description	Deck Description	Super-structure Description
1	PA	4800	1982	7	4	5	---	---	---
2	PA	4300	1973	5	5	5	Bit.-some heavy cracks	Conc. hair-line cracks & some spalls	Conc. slab hairline cracks & spalls
3	PA	2950	1983	5	4	4	Bit.-heavily cracked	Conc. very heavily disin. many rebars exposed	---
4	CA	2092	1976	7	7	3	---	---	---
5	CA	820	1976	7	7	4	---	---	---
6	IL	4650	1983	5	5	5	---	---	---
7	IL	350	1978	6	6	4	---	---	---
8	OH	3780	1980	6	7	6	Fair Condition	Fair Condition	---
9	OH	920	1982	4	5	4	Major Deficiency	Major Deficiency	---
10	OH	700	1980	4	4	4	Fair Condition	Critical Condition	---
11	OH	470	1982	5	8	5	Fair Condition	Fair Condition	---
12	NY	120	---	5	4	4	Minor Deterioration	NA	Minor to serious det.
13	NY	200	---	6	3	3	Minor det. to new condition	NA	Serious deterioration
14	NY	1928	1973	6	4	4	Minor Deterioration	NA	Minor to serious det.
15	NY	1000	---	5	4	4	Minor det. to new condition	NA	Minor to serious det.

CHAPTER FIVE

SENSITIVITY STUDIES

To assess the influence of variations in the statistical parameters of the design variables on the safety index and the partial safety factors (load and resistance factors), sensitivity studies were conducted for a family of hypothetical T-beam bridges (designed according to AASHTO specifications). These studies also show the range of the safety index.

GENERATION OF HYPOTHETICAL BRIDGES

In order to investigate the sensitivity of the safety index to various parameters, such as coefficients of variation and bias coefficients of resistance and load effects, a family of hypothetical, simply supported, reinforced concrete T-beam bridges was generated. These bridges were designed in accordance with AASHTO load factor design (*1*), i.e.,

$$0.9\,R = 1.3\,DL + 2.167\,LL \qquad (37)$$

where R = resistance, DL = dead load, and LL = live load.

Four T-beam bridges, each having six girders and carrying three traffic lanes, were designed for span lengths of 30, 45, 60, and 80 ft. A typical cross section is shown in Figure 21. From these four hypothetical bridges, a relationship between dead load to live load ratio, DL/LL, and span length, SL, was established. The following expression for DL/LL was obtained by fitting a parabolic curve to the data from these four T-beam bridges, as shown in Figure 22.

$$\frac{DL}{LL} = 0.6967 - 0.007620 \cdot SL + 0.0002554 \cdot (SL)^2 \qquad (38)$$

where SL is the span length. Since only linear behavior of the bridge is considered, the design Eq. 37 may be normalized by the maximum live load moment,

$$0.9\,\frac{R}{LL} = 1.3\,\frac{DL}{LL} + 2.167 \qquad (39)$$

By assuming that Eq. 38 is valid for span lengths up to about 100 ft, a whole family of T-beam bridges with DL/LL ranging from 0.5 to 2.5 was generated; thus, every point on the fitted curve of Figure 22 represents a hypothetical T-beam bridge and has its resistance dictated by Eq. 39.

Similarly, for slab bridges, a linear relation is found for the dead-to-live load ratio in terms of span length (Fig. 22): $(DL/LL) = -0.4873 + 0.05877\,(SL)$.

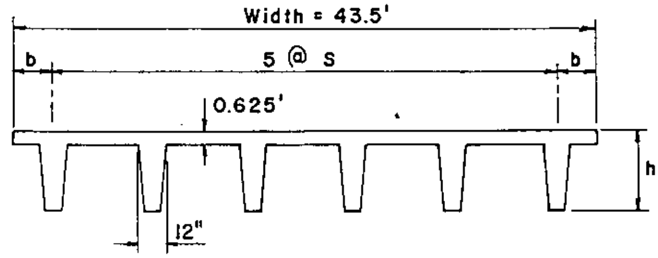

Figure 21. Typical cross section of hypothetical reinforced concrete T-beam bridges.

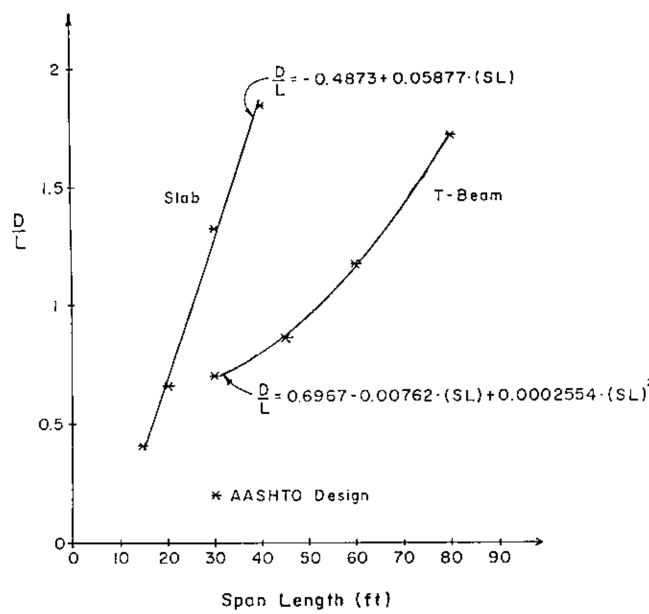

Figure 22. Dead to live load ratio vs. span length for hypothetical reinforced concrete T-beam and slab bridges.

SENSITIVITY OF DEAD LOAD UNCERTAINTIES

The total dead load effect is typically composed of contributions from structural concrete, railing, and asphalt concrete (A.C.) overlay. The effect of each of these three dead load components may be treated as a separate random variable. Coefficients of variation for the dead load effects of the structural concrete and railing are typically much smaller than that of the A.C. overlay, which may have a coefficient of variation as high as 0.20.

For uncorrelated random variables, the variance of the sum, Y, of the variables, X_i, equals the sum of the variances in each of the variables. Thus if $Y = \Sigma X_i$, then

$$\sigma_Y^2 = \sum \sigma_{xi}^2 \qquad (40)$$

The coefficient of variation of Y, V_Y, may be written as a weighted mean squared average of the coefficient of variation for each X_i, V_{xi},

$$V_Y = \left[\sum (X_i/Y)^2\, V_{xi}^2\right]^{1/2} \qquad (41)$$

where the weighting, X_i/Y, is the percentage contribution from different variables X_i, which in this case is the percentage of the total dead load effect contributed by each of the dead load components.

As indicated in Figure 23, more than 70 percent of the total dead load moment is caused by the weight of the structural concrete section. The A.C. overlay contributes to at most 20 percent. Similar observations were made for existing bridges, which are also included in Figure 23. These data indicate that the most important factor in the dead load effect is the structural concrete.

In the subsequent sensitivity analyses on the hypothetical bridges, all of the basic variables mentioned in Chapter Three are lumped into three groups: resistance, dead load effect, and live load effect. The following statistical data were used for a benchmark comparison:

$$V_R = 0.12,\ V_D = 0.10,\ V_L = 0.30$$
$$\delta_R = \delta_D = \delta_L = 1.0 \qquad (42)$$

where δ is the bias coefficient and V is the coefficient of variation.

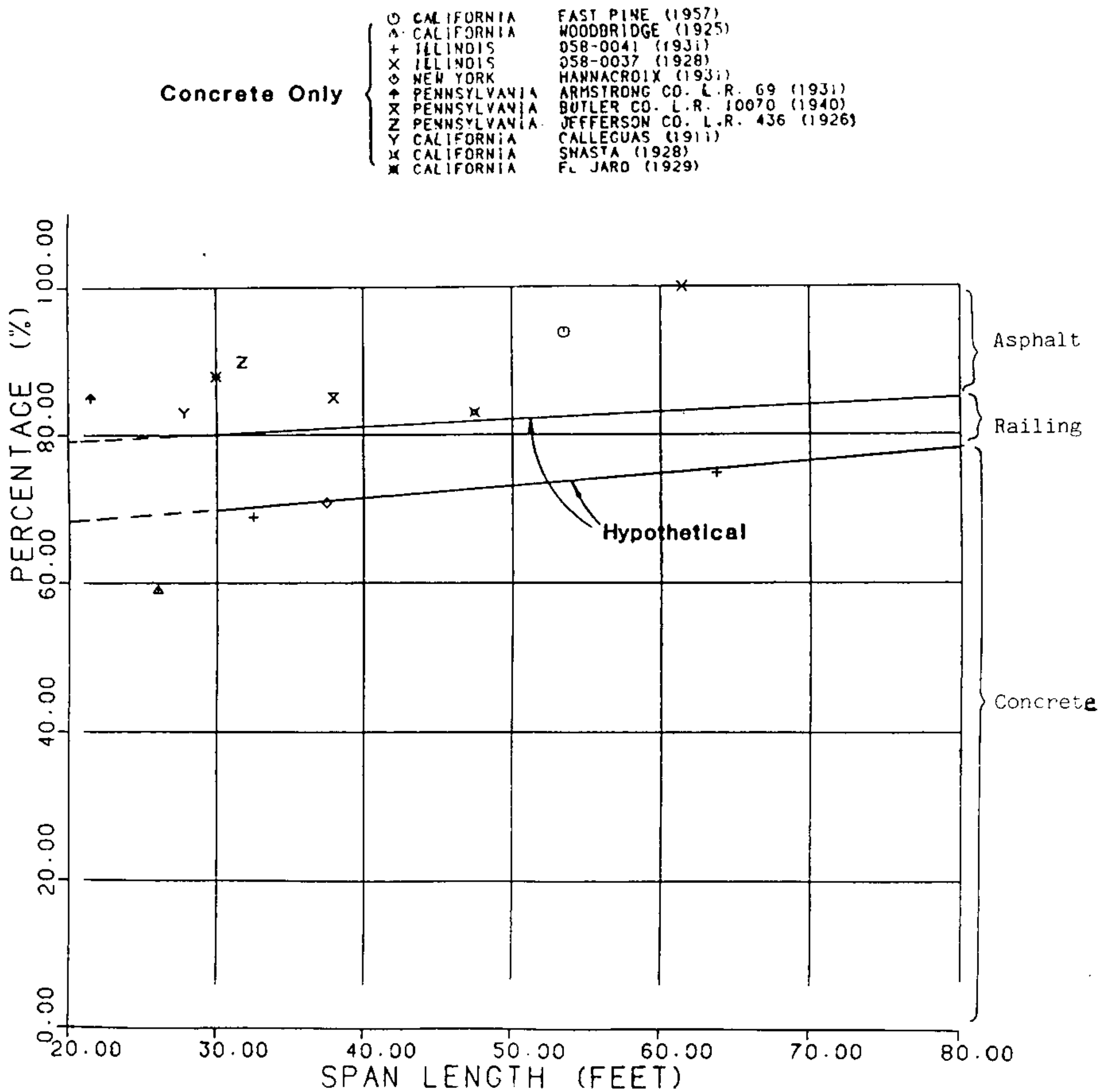

Figure 23. Percentage of dead load contributions (concrete structural section, railing, and A.C. overlay) vs. span length for hypothetical reinforced concrete T-beam bridges.

SENSITIVITY OF β TO STATISTICAL DISTRIBUTION TYPES

The mean value, the coefficient of variation and the type of statistical distribution of the random design variables are needed to conduct a second moment statistical analysis. Selecting the type of statistical distribution is usually the most difficult task. In practice, a particular type of distribution may be selected only occasionally on the basis of theoretical or experimental evidence. Normal and log-normal distributions are two of the most commonly assumed types (*4,7,19,25,27,44,56*).

A normal distribution is used as the distribution of the sum of independent random variables (Central Limit Theorem), and a log-normal distribution is used for the distribution of the product of independent random variables. Figure 24 illustrates the typical differences in the safety index that result from these two distributions. Log-normal distributions have been used exclusively for all random variables appearing in the proposed rating procedure (*25,44*).

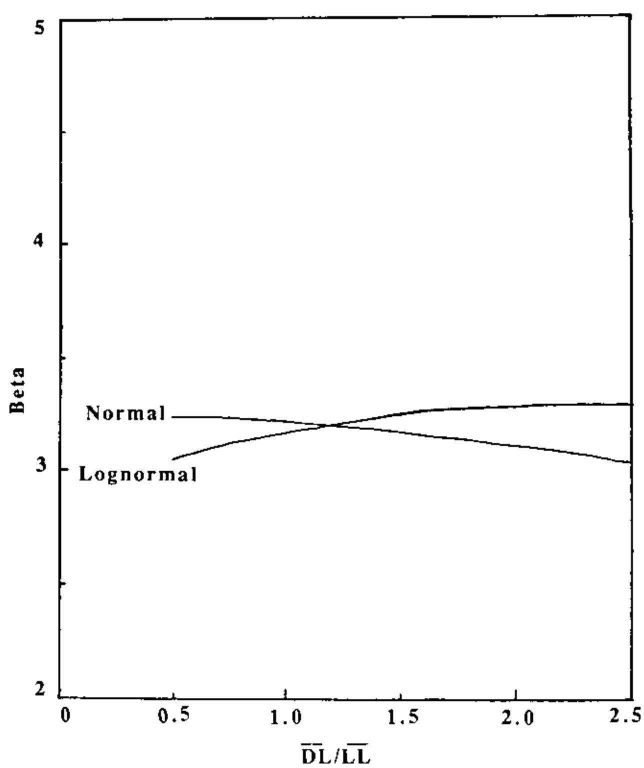

Figure 24. Sensitivity of β for normal and log-normal distribution types.

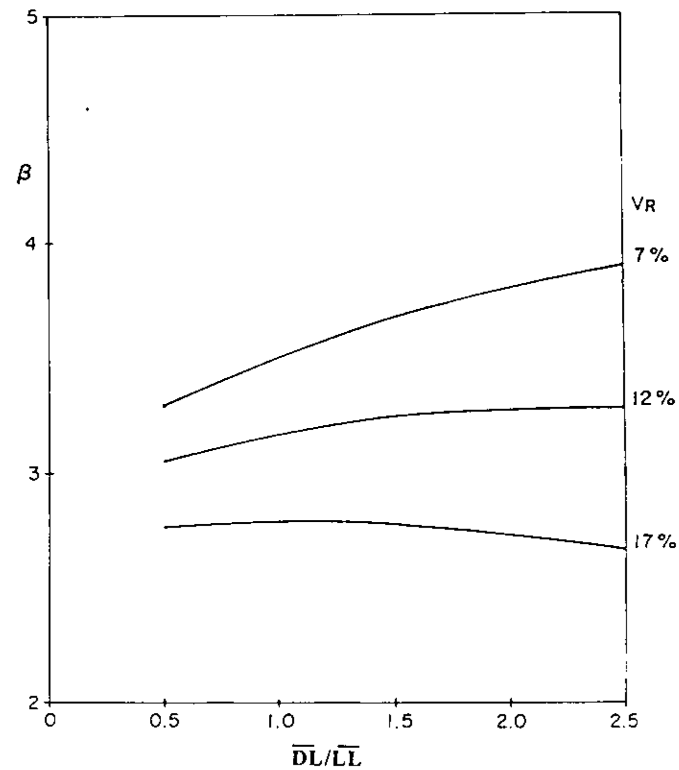

Figure 25. Sensitivity of β due to uncertainty in resistance, V_R, for hypothetical T-beam bridges.

SENSITIVITY OF β TO UNCERTAINTIES AND BIASES IN RESISTANCE, DEAD LOAD AND LIVE LOAD

Using the statistic data given in Eq. 42, the safety indices were computed for the family of hypothetical T-beam bridges with DL/LL ratios from 05. to 2.5. Figures 25, 26, and 27 show the sensitivity of the safety index to incremental changes of 5 percent in the coefficients of variation for resistance, dead load and live load effect, respectively. In the typical ranges of V_R, V_D and V_L, the following is observed:

1. By Changing V_R by ± 5 percent from the typical value while keeping V_D and V_L at the typical values, the change in β varies from ± 0.3 for $DL/LL = 0.5$ to ± 0.6 for $DL/LL = 2.5$ (see Fig. 25).

2. The sensitivity of β resulting from a change of ± 5 percent in V_D is negligible at short span lengths that are represented by low DL/LL ratios. However, the sensitivity increases as the span length, and thus the DL/LL ratio increases. Even at a DL/LL ratio of 2.5, however, the change in β is still small at about ± 0.1 to ± 0.2.

3. The sensitivity of β resulting from a change of 5 percent in V_L decreases as span length, and thus the DL/LL ratio, increases. At DL/LL ratios of 0.5 and 2.5, the changes in β are approximately 0.5 and 0.2, respectively.

Based on these observations the following conclusions are made regarding the sensitivity of the safety index to variations in the coefficients of variation for the three random variables:

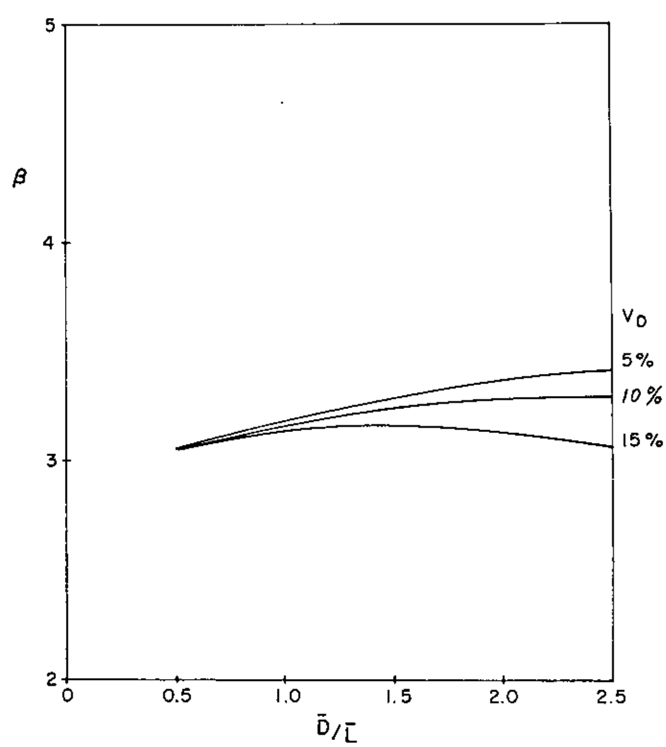

Figure 26. Sensitivity of β due to uncertainty in dead load effect, V_D, for hypothetical T-beam bridges.

1. The coefficient of variation V_R has the most consistent influence on the safety index.

2. The influence of V_L is larger than that of V_R at short span lengths, but the influence decreases as the span length increases.

3. The coefficient of variation V_D has the least influence on β. Even at longer span lengths where dead load effect is usually dominant, the sensitivity of β due to V_D is comparable to that due to V_L.

Figure 28 shows the sensitivity of the safety index caused by various biased estimates. Similar to the coefficient of variation, the bias in resistance has the largest effect. However, the effect of bias in dead load is comparable to that of live load.

This fact indicates that the greatest benefits may be obtained by focusing the efforts on research and field measurement of resistance and live load modeling. As mentioned earlier, the prediction of future live load growth is the most difficult problem for design. However, when evaluating an existing, twenty-year-old bridge, engineers are in a better position to predict the future live load for the much shorter rating period than they are when designing a new bridge. In addition, the resistance in an existing bridge with regular maintenance and inspection records can be more easily predicted and observed.

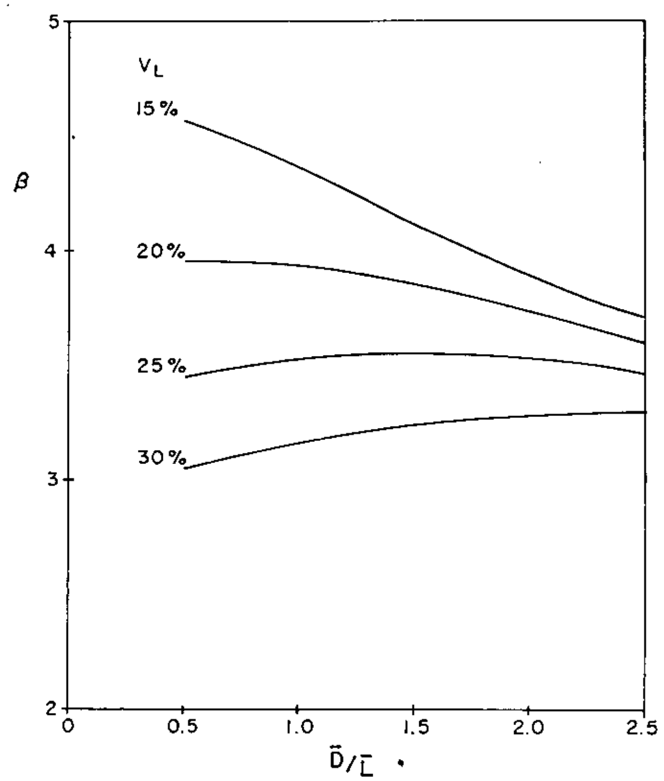

Figure 27. Sensitivity of β due to uncertainty in live load effect, V_L, for hypothetical T-beam bridges.

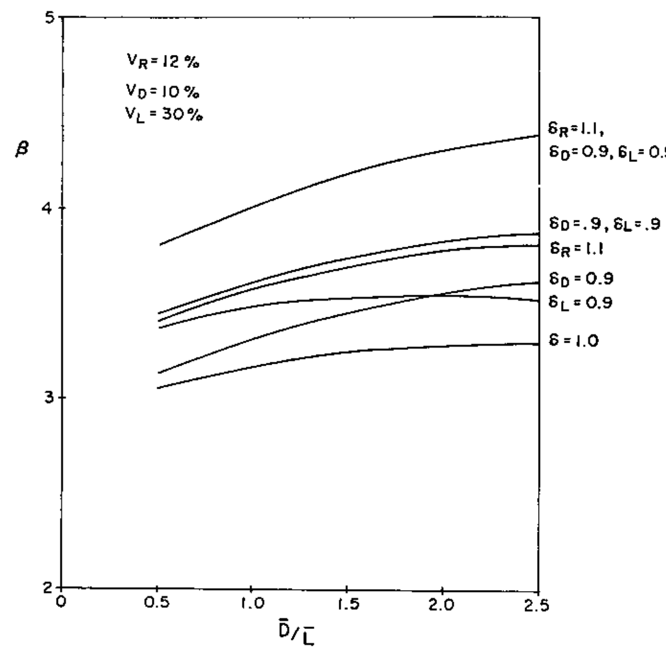

Figure 28. Sensitivity of β due to biased estimates of nominal values of resistance, dead load, and live load for hypothetical T-beam bridges.

CHAPTER SIX

CALIBRATION OF PARTIAL FACTORS

RELIABILITY-BASED STRENGTH EVALUATION METHODS

As in design, the reliability analysis method for the strength evaluation of existing bridges may be implemented at different levels of sophistication. The commonly used methods may be labeled as the Level 2 method and Level 1 method. In the Level 2 method, the reliability analysis of each bridge structure being evaluated will be conducted individually based on the specific statistics of resistances and load conditions associated with the bridge under consideration. Different approaches have been developed to actually implement this analysis. The advanced Second-Moment method, as described in Chapter Two, is the one commonly used in practice.

The Level 1 method is a simple, deterministic method. In this method, structural reliability is not considered for each bridge structure individually. Instead, structural components of similar mechanical actions and with similar statistics for load and resistance are considered as a group. A set of resistance and load factors is selected such that, when the limit state equation is satisfied, i.e., Factored resistance = Factored load effects, the structural reliability on the average will correspond to a prescribed target value.

Because of the extensive statistical data required and the complexities involved in the numerical implementation, the Level 2 method is only used for projects of special importance and for calibrating a deterministic Level 1 procedure.

LIMIT-STATE CHECKING FORMAT

In the traditional working stress method, the design check is based on a single safety factor, i.e.,

$$R_n \geq (\text{Safety Factor})\,[D_n + L_n] \qquad (43)$$

Since uncertainties associated with resistance and different components of load effects vary considerably, the resulting safety margin varies with the ratio of load effects. This is undesirable because an objective reliability measure cannot be established. (The current AASHTO load factor rating at operating level is essentially of this format with a safety factor equal to 1.3/0.9 = 1.44.)

The Level 1, load and resistance factor checking format, however, provides several partial safety factors for the treatment of the respective uncertainties associated with each design variable, i.e.,

$$\phi R_n \geq \gamma_D D_n + \gamma_L L_n \qquad (44)$$

In addition, these factors are calibrated to a target reliability which will be selected based on past performances of the type of structures under consideration.

To illustrate how uncertainties of individual variables can be accounted for in the selection of load and resistance partial factors, the simple mean-value reliability method will be used (*4*) in this section. By assuming log-normal distributions for both resistance R and total load effect Q, the safety checking format may be written as:

$$\overline{R} \geq \overline{Q} \exp \left[\beta \sqrt{V_R^2 + V_Q^2}\right] \qquad (45)$$

By introducing a separation constant α (with value from 0.55 to 0.75) to approximate the radical $\sqrt{V_R^2 + V_Q^2}$, load and resistance parameters may be decoupled, i.e.,

$$\exp[-\alpha\,\beta\,V_R] \cdot \overline{R} \geq \exp[\alpha\,\beta\,V_Q] \cdot \overline{Q} \qquad (46)$$

(Factored resistance) $\geq$ (Factored load effect)

The resistance factor, ϕ, may therefore be expressed as a function of bias coefficient δ_R, coefficient of variation V_R, and the safety index β, i.e.,

$$\phi = \delta_R \exp[-\alpha\,\beta\,V_R] \qquad (47)$$

For a safety index β of 2.8, for example, the resistance factors for various sets of (δ_R, V_R) are tabulated in Table 21(a).

The general trend is that the resistance factor, or the capacity reduction factor, decreases as: (1) the uncertainty, V_R, increases; and (2) the nominal (characteristic) resistance becomes an unconservative estimate of the mean resistance (i.e., decreasing δ_R). In a similar fashion, the load factor may be expressed as:

$$\gamma_Q = \delta_Q \exp[\alpha\,\beta\,V_Q] \qquad (48)$$

However, in this simple illustration, it would be difficult to separate the dead load and live load factors. Instead we can look at the extreme cases, i.e., dead load only and live load only. For different sets of (δ_Q, V_Q), the load factors are given in Table 21(b).

Load factors increase as: (1) the uncertainty, (V_Q, increases; and (2) the nominal (characteristic) load effect becomes a conservative estimate of the mean load effect (i.e., increasing δ_Q). Assuming that the contribution of live load effect to Q is minimal and can be neglected, the dead load factor can be estimated at 1.2 to 1.3. On the other hand, when dead load effect is insignificant, V_Q (in the range of 0.20 to 0.30) is mostly contributed by live load effects. The live load factor may vary from 1.4 to 1.9.

Table 21(a). Sensitivity of resistance factor.

δ_R	V_R	ϕ (β = 2.8) (α = 0.55)	(α = 0.75)
1.1	0.10	0.94	0.89
	0.15	0.87	0.80
	0.20	0.81	0.72
	0.25	0.75	0.65
1.1	0.15	0.87	0.80
1.0		0.79	0.73
0.9		0.71	0.66

Table 21(b). Sensitivity of load factor.

δ_Q	V_Q	γ_Q (β = 2.8) (α = 0.55)	(α = 0.75)	Remarks
1.0	0.10	1.17	1.23	Dead Load Dominates
	0.15	1.26	1.37	
	0.20	1.36	1.52	
	0.25	1.47	1.69	Live Load Dominates
	0.30	1.59	1.88	
0.8	0.20	1.09	1.22	
0.9		1.22	1.37	
1.0		1.36	1.52	
1.1		1.50	1.67	
1.2		1.63	1.83	

The above illustrations, based on the mean-value method, is only qualitative, and is presented to demonstrate the influence of bias coefficient and COV on load and resistance factors. In the rest of the chapter, advanced first-order second-moment will be used to select the load and resistance factors.

GENERAL CALIBRATION PROCEDURE

The objectives of code calibration are generally stated as achieving uniform reliability and economic designs in all applications covered by the code. In practice, economic considerations are taken into account implicitly by a judicious selection of the target reliability level. To establish the target reliability level, heavy emphasis must be placed on professional experiences and engineering judgments, and the performance of bridges subjected to known loads which have been designed in accordance with past and current design provisions. The current code of standard practices may often be seen as the best documentation of such experiences and judgments. The load and resistance partial factors are then selected to account for the uncertainties of the design variables, and to provide uniformity in the safety index for a whole range of structures.

For given load and resistance statistics and a specified target safety index, the required nominal resistance may be evaluated by a Level 2 design procedure for each of the existing structures. In the process, the most critical point on the limit state surface, known as the design (or checking) point, is determined such that

$$R^* = D^* + L^* \tag{49}$$

Equivalently, the Level 1 criteria are

$$\phi R_n = \gamma_D D_n + \gamma_L L_n \tag{50}$$

where

$$\phi = \frac{R^*}{R_n},\ \gamma_D = \frac{D^*}{D_n} \text{ and } \gamma_L = \frac{L^*}{L_n} \tag{51}$$

where the subscript n designates nominal values.

The partial factors that appear in Eq. 50 may be scaled by a constant to yield an equation with only two independent variables and still give the same Level 1 criteria. If the two independent variables are known, the partial factors in Eq. 51 can be determined if one of the factors is determined a priori. Therefore, for a given ϕ value, the load factors, γ_D and γ_L, are determined.

Following the Level 2 procedure, different sets of load factors may be determined for structures with different loading statistics. In the Level 1 method, by using a constant set of partial factors, the safety index of a given structure will deviate from the target safety index, β_o. The calibration process then proceeds by adjusting the partial factors to make these deviations from β_o (in terms a weighted averaged value) minimized. Another approach used by the Canadian Standard Association (*10,47*) in determining the partial factors is based on minimizing the expected cost of failure.

In rating existing structures, resistance and inherent safety levels exist for each structure and will not be changed by simply selecting different partial factors. However, by selecting appropriate partial factors, the rating may be made to reflect the inherent safety level more consistently. This is the purpose of calibration for rating.

The rating factor is defined as

$$RF = \frac{\phi R - \gamma_D D}{\gamma_L L} \tag{52}$$

which is a measure of undercapacity ($RF < 1$) or overcapacity ($RF > 1$) of the bridge component to carry the rating vehicle. If an optimal set of partial factors have been determined, the safety index of each bridge, when evaluated for the modified (increased or reduced) live load effect $RF \cdot L$ should be approximately equal to the target value β_o. Calibration for rating involves the following trial and error steps:

1. Selecting a set of partial factors ϕ, γ_D and γ_L.
2. Determining a rating factor RF_j for each bridge, j, according to Eq. 52.
3. Reevaluating the safety index $\hat{\beta}_j$ of each bridge subjected to the modified live load effect $RF_j \cdot L_j$.
4. Examining the weighted average difference between calculated safety indices and the target value

$$I\left(\frac{\phi}{\gamma_L}, \frac{\gamma_D}{\gamma_L}\right) = \left\{\frac{1}{N-1}\sum_{j=1}^{N}(\hat{\beta}_j - \beta_o)^2 W_j\right\}^{1/2} \tag{53}$$

where W_j = a weighting factor that can be used to reflect the relative importance of the bridge. Notice that the weighted average difference can be evaluated in terms of the two independent variables ϕ / γ_L and γ_D/γ_L.

5. Iteration of the foregoing process for different combinations of partial factors such that the weighted average difference (Eq. 53) is minimized.

A variation of this calibration procedure which avoids repeated reevaluation of the safety indices, $\hat{\beta}_j$ (step 3) is based on the observation that there exists a functional relationship between RF_j and $\hat{\beta}_j$, the safety index of the bridge subjected to the rating vehicle loading. Therefore, a bridge structure with a high safety index for a specified live load effect should also have a high rating factor for that load effect. This relationship may not be linear, but it must be monotonical. In addition, the rating factor should be close to 1 for bridges with a safety index close to $\hat{\beta}_o$. A possible form of the equation relating RF_j to the safety index is:

$$\beta(RF_j) = \beta_o + a\,(RF_j - 1) + b\,(RF_j^2 - 1) \quad (54)$$

where a and b are constants, which may be determined by the least square best fit for an ensemble of bridges and a set of partial factors. For different sets of partial factors, the least square fitting will give different mean square differences for β_j and $\beta(RF_j)$:

$$I\left(\frac{\phi}{\gamma_L}, \frac{\gamma_D}{\gamma_L}\right) = \left\{\frac{1}{N-1}\sum_{j=1}^{N} [\beta_j - \beta(RF_j)]^2\, W_j\right\}^{1/2} \quad (55)$$

By trial and error the mean square difference can be minimized to yield the optimal set of partial factors in terms of the two ratios, ϕ/γ_L and γ_D/γ_L. This method was used in this study.

TARGET SAFETY INDEX, β_o

Currently, bridges are rated at inventory (design) level and operating level. The latter is intended to rate the bridge at maximum live load carrying capacity. Both the "allowable stress" method and the "load factor" method may be used. A rating factor of 1 at operating level indicates that the rating vehicle is the maximum live load the bridge may carry. *Therefore, the operating rating factor was used as the criteria for selecting the target safety index.*

The site specific conditions and the safety indices for the exiting bridges discussed in Chapter Four are given in Tables 22(a) and 22(b) for T-beam and slab bridges, respectively. Figures 29 and 30 show, for T-beam and slab bridges, respectively, the correlations between safety indices and rating factors (operating level). The rating factors for each bridge were provided by the state agency. Some are rated by the allowable stress method, some are rated by the load factor method. A few bridges are rated by both methods. Contrary to the conventional belief, the working stress rating may in some cases be higher than the load factor rating (as shown in Figs. 29 and 30). This is because the allowable stress to yield stress ratio of steel reinforcement for earlier years and for unknown grades may be higher than that of steel used in recent years. This results in lower factors of safety for many older bridges.

Table 22(a). Site specific conditions and safety indices of T-beam bridges.

Bridge No.	ADT (Year)	Truck Traffic Category	Deck Condition	Impact	Beta
TB1 CA	--	L	8	L	6.90
TB2 CA	2000 (1980)	L	6	L	4.36
TB4 IL	7700 (1983)	M	6	L	5.74
TB6 PA	14600 (1982)	M	5	H	3.04
TB7 PA	1400 (1973)	L	4	H	1.97
TB8 PA	3750 (1973)	L-M	4	H	4.81
TB9 CA	5894 (1974)	M	6	L	2.91
TB10 CA	1000 (1975)	L	5	H	5.66
TB11 CA	2600 (1976)	L	6	L	3.52
TB12 PA	2800 (1973)	L	5	H	6.24
TB13 NY	943 (1965)	L	3	H	3.89
TB14 NY	2314 (1974)	L	-	H	4.93
TB15 NY	500 (1977)	L	4	H	1.63
TB16 NY	1200 (1977)	L	5	H	2.65
TB17 NY	2100 (1974)	L	5	H	2.00
TB18 OH	3560 (1983)	L	5	H	4.27
TB19 OH	2320 (1980)	L	5	H	4.00
TB20 OH	2700 (1980)	L	4	H	5.40
TB21 OH	3970 (1980)	L	6	L	5.36
TB22 IL	1000 (1983)	L	6	L	5.08
TB23 IL	2700 (1983)	L	5	H	3.47

ADT = Average Daily Traffic
ADTT= Average Daily Truck Traffic
= (15% - 25%) ADT

Table 22(b). Site specific conditions and safety indices of slab bridges.

Bridge No.	ADT (Year)	Truck Traffic Category	Deck Condition	Impact	Beta
S1 PA	4800 (1982)	L-M	7	L	4.50
S2 PA	4300 (1973)	L	5	H	2.65
S3 PA	2950 (1983)	L	5	H	2.15
S4 CA	2092 (1976)	L	7	L	3.21
S5 CA	820 (1976)	L	7	L	3.71
S6 IL	4650 (1983)	L	5	H	5.37
S7 IL	350 (1978)	L	6	L	3.27
S8 OH	3780 (1980)	L	6	L	3.80
S9 OH	920 (1982)	L	4	H	4.01
S10 OH	700 (1980)	L	4	H	2.90
S11 OH	470 (1982)	L	5	H	2.84
S12 NY	120 (----)	L	5	H	4.61
S13 NY	200 (----)	L	6	L	5.15
S14 NY	1928 (1973)	L	6	L	3.66
S15 NY	1000 (----)	L	5	H	4.66

ADT = Average Daily Traffic
ADTT= Average Daily Truck Traffic
= (15% - 25%) ADT

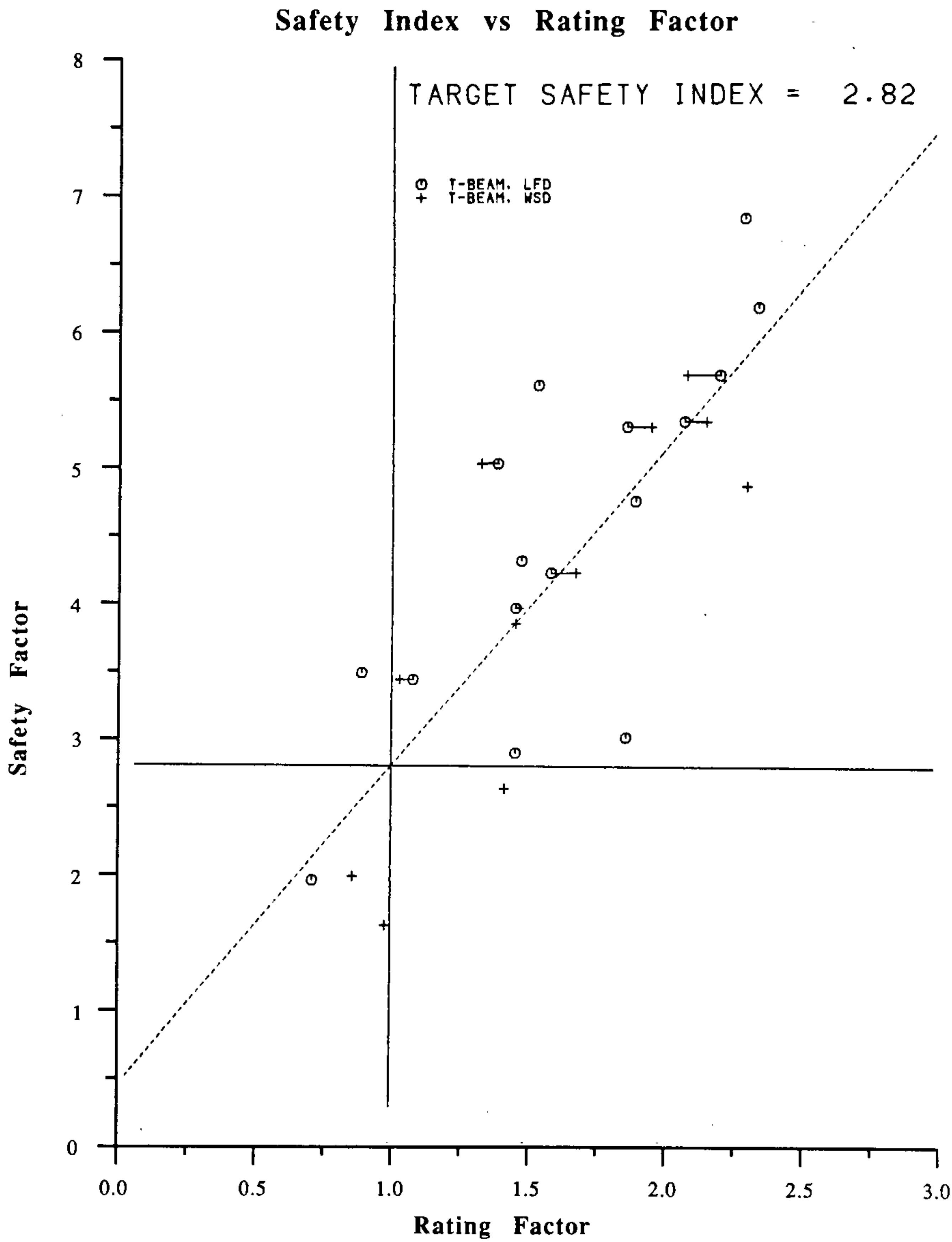

Figure 29. Safety index vs. rating factor for existing reinforced concrete T-beam bridges.

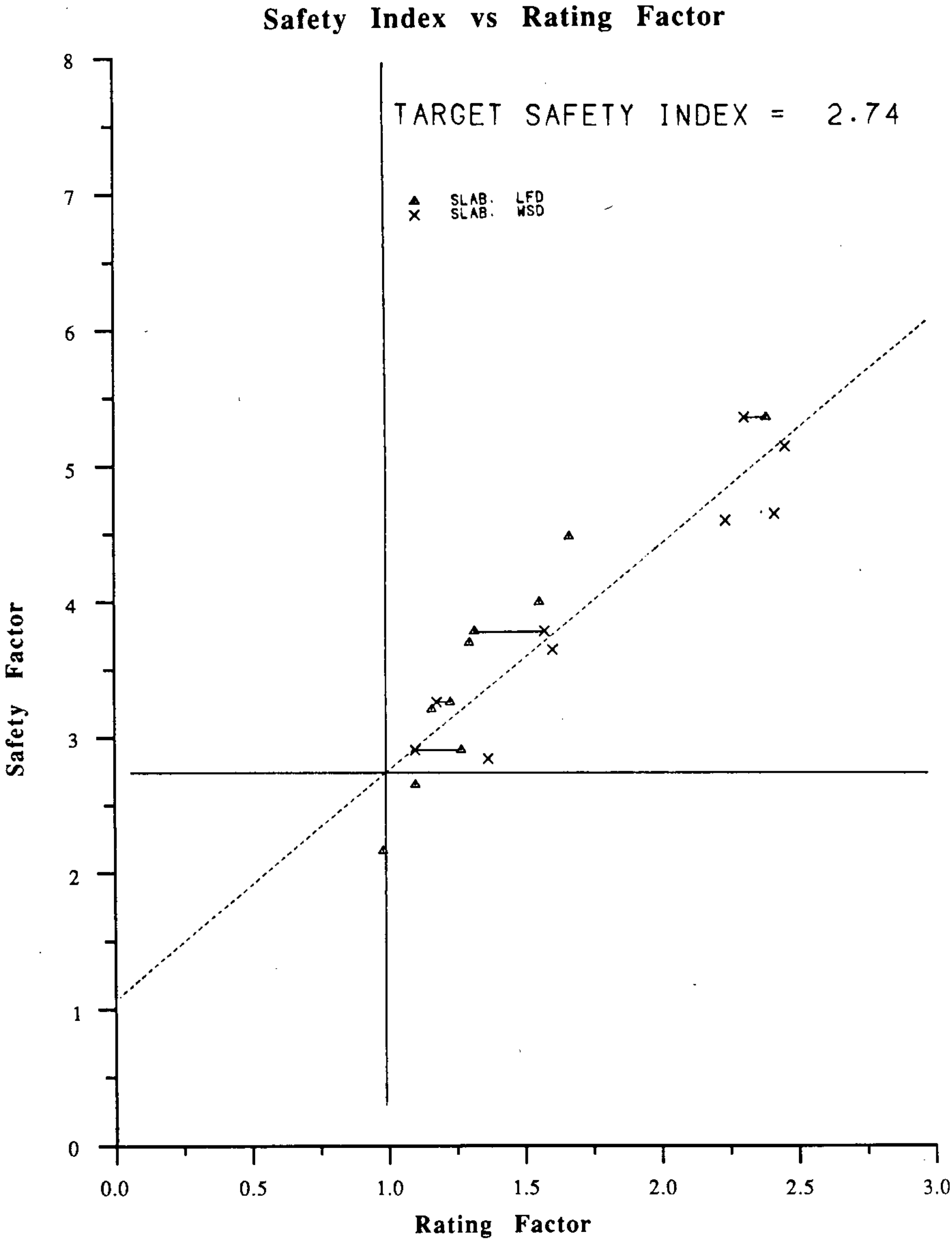

Figure 30. Safety index vs. rating factor for existing reinforced concrete slab bridges.

The safety indices, β_j, have been evaluated based on available statistical data of live load effect as outlined in Chapter Three. Figures 29 and 30 also show that the scattering in the T-beam bridges is much larger than that in the slab bridges. The least square best fits are

$$\beta(RF_j) = 0.47 + 2.35\ RF_j, \text{ for T-beam bridges} \quad (56)$$

and

$$\beta(RF_j) = 1.07 + 1.67\ RF_j, \text{ for slab bridges} \quad (57)$$

Corresponding to a RF of 1, the safety indices obtained from these equations are 2.82 and 2.74 for T-beam and slab bridges, respectively. Inasmuch as the main interest for bridge evaluation is in rating factors close to 1, from this standpoint the two bridge types are not too different. Therefore, for the purpose of this project, all bridges are lumped together into one data base. Figure 31 shows the correlation between β_j and RF_j for all 36 bridges. The least square best fit is

$$\beta(RF_j) = 0.75 + 2.04\ RF_j \quad (58)$$

The target safety index, β_o, was selected to be 2.80. Noting the scattering of β_j for RF_j close to 1, a safety index of 2.4 and above should be an acceptable value of β_j for calibration purposes.

MAINTENANCE, INSPECTION, AND FIELD MEASUREMENT

Item 59 of the NBIF is the condition rating of the superstructure by which the state bridge engineers report the condition of the bridge superstructure on a scale of 0 to 9 (see Table 23). If this condition rating is conducted systematically and consistently, it may be used as an indicator in classifying different resistance categories. For deteriorated structures, the efforts in preventive maintenance, inspection and field measurements may be used to further refine the classification.

To quantify the resistance statistics, the coefficient of variation, V_R, and bias coefficient, δ_R, for each category need to be determined. Because of the lack of documented data at the present time, values for V_R and δ_R were assumed.

On the basis of the superstructure condition rating, existing bridges may be classified into three broad categories to reflect the degree of deterioration: (1) good or fair ($\geq$ 7); (2) deteriorated (4,5,6); (3) seriously deteriorated to potentially hazardous ($\leq$3).

It is assumed that for bridges in good or fair condition,

$$V_R = 10 \text{ percent, and } \delta_R = 1.1 \quad (59)$$

regardless of the effort of maintenance and inspection. This is consistent with values in other studies (*9, 37*).

For deteriorated structures, three additional factors are considered in classifying structures: (1) strength evaluation based on plans and field measurements; (2) inspection frequency (frequent and normal (2 year intervals)); and (3) preventive maintenance (implemented, not implemented).

By conducting field measurements, presumably the degree of deterioration and its locality may be determined and more realistic estimates of the nominal resistance can be made. In these cases, the same bias coefficient of 1.1 as for structures in good or fair conditions is assumed. For rating based on plans, reductions of 0.1 and 0.2 are assumed for the two categories of deteriorated structures.

Further, it is assumed that the effect of frequent inspection and preventive maintenance may be reflected in the scattering of the resistance of deteriorated structures. Coefficients of variation up to 20 percent and 25 percent were assumed for the two categories of deteriorated structures with normal inspection intervals (2 years) and no preventive maintenance.

Table 24 lists the hierarchical classification of the resistance categories and associated statistical values.

Table 23. Description of superstructure condition rating (NBIF).

Rating	Equivalent Rating Conditions Descriptions 1978	1972
N	Not applicable	Not applicable
9	New condition	New condition
8	Good condition - no repairs needed	Good condition - no repair necessary
7	Generally good condition - potential exists for minor maintenance	Minor items in need of repair by maintenance forces
6	Fair condition - potential exists for major maintenance	Major items in need of repair by maintenance forces
5	Generally fair condition - potential exists for minor rehabilitation	Major repair - contract needs to be let
4	Marginal condition - potential exists for major rehabilitation	Minimum adequate to tolerate present traffic, immediate rehabilitation necessary to keep open
3	Poor condition - repair or rehabilitation required immediately	Inadequacy to tolerate present heavy load - warrants closing bridge to trucks
2	Critical condition - the need for repair or rehabilitation is urgent. Facility should be closed until the indicated repair is complete.	Inadequacy to tolerate any live load - warrants closing bridge to all traffic
1	Critical condition - facility is closed. Study should determine the feasibility for repair	Bridge repairable, if desirable to reopen to traffic
0	Critical condition - facility is closed and is beyond repair	Bridge conditions beyond repair - danger of immediate collapse

RESISTANCE FACTORS

In establishing the target safety index site specific live load conditions (e.g., truck weight control unenforced, and mostly light truck traffic volume) were used. Although an initial review of the NBIF indicated a variation in superstructure condition ratings, subsequent contacts with the individual states, however, revealed a difference in practice relative to assigning these ratings. Therefore, these ratings were not felt to be totally reliable and, because none of the bridges were posted with a load limit, it was assumed that all bridges in the data base were in good

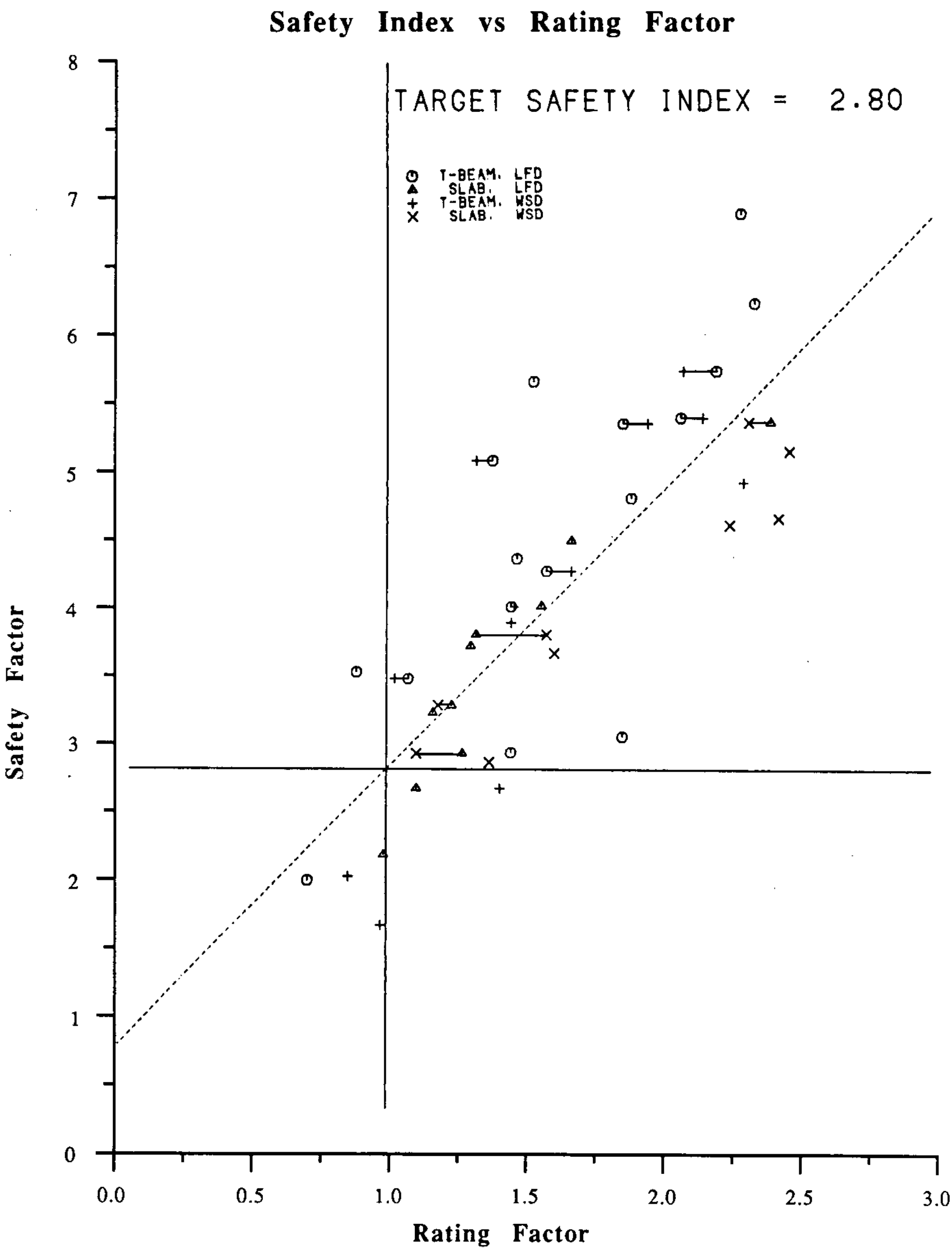

Figure 31. Target safety index determined from combined existing reinforced concrete T-beam and slab bridges.

Table 24. Resistance categories based on superstructure condition, inspection frequency, and preventive maintenance.

Superstructure Condition (r)	Rating based on Plans	Rating based on Field Measurement	Inspection Frequency: Frequent	Inspection Frequency: Normal (2 yrs.)	Preventive Maintenance: Yes	Preventive Maintenance: No	Resistance V_R	Resistance δ_R	ϕ
Good or Fair $r \geq 7$	--	--	--	--	--	--	10%	1.1	0.94
Deteriorated (r=4,5,6)		X	X		X		10%	1.1	0.94
		X	X			X	15%	1.1	0.84
		X		X	X		15%	1.1	0.84
		X		X		X	20%	1.1	0.76
	X		X		X		10%	1.0	0.84
	X		X			X	15%	1.0	0.76
	X			X	X		15%	1.0	0.76
	X			X		X	20%	1.0	0.67
Seriously Deteriorated to Potentially Hazardous ($r \leq 3$)		X	X		X		15%	1.1	0.84
		X	X			X	20%	1.1	0.76
		X		X	X		20%	1.1	0.76
		X		X		X	25%	1.1	0.67
	X		X		X		15%	0.9	0.67
	X		X			X	20%	0.9	0.60
	X			X	X		20%	0.9	0.60
	X			X		X	25%	0.9	0.54

or fair condition. For this data base, it was found that the mean square difference in β_j and $\beta(RF_j)$, Eq. 55, is minimized at the following point

$$\frac{\phi}{\gamma_L} = 0.59, \text{ and } \frac{\gamma_D}{\gamma_L} = 0.77 \qquad (60)$$

which corresponds to a target safety index of 2.8. The trial and error process is shown in Figure 32.

To assess the sensitivity of the resistance factors, the resistance statistics of the original data base were arbitrarily varied. Based on the varied data base, a new target safety index was established. Assuming the ratio of two load factors, γ_D/γ_L, remained the same, the resistance factor was recalibrated corresponding to the new target safety index. The following two sets of resistance statistics were considered:

$$V_R = 15\%,\ \delta_R = 1.1\ (\beta_o = 2.37) \qquad (61a)$$

$$V_R = 10\%,\ \delta_R = 1.0\ (\beta_o = 2.29) \qquad (61b)$$

In both cases the value obtained for the resistance factor was essentially unchanged. Results of these two sensitivity analyses are shown in Figure 33. Note that either V_R or δ_R has been varied to a rather large extent. Therefore, it may be concluded that small deviations in the assumed values for resistance statistics will not significantly change the resulting resistance factors.

For the basic data base where structures are in good or fair condition, a resistance factor of 0.9 or higher is acceptable. Based on the recent Probability Based Load Criterion for American National Standard A58, a dead load factor of 1.2 is deemed appropriate. Therefore, from Eq. 60,

$$\phi = 0.92 \qquad \gamma_L = 1.56 \qquad \text{for } \gamma_D = 1.2 \qquad (62)$$

Rounding the live load factors to 1.6, the corresponding resistance factor is

$$\phi = 0.94 \qquad (63)$$

To account for the efforts in maintenance, inspection, and field measurement, as described earlier, the resistance factors were determined using the assumed resistance statistics, the same target safety index of 2.8, and the same load factors. The new resistance to live load factor ratios, ϕ/γ_L, obtained from this procedure are given in Table 25. Also, the trial and error process used to obtain these ratios is illustrated in Figure 34. The resistance factors corresponding to the hierarchical classification in Table 24 are shown in Figure 35.

LOAD FACTORS

In principle, resistance and load factors are coupled. However, the variations in load factors due to variations in the resistance statistics are small. For practical purposes, it can be assumed that the influence from one to the other is negligible.

Assuming different weight control (unenforced and enforced) and truck traffic volume (light, moderate, and heavy), a total of six live load categories was defined. For each of these live load categories, the live load statistics as described in Chapter Three were used as tentative values in the calibration of live load factors. Live load category 1, which assumes unenforced vehicle weight control and light truck traffic volume, is essentially the site-specific live load environment for most bridges in the statistical data base that was used to establish the target safety index and to determine resistance factors. For this live load environment, the following load factors are deemed appropriate for $\phi = 0.94$,

$$\gamma_D = 1.2 \text{ and } \gamma_L = 1.6 \qquad (64)$$

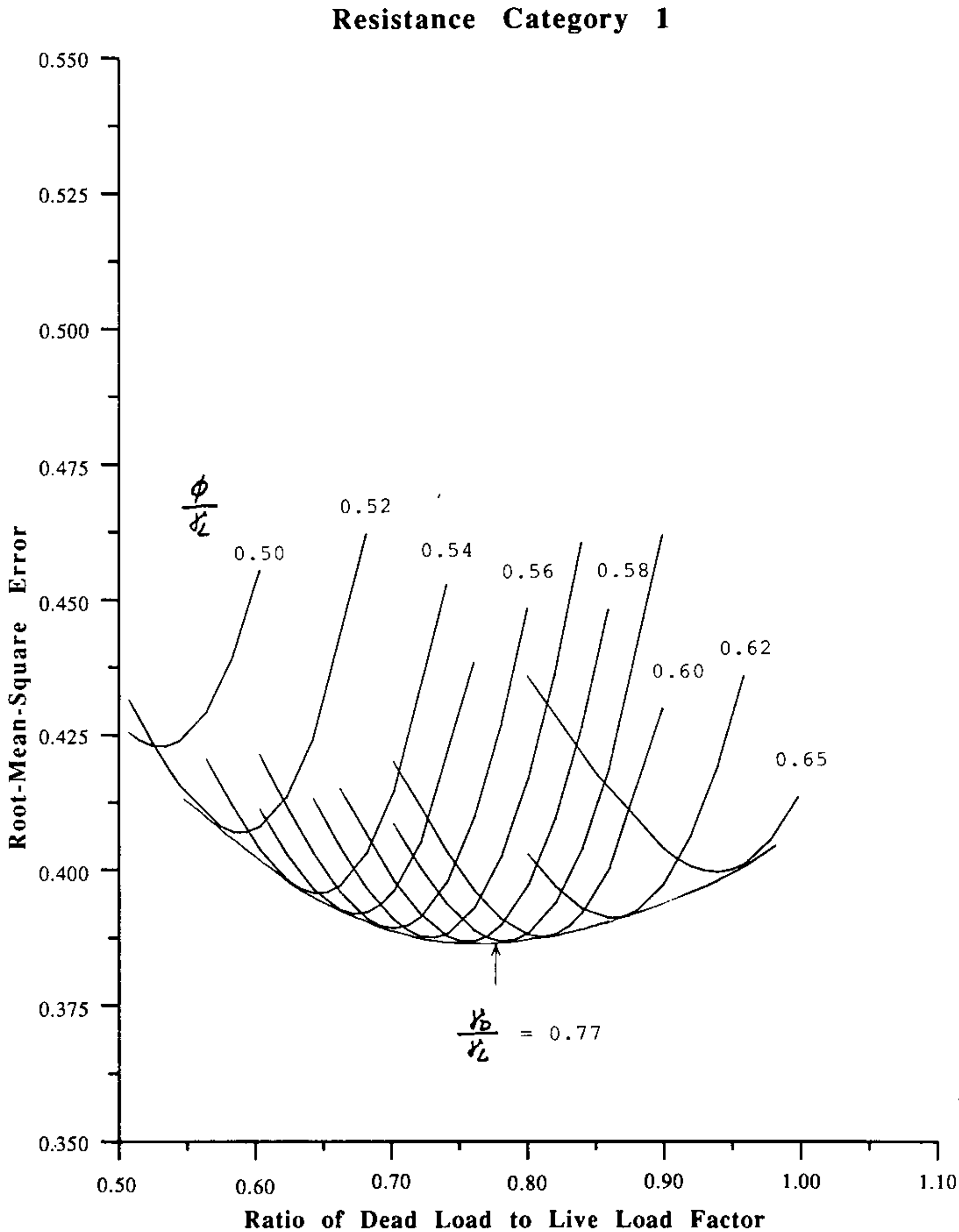

Figure 32. Trial and error calibration.

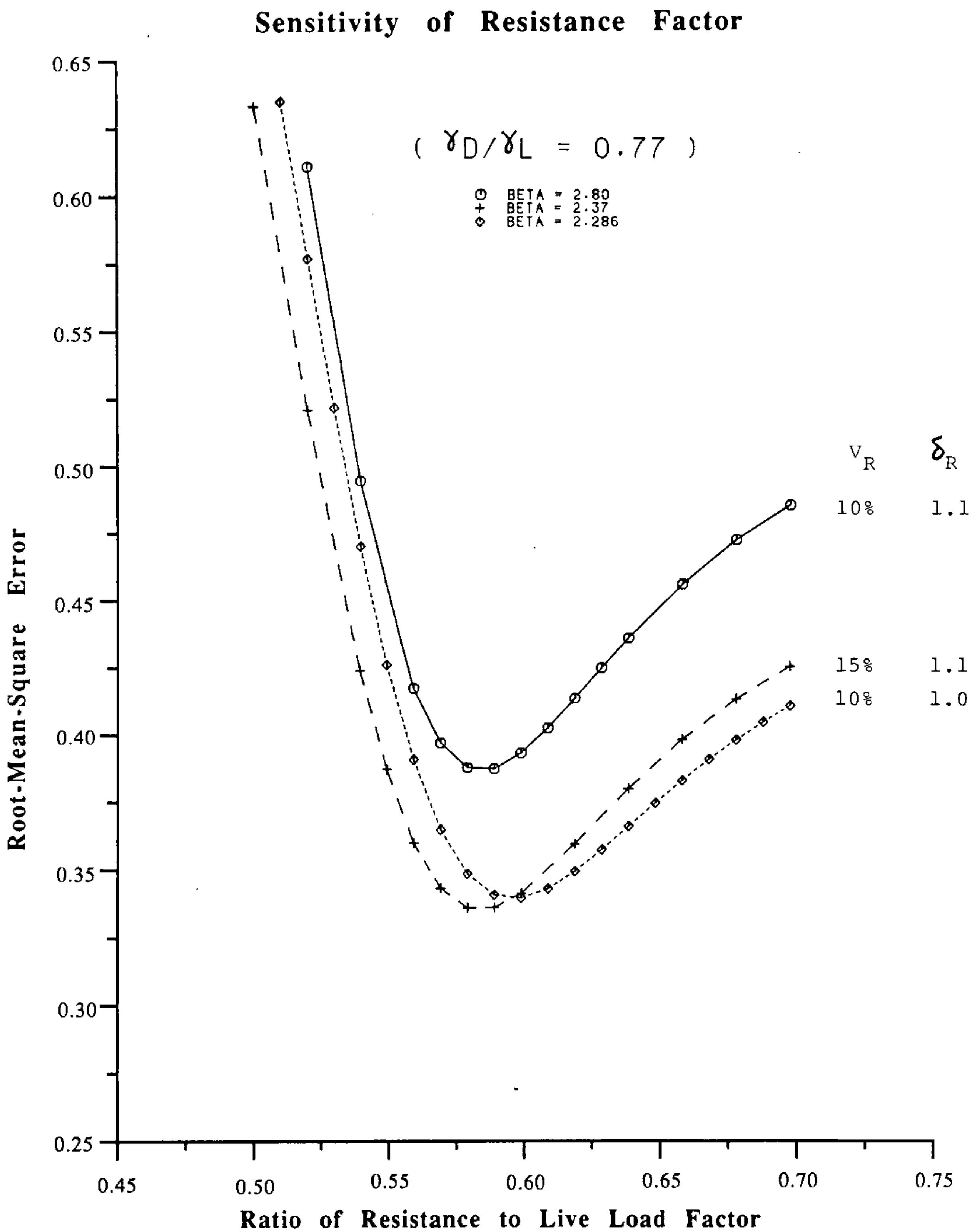

Figure 33. Sensitivity of the resistance factors (γ_D/γ_L = 0.77).

It was assumed further that a constant dead load factor may be used for all live load categories, i.e., $\gamma_D = 1.2$. Given ϕ and γ_D, the live load factors were determined and are given in Table 26 and Figure 36.

DISCUSSION

One way to assess the Level 1 load factor criteria rating is the correlation of the rating and the safety level for all bridges in the data base. Figure 31 shows such a correlation for rating based on current AASHTO methods. Considerable scattering exists. New rating factors may be determined using the same site conditions for each bridge and the partial factors determined in previous sections. The correlation between the new ratings and the safety level is shown in Figure 37. This correlation is much better than the current practice. The better correlation can be attributed to two factors: first, the partial factors used; and second, the improved live load effect. As shown in Chapter Three, the maximum longitudinal moment caused by controlling one of the two measured trucks suggested by Moses could be substantially different from the HS loading. It is important, for

Table 25. Resistance to live load factors, ϕ/γ_L, for different resistance statistics ($\gamma_D/\gamma_L = 0.77$).

δ_R	V_R 10%	15%	20%	25%
1.1	0.59	0.53	0.47	0.41
1.0	0.52	0.48	0.42	0.37
0.9	---	0.43	0.38	0.34

Table 26. Live load factors based on live load categories.

Live Load Category	Weight Control	Truck Traffic Volume	γ_L
1	Unenforced	Light	1.60
2		Moderate	1.84
3		Heavy	1.93
4	Enforced	Light	1.48
5		Moderate	1.73
6		Heavy	1.82

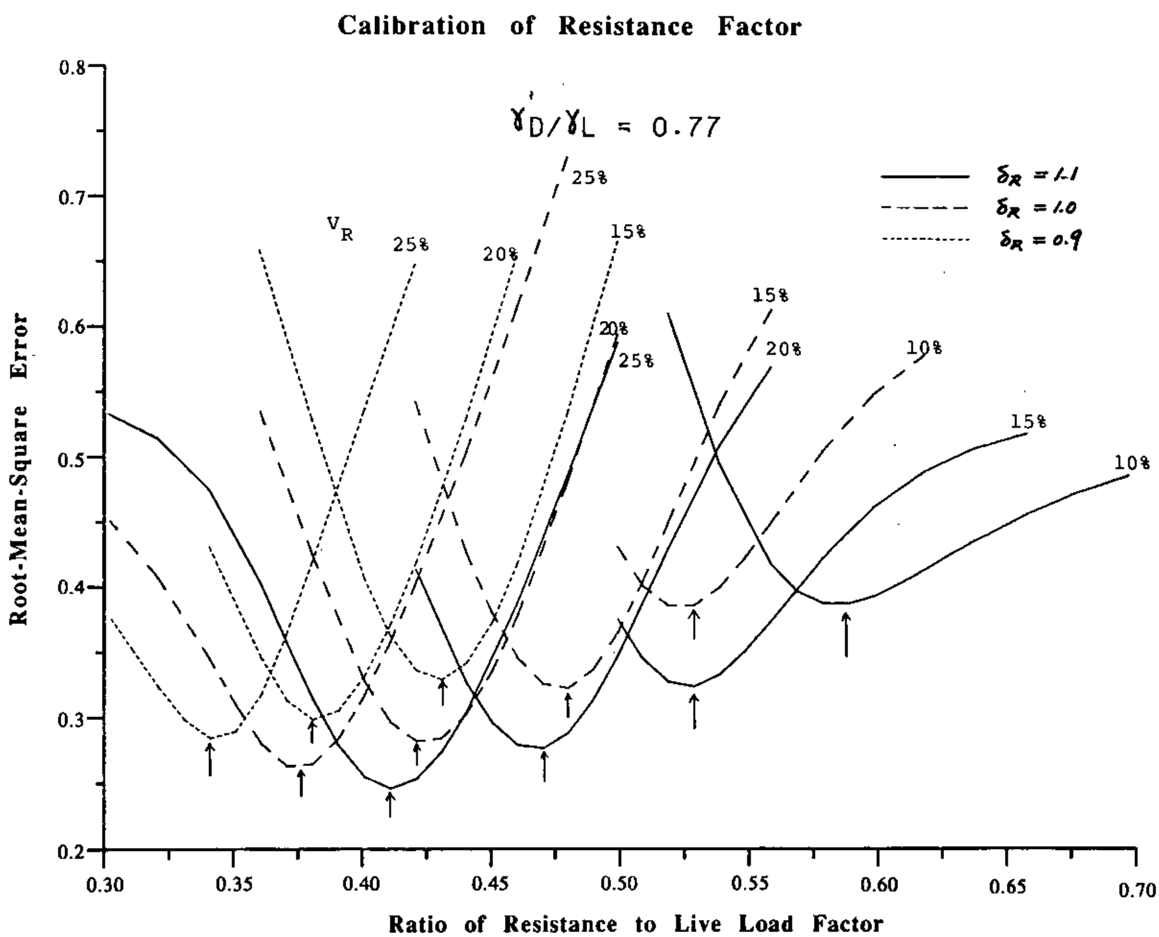

Figure 34. Calibration of resistance factors (γ_D/γ_L) = 0.77).

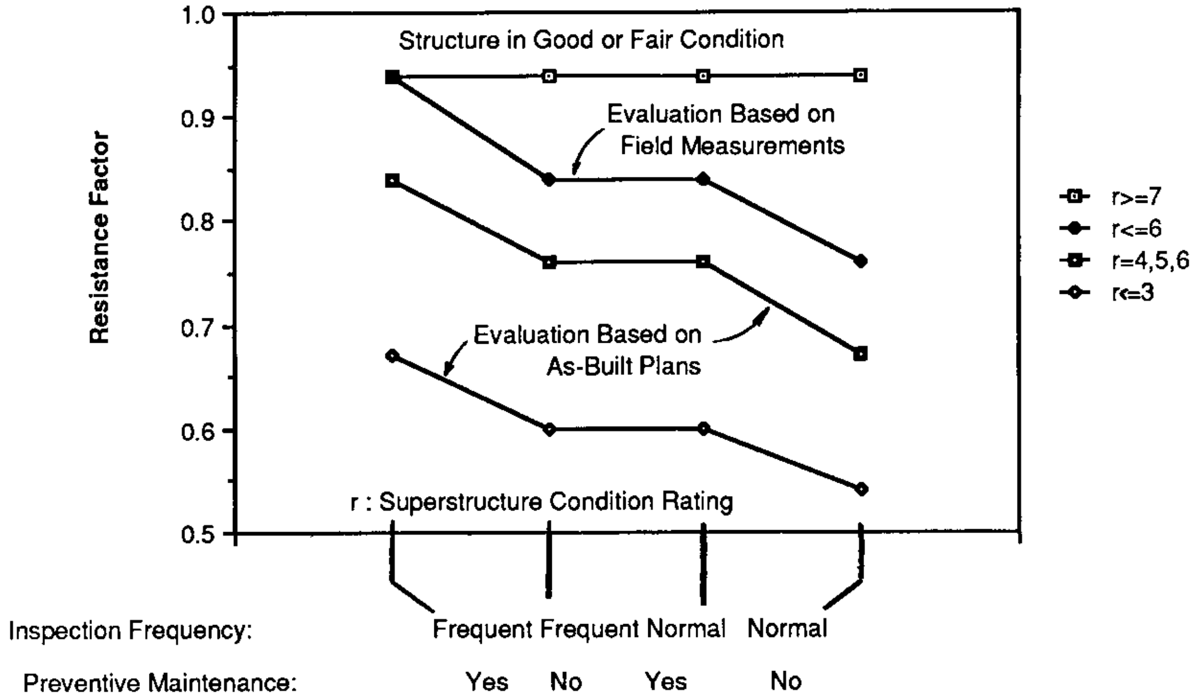

Figure 35. Resistance factors based on superstructure condition, inspection, and preventive maintenance.

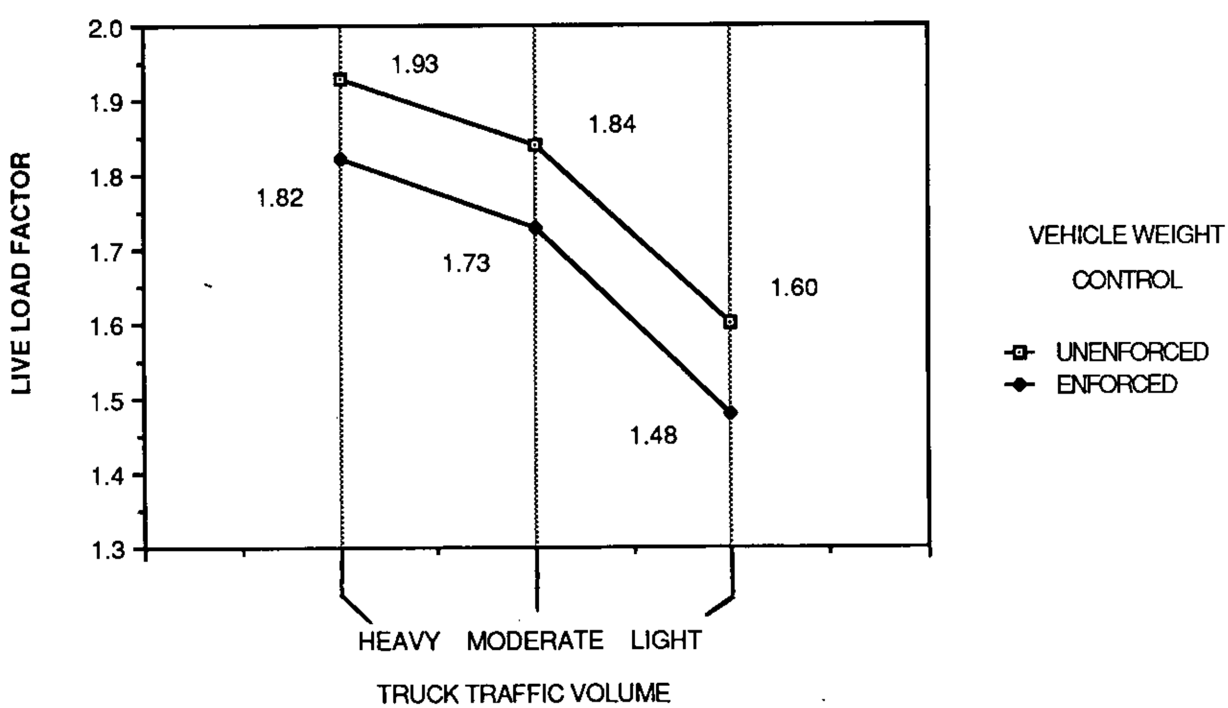

Figure 36. Live load factors based on vehicle weight control and truck traffic volume.

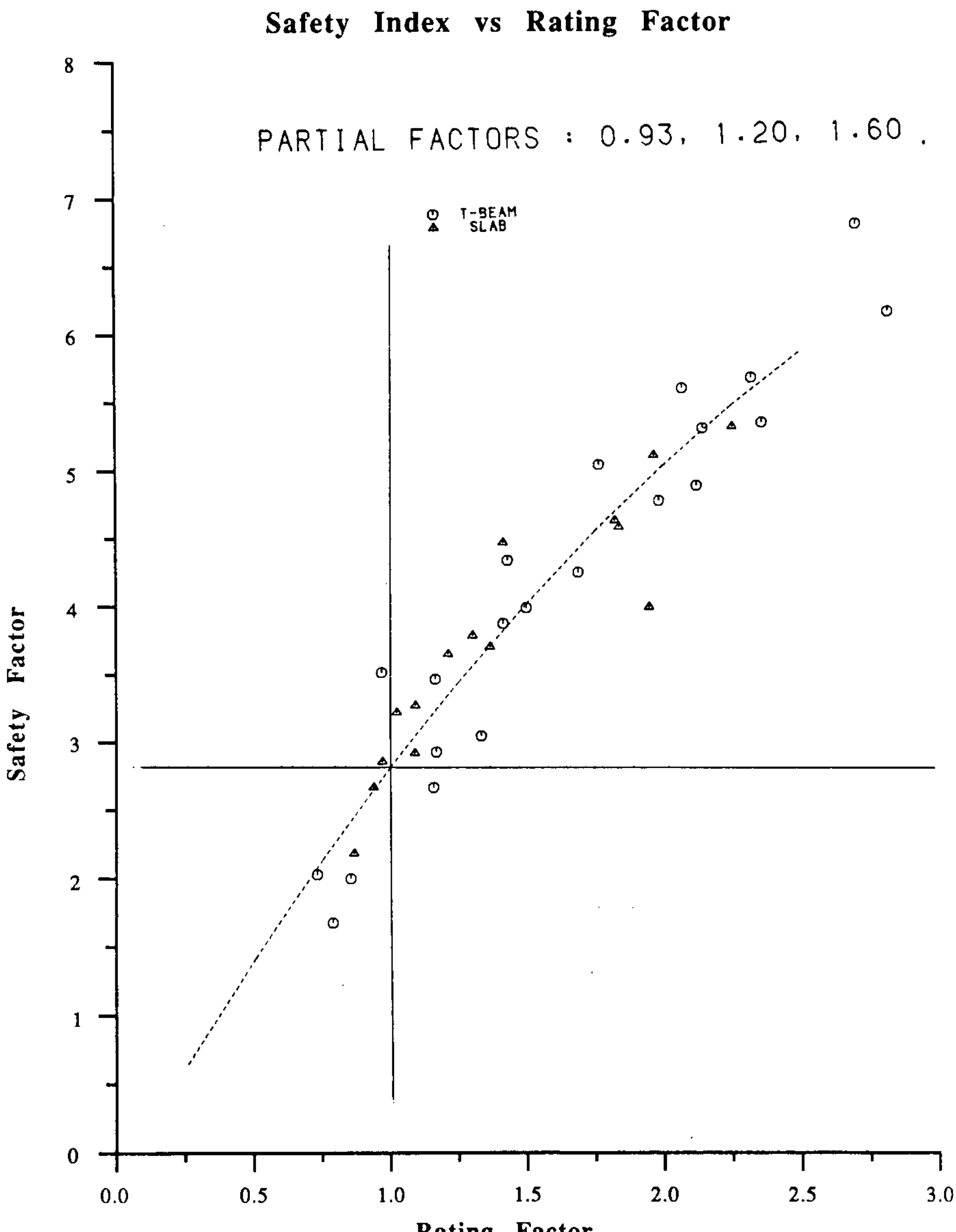

Figure 37. Correlation of the safety index and the rating factor using proposed partial factors.

calibration purposes, that the rating should be conducted based on the same trucks used in evaluating the safety indices. As demonstrated in Chapter Three (Fig. 19), HS20 rating or AASHTO legal truck ratings may be converted to the rating factor corresponding to the measured "average" trucks.

For bridges with rating less than 1, the maximum and minimum safety indices, β_j, evaluated for the reduced live load effect, $RF_j \cdot L_j$, are 3.63 and 2.39, respectively.

EXAMPLE PROBLEMS

Appendix A includes the guidelines for rating reinforced concrete bridge structures.

Example 1—Woodbridge Bridge, California (1925)

This is a single-span, simply-supported, two-lane, five-girder reinforced concrete T-beam bridge. The ultimate resistance and dead load moment at midspan are, respectively, $M_U = 493$ kip-ft, and $M_D = 124$ kip-ft.

The live load girder moment at midspan for the HS20 truck loading with the AASHTO impact effect is $M_L = 147$ kip-ft.

The rating factors based on the current AASHTO "load factor" method are as follows. At inventory level, $\frac{0.9\,(493) - 1.3\,(124)}{1.3\,(5/3)\,(147)} = 0.89$; and at operating level, $\frac{0.9\,(493) - 1.3\,(124)}{1.3\,(147)} = 1.48$.

The inventory level rating indicates that under the current design loading and design specification, the bridge does not satisfy the design limit state. However, the operating level rating indicates sufficient strength exists to carry the HS20 truck loads if the uncertainty of the truck load is no more than that of the dead load. Using the newly calibrated partial factors, the bridge may be rated as follows:

1. The bridge is in good or fair condition; $\phi = 0.94$ (Table A-2).
2. The dead load factor is 1.2 (Table A-4).
3. The truck traffic at the bridge site is considered light and the truck weight control is not enforced. The live load factor for this situation is 1.6 (Table A-7). Assuming smoothed roadway with an impact factor of 1.1, the nominal HS20 girder moment is 147 (1.1/1.3) = 124.4. Therefore, $RF_{HS20} = \frac{0.94(493) - 1.2(124)}{1.6(124.4)} = 1.58$.

With the assumption that the bridge is subjected to different truck traffic volumes and truck weight control enforcement (Table A-7), the following HS20 rating factors, showing the variation with live load conditions, were obtained:

	Truck Weight Control	
Truck Traffic Volume	Unenforced (*RF*)	Enforced (*RF*)
Light	1.58	1.71
Moderate	1.37	1.46
Heavy	1.31	1.39

The increase in rating due to weight control enforcement is about 6 to 8 percent.

The dead load effect includes the following components:

Structural Concrete	72.67	58.5%
A.C. Overlay	32.96	26.5%
Railing	18.59	15.0%
$M_D =$	124.22 kip-ft	

The contribution due to A.C. overlay is relatively high in this bridge. Assume that a dead load factor of 1.4 is necessary for the overlay material and maintain the same factor of 1.2 for structural concrete and railing. The rating factor is

$$RF_{HS20} = \frac{0.94(493) - 1.2(72.67 + 18.59) - 1.4(32.96)}{1.6(124.4)} = 1.55$$

which is only 2 percent less than the case with no consideration given to the different dead load categories.

To obtain rating factors corresponding to controlling trucks suggested by Moses, *RF*, the conversion factors can be obtained from Figure 19 based on span length (26 ft) for different truck traffic.

$$RF = \begin{cases} 1.09 & \text{light traffic} \\ 0.88 \; * \; RF_{HS20} & \text{moderate traffic} \\ 0.79 & \text{heavy traffic} \end{cases}$$

For longer span lengths, however, the HS20 rating is usually more conservative than the rating for the controlling trucks suggested by Moses even for the heavy truck traffic category.

Example 2—Lishakill Bridge, New York (1911)

This is a simple-span five-girder T-beam bridge with a clear span length of 36 ft. The width of the roadway pavement is 18 ft. The center-to-center girder spacing is 64 in. This bridge has the highest ratio of dead load effect to live load effect, 1.44, among the T-beam bridges in the data base.

The ultimate resistance and dead load effect at midspan are, respectively: $M_U = 600$ kip-ft, and $M_D = 303$ kip-ft.

For the HS20 truck and AASHTO impact (1.3), the live load effect at midspan is $M_L = 211$ kip-ft. The ratings according to the AASHTO load factor method are 0.32 at inventory level and 0.53 at operating level.

Based on the "allowable stress" method, the bridge was rated by the state as 0.21 and 0.86 at inventory and operating level, respectively.

Assuming the bridge is in fair condition (Table A-2), has unenforced vehicle weight control and light truck traffic volume (Table A-7), and a smooth roadway, we can rate the bridge for the live load moment of (211 × 1.1/1.3) = 178 kip-ft, as

$$RF_{HS20} = \frac{0.94(600) - 1.2(303)}{1.6(178)} = 0.70$$

The rating factor corresponding to the controlling truck suggested by Moses is

$$RF = \frac{1}{0.79} RF_{HS20} = 0.89 \text{ (Fig. 19)}$$

which is 27 percent higher than the HS20 rating.

Example 3—Adams County Route 36 (Bridge No. 001-0005) Illinois (1924)

This is a four-girder, simple-span T-beam bridge with a span length of 51.5 ft and a girder spacing of 6.58 ft. The ultimate girder resistance at midspan is M_U = 1,678.8 kip-ft.

The dead load moment is composed of

Structural Concrete	359.8 kip-ft
A.C. Overlay	31.1
Railing	138.5
	M_D = 529.4 kip-ft

The HS20 live load moment (plus impact, 1.28) at an interior girder is M_L = 455.4 kip-ft. The AASHTO load factor ratings are 0.83 at inventory level and 1.39 at operating level.

Based on the partial factors for fair superstructure condition, smooth roadway (with impact 1.1), light truck traffic, and unenforced vehicle weight control, the rating factor is computed, for the live load moment of 391.4 kip-ft (=455.4 × 1.1/1.28), as

$$RF_{HS20} = \frac{0.94(1678.8) - 1.2(529.4)}{1.6(391.4)} = 1.51$$

In this bridge, the railing constitutes about 26 percent of the total dead load effect. Assuming that a dead load factor of 1.05 is sufficient for railing, and maintaining the same factor 1.2 for structural concrete and overlay, the rating factor may be determined as follows

$$\frac{0.94(1{,}678.8) - 1.2\,(359.8 + 31.1) - 1.05(138.5)}{1.6(391.4)} = 1.54$$

There is only about a 2 percent increase in the rating factor.

Example 4—Deteriorated Structure

The Woodbridge Bridge in Example 1 is used as an illustration. Assume that some deterioration in the bridge structure was reported and the superstructure condition rating of the bridge is 5. Based on the as-built plans, the ultimate capacity of an interior girder is M_U = 493 kip-ft. The load effects are M_D = 124 kip-ft and $M_L(I+1)$ = 124.4 kip-ft (smooth roadway).

The new ratings are calculated for the following assumptions:

1. The bridge is under the normal biennial inspection program. No preventive maintenance has been done. From Figure 35 (Table A-2), the resistance factor for this situation is 0.67. Therefore,

$$RF_{HS20} = \frac{0.67(493) - 1.2(124)}{1.6(124.4)} - 0.91$$

2. The bridge is inspected annually to observe the rate of deterioration. However, no preventive measure has been taken so far to retard further deterioration. The resistance factor for this case is 0.76. Therefore, RF_{HS20} = 1.13.
3. The bridge has been inspected annually since the first signs of deterioration were observed. In addition, preventive measures were taken to repair and to retard further deterioration. A resistance factor of 0.84 may be used in this situation. Therefore, RF_{HS20} = 1.33.

Example 5—Lovekin Boulevard Undercrossing (1972)

This is a 121.3-ft single-span, 40-ft wide, four-cell box-girder bridge. The ultimate resistance and dead load moment at midspan for the total cross section are, respectively: M_U = 42,100 kip-ft and M_D = 19,282 kip-ft.

The live load moment at midspan for an HS20 truck which included the AASHTO impact (1.20) and distribution factor ($S/7$) is M_L = 6,549.4 kip-ft.

The rating factors at inventory and operating level based on the current AASHTO load factor method are, respectively:

$$RF_{HS20} = \frac{0.90\,(42{,}100) - 1.3\,(19{,}282)}{1.3\,(5/3\,(6{,}549.4)} = 0.90$$

$$RF_{HS20} = \frac{0.90\,(42{,}100) - 1.3\,(19{,}282)}{1.3\,(6{,}549.4)} = 1.51$$

The inventory level rating indicates that under the current design loading and specifications, the bridge does not satisfy the design limit state. However, the operating level rating indicates that there is sufficient strength to carry the HS20 truck. Using the proposed calibrated partial factors, the bridge may be rated as follows:

1. The bridge is in good or fair condition; ϕ = 0.94 (Table A-2).
2. The dead load factor is 1.2 (Table A-4).
3. The truck traffic at the bridge site is considered light and the truck weight control is not enforced. For smooth roadway, the live load effect is (6,549 × 1.1/1.2) 6,003.6 kip-ft. The live load factor for this situation is 1.6 (Table A-7). Thus,

$$RF = \frac{0.94\,(42{,}100) - 1.2\,(19{,}282)}{1.6\,(6{,}003.6)} = 1.71$$

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

Reported herein is a study pertaining to the strength evaluation of existing reinforced concrete bridges. The emphasis is placed on the probabilistic calibration of the load and resistance factors in the Level 1 limit state format. This format is essentially the same as the current AASHTO Load Factor Method. However, by using the probabilistic calibration procedure, various aspects of the load and resistance variables may be accounted for systematically. Also, the inherent differences between the design of a new bridge and the evaluation of an existing bridge may be considered automatically by using different statistical data bases. Only the flexural strength of the superstructure was considered.

For the load environment, site-specific live load parameters based on weigh-in-motion field measurements were used. These parameters include truck traffic volume, truck weight control enforcement, characteristic truck weight, and roadway roughness and/or bump.

The data available for this study are based on measurements conducted in 10 states. The live load model was formulated such that individual parameters may be identified and easily updated whenever additional information becomes available or when special conditions exist in any single state that warrant recalibration.

The following factors were considered in evaluating resistance: degree of deterioration in the structure, frequency of inspection, preventive maintenance, and basis of ultimate resistance evaluation.

For each resistance category in the hierarchical classification, a pair of values was assumed for V_R and δ_R. With the exception of bridges in good or fair condition, these statistical values were based on information available in the literature or subjective engineering judgments.

Although there is a paucity of available data on which to rigorously calibrate all the variations in this procedure, it is believed that the formulation of the procedure developed herein provides a sound basis for future improvements. In addition, it is enough of an improvement in current practice to warrant AASHTO acceptance as a guide specification.

By using the site-specific live load model, a target safety index of 2.8 was established based on current AASHTO rating at operating level. It should be noted that in selecting this target safety index, the high system redundancy inherent in typical reinforced concrete bridges has been implicitly considered. The corresponding system safety index will be higher. Because of the complexities involved in a complete system reliability analysis, it is difficult to ascertain this system reliability. However, a component safety index of 2.8, which is based on the operating level rating of existing bridges, is deemed sufficient to ensure the appropriate overall system reliability.

The limit state criteria may be written as

$$\phi R = \text{Factored loads} \qquad (65)$$

If the load factors were kept at the same level as in the current AASHTO specification, the nominal central safety factor would be proportional to $1/\phi$. The adoption of a higher ϕ factor (0.94) for bridges in good or fair condition only reduces the central safety factor by 4 percent. Judging from the past performance of bridges, this change is minimal. For deteriorated structures, lower ϕ values are suggested for different inspection and maintenance efforts.

The right-hand side of the limit state equation may be expressed in a format similar to the current AASHTO load factor method,

$$\text{Factored load effect} = \gamma_D (D + q\,L) \qquad (66)$$

where q is the ratio of γ_L and γ_D. Written in this form, γ_D represents the basic uncertainties common to both dead load and live load effects, while q is the additional uncertainty solely related to the live load. The dead load factor chosen in this study is 1.2, which is about 9 percent less than the current design value. The live load factors determined in this study are for different live load categories based on available live load statistics. The corresponding q factors are as follows:

Weight Control Enforcement	Truck Traffic Volume		
	Light	Moderate	Heavy
No	1.33	1.53	1.61
Yes	1.23	1.44	1.52

In designing a new reinforced concrete bridge, there is much higher uncertainty about the truck traffic volume and weight control enforcement that could exist during the life of the bridge. It is necessary, therefore, to use a conservative value for the q factor. It is interesting to note that the current AASHTO design value of $5/3$ is very close to 1.61 for the most severe live load condition in this study. For existing bridges, truck traffic conditions are known or may be obtained. It is justified to use a smaller value for the q factors as obtained from the probabilistic calibrations. By choosing the q factor (or live load factor) according to the site-specific truck traffic condition, rating of existing bridges may be conducted more rationally. It has been shown that for all bridges in the data base with light truck traffic, the new rating factors obtained are close to the current AASHTO operating level rating. In addition, if the truck load is completely under control, such as in the case where the bridge is temporarily closed to allow a particular overload truck of known weight and wheel configuration to cross at crawl speeds, it may be feasible to reduce q further.

During the course of this project, various aspects related to the strength evaluation of existing reinforced concrete bridges were reviewed. Some of the more important research needs that could be identified include the following:

1. At present, the knowledge about deteriorated, hazardous structures is lacking. It is important that bridge inspections may distinguish between cosmetic-type deterioration and structural deterioration. The superstructure condition rating as currently implemented in the NBIF should be rated systematically and should reflect consistently the degree of deterioration.

2. For seriously deteriorated structures, more full-scale load testing is recommended. On the one hand, the test results could be directly used to rate the particular structures. The results, if documented properly, could also be used to establish a statistical data base of resistance of deteriorated structures. This could be used for further refined calibration.

3. The FHWA-computerized National Bridge Inventory File is a valuable data base. However, more useful information of a bridge could be included. To maximize its use in rating, it is recommended that the following items be included: (a) resistance and load effects (shear and moment), and (b) failure mode corresponding to the rating factor reported. With this additional information, the centralized NBIF data base can be very useful not only for *administrative* purposes but also for such *technical* purpose as selecting a target safety index for future code revisions.

4. Among the live load parameters, the girder distribution factor needs further study. In the current AASHTO code, the girder distribution is given in terms of number of wheel lines. It is implicit in the formula for the two lanes loaded case that two identical trucks occupy the bridge simultaneously. When applied to the rating of a special overload truck, the AASHTO load distribution factor becomes unnecessarily conservative because the possibility of two identical overload trucks side-by-side on the bridge is unlikely. For this case, guidelines are needed so that the live load effect distributed to a girder due to a standard truck and a special overload truck can be evaluated. An ongoing research project (NCHRP Project 12-26) on wheel load distribution will take this into account.

5. More field measurements of highway bridge live loadings should be conducted both for areas with high percentage of overload permit trucks and for more typical rural area and secondary roads. From these measurements, site-specific live-load parameters may be identified. With a more extensive data base, the values used in this study may be verified or revised. For a certain geographical locality, it may be advantageous to establish regional live load statistics instead of using the national average.

In addition, field measurement data obtained from other measurement methods should also be surveyed to avoid any potential bias from a single measurement method. With these data available, a loading code, similar to the NBS A58 loading code (*19*), can be developed specifically for bridges, which is applicable to all bridge types.

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APPENDIX A

GUIDELINES

SECTION 1

INTRODUCTION

1.1 PURPOSE

These guidelines establish a methodology for determining the live load capacity of existing reinforced concrete bridges.

1.2 SCOPE

The methodology is presented in a limit states format using both load and resistance factors. This format provides a basis on which probability theory and engineering judgment can be rationally combined. The approach is general enough to allow for independent consideration of each of the major variables that can affect the determination of the live load capacity of a bridge. The numerical values assigned to the load and resistance factors are based on probabilistic calibration with the current AASHTO practice utilizing a set of existing bridge structures, available statistical data, and engineering judgments such that, on the average, a safety index of 2.8 may be attained. These values are subject to change as more statistical data become available.

1.3 APPLICABILITY

This methodology was developed for existing, reinforced concrete bridges consisting of rigid frames, simple spans, or continuous spans with right or skewed supports. The applicable bridge types include slab, girder, T-beam, and box-girder bridges with short-to-medium span lengths. These guidelines are intended for bridges with known structural details.

SECTION 2

SYMBOLS AND DEFINITIONS

The following symbols and definitions apply to these Guidelines:

D_i Nominal dead load effect of element i
I "Impact factor" used to approximate the dynamic effects of moving vehicles
L Nominal live load effects for the rating vehicle
R Nominal resistance
RF Rating factor
γ_{Di} Dead load factor for element i
γ_L Live load factor
ϕ Capacity reduction factor to account for uncertainties in resistance due to variations in dimensions, material properties, and theory.

SECTION 3

LIMIT STATES EVALUATION

3.1 GENERAL

The determination of bridge live load capacity requires some knowledge of both the physical properties of the bridge and the applied loadings. This knowledge allows the prediction of nominal strengths and loads from established procedures. Because of the uncertainties in this knowledge, actual strengths and loads may vary from their nominal values. In addition, there are uncertainties both in the structural analysis and in determining the amount and extent of deterioration.

One method of accounting for these uncertainties is through a limit states evaluation developed on the concept of consistent structural reliability.

3.2 DEFINITION OF LIMIT STATES EVALUATION

When a structure or structural element becomes unfit for its intended purpose, it is said to have reached its limit state. Limit states fall into two categories: safety limit states and serviceability limit states. Structure reliability is the probability that a given structure will perform satisfactorily by not reaching its limit state over a specified time period.

Safety limit states correspond to the ability of the structure or structural component to support the applied loads. Serviceability limit states either restrict the normal use of a bridge or affect its durability. The acceptable level of structure reliability will vary depending on the type of limit state.

3.3 THE BASIC RATING EQUATION

The basic rating equation used in these guidelines is

$$RF = \frac{\phi R - \sum_{i=1}^{m} \gamma_{Di} D_i}{\gamma_L L(1+I)} \qquad \text{(A-1)}$$

where:

RF = rating factor (the portion of the rating vehicle allowed on the bridge);
ϕ = capacity reduction factor;
m = number of elements included in the dead load;
R = nominal resistance;
γ_{Di} = deadload factor for element i;
D_i = nominal dead load effect of element i;
γ_L = live load factor for rating vehicle;
L = nominal live load effect for the rating vehicle(s) (see Commentary, Appendix B, Section 3.3); and
I = live load impact factor.

This equation should be evaluated for critical components at both safety and serviceability limit states. The following major sections will discuss methods for determining each of the variables in this equation.

3.4 EVALUATION PROCESS

The limit states evaluation process described in these guidelines consists of the following steps:

Step 1. Collection of information—field inspection, office records, and special testing.
Step 2. Selection of rating vehicle—
Step 3. Analysis—identification of critical limiting states(s), determination of nominal load effects, and determination of nominal resistance.
Step 4. Selection of load and resistance factors—charts and tables in these Guidelines.
Step 5. Determination of rating factors—

The results of the structural strength evaluation may be used to determine restrictions on the use of the bridge by normal traffic load (load limit posting), the maximum weight of the occasional overload vehicle allowed to mix with normal traffic (unsupervised overload permits), or the absolute maximum weight of the vehicle allowed on the bridge under controlled circumstances (supervised overload permit). In addition, a substandard live load capacity may be used as justification for future repairs and/or replacement.

Figure A-1 presents a flowchart of the evaluation process.

SECTION 4

RESISTANCE

4.1 GENERAL

Resistance shall be calculated for the probable limiting mode(s) for both the safety and serviceability limit states as described in Sections 4.2 and 4.3. In most short-to-medium span reinforced concrete bridges, flexure in the primary load-carrying members (i.e., girders or longitudinal strip of a flat slab bridge) is the probable limiting mode; thus, nominal resistance is the flexural capacity of these members. Occasionally, shear in the girders or bending and shear in other structural components, such as slabs or bent caps, are more likely limiting modes. This is particularly true when these components are badly deteriorated. Because shear failures can be catastrophic, care should be taken not to overlook the possibility of a shear failure. When in doubt as to the most probable limiting mode, the rating equation should be evaluated for each potential limiting mode.

Nominal resistance shall be modified by a capacity reduction factor. This factor, which accounts for uncertainties in calculating nominal resistance, is described in Section 4.4.

Only the resistance factor for flexural behavior is considered. The resistance factor for shear currently used in AASHTO specifications may be used.

4.2 RESISTANCE—SAFETY LIMIT STATES

The nominal resistance of reinforced concrete members at the safety limit state will be the ultimate strength of the given member. Nominal strength calculations shall take into consideration the observable effects of deterioration, such as loss of concrete or steel cross-sectional area, loss of composite action,

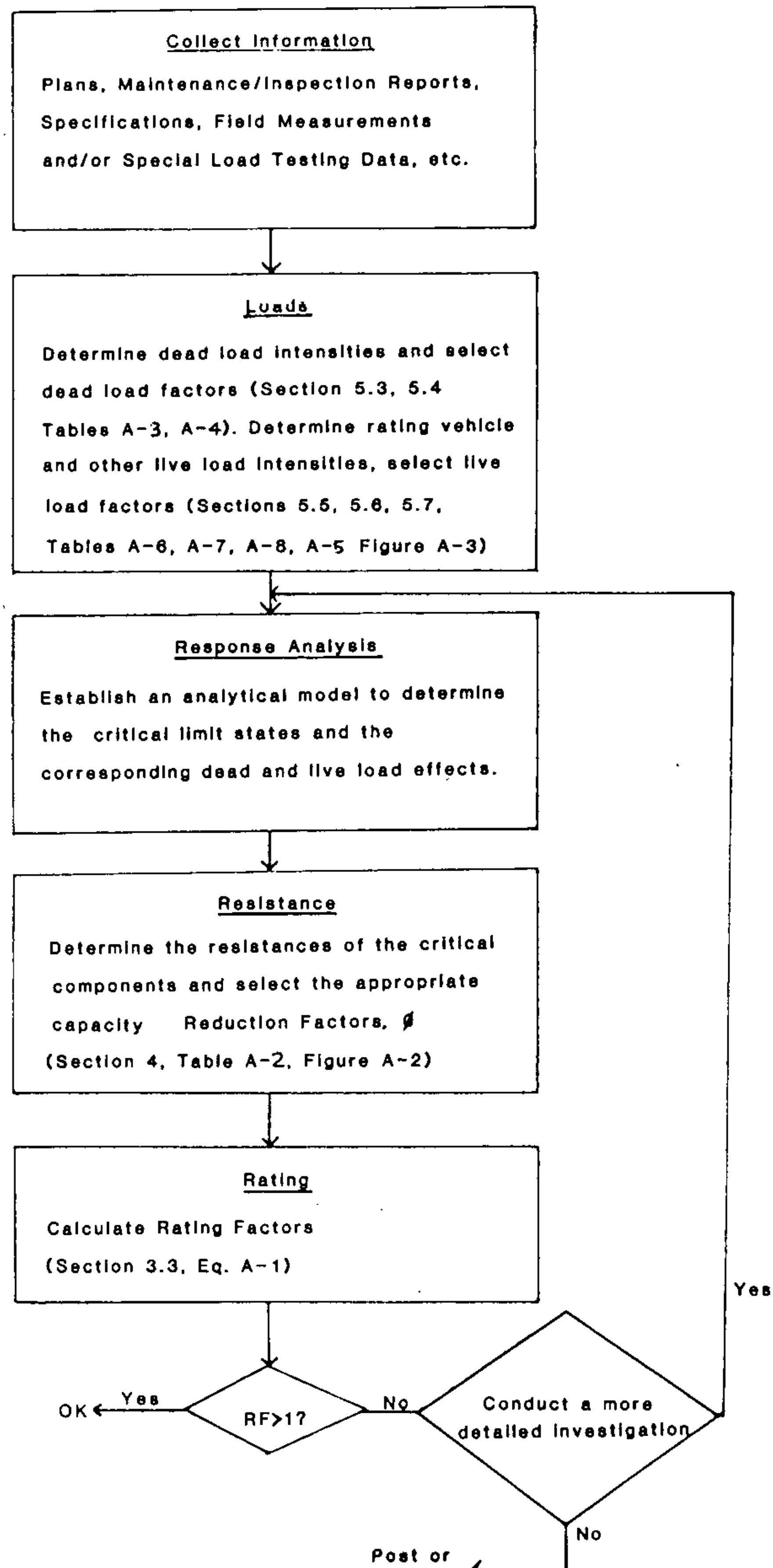

Figure A-1. Flowchart of the evaluation process.

or reduced material strengths. Alternately, nominal strengths may be determined from plan dimensions with deterioration accounted for by reduced resistance factors.

4.2.1. Concrete

The strength of sound concrete shall be assumed to be equal to either the values taken from the plans and specifications or the average of construction test values. When these values are not available, the ultimate stress of sound concrete may be assumed to be 3,000 psi. A reduced ultimate strength shall be assumed (no less than 2,000 psi, however) for unsound or deteriorated concrete unless evidence to the contrary is gained by field testing.

4.2.2. Reinforcing Steel

The area of tension steel to be used in computing the ultimate flexural strength of reinforced concrete members shall not exceed that available in the section or 75 percent of the steel reinforcement required for a balanced condition. The steel yield stresses to be used for various types of reinforcing steel are given in Table A-1.

4.3 RESISTANCE—SERVICEABILITY LIMIT STATES

The nominal resistance for reinforced concrete members at the serviceability limit state shall be computed by using the elastic theory of flexure. The allowable steel stress limitations will be based on fatigue and crack control requirements as described in the current AASHTO specifications. The following conditions apply to the evaluation of serviceability limit states:

1. Restrictions of nonpermit traffic (i.e., posting) will not be required to maintain serviceability.
2. Fatigue stress limitations do not need to be considered for occasional overload trucks.
3. Frequent inspections are recommended for bridges subjected to live loadings that produce steel stresses beyond the recommended allowable stresses for serviceability.

Other serviceability limit states do not need to be considered except at the discretion of the engineer.

4.4 CAPACITY REDUCTION FACTOR

The nominal resistance shall be multiplied by a capacity reduction factor to account for uncertainties due to variations in material strengths, dimensional tolerances, accuracy of the methods used to calculate nominal resistance, current superstructure condition, and future deterioration. The capacity reduction factor also accounts for variations in inspection and maintenance efforts that affect the ability to detect and/or prevent future deterioration or distress that can potentially result in losses in live load capacity. The resistance categories are classified hierarchically as shown in Table A-2 according to the following: degree of deterioration, evaluation based on as-built plans or field measurements, inspection frequency, and preventative maintenance.

Table A-1. Yield stress of reinforcing steels.

Reinforcing Steel	Yield Stress F_y (psi)
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Intermediate Grade and unknown after 1954 (Grade 40)	40,000
Hard Grade (Grade 50)	50,000
Grade 60	60,000

Table A-2. Resistance categories.

Superstructure Condition* (r)	Rating based on: Plans	Field Measurement	Inspection Frequency: Frequent	Normal (2 yrs.)	Preventive Maintenance: Yes	No	ϕ
Good or Fair $r \geq 7$	—	--	--	—	--	—	0.94
		X	X		X		0.94
		X	X			X	0.84
		X		X	X		0.84
		X		X		X	0.76
Deteriorated	X		X		X		0.84
(r=4,5,6)	X		X			X	0.76
	X			X	X		0.76
	X			X		X	0.67
		X	X		X		0.84
Seriously		X	X			X	0.76
Deteriorated		X		X	X		0.76
to Potentially		X		X		X	0.67
Hazardous	X		X		X		0.67
($r \leq 3$)	X		X			X	0.60
	X			X	X		0.60
	X			X		X	0.54

* Item 59 from National Bridge Inventory File

Capacity reduction factors for safety limit states shall be taken from Figure A-2 for flexure. The capacity reduction factor for serviceability limit states shall equal 1.0.

SECTION **5**

LOADS

5.1 GENERAL

Bridges shall be evaluated for their ability to carry dead load and traffic live load, including the dynamic effects of moving vehicles. Normally, other types of loadings such as wind, earthquakes, and thermal forces shall not be considered except as determined by the Engineer.

5.2 COMBINATION OF LOADS

The load factors described herein apply to AASHTO Group I loading as described in Sec. 1.2.22 of the AASHTO *Standard Specifications for Highway Bridges.* Loading combinations involving live loads other than traffic live load and vehicular impact require the use of the load factors for Group II through X given in Table 1.2.22 of the AASHTO *Standard Specifications for Highway Bridges.* These combinations shall be considered for the safety limit states only.

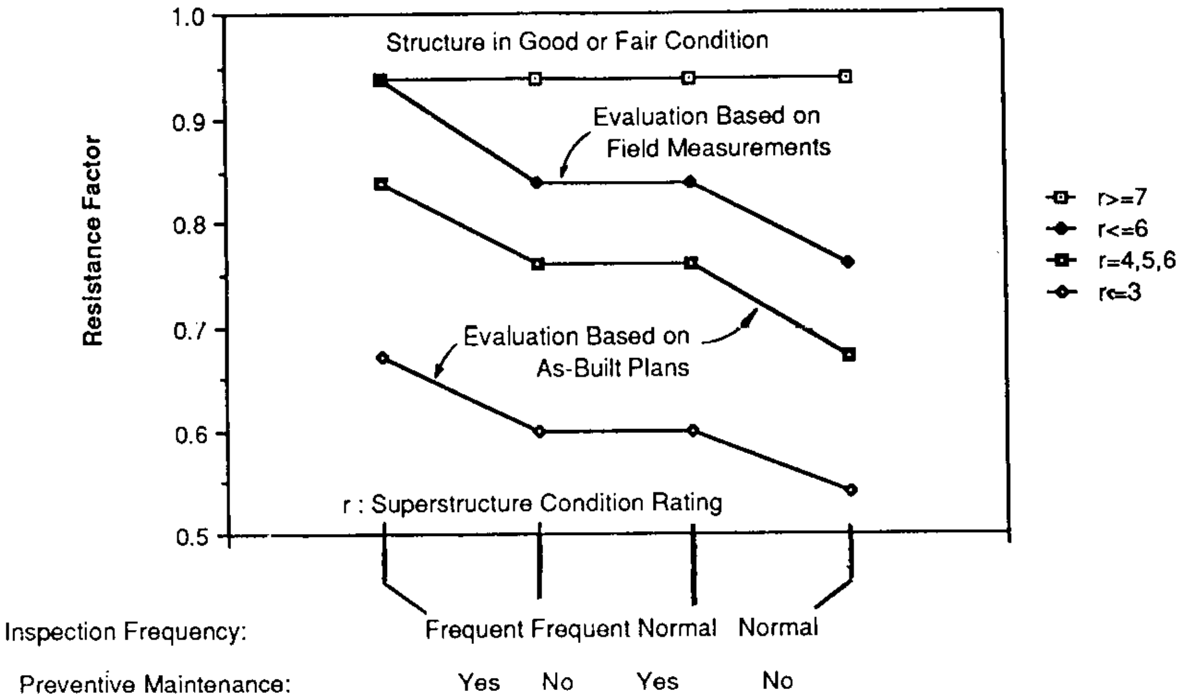

Figure A-2. Capacity reduction factor—flexure.

5.3 DEAD LOADS

Dead loads, which shall be determined from dimensions on the plans or from field measurements, shall include the weights of each of the permanent parts and appendages of the bridge. Particular care should be given to include any additional dead load resulting from modification to the bridge. The minimum unit weights of materials to be used in computing the dead load shall be taken from Table A-3.

5.4 DEAD LOAD FACTORS

Partial dead load factors have been established to reflect the various degrees of control used in producing the structural and nonstructural components of the bridge.

Separate dead load factors may be applied for various types of components according to Table A-4.

For serviceability limit states, the dead load factor shall be equal to 1.0

5.5 LIVE LOADS

The traffic live load used for determining the live load capacity shall be representative of the actual vehicles using the bridge. In most cases, the typical vehicles shown in Figure A-3 or the standard AASHTO "H" or "HS" vehicle will satisfy these criteria. The axle spacings and weights chosen for the typical vehicle types shown in Figure A-3 were selected from actual maximum legal loads conforming closely with the regulations of most states. In some cases it may be necessary to adjust the given axle weights and spacings to conform with allowable maximum legal weights and lengths, which vary from state to state. Because axle weights and spacings for overload vehicles may vary considerably from those of typical legal loads, some jurisdictions may find it useful to develop special vehicle configurations for the purpose of issuing overload permits.

In computing the rating factor for normal loading, only one vehicle shall normally be considered in any one lane. The number of traffic lanes to be loaded shall be in conformance with

Table A-3. Dead load unit weights.

MATERIAL	UNIT WEIGHT (lbs. per cu. ft.)
Asphalt surfacing	144
Concrete, plain or reinforced (normal weight)	150
Steel	490
Cast Iron	450
Timber (treated or untreated)	50
Earth (compacted), sand, gravel or ballast	120

Table A-4. Dead load factors.

	γ_D
Structural Concrete	1.20
Factory Produced Component	1.05
Asphalt Wearing Surface	1.40

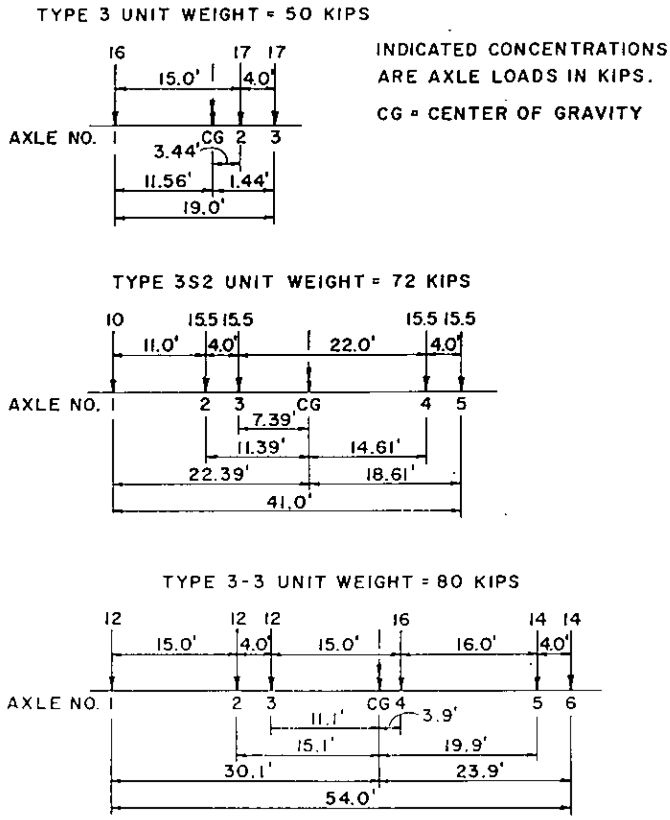

Figure A-3. Typical legal load types.

the current AASHTO *Standard Specifications for Highway Bridges,* except that the minimum roadway width for two lanes shall be reduced to 18 ft. When the engineer determines that conditions of traffic movement and volume would warrant it, fewer traffic lanes than specified previously may be considered.

Unless special circumstances exist that would make loading by multiple overload vehicles probable, only one overload vehicle needs to be considered on the bridge at any one time. Unless special traffic control is provided, one legal vehicle shall be assumed to occupy the lane adjacent to the overload vehicle. When the engineer determines the conditions of traffic movement and volume would warrant it, the legal vehicle may be eliminated.

The gage distance between wheels of typical legal vehicles shall be taken as 6 ft. The spacing, center-to-center, of adjacent wheels of passing vehicles shall be taken as 3 ft and the center of the outside wheel of any vehicle shall be considered as applied at a distance not less than 1.5 ft from the face of the curb.

The probable maximum sidewalk loadings should be used in calculations for safe load capacity ratings. This loading will vary from bridge to bridge, depending generally on its location. Because of this variation, the Engineer must use his judgment to make the final determination of the unit loading to be used. This loading will not exceed the design sidewalk loading given in the AASHTO *Standard Specifications for Highway Bridges.*

5.6 LIVE LOAD FACTORS

Live load factors account for the variability of traffic live load, including overweight vehicles and/or axles, the presence of multiple vehicles, different axle spacings, and variations in accuracy in the analysis of load effects.

The degree of refinement or sophistication used to determine the distribution of live loads to the load-carrying components shall be categorized into one of the three levels shown in Table A-5.

The methodology described in these guidelines makes additional allowances for the traffic volume on the bridge and the enforcement of weight limit control. Six live load categories may be classified as shown in Table A-6.

Live load factors to be used in evaluating safety limit states shall be taken from Table A-7. For serviceability limit states, live load factors shall be equal to 1.0.

Table A-5. Live load distribution levels.

LEVEL	DESCRIPTION
1	Grillage analogy, orthotropic plate theory, finite element, or specially prepared influence surfaces developed by using one of these methods
2	Load distribution based on formulas which have been derived for specific loads, such as AASHTO design live load distribution factors for AASHTO design loads
3	Load distributions based on formulas which are not specifically intended for the loading under consideration.

Table A-6. Live load categories.

Weight Limit Control	Traffic Volume	Category
Unenforced	Light (200 Trucks/Day)	1
	Moderate (2000 Trucks/Day)	2
	Heavy (10,000 Trucks/Day)	3
Enforced	Light	4
	Moderate	5
	Heavy	6

Table A-7. Live load factors.

Live Load Category	Live Load Distribution Levels 1*	2	3*
1	1.44	1.60	1.76
2	1.65	1.84	2.02
3	1.74	1.93	2.12
4	1.33	1.48	1.63
5	1.56	1.73	1.90
6	1.64	1.82	2.00

*Derived from Level 2 by ±10% adjustment.

5.7 IMPACT

The dynamic effects of moving live loads shall be included in the evaluation of both the safety and serviceability limit states. The dynamic effect traditionally referred to as the "impact factor" is applied to the live load to account for the dynamic interaction between the bridge and the suspension system of the moving vehicle.

The dynamic effects of moving live loads shall be included in the evaluation Group A components as described in Sec. 1.2.12 of the AASHTO *Standard Specifications for Highway Bridges.*

A dynamic load test, using design or rating live load moving at normal speed across the bridge, may be used to establish values for dynamic load allowances. In the absence of these test data, the impact factors shall be determined according to the roadway roughness as shown in Table A-8.

Table A-8. Impact factors.

Roadway	I
Rough surface; bump in approaches or expansion joints	0.20
Smooth surface	0.10

SECTION 6

DETERIORATION

6.1 GENERAL

From the collected field data, the degree of deterioration needs to be determined so that the revised load-carrying capacity of the structure can be determined.

6.2 MOMENT CAPACITY

Using the plots of deterioration, determine the cross section of the girder having the greatest percentage loss in moment capacity due to deterioration at the highly stressed region, e.g., within the center ½ of a simply-supported span. This may require several trial calculations. Assume that the moment capacity of the highly stressed region of the girder has been reduced by this same percentage.

When calculating the moment capacity, all of the concrete in the compression area from the girder's neutral axis is considered active.

The deteriorated concrete compression section is compensated for by reducing the depth of the slab an amount equivalent to the area lost from spalls and the areas above detectable undersurface fracture planes. Assume that the undersurface plane is level with the top of the top layer of reinforcing steel. Also assume that the plane of an undersurface fracture will travel up to the deck surface at a 45-deg angle from the edge of the detectable fracture.

When the steel section loss is too small to determine, or when there is visible evidence of corrosion on the concrete (e.g., rust stains) at a covered bar, assume that the bar has lost 3 percent of its cross section.

Reinforcement at a section should be considered effective only to the extent that stress in the steel can be developed due to bar anchorage. When the full yield stress of the steel cannot be developed, the effective steel area used to calculate flexural capacity shall be reduced in proportion to the reduction in the amount of steel stress that can be developed.

When more than half of the perimeter of a steel reinforcing bar is exposed, assume that no bond exists between the bar and the concrete over the length of exposure. When less than half of the perimeter is exposed, the bond should be reduced in proportion to the percentage of the perimeter exposed. When a longitudinal crack greater than 0.1 in. in width occurs in the concrete along a line parallel to the bar, assume that the bond is reduced by 10 percent over the length of the crack. When a longitudinal crack less than 0.1 in. in width exists, a bond reduction shall be assumed to vary linearly with the width of the crack from 10 percent at a width of 0.1 in. to no reduction at a crack width of 0.01 in. or less.

6.3 SHEAR CAPACITY

When the field report shows the following conditions existing in the region of high deterioration in the girder, the girder shear capacity should be reduced as indicated:

1. *Diagonal cracks*—reduce shear capacity of the concrete based on the width of the crack up to a maximum of 100 percent for crack widths of ¼ in. or more.
2. *Rust stains*—reduce area of shear steel by 5 percent.
3. *Vertical displacement across a crack(s)*—neglect shear capacity of concrete and reduce area of shear steel by 10 percent.
4. Spalled concrete along diagonal cracks—neglect shear capacity of concrete.

APPENDIX B

COMMENTARY

SECTION 1

INTRODUCTION

1.1 PURPOSE

The guidelines were developed to improve the procedures for evaluating the structural strength of existing reinforced concrete bridges. Unlike steel, reinforced concrete seldom fails catastrophically; however, gradual deterioration due to environmental factors can affect the structural capacity of reinforced concrete bridges. In addition, test results have indicated that reinforced concrete bridges generally have a greater structural capacity than indicated by the most commonly used analytical methods.

The guidelines address several of the discrepancies and shortcomings of existing evaluation procedures. Many questions still remain, but the methodology is developed within a framework that provides for the systematic improvement of the evaluation process. Moreover, the methodology can be used in conjunction with a wide range of engineering practices. The use of improved techniques is encouraged since credit is given to the improved reliability that can be obtained

One of the primary goals of the guidelines is to clarify the procedures used to evaluate the effects of deterioration. A more comprehensive field inspection procedure is included in Appendix D of this report for bridges that are seriously deteriorated.

1.2 SCOPE

The limit states approach using partial load and resistance factors was chosen as the basis for the proposed methodology because it conforms closely to the current AASHTO load factor design, while still allowing for a systematic consideration of the differences involved in bridge evaluation. This approach allows each variable to be addressed separately, researched (if needed), and proportionally weighed in the overall rating process. Thus, it provides a framework into which future developments can be incorporated.

Load and resistance factors can be calculated from the coefficient of variation of actual load effects and resistance, the ratio of the mean value to nominally determined values, and the desired safety level. Therefore, as more data become available on the distribution of actual load effects and resistances, new and more realistic load and resistance factors can be developed.

The definition of limit states themselves is likely to change with time as more experience is gained. By documenting the guidelines in a limit states format, new limit states can be systematically and rationally included with a minimum of disruption to evaluation practices.

The methods for calculating nominal resistance at the various limit states, the suggested techniques for determining nominal load effects, and the values for load and resistance factors suggested in the methodology were derived from the information available at the time of publication. It is almost certain that improvements will be made as experience is gained from using the methodology. Nothing in the guidelines should be interpreted as opposing the introduction of improvements provided they can be justified by the principles of good structural engineering and a sound statistical base.

The guidelines have been written in specification format with the intent that they will be reviewed, scrutinized, and modified so that they can eventually be included in the AASHTO provisions.

1.3 APPLICABILITY

Although the guidelines will be developed for a specific, narrow range of bridges, many of the principles involved are applicable to the evaluation of all types of bridges. Ideally, any type of bridge could be evaluated by the limit states approach. If evaluation procedures were developed so this could be done, progress would be made toward load limit evaluations based on equal probabilities of failure.

The guidelines were developed specifically for bridges in the United States. Current American bridge evaluation practices were considered in developing the methodology. Although the basic concepts can be universally applied, it would be prudent to consider the effects of local practices on the load limit values obtained by the methodology before applying these guidelines to bridges outside the United States.

SECTION 2

SYMBOLS AND DEFINITIONS

The following symbols and definitions apply to these Guidelines and Commentary:

A Random variable reflecting the uncertainties in transforming the nominal dead loads assigned to each member prior to carrying out the structural analysis, normally referred to as the "dead load distribution"; this variable is equal to 1.0 if the mean of all the assumptions equals the correct answer

D_i Nominal dead load effect of element i

I "Impact factor" used to approximate the dynamic effects of moving vehicles

L_j Nominal live load effects for load j other than the rating vehicle

L_R Nominal live load effects for the rating vehicle

m Total number of elements contributing dead load to the structure
n Total number of live loadings contributing to the live load effects other than the rating vehicle or vehicles
Q_k Effect of load k
R_n Nominal resistance
RF Rating factor
V_R Coefficient of variation of the resistance
V_D Coefficient of variation of the dead load
V_L Coefficient of variation of the live load
β Safety index
γ_{Di} Dead load factor for element i
γ_{Lj} Live load factor for load j other than the rating vehicle
γ_{LR} Live load factor for the rating vehicle
δ_D Bias coefficient to transform the calculated or nominal dead loads to mean dead loads
δ_L Bias coefficient to transform the calculated or nominal live loads to mean live loads
δ_R Bias coefficient to transform the calculated or nominal resistance to mean resistances
ϕ Capacity reduction factor to account for uncertainties in resistance due to variations in dimensions, material properties, and theory.

SECTION 3

LIMIT STATES EVALUATION

3.1 GENERAL

The evaluation of a structure is based on the simple principle that the available capacity of a structure to carry loads must exceed the capacity required to support the applied loadings. To perform an evaluation, therefore, it is necessary to know something about the available capacity, the applied loading, and the response of the structure to that loading. Knowledge and information with respect to each of these items is seldom complete; and, therefore, evaluation can seldom be done precisely.

To compensate for this lack of knowledge and information, engineers have used safety factors to ensure that failure does not occur. Within the United States, until very recently, safety factors in bridge evaluation were included in the allowable stresses specified by AASHTO. These allowable stresses have undergone an evolutionary process and have tended to assume values which, from experience, have resulted in load capacity evaluations that have a maximum probability of exceeding a limit state which is both socially and economically acceptable. This approach has the drawback that all uncertainties relating to available capacity, loadings, and structure response are accounted for by the allowable stress. This has resulted in different probabilities of failure for various types of structures.

AASHTO has also adopted an alternate load factor evaluation approach for bridges. In this approach, safety factors are included in the form of load factors which account for the uncertainties of loadings and structural response. In determining structural resistance the ultimate capacity is assumed. In the case of reinforced concrete, resistance factors are also used which reflect the uncertainties related to the material properties, section dimensions, and the assumptions of ultimate strength theory. The load factor evaluation is generally accepted as a more rational approach than the allowable stress method. (*B.1, B.2*).

3.2 DEFINITION OF LIMIT STATES EVALUATION

Limit states design and evaluation have become very popular in recent years. The trend in the development of new design evaluation codes and standards is toward this approach (*B.2, B.3*).

Within the framework of a limit states approach, structural failure is defined in terms of its impact on the users of the structure. For example, failure may be considered to occur when the safety of a user is placed in jeopardy. When a structure is loaded to the point that such a failure is imminent, it is said to have reached its *ultimate* or *safety limit state.* This type of failure is obviously very serious. On the other hand, failure may be defined as excessive deflections or vibrations. This type of failure does not affect safety, but can affect the way a structure performs in service. These less serious failures occur when a structure reaches what is called a *serviceability limit state.*

When presented in a load and resistance factor format, the limit states approach can be used to consider the relative seriousness of each type of structural failure because the load and resistance factors can be adjusted to yield different probabilities of failure. A serious failure can, therefore, be given a low probability of occurrence, while less serious failures can be assigned higher probabilities of occurrence. This approach is intended to prevent both excessive construction costs in new structures and overly restrictive load limits on existing structures.

The values for load effect and resistance are nominally determined by established methods. The relationship between these nominal values and the distribution of actual values for load effect and resistance is used along with probability theory to select load and resistance factors. Theoretically if load and resistance factors are considered separately, the probability of failure will be less dependent on the structure type.

The following section will illustrate in greater detail the development of the basic limit states rating equation used in the guidelines.

3.3 THE BASIC RATING EQUATION

The basic structural engineering equation states that the resistance of a structure must equal or exceed the demand placed on it by loads. Stated mathematically, $R \geq \sum_k Q_k$, where $R =$ resistance, and $Q_k =$ effect of load k.

The solution of this simple equation encompasses the whole art and science of structural engineering including the disciplines of strength of materials, structural analysis, and load determination. This equation applies to design as well as evaluation. In structural evaluation, the objective is to determine the maximum allowable live load. In the case of bridge evaluation, this usually means the maximum vehicle weight.

Any rational and tractable approach to the analytical solution of the basic structural engineering equation requires that the modes of failure be identified to establish the resistance. The location, types, and extent of the critical failure modes must be determined. The equation must be solved for each of these potential failure modes.

Since neither resistance nor the load effect can be established with certainty, safety factors must be introduced that give adequate assurance that the limit states are not exceeded. This may be done by stating the equation in a load and resistance factor format.

Separate load or resistance factors that will account for each of the major sources of uncertainty may be introduced to the equation. The basic rating equation used in the guidelines is simply a special form of the basic structural engineering equation with load and resistance factors introduced to account for uncertainties that apply to the bridge evaluation problem. It is written as follows:

$$RF = \frac{\phi R - \sum_{i=1}^{m} \gamma_{Di} D_i - \sum_{j=1}^{n} \gamma_{Lj} L_j (1+I)}{\gamma_{LR} L_R (1+I)} \qquad \text{(B-1)}$$

where

RF = rating factor (the portion of the rating vehicle allowed on the bridge);
ϕ = capacity reduction factor;
m = number of elements included in the dead load;
R_n = nominal resistance;
n = number of live loads other than the rating vehicle;
γ_{Di} = dead load factor for element i;
D_i = nominal dead load effect of element i;
γ_{Lj} = live load factor for live load j other than the rating vehicle(s);
L_j = nominal traffic live load effects for load j other than the rating vehicle(s);
γ_{LR} = live load factor for rating vehicle;
L_R = nominal live load effect for the rating vehicle; and
I = live load impact factor.

The concept of a rating factor RF is introduced into this equation for convenience. The rating factor is defined as the ratio between the maximum permitted traffic live load effect and the effect of the nominal rating vehicle. Thus, if the rating factor equals or exceeds unity, the bridge live load capacity is sufficient to support the rating vehicle. However, if the rating factor is less than unity, the rating vehicle may overload the bridge.

The maximum permitted traffic live load effect will be the total resistance minus the effect of loadings other than the rating vehicle. This will include dead loads, nonvehicular live loads, and, in the case of unsupervised permit loading, the vehicular live load and the impact of normal traffic that could mix with the rating vehicle. This may be written as follows:

Rating				Dead		Other
Vehicle	=	Capacity	−	Load	−	Live Load
Effects				Effects		Effects

The basic rating equation, as stated earlier, is in a more general format than the current AASHTO practice. In the current AASHTO specification, the live load effects are computed based on a wheel line distribution factor that considers implicitly more than one vehicle on the bridge. This is a special case in the proposed rating equation in which $L_j = 0$ and L_R is computed for the standard rating vehicles. The above rating equation may be particularly advantageous when rating for an overload special permit vehicle. In this case, the concurrent probability of two or more such overload vehicles is practically zero. Therefore, L_R is the load effect due to a single rating vehicle, and L_j is the load effect due to the more common trucks. However, a simple formula for evaluating such single vehicle load effect is not available. It is envisioned that in the ongoing NCHRP Project 12-26 "Wheel Load Distribution," simplified procedures will be developed to evaluate the maximum load effect in a bridge component due to a single vehicle.

The load and resistance factors included in the basic rating equation are selected to yield an acceptable reliability level in terms of safety index. Each of these factors accounts for certain aspects of the uncertainty in calculating normal load and resistance effects. They are discussed in more detail in Sections 4 and 5.

3.4 EVALUATION PROCESS

Each of the steps in the evaluation process may be performed in any one of several ways. Therefore, the guidelines are general enough to accommodate the practices of several different engineers or agencies. The load and resistance factors presented in the guidelines were developed on the principle that the accuracy of an evaluation was dependent, in part, on the methods used to perform the evaluation.

For economic reasons, it is desirable to keep the evaluation effort to a minimum. If the capacity of a bridge can be shown to be sufficient by making some approximations, there is no need to resort to an expensive evaluation procedure. On the other hand, if the sufficiency of a bridge cannot be reliably established using a more approximate method, then an engineer may wish to resort to a more sophisticated approach in order to demonstrate the sufficiency of the bridge. Therefore, the evaluation process outlined in the guidelines is a cyclic process in which one or several of the steps may be repeated.

The evaluation process may also vary depending on how the results of the evaluation will be used. Often this may mean that one or more of the steps in the process will be repeated. In the case where the results of evaluation will be used to issue an overload permit, for example, the selection of a rating vehicle, parts of the analysis, and selection of load factors, will vary. At least those steps that are related to these items must be repeated.

SECTION 4

RESISTANCE

4.1 GENERAL

The determination of structural resistance is one of the primary tasks in the evaluation process. In a limit states approach it is necessary to define the limit states at which resistance will be determined. These limit states should provide for similar structural performance regardless of the material or structure type.

4.2 RESISTANCE—SAFETY LIMIT STATES

Safety limit states are those states corresponding to the maximum load-carrying capacity of a structure or component. These limit states should have a very low probability of occurrence because they can lead to loss of life as well as to major financial losses. They include:

1. Loss of equilibrium of all or part of the structure considered as a rigid body (e.g., overturning, sliding, uplift).
2. Loss of load-bearing capacity of members due to insufficient material strength, buckling, fatigue, fire, corrosion, or deterioration.
3. Overall instability of the structure (e.g., P-delta effect, wind flutter, seismic motions).
4. Very large deformation (e.g., transformation into a mechanism).

In the case of reinforced concrete structures subjected to traffic live loads, the safety limit state is assumed to occur when an individual component, such as a girder, reaches its ultimate capacity and forms a plastic hinge. In most cases, this state does not present a serious threat to safety. The actual threat to safety occurs when enough plastic hinges are formed within the structure to result in a collapse mechanism. Many studies have shown that this will normally occur at a loading significantly above the load at which the first plastic hinge was formed. This is because all reinforced concrete structures have a high level of structural redundancy. Possible exception to this may be a simply-supported span with seriously deteriorated support conditions and resulting in an unstable condition. Therefore, what is currently defined as the safety limit state would in most cases be more appropriately called a "severe damage" limit state.

Determination of the true safety limit state involves very complicated and difficult analytical procedures. In most cases the use of these procedures for routine evaluation of bridges is not economically feasible. Until simplified methods are developed, the ultimate member capacity may be used as a lower bound of the true safety limit state. This may be either the ultimate capacity in shear or in flexure.

Different methods for considering the observable effects of deterioration were studied in developing the guidelines. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths. An alternate, but less reliable approach is to calculate resistance based on plan dimensions and use a smaller capacity reduction factor.

The capacity reduction factors rely in part on the control of future deterioration that can be obtained from various levels of inspection and maintenance. The development of these factors is discussed further in Section 4.4. More discussion of the effects of deterioration is included in Section 6. A field inspection procedure which is more comprehensive than the current practice is included in Appendix D for bridges with serious deterioration.

4.3 RESISTANCE—SERVICEABILITY LIMIT STATE

Serviceability limit states either restrict the normal use of the bridge or affect its durability. They include:

1. Excessive deflection or rotation affecting the use or appearance of the structure or nonstructural components.
2. Excessive local damage (e.g., cracking, splitting, spalling, local yielding, slip of connection) affecting the use, durability, or appearance of the structure.
3. Excessive undesirable vibrations.

The most important serviceability limit states in bridge evaluation are those that tend to affect the durability of the structure and shorten its useful life. Two types of serviceability failures are considered critical for reinforced concrete.

One of these critical serviceability failures is fatigue in the reinforcing steel. This will occur when a large number of repetitive live loads result in a large number of stress reversals in the steel. The critical number of load repetitions is only likely to occur due to normal traffic. Therefore, a restriction of permit vehicles because of fatigue serviceability criteria is usually not warranted. In addition, the restriction of normal traffic because of any serviceability criteria is not necessary because such a load restriction in itself restricts the normal use (i.e., serviceability) of the bridge. Therefore, load limit posting, in this case would, in effect, preserve one type of serviceability at the expense of another. Inasmuch as evaluation of the serviceability limit state for fatigue is not used to restrict live loadings, its primary function is to alert the Engineer to a potential problem that will warrant more frequent field inspections or correction.

Crack control is the other critical serviceability limit state that is considered in evaluating existing reinforced concrete bridges. The effect that crack width has on the rate of deterioration of structures exposed to severe environments is still unknown. However, there is some concern that excessive crack width can cause an increase in the rate of deterioration, although several other factors not associated with the level of live loading also play a role.

Crack control has been a problem in bridges using higher strength reinforcement designed by load factor methods. The increased levels of steel strain can cause excessive cracks to form. The crack control criteria used in the guidelines are derived from the AASHTO design criteria.

Serviceability criteria for crack control should not be used to restrict normal traffic, but they should be used when issuing overload permits. A probabilistic approach was not used in developing the AASHTO serviceability criteria, although the format of serviceability evaluation lends itself to future calibration based on probabilistic techniques. Future research in this area should be directed toward further investigation of the true effect of serviceability criteria, such as the effect of crack width on the durability of the bridge and on calibration of serviceability evaluation to acceptable levels of reliability.

4.4 CAPACITY REDUCTION FACTOR

A capacity reduction factor, ϕ, is included in the basic rating equation to account for variation in the calculated resistance. It takes into consideration the dimensional variations of the structure, differences in material properties, current condition and future deterioration, and the inaccuracies in the theory for calculating resistance. Because different maintenance and inspection practices provide different levels of control over future deterioration, these practices are considered in selecting the capacity reduction factor.

The capacity reduction factors (or resistance factors) vary depending on the degree of deterioration in the superstructure. For bridge structural components in good or fair condition, the following resistance statistics are generally accepted: $V_R = 10$ percent and $\delta_R = 1.1$.

As the degree of deterioration increases, the resistance may become more uncertain, i.e., larger coefficient of variation. In addition, the computed nominal resistance may become less conservative, i.e., smaller bias coefficient. In the lack of documented data, V_R and δ_R for deteriorated structures may be estimated based on engineering judgments.

For the deteriorated structures, if the resistances were evaluated based on the "as-built" plans with no consideration given to the current conditions, the bias coefficients were set at 1.0 and 0.9 for moderately deteriorated and seriously deteriorated structures, respectively.

Adjustments were also made to the coefficient of variation, V_R, to account for the degree of deterioration, inspection frequency and preventive maintenance. By conducting more frequent inspections and/or preventive maintenance measures, relevant data may be gathered with regard to the type of deterioration and the locality of deterioration. Based on these considerations, the coefficients of variation are assigned according to Table B-1.

Based on these resistance statistics, probability calibrations were carried out for a target safety index of 2.8. Assuming load factors remain unchanged, the capacity reduction factors were determined according to Table B-2. The capacity reduction factors vary from 0.94 to 0.54 for the worst case where the structure is seriously deteriorated, rating is based on the "as-built" plans, inspection is conducted only biennially, and no preventive maintenance has been done.

These capacity reduction factors apply to the flexural actions only.

Table B-1. Coefficients of variation, V_R, for deteriorated structures.

Inspection Frequency	Frequent		Normal	
Preventive Maintenance	Yes	No	Yes	No
Moderately Deteriorated	10%	15%	15%	20%
Seriously Deteriorated	15%	20%	20%	25%

SECTION 5

LOADS

5.1 GENERAL

Loads consist of concentrated or distributed forces that are applied directly to the bridge or result from deformations or the constraint of deformations. For bridge evaluation the most important loads are dead load and vehicular live load plus its accompanying dynamic effects, because each of these loadings induces high superstructure stresses.

5.2 COMBINATION OF LOADS

Loadings other than dead load and traffic live load usually do not result in significant bending or shear in the superstructure. Because the critical mode of failure for traffic live load almost always occurs in the superstructure, other types of loads will seldom affect the live load capacity of the bridge. When other combinations of loads can affect the capacity of the bridge, such as when substructure components can fail because of traffic live loading, the AASHTO load factors for design will be used. Reduction of live load intensity according to Sec. 1.2.9 of the AASHTO *Standard Specifications for Highway Bridges* will be used in these cases.

Table B-2. Resistance categories and capacity reduction factors.

Superstructure Condition (r)	Rating based on Plans	Field Measurement	Inspection Frequency Frequent	Normal (2 yrs.)	Preventive Maintenance Yes	No	Resistance V_R	δ_R	ϕ
Good or Fair r ≥ 7	--	--	--	--	--	--	10%	1.1	0.94
Deteriorated (r=4,5,6)		X	X		X		10%	1.1	0.94
		X	X			X	15%	1.1	0.84
		X		X	X		15%	1.1	0.84
		X		X		X	20%	1.1	0.76
	X		X		X		10%	1.0	0.84
	X		X			X	15%	1.0	0.76
	X			X	X		15%	1.0	0.76
	X			X		X	20%	1.0	0.67
Seriously Deteriorated to Potentially Hazardous (r = 3)		X	X		X		15%	1.1	0.84
		X	X			X	20%	1.1	0.76
		X		X	X		20%	1.1	0.76
		X		X		X	25%	1.1	0.67
	X		X		X		15%	0.9	0.67
	X		X			X	20%	0.9	0.60
	X			X	X		20%	0.9	0.60
	X			X		X	25%	0.9	0.54

5.3 DEAD LOADS

Dead load can usually be determined more accurately than any other type of loading. One major source of error is failure to consider some of the elements that will contribute to dead load. Some items that are often overlooked are wearing surfaces, parapets and railings, utilities, light standards and signs, and structure modifications not shown on plans. Other items that can affect the calculation of dead load are dimensional variations in the concrete section and variations in the unit weight of material.

5.4 DEAD LOAD FACTORS

Dead load factors are used to account for variations in dimensions, unit weights, and methods of calculating dead load effect. The variation in the dead load of different components will depend on the accuracy with which the components can be manufactured or measured. Factory-produced girders cast-in-steel forms obviously have less variation than an asphalt overlay placed on the bridge deck.

Three categories of dead load intensity were considered: structural section D_1, parapets and railings D_2, and asphalt overlay material D_3. The total dead load effect is defined $DL = C_D (A_1D_1 + A_2D_2 + A_3D_3)$, where C_D is a deterministic constant converting load intensities (weight per unit length) to load effect (moment or shear), and A_i is the analysis uncertainty variable associated with D_i.

In the reliability analysis, the coefficients of variation and the bias coefficients were assigned to D_i and A_i according to Table B-3.

A dead load factor of 1.2 was chosen for the total dead load effect. For reinforced concrete bridges, the self-weight of structural section accounts for more than 70 percent of the total dead load. For this type of bridge, it is not essential to assign different dead load factors for different dead load categories. However, for future enhancement of the guidelines to include other types of bridges, three dead load factors, as shown in Table B-4, were assigned to account for the different uncertainty levels.

Table B-4. Dead load factors.

Dead Load Category	γ_D
Structural Section	1.20
Factory Fabricated Components	1.05
A/C Overlay	1.40

5.5 LIVE LOAD

Highway vehicles come in a wide variety of sizes and configurations. No single vehicle can accurately reflect the effects of all of these vehicles. Because it is necessary to limit the number of vehicle configurations to a manageable level to keep the evaluation process from becoming too cumbersome, the effect of the actual traffic live loads will vary from predicted values. This variation will usually be greater than the variation in dead load effect. To minimize this difference, it is necessary to select a rating vehicle with axle spacings and relative axle weights similar to actual vehicles. Because overload vehicles typically have very different axle configurations, it is very important that this be considered when issuing permits.

The guidelines specify the number of vehicles to be considered on the bridge at any one time. These numbers are based on an estimate of the maximum likely number of vehicles under typical traffic situations. When unusual conditions exist, adjustments to the specified number of vehicles should be made.

Judgment must also be exercised with regard to sidewalk loadings. The likelihood of the maximum truck loading occurring at the same time as the maximum sidewalk loading is small. A unit loading for the sidewalk for the purposes of load limit evaluation will generally be less than the design unit loading.

Table B-3. Statistics of dead loads.

	Coefficient of Variation	Bias Coefficient
Analysis Variable (A_i)	5%	1.0
Structural Section (D_1)	5%	1.05
Parapets and Railings (D_2)	3%	1.03
A/C Overlay (D_3)	20%	1.05

5.6 LIVE LOAD FACTORS AND IMPACT FACTORS

Live load factors are used in strength evaluations to account for variations in the maximum live load effect that is likely to occur during the rating period of the bridge. These factors must be established based on a probabilistic live load model which accounts for gross vehicle weight, multiple presence, and distribution of load effects.

Each of these factors may depend on the vehicle weight control enforcement and the truck traffic volume at the bridge site. If weight limits are strictly enforced, the variations in the maximum live load effect will be less. If the truck traffic at a site is heavy, higher variations may be expected. These considerations may be incorporated into the probabilistic calibration procedure by assigning appropriate coefficients of variation to the associated variables.

In the reliability analysis, the following live load model for flexure as suggested by Moses was used: $LL = a\, W H g\,(1+I)$, where LL = maximum live load moment in a bridge component, a = deterministic constant, W = characteristic vehicle weight (95 percentile value), H = multiple presence random variable, g = distribution random variable, and I = impact factor.

The distribution factors as defined in the current AASHTO specifications were used. The AASHTO empirical formula was compared with more accurate analytical results. The bias of the AASHTO "S-over" formula for T-beam bridges was estimated, according to Table B-5, as a function of girder spacing, S. Similarly for slab bridges, the bias coefficients of AASHTO distribution factor, i.e. $1/E$, were tabulated, according to Table B-6, as a function of span length, L. The coefficient of variation is assumed to be 5 percent.

Based on limited measurements of live load effects in highway bridges, Moses suggested tentative values for the mean and the coefficient of variation for H. As shown in Tables B-7 and B-8, these values vary with the truck traffic volume, the weight limit enforcement, and the span length. Also based on the field measurements, two characteristic vehicle configurations were identified as shown in Figure B-1 to represent single body trucks and semi-tractor trailers on the nation's highway bridges. The 95th percentile truck weights for different truck traffic volumes are given in Table B-9.

The dynamic (impact) effect $(1+I)$ was also considered as random variables. Based on field measurements of actual truck traffic conducted by Moses, it was observed that the impact is influenced more by the roadway roughness or bumps in approaches or expansion joints rather than the dynamic amplification (or resonance). The mean value of the impact effect $(1+I)$ was determined:

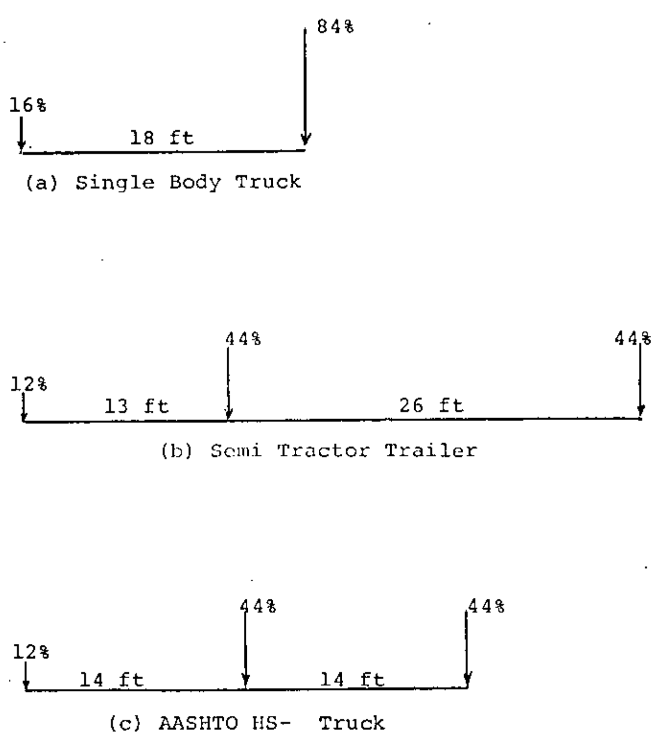

Figure B-1. Representative vehicle configurations.

Table B-5. Bias coefficients for wheel load distribution in "T" beam bridges.

S (ft)	4	5	6	7	8	9
g	1.166	1.076	1.076	1.027	0.979	0.949

Table B-6. Bias coefficients for wheel load distribution in slab bridges.

L (ft)	15	25	40	50	60
Two Lane	0.946	1.062	1.236	1.352	1.352
Four Lane	1.008	1.132	1.317	1.441	1.441

Table B-7. Mean multiple presence factor.

Traffic Volume \ Span Length	40 ft	$\bar{H}$ 40 ft-100 ft	>100 ft
Light (200 T/day)	2.2	2.42	2.64
Moderate (2,000 T/day)	2.4	2.64	2.88
Heavy (10,000 T/day)	2.5	2.75	3.00

Table B-8. Coefficients of variation for the multiple presence factor.

Weight Control \ Span Length	40 ft	V_H 40 ft-100 ft	>100 ft
Enforced	0.20	0.17	0.15
Unenforced	0.23	0.20	0.18

Table B-9. Characteristic truck weight (95-percentile value).

Category	W kips Single	Semi
Light (200 T/day)	38	68
Moderate (2,000 T/day)	47	75
Heavy (10,000 T/day)	52	82

$$\text{for smooth roadway } \overline{(1+I)} = 1.10$$

$$\text{for rough roadway } \overline{(1+I)} = 1.20$$

The coefficient of variation for $(1+I)$ is 8 percent.

Based on these statistics, the live load factors for the distribution Level 2 were determined for different truck traffic volumes and weight limit controls, as given in Table A-7 in the Guidelines. For live load distribution determined by approaches of distribution Level 3 or Level 1, the live load factors were estimated by adjusting those corresponding to the distribution Level 2 by ± 10 percent. This is based on a study of 70 R.C.

T-beam bridges (the on-going NCHRP 12-26 Project, "Wheel Load Distribution in Highway Bridges"), which shows that the AASHTO wheel load distribution formula is, on the average, about 5 percent unconservative.

SECTION 6

DETERIORATION

6.1 GENERAL

Deterioration in a superstructure does not necessarily reduce its load-carrying capacity. For instance, a spall in the deck near the supports of a simple-span bridge or loss of section in a bottom layer of reinforcing steel near the supports will not necessarily affect the capacity of the span, because the deterioration will have occurred in an area which will be subjected to low stresses during loading. However, if similar deterioration occurs at the highly stressed, midspan section of a simple-span bridge, it will directly affect the loading capacity of the bridge.

It, therefore, follows that both the degree and location of deterioration must be considered when determining the effects of deterioration on a superstructure's load-carrying capacity.

The effects of deterioration on the performances of each of the different components of the structural system vary, primarily because of the different roles of the components in the system. For instance, deck slabs serve a dual function: (1) they distribute wheel loads to the girder system, and (2) they serve as a flange of the girder system in positive bending in transmitting these loads to the substructure. Thus, both of these functions should be examined independently when considering the effects of deterioration on the performance of deck slabs.

It has long been suspected that the AASHTO specifications for conventional deck design are far too conservative.

To obtain test data on deck slabs, Batchelor, Hewitt, and Csagoly tested a number of ⅛th scale models of an 80-ft prototype steel "I" beam, concrete deck bridge (*B.4*). Their conclusions included the following: (1): "...conventionally reinforced deck slabs have very high factors of safety against failure by punching and are wastefully reinforced," and (2) "...0.2 percent isotropic top and bottom reinforcement is recommended as the maximum reinforcement. This amounts to reduction of 66 percent of the current reinforcement requirements."

Csagoly, Holowka, and Dorton (*B.5*) loaded the deck slabs of 32 bridges with a point load (representing a dual-tired wheel) of 100 kip, 10 times larger than the allowable legal load. In the summary of their report, they state: "Even under this heavy overload, combined with the deterioration of some decks, no failure of restrained decks was ever observed."

Fullarton and Edmonds (*B.6*) tested a structure that was scheduled to be demolished by using two jacks reacting to beams supported under the superstructure. They found that "the maximum load of six times an HS wheel load, which was limited by the capacity of the reaction frame, caused no significant distress to the deck slab."

There have been many other deck-loading projects with similar results.

Many deteriorated decks in actual service have functioned without failure, some with fully exposed reinforcing steel. Serious problems have only developed when there was a combination of very little concrete section left and badly corroded reinforcing steel.

With the proven large capacity for wheel loads inherent in a standard deck, it is unrealistic to adopt a rating factor for it as a load carrier to the girder system. It would have to suffer a very high degree of deterioration before warranting penalization. Therefore, no deterioration effect will be considered in the guidelines for the deck acting as a distributor of loads to the girder system, although very deep hazardous pot holes could seriously affect the load-carrying capacity of bridge deck slabs.

Because deterioration in the deck does have a negative effect on the girder's capacity, deterioration must be considered for a deck slab's functioning as a girder flange. However, isolated, noncontinuous potholes or other cosmetic surface deteriorations have negligible effect on resistance.

6.2 MOMENT CAPACITY

Generally, it is the deterioration within the cross-sectional (two-dimensional) area of the system that affects the load-carrying capacity of a girder rather than in the overall (three-dimensional) volume. One exception to this occurs when there is total loss of both cover and bond of reinforcing steel over a significant length of the bar. The significance of this exception is demonstrated in work by Minkarah and Ringo (B.7). They tested 40 reduced-scale (5in. by 10in. by 9.5ft) concrete beams under various cover and bond variables. They found that when the tensile reinforcing steel was devoid of cover and bond for 32 percent of its length within the maximum positive moment zone of the beam, the beam had the same coefficient of strength, *CS*, as the control beam with no loss of cover. (The coefficient of strength is the ratio of the maximum experimental load on the test beam to the load capacity of the control beam.) With 42 percent of the bar devoid of cover and bond, the *CS* was 0.930. Even with 63 percent devoid, the average *CS* of the two beams tested was 0.831. In other words, even when the cover and bond on the reinforcing steel was absent for as much as 63 percent of the span length, only 17 percent of the beam's strength was lost.

In actual practice, it is rare to find total cover and bond loss occurring over more than 30 percent of the span length of a reinforcing bar in tension. Since this corresponds to a *CS* of 1.0, the deterioration of cover and bond loss on the steel should be considered insignificant unless it affects the anchorage near the ends of individual reinforcing bars.

A loss of concrete (spall) in a compression area results in a redistribution of the stresses in that area. This action causes some stress concentrations at section changes, which is undesirable. Once the stresses are redistributed, they maintain a fairly uniform pattern until another change in section is reached. Consequently, the continuous section resulting from a long, longitudinal spall can be less detrimental than the many changes in section resulting from several smaller spalls.

Undersurface fractures are considered spalls. At the time a survey is made, the concrete above the fracture plane is undoubtedly carrying some compressive forces. From experience, however, it is known that probably within one year the area will develop from an incipient spall to a full spall. Consequently, it is best to treat it as a full spall which is its anticipated near-term worst condition.

Engineering experience must be relied on to determine the future size of an incipient spall. When "sounding" incipient spalls formed by an undersurface fracture, there is no way of knowing if the fracture plane is approaching the surface, is some place between the surface and top steel, or is still on its typical undulating pattern between the tops of parallel reinforcing bars. Until methods are found to determine the path, it should be assumed for the deterioration calculation that the path is along the top of the top reinforcing steel mat and is directed upward on a 45 degree angle to the deck surface at the edge of the detectable fracture. The perimeter of the plotted area of an undersurface fracture should therefore be widened by two times the thickness of the cover over the reinforcing steel to estimate the width of the area expected to be lost because of deterioration. Assume the thickness of the spalled area to be the same as the cover over the steel.

Theoretically, the deterioration effect on the concrete stress is different when the deterioration is in compressive concrete rather than in steel under tension. When steel loses cross section, the moment arm between it and the force resultant of the compressive stresses in the concrete changes very little. On the other hand, when concrete spalls, the resultant compressive force moves towards the steel, which can in some cases cause a significant change in the stresses, especially in the concrete. The shortened moment arm results in a higher compressive force acting on a smaller area. When the spalled area is narrow (with respect to the width of the deck), this increased stress is insignificant because of both the low ratio of the spalled area to the nonspalled area in the cross section and the natural readjustment of stresses that takes place during high stress loading. As the ratio of spalled to nonspalled areas increases and there is less unaffected area for stress readjustment, concrete strength may affect the capacity of the section. This will be taken into account in the calculations by limiting the effective reinforcement to that available in the section or 75 percent of that required for balanced condition (*B.8*).

To simplify calculations, the loss of the concrete compression section is compensated for by reducing the depth of the slab an amount equivalent to the area lost due to deterioration.

Except for shear-type cracks in a high shear region of the girder and cracks along lines of reinforcing that can affect bond, concrete cracking is not believed to have a strong effect on load capacity.

Several studies on actual steel loss due to deterioration caused by salt have shown that the "typical" corroded bar has lost less than 3 percent of its cross-sectional area. Unless hard data, in the form of actual measurements, can be collected, it should be assumed that where there is an indication of steel corrosion the steel involved has lost 3 percent of its section. It should also be assumed that "indication" includes either large cracks in, or rust stains on, the concrete covering the reinforcing steel.

Tables B-10, B-11, and B-12 illustrate the effects the loss of concrete compressive area and steel tensile area have on the ultimate moment capacity of a beam. Calculations in the tables are based on the T-beam section shown in Figure B-2.

Table B-10 shows the percent reduction in moment capacity for each ½ in. reduction in slab depth. Note that if there is no steel loss, the deck can suffer a 29 percent loss in cross-section area due to deterioration with only a 5.46 percent reduction in moment capacity. Generally, with deterioration caused by deicing salt, there is no bottom girder steel loss in the midspan area. Steel corrosion in this area is, however, common in a marine environment.

Because of its insignificant influence on the moment capacity of a girder, deterioration occurring in a low stress zone should not be considered when rating a structure.

Deterioration in a high stress area, on the other hand, greatly influences the moment capacity of a girder.

Although Bakht and Csagoly's report (*B.9*) did not state the degree of deterioration on the field load testing they did on some 32 bridges, they stated that "the testing program, besides helping to understand the structural behavior of bridges, has saved several bridges which otherwise were condemned by conventional theoretical standards."

Table B-10. Loss of concrete compressive area (see Fig. B-2 for section properties).

Slab Lost Depth (in)	% Deterioration	Full Section Moment Cap.	Reduced Sect. Moment Cap.	% Cap Reduction
1/2	4.77	2946	2921	0.85
1	9.67	2946	2895	1.73
1-1/2	14.55	2946	2868	2.65
2	19.45	2946	2841	3.56
2-1/2	24.22	2946	2814	4.48
3	29.11	2946	2785	5.46

Table B-11. Loss of steel tensile area (see Fig. B-2 for section properties).

Number of Bars Lost	% Deterioration	Full Area Mom. Cap.	Reduced Area Mom. Cap.	% Cap Reduction
1/2	5	2946	2789	5.33
1	10	2946	2632	10.66
1-1/2	15	2946	2475	15.99
2	20	2946	2317	21.35

Table B-12. Loss of concrete compressive area plus loss of steel tensile area.

Lost (in)	No. of Bars Lost	% Deterioration	Full Section Mom. Cap.	Reduced Section Mom. Cap.	% Cap Reduction
1/2	1/2	9.77	2946	2765	6.14
1	1	19.67	2946	2587	12.19
1-1/2	1-1/2	29.55	2946	2411	18.16
2	2	39.45	2946	2237	24.07

In reporting the results from field tests on five short-span bridges, Kissane, Beal, and Sanford (*B.10*) noted the following: "Findings include: 1) induced strain and deflections are much lower than expected from analytical methods, 2) concrete deterioration does not cause a noticeable change in measured structural response. . ."

Beal (*B.11*) reported on tests conducted by the State of New York on a reinforced concrete T-beam bridge that was considered in poor condition. An inspection conducted just prior to the tests indicated that the bridge was assigned a condition rating of 2–3 for its primary members on a scale from 1 (potentially

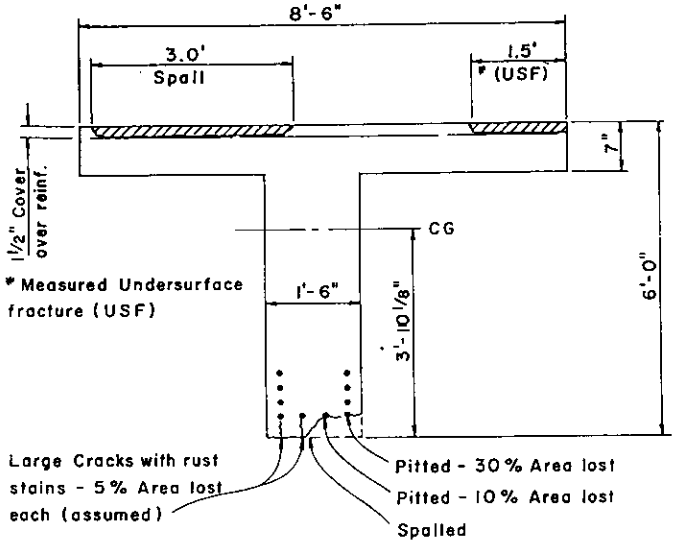

Figure B-2. Typical T-beam cross section.

hazardous) to 7 (new condition). Tension steel in the girder stems was exposed and there was extensive longitudinal cracking on the vertical faces of all the girder stems. Cores taken in the deck indicated that the deck was highly fractured throughout. The borings indicated that a 4-in. concrete wearing surface was substantially disintegrated. Exposed rusted rebars in the girder stems were estimated to have a 1 to 2 percent loss in cross-sectional area. Beal concluded that there was no evidence to suggest that the section was weakened either by apparent concrete deterioration, the loss of rebar cross-section, or the loss of rebar cover.

By close exploration and evaluation, an engineer should be able to determine the percent deterioration in a given cross section with a high degree of accuracy. If in error, the inaccuracy will probably be off on the high side because the tendency of investigators is to be conservative in their measurements.

From the available information, it appears that more than an adequate margin of safety will be provided by using the moment capacity as calculated from the field data for evaluating deterioration.

6.3 SHEAR CAPACITY

Normally, the shear capacity of a structure is not adversely affected by deterioration, but, when hard evidence is present to indicate that it is, an evaluation of the remaining capacity of the girder is difficult to determine. For hard evidence to indicate that the shear capacity of a structure is adversely affected by deterioration, one or more of the following symptoms must occur in the high shear area of the girder: wide diagonal cracks, rust stains, vertical displacement across a crack(s), and concrete spalling along a crack.

The recommended reductions in normal shear capacity shown in Section 6.3 of the guidelines for each of these symptoms were estimated by using sound engineering judgment.

SECTION 7

REFERENCES

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B.2 MACGREGOR, J. G., "Safety and Limit States Design for Reinforced Concrete." *Canadian J. Civ. Eng.* Vol. 3 (1976) pp. 484–513.

B.3 CANADIAN STANDARDS ASSOCIATION, "Design of Highway Bridges Supplement No. 1-1980." *CSA Standard Can3-S6-M78* (Apr. 1980).

B.4 BATCHELOR, BARRINGTON, ET AL., "Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges." *Proc., Transportation Research Record 664,* Vol. 1 (Sept. 1978) pp. 162–170.

B.5 CSAGOLY, P., HOLOWKA, M., and DORTON, R., "The True Behavior of Thin Concrete Bridge Slabs." *Proc., Transportation Research Record 664,* Vol. 1 (Sept. 1978) pp. 171–179.

B.6 FULLARTON, D. H. and EDMONDS, F. D., "Destructive Testing of the Mangateweka Stream Bridge." *Report No. 5-78/1,* New Zealand Ministry of Works and Development (Apr. 1978).

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B.8 AASHTO, *Manual for Maintenance of Bridges* (1983) pg. 38.

B.9 BAKHT, BAIDAR and CSAGOLY, P. F., "Bridge Testing." Ontario Ministry of Transportation and Communication (Aug. 1979).

B.10 KISSANE, R. J., SANFORD, J. A., and BEAL, D. B., "Load Rating of Short-Span Highway Bridges." *Research Project 156-1,* FHWA (May 1979).

B.11 BEAL, D. B., "Field Tests of Hannacroix Creek Bridge." Engineering Research and Development Bureau, New York State Department of Transportation, State Campus (Feb. 1982).

APPENDIX C

EXAMPLES

This appendix presents a selected group of examples illustrating the use of the proposed rating procedures. The following table identifies the characteristic features of the reinforced concrete T-beam and slab example problems treated in the remainder of this appendix.

Example	State	No. Spans	Year Built	Span Length (ft)
		T-Beam		
1	AR	3	1963	55, 31
2	CA	1	1925	26
3	NY	1	1928	25
4	NY	1	1928	30
		Slab		
5	AR	3	1957	35
6	PA	1	1926	19
7	PA	1	1941	8
8	OH	1	1930	25

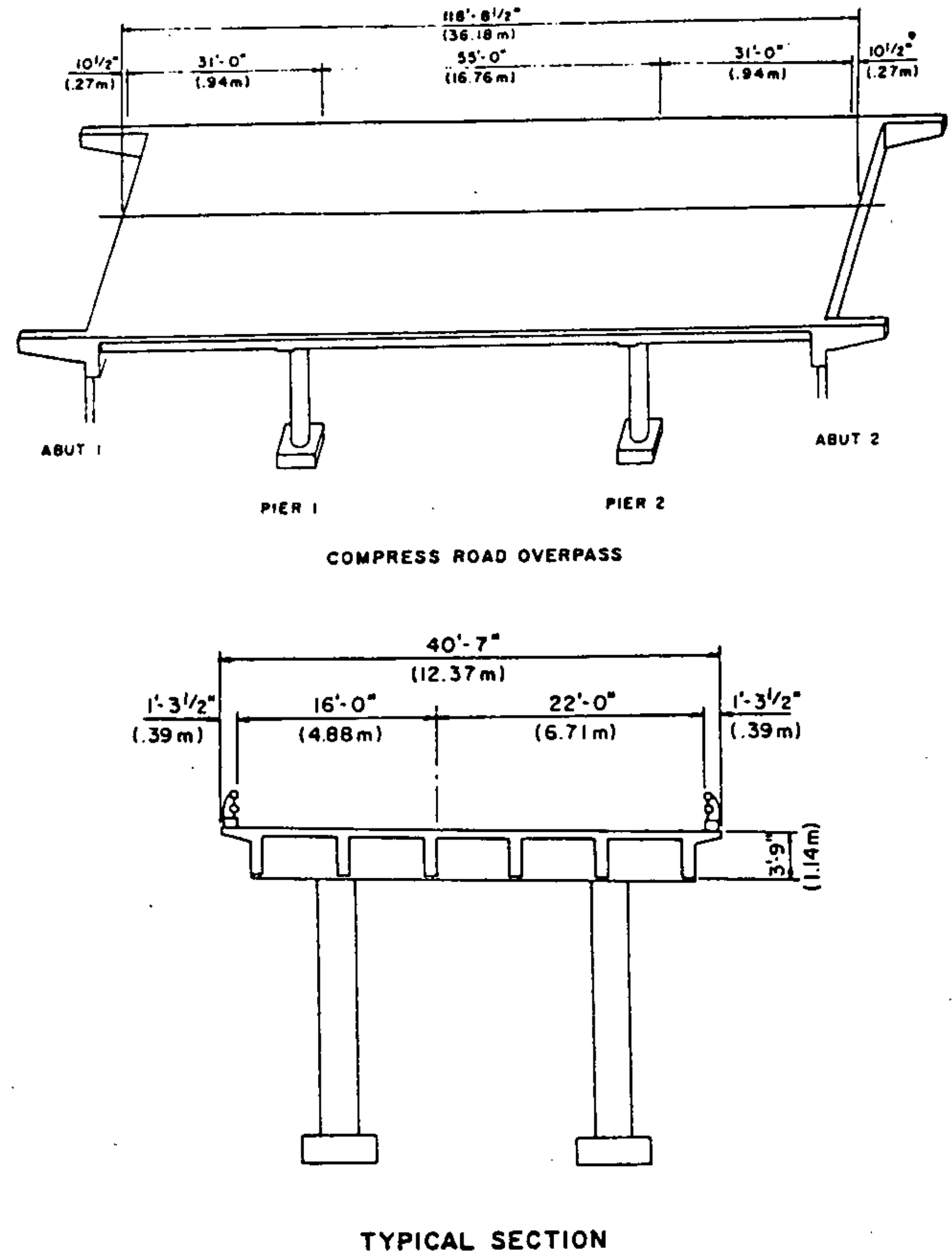
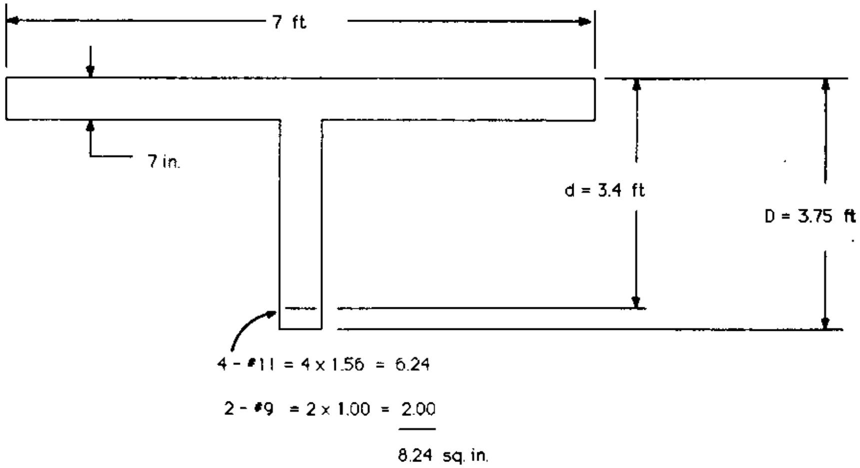


Figure C-1. Example 1—Casa Grande Highway Bridge on Route I-8 over 280, Yuma County, Arizona.

Span 2, 0.5 point-positive



f'_c = 3,000 psi

f_y = 40,000 psi

Resistance

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(8.24)(40)}{(0.85)(3)(84)} = 1.54 \text{ in.} = 0.13 \text{ ft.}$$

$$M_u = [A_s f_y (d - \frac{a}{2})] = [(8.24)(40)(3.4 - \frac{0.13}{2})]$$

$$M_u = 1{,}100 \text{ k-ft}$$

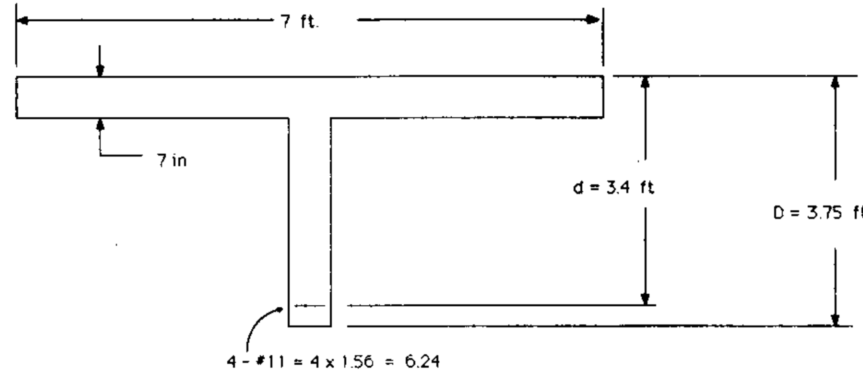
Dead Load

183 k-ft (BDS)

Live Load (HS20)

AASHTO
297 k-ft (BDS)
Proposed Rating Procedure
256 k-f (BDS) (I+1) = 1.1
(Assume smooth surface)

Span 3, 0.6 point-positive



Resistance

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(6.24)(40)}{(0.85)(3)(84)} = 1.17 \text{ in.} = 0.10 \text{ ft.}$$

$$M_u = [A_s f_y (d - \frac{a}{2})] = [(6.24)(40)(3.4 - \frac{0.10}{2})]$$

$$M_u = 836 \text{ k-ft}$$

Dead Load

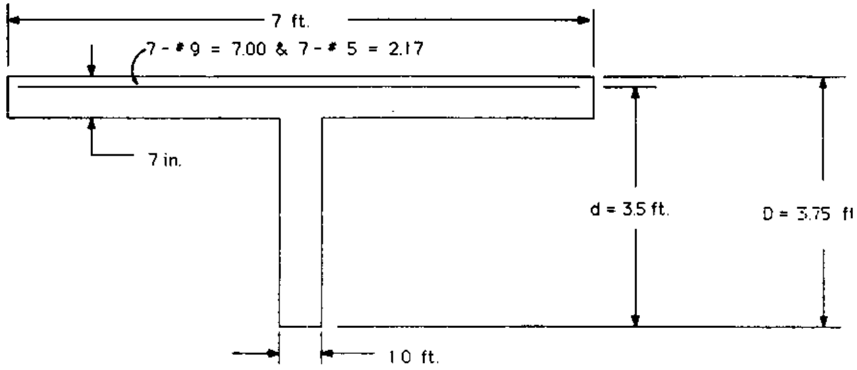
88 k-ft (BDS)

Live Load (HS20)

AASHTO
244 k-ft (BDS)
Proposed Rating Procedures
206. k-ft (BDS) (I+1) = 1.1

Figure C-1. Continued

Span 2, 1.0 point-negative



Resistance

$$a = \frac{A_s\ f_y}{0.85\ f'_c\ b} = \frac{(9.17)(40)}{(0.85)(3)(12)} = 11.99\text{ in.} = 1.00\text{ ft.}$$

$$M_u = [A_s\ f_y\ (d - \frac{a}{2})] = [(9.17)(40)(3.5 - \frac{1.00}{2})]$$

$$M_u = 1{,}100\text{ k-ft}$$

Dead Load

232 k-ft (BDS)

Live Load (HS20)

AASHTO
213 k-ft (BDS)
Proposed Rating Procedures
182 k-ft (BDS) (I+1) = 1.1

Span 2, 0.5 point-positive

$$RF = \frac{\phi\ M_u - \gamma_D\ M_{DL}}{\gamma_L\ M_{LL}}$$

$$= \frac{\phi\ 1100 - {}_D\ 183}{\gamma_L\ 256} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	0.99	1.82	0.54	0.80
	0.60	1.16		0.60	0.94
	0.67	1.37		0.67	1.11
	0.76	1.62		0.76	1.32
	0.84	1.86		0.84	1.51
	0.94	2.15		0.94	1.75
1.60	0.54	0.92	1.84	0.54	0.80
	0.60	1.08		0.60	0.94
	0.67	1.26		0.67	1.10
	0.76	1.51		0.76	1.31
	0.84	1.72		0.84	1.50
	0.94	1.98		0.94	1.73
1.73	0.54	0.85	1.93	0.54	0.75
	0.60	1.00		0.60	0.89
	0.67	1.17		0.67	1.04
	0.76	1.39		0.76	1.25
	0.84	1.59		0.84	1.43
	0.94	1.84		0.94	1.65

Inv. (AASHTO)	Oper. (AASHTO)
1.17	1.95

Figure C-1. Continued

Span 3, 0.6 point-positive

$$RF = \frac{\phi\, M_u - \gamma_D\, M_{DL}}{\gamma_L\, M_{LL}}$$

$$= \frac{\phi\, 836 - \gamma_D\, 88}{\gamma_L\, 206} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	1.14	1.82	0.54	0.92
	0.60	1.30		0.60	1.05
	0.67	1.49		0.67	1.21
	0.76	1.74		0.76	1.41
	0.84	1.95		0.84	1.59
	0.94	2.23		0.94	1.81
1.60	0.54	1.05	1.84	0.54	0.91
	0.60	1.20		0.60	1.04
	0.67	1.37		0.67	1.20
	0.76	1.61		0.76	1.40
	0.84	1.81		0.84	1.58
	0.94	2.06		0.94	1.80
1.73	0.54	0.97	1.93	0.54	0.86
	0.60	1.11		0.60	0.99
	0.67	1.28		0.67	1.15
	0.76	1.48		0.76	1.33
	0.84	1.67		0.84	1.50
	0.94	1.91		0.94	1.71

Inv. (AASHTO)	Oper. (AASHTO)
1.21	2.01

Span 2, 1.0 point-negative

$$RF = \frac{\phi\, M_u - \gamma_D\, M_{DL}}{\gamma_L\, M_{LL}}$$

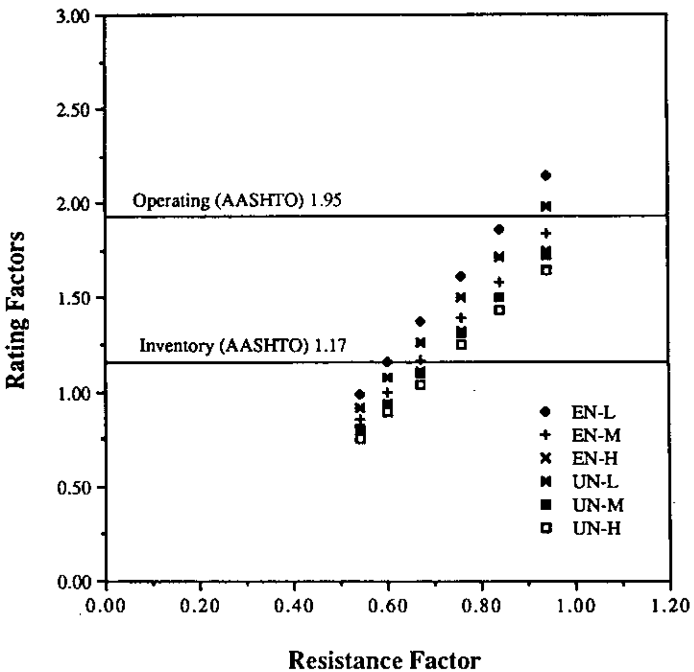
$$= \frac{\phi\, 1100 - \gamma_D\, 232}{\gamma_L\, 182} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	1.17	1.82	0.54	0.95
	0.60	1.42		0.60	1.15
	0.67	1.70		0.67	1.38
	0.76	2.07		0.76	1.69
	0.84	2.40		0.84	1.95
	0.94	2.81		0.94	2.28
1.60	0.54	1.09	1.84	0.54	0.95
	0.60	1.31		0.60	1.14
	0.67	1.58		0.67	1.37
	0.76	1.92		0.76	1.66
	0.84	2.21		0.84	1.93
	0.94	2.60		0.94	2.26
1.73	0.54	1.01	1.93	0.54	0.90
	0.60	1.22		0.60	1.09
	0.67	1.45		0.67	1.31
	0.76	1.77		0.76	1.59
	0.84	2.05		0.84	1.84
	0.94	2.40		0.94	2.15

Inv. (AASHTO)	Oper. (AASHTO)
1.49	2.49

Figure C-1. Continued



NOTE: EN-L (Enforced-Light) UN-L (Unenforced-Light)
EN-M (Enforced-Moderate) UN-M (Unenforced-Moderate)
EN-H (Enforced-Heavy) UN-H (Unenforced-Heavy)

Figure C-2. Example 1—proposed rating factors for span 2, 0.5 point-positive moment.

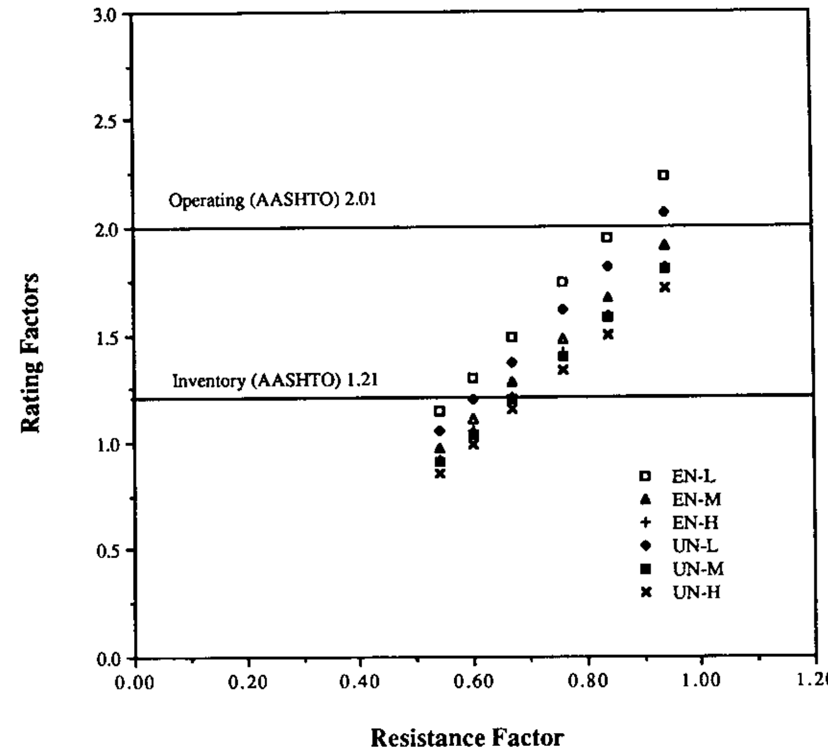


Figure C-3. Example 1—proposed rating factors for span 3, 0.6 point-positive moment.

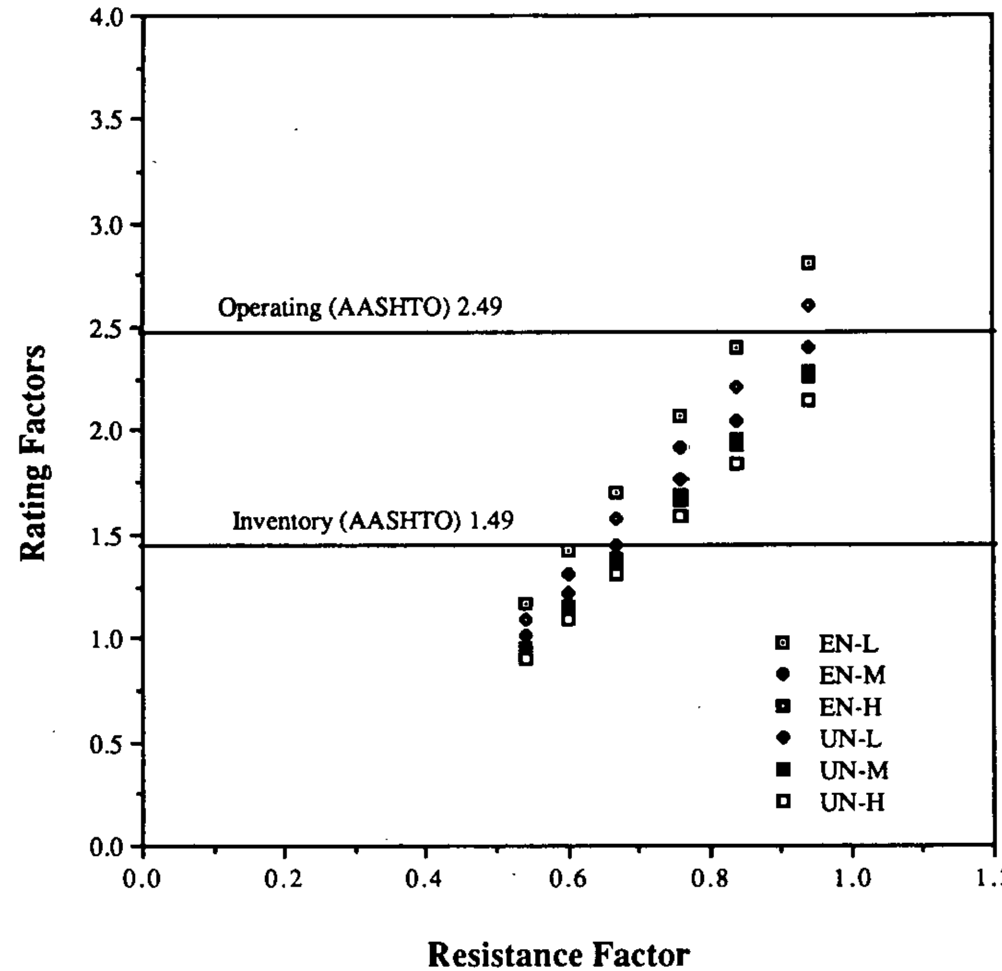


Figure C-4. Example 1—proposed rating factors for negative moment.

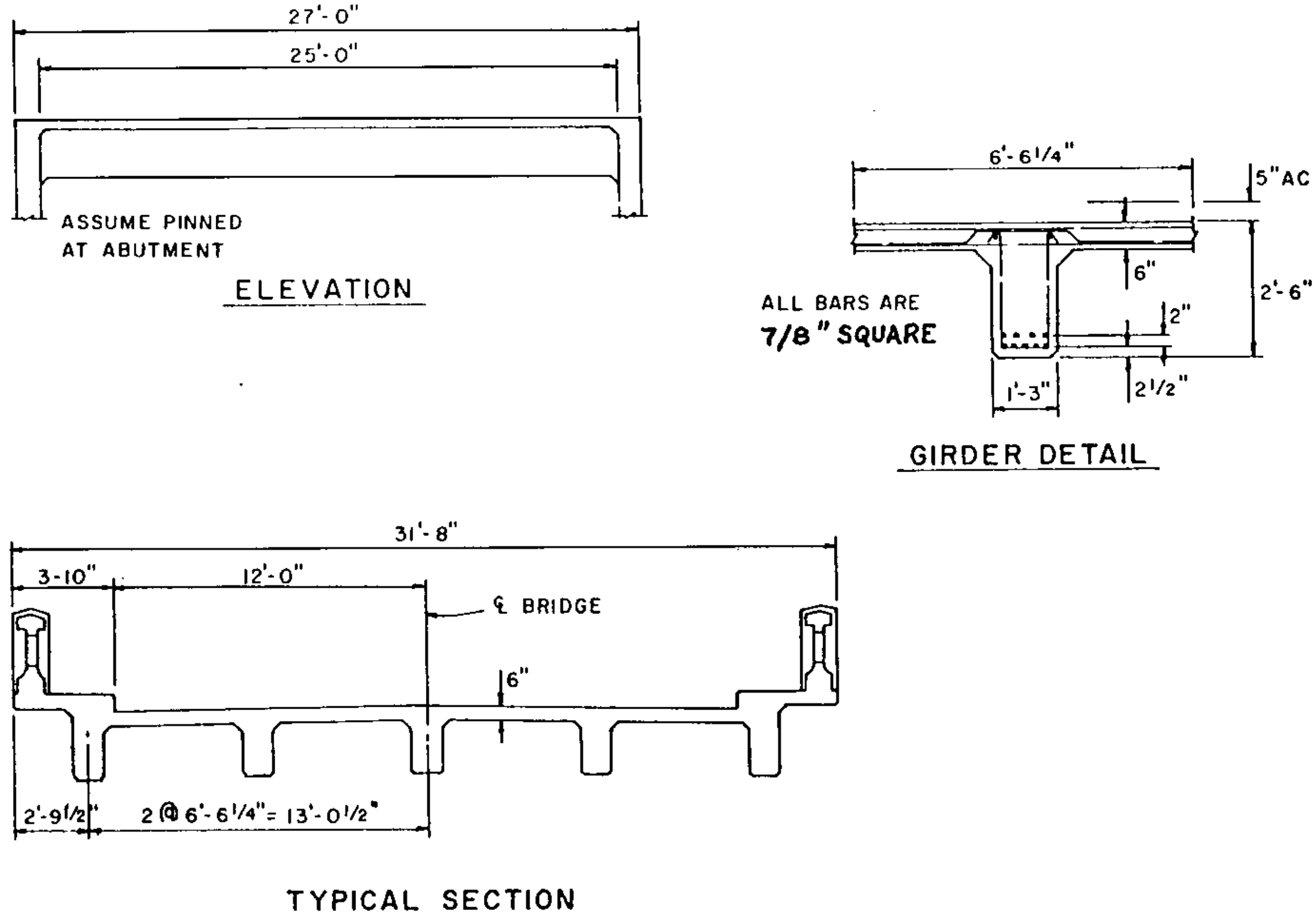


Figure C-5. Example 2—Woodbridge irrigation canal, California.

Resistance

Year Built-1925

$$M_u = A_s f_y (d - \frac{a}{2})$$

$$= (6.89)(33)(2.22 - 0.5 \times 0.095)$$

$$= 494.0 \text{ k-ft}$$

f'_c = 3,000 psi

f_y = 33,000 psi (reinforcing steel unknown)

Dead Load

		W(k/ft)	M_{DL} (k-ft)
Struc. Conc. = 0.150[0.5x6.52x2x1.25]	=	0.86	$\frac{(0.86)(26)^2}{8} = 72.7$
Rail	=	0.22	$\frac{(0.22)(26)^2}{8} = 18.6$
AC Overlay = (0.144)(0.42)(6.52)	=	0.39	$\frac{(0.39)(26)^2}{8} = 33.0$
			124.3

Live Loads

AASHTO

App. A - AASHTO HS20

$$M_{LL} = \frac{(222.2)}{2}(1.09)(1.30)$$

$$= 157.4$$

$$g = \frac{S}{6} = \frac{6.52}{6} = 1.09$$

$$I+1 = 1 + \frac{50}{26+125} = 1.33$$

$$\leq = 1.30$$

Proposed Rating Procedures

$$M_{LL} = \frac{(222.2)}{2}(1.09)(1.1)$$

$$= 133.2$$

Proposed Rating Procedures

(Assume smooth surface)
I+1 = 1.1

Figure C-5. Continued

Span 1, 0.5 point

$$RF = \frac{\phi M_u - \gamma_D M_{DL}}{\gamma_L M_{LL}}$$

$$= \frac{\phi(494) - \gamma_D (124.3)}{\gamma_L (133.2)} \qquad \gamma_D = 1.2$$

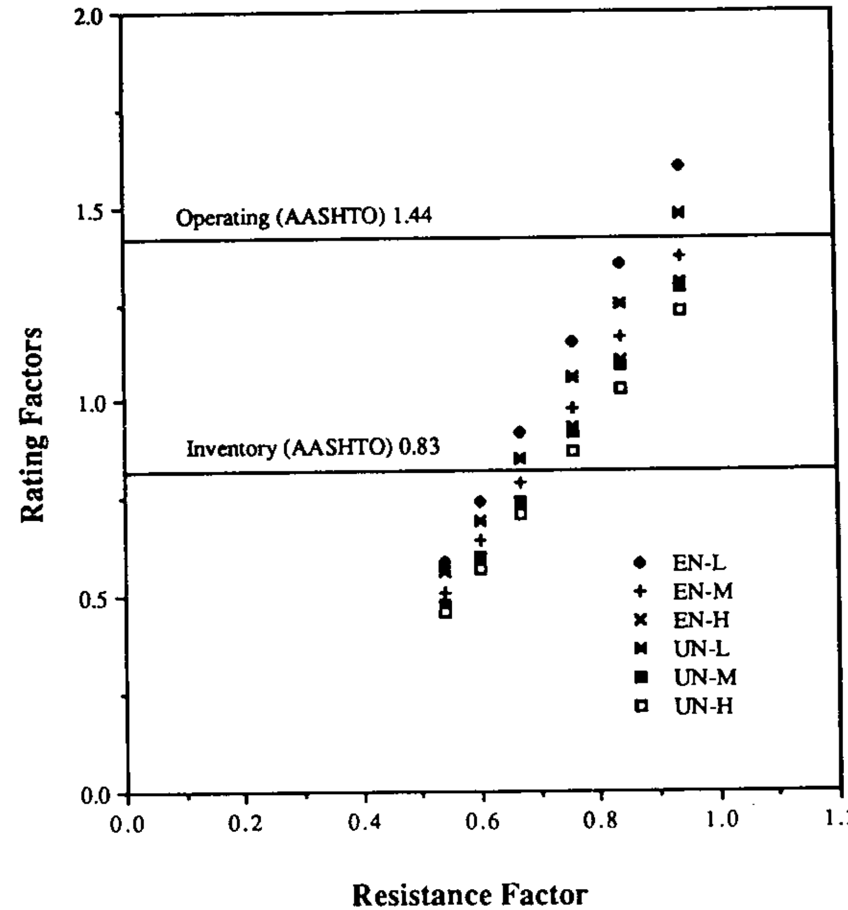
Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	0.59	1.82	0.54	0.48
	0.60	0.74		0.60	0.60
	0.67	0.92		0.67	0.74
	0.76	1.15		0.76	0.93
	0.84	1.35		0.84	1.10
	0.94	1.60		0.94	1.30
1.60	0.54	0.56	1.84	0.54	0.48
	0.60	0.69		0.60	0.60
	0.67	0.85		0.67	0.74
	0.76	1.06		0.76	0.92
	0.84	1.25		0.84	1.09
	0.94	1.48		0.94	1.29
1.73	0.54	0.51	1.93	0.54	0.46
	0.60	0.64		0.60	0.57
	0.67	0.79		0.67	0.71
	0.76	0.98		0.76	0.87
	0.84	1.16		0.84	1.03
	0.94	1.37		0.94	1.23

Inv. (AASHTO) 0.83

Oper. (AASHTO) 1.44

Figure C-5. Continued



C-6. Example 2—proposed rating factors for span 1, 0.5 point-positive moment.

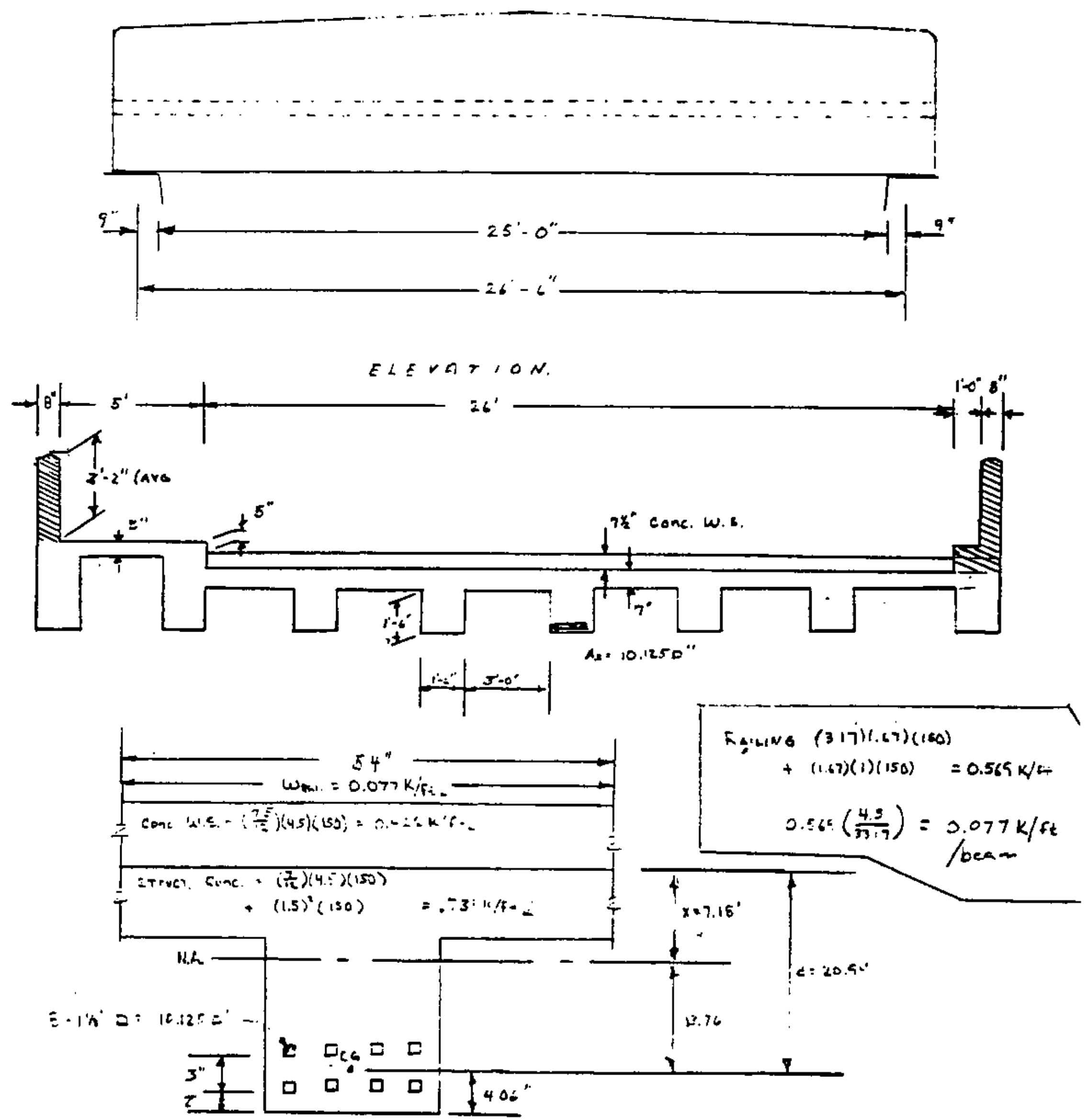


Figure C-7. Example 3—Route 41 over Trout Brook Tributary Town of Golden, Courtland County, New York.

BIN 1024880 - NY Rte 41 over Trout Brook Tributary, Town of Golden, Cortland County

Resistance Factor	Description	RF(1) f_y=40 ksi	RF(1) f_y=33 ksi
0.76	Field Meas. Norm. Insp., No PM	2.72	2.15
0.84	Field Meas. Norm. Insp., PM Field Meas. Norm. Insp., No PM	3.09	2.44
0.94	Field Meas. Freq. Inspe., PM	3.57	2.83
AASHTO Load Factor	Inventory Operating	2.07 3.45	1.63 2.72
NYS DOT LR Prog.(2)	Inventory Operating	1.10 1.86	

(1) f_y = 40 ksi specified on NYS Standard Sheet issued in 1928 for T-beam bridges.

f_y = 33 ksi specified in NCHRP Project 10-15(1) for unknown reinforcing steel prior to 1954.

(2) NYS DOT Load Rating Program - Uses Allowable Stress Method. Assumes f_y = 30 ksi.

Figure C-7. Continued

Resistance M_u

$$M_u = A_s f_y \left(d - \frac{a}{2}\right)$$

f_y = 40 ksi f'_c = 3 ksi

$$= (9.872)(40)\left(20.94 - \frac{2.87}{2}\right)$$

Tension steel reduced 5% due to deterioration[1]

$$M_u = 641.85 \text{ ft-k}$$

$$A_s = \left(\frac{10.125}{2}\right)(.95) + \left(\frac{10.125}{2}\right)$$

$$= 9.872 \text{ in}^2$$

@ f_y = 33 ksi M_u = 536.31 ft-k

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(9.872)(40)}{0.85(3)(54)}$$

$$a = 2.87 \text{ in}$$

Dead Load M_{DL}

Struct. Concrete	$\frac{(0.731)(25)^2}{8}$ =	57.1
Wear. Surface	$\frac{(0.422)(25)^2}{8}$ =	33.0
Rail	$\frac{(0.077)(25)^2}{8}$ =	6.0
		96.1 ft-k

(1) A 5% reduction in steel is the minimum specified in the Project 10-15(1) report. A more realistic reduction of 2%-3% was observed for this structure.

Figure C-7. Continued

Live Load M_{LL}

AASHTO

$$M_{LL} = \left(\frac{207.4}{2}\right)(.75)(1.3)$$

App. A - AASHTO-Max. Mom. Tables
$M_{MAX} = 207.4$ ft-k

$$M_{LL} = 101.1 \text{ ft-k}$$

$$g = \frac{S}{6} = \frac{4.5}{6} = 0.75$$

Proposed Rating Procedures

$$M_{LL} = \frac{(207.4)}{2}(0.75)(1.1)$$

$$= 85.6$$

$$I+1 = 1 + \left(\frac{50}{S+125}\right) = 1 + \frac{50}{150} = 1.33$$

$$= 1.3 \qquad \text{Use 1.3 MAX}$$

Proposed Rating Procedures
(Assume smooth surface)
$I+1 = 1.1$

Reduction Factors

$\phi = 0.76$ Table A-3 (Rated 4, Normal Insp., No PM)
$\gamma_D = 1.2$ Table A-5 (Structural Concrete)
$\gamma_L = 1.6$ Table A-8 (Dist. Level 2, Category 20 Unenforced, light traffic).

Rating Factor (RF)

$$RF = \frac{\phi M_u - \gamma_D M_{DL}}{\gamma_L M_{LL}}$$

	ϕ = 0.76	0.84	0.94
$RF = \frac{0.76(641.85) - 1.2(96.1)}{1.6(85.6)} =$	2.72	3.09	3.57
@ $f_y = 33$ ksi	2.14	2.44	2.83

Figure C-7. Continued

AASHTO Load Factor

$$\text{Inventory} = \frac{0.9(M_u) - 1.3(M_{DL})}{5/3\ 1.3\ (M_{LL})}$$

$$= \frac{0.9(641.85) - 1.3(96.1)}{5/3\ 1.3(101.1)} = 2.07$$

$$\text{Operating} = \frac{0.9(M_u) - 1.3(M_{DL})}{1.3\ (M_{LL})}$$

$$= \frac{0.9(641.85) - 1.3(96.1)}{1.3(101.1)} = 3.45$$

@ $f_y = 33$ ksi Inv. = 1.63
Oper. = 2.72

Figure C-7. Continued

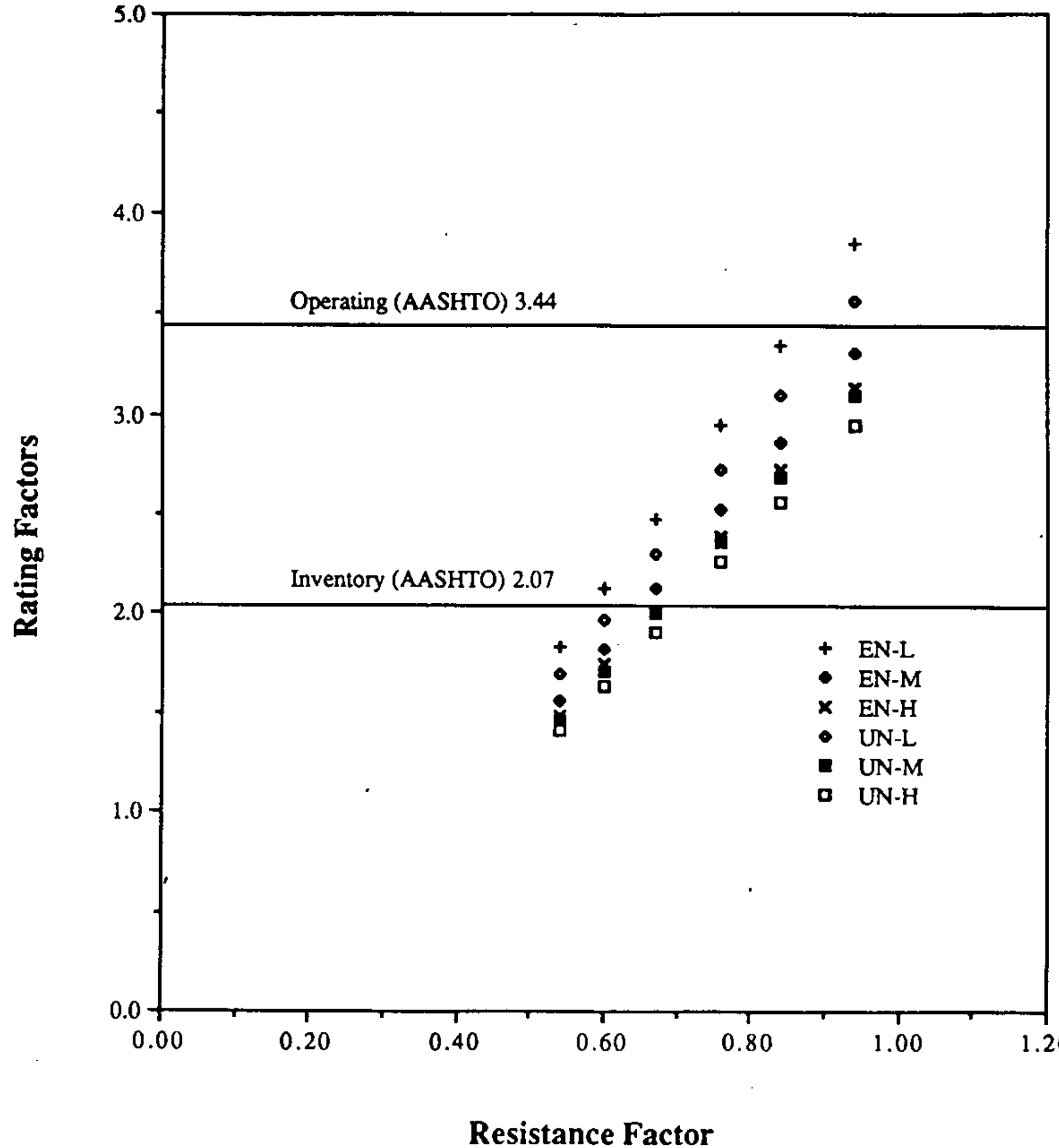


Figure C-8. Example 3—proposed rating factors for span 1, 0.5 point-positive moment.

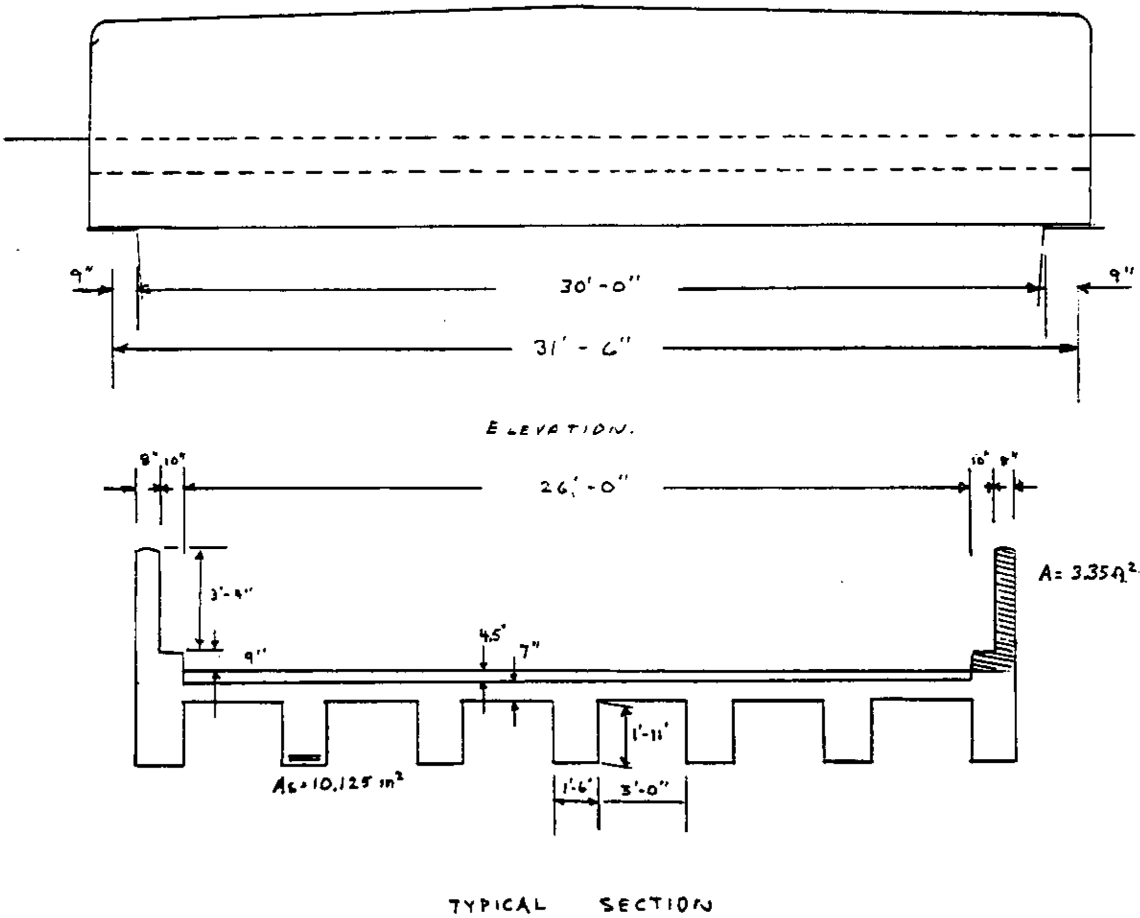


Figure C-9. Example 4—Route 145 over unknown creek, Albany County, New York.

BIN 1038230 - Rte 145 over Unknown Creek, Albany County

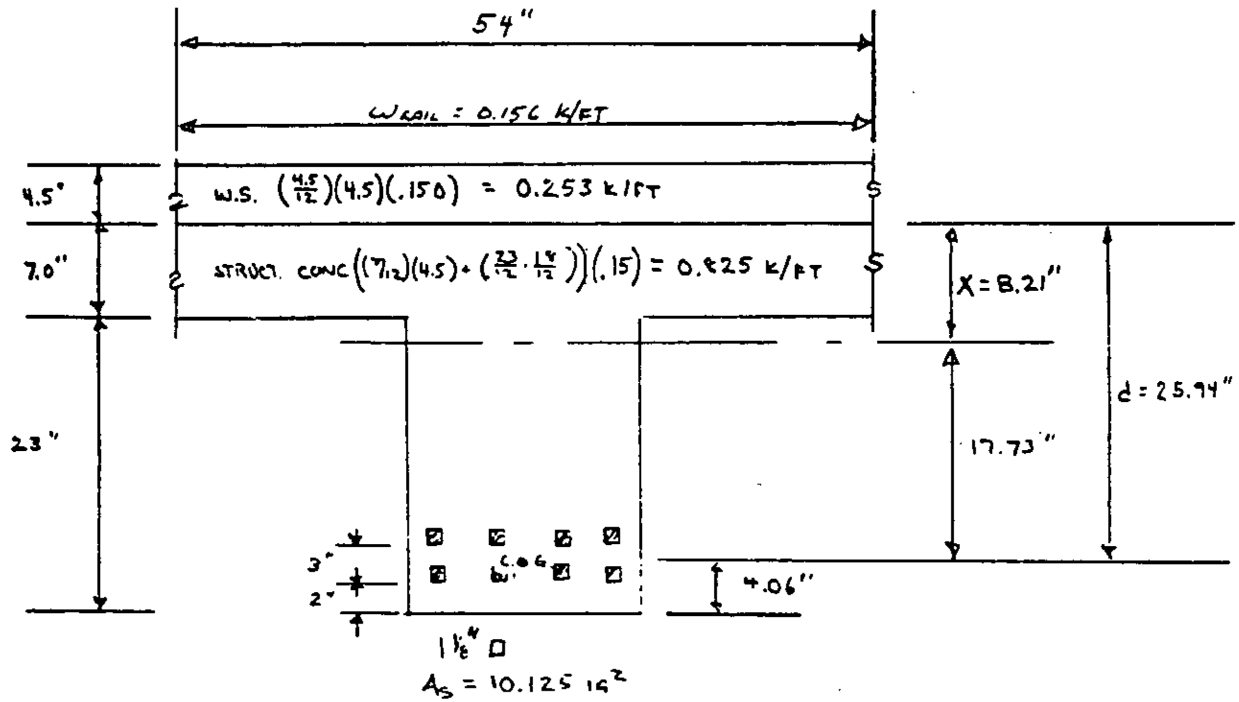
Resistance Factor	Description	RF(1) f_y=40 ksi	RF(1) f_y=33 ksi
0.76	Field Meas. Norm. Insp., No PM	2.40	1.84
0.84	Field Meas. Norm. Insp., PM	2.74	2.14
	Field Meas. Freq. Insp., No PM		
0.94	Field Meas. Freq. Inspe., PM	3.18	2.49
AASHTO Load Factor	Inventory	1.83	1.42
	Operating	3.05	2.37
NYS DOT(2) LR Prog.	Inventory	1.41	
	Operating	2.37	

(1) f_y = 40 ksi specified on NYS Standard Sheet issued in 1928 for T-beam bridges.

f_y = 33 ksi specified in NCHRP Project 10-15(1) for unknown reinforcing steel prior to 1954.

(2) NYS DOT Load Rating Program - Uses Allowable Stress Method. Assumes f_y = 30 ksi.

Figure C-9. Continued



$$\text{DL Rail} \quad 2(3.35)(0.15) = 1.005 \text{ k/ft} \left(\frac{4.5}{29}\right) = 0.156 \text{ k/ft/beam}$$

Figure C-10. T-beam cross section.

Resistance M_u

$$M_u = A_s f_y \left(d - \frac{a}{2}\right) \qquad f_y = 40 \text{ ksi} \quad f'_c = 3 \text{ ksi}$$

$$= (9.872)(40)\left(25.94 - \frac{2.87}{2}\right)$$

Tension steel reduced 5% due to deterioration[1]

$$M_u = 806.38 \text{ ft-k}$$

$$A_s = \left(\frac{10.125}{2}\right)(.95) + \left(\frac{10.125}{2}\right)$$

$$= 9.872 \text{ in}^2$$

@ $f_y = 33$ ksi $M_u = 672.11$ ft-k

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(9.872)(40)}{0.85(3)(54)}$$

$$a = 2.87 \text{ in}$$

Dead Load M_{DL}

Struct. Concrete $\dfrac{(0.825)(30)^2}{8} = 92.8$

Wear. Surface $\dfrac{(0.253)(30)^2}{8} = 28.5$

Rail $\dfrac{(0.156)(30)^2}{8} = 17.6$

138.9 ft-k

(1) A 5% reduction in steel is the minimum specified in the Project 10-15(1) report. A more realistic reduction of 2%-3% was observed for this structure.

Figure C-10. Continued

Live Load M_{LL}

$$M_{LL} = \left(\frac{282.1}{2}\right)(.75)(1.3)$$

App. A - AASHTO-Max. Mom. Tables
$M_{MAX} = 282.1$ ft-k

$$M_{LL} = 137.5 \text{ ft-k} \qquad g = \frac{S}{6} = \frac{4.5}{6} = 0.75$$

Proposed Rating Procedures

$$M_{LL} = \frac{(282.1)}{2}(0.75)(1.1) \qquad I+1 = 1 + \left(\frac{50}{S+125}\right) = 1 + \frac{50}{150} = 1.33$$

$$= 116.4 \qquad\qquad = 1.3 \quad \text{Use 1.3 MAX}$$

Proposed Rating Procedures
(Assume smooth surface)
$I+1 = 1.1$

Reduction Factors

$\phi = 0.76$ Table A-3 (Rated 4, Normal Insp., No PM)
$\gamma_D = 1.2$ Table A-5 (Structural Concrete)
$\gamma_L = 1.6$ Table A-8 (Dist. Level 2, Category 20 Unenforced, light traffic).

Rating Factor (RF)

$$RF = \frac{\phi M_u - \gamma_D M_{DL}}{\gamma_L M_{LL}}$$

	ϕ = 0.76	0.84	0.94
$RF = \dfrac{0.76(806.38) - 1.2(138.9)}{1.6(116.4)} =$	2.40	2.74	3.18
@ $f_y = 33$ ksi	1.84	2.14	2.49

Figure C-10. Continued

AASHTO Load Factor

$$\text{Inventory} = \frac{0.9(M_u) - 1.3(M_{DL})}{5/3\ 1.3\ (M_{LL})}$$

$$= \frac{0.9(806.38) - 1.3(138.9)}{5/3\ 1.3(137.5)} = 1.83$$

$$\text{Operating} = \frac{0.9(M_u) - 1.3(M_{DL})}{1.3\ (M_{LL})}$$

$$= \frac{0.9(806.38) - 1.3(138.9)}{1.3(137.5)} = 3.05$$

@ f_y = 33 ksi   Inv. = 1.42
Oper. = 2.39

Figure C-10. Continued

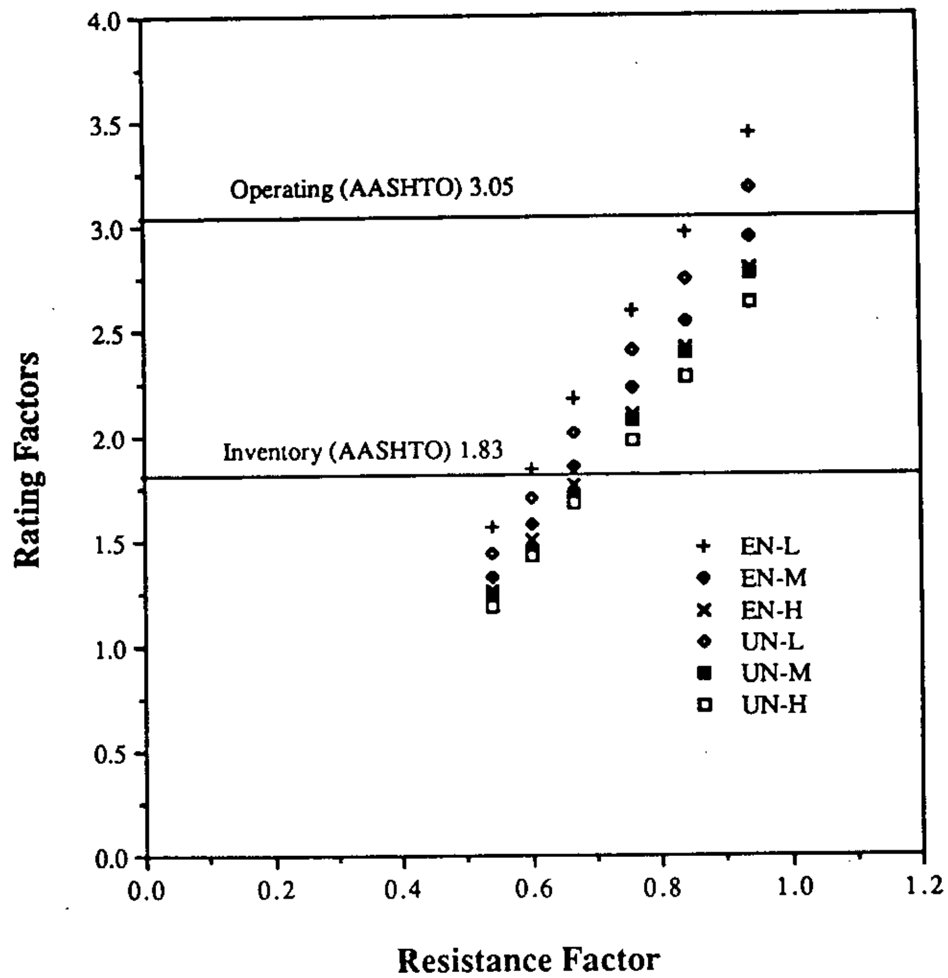
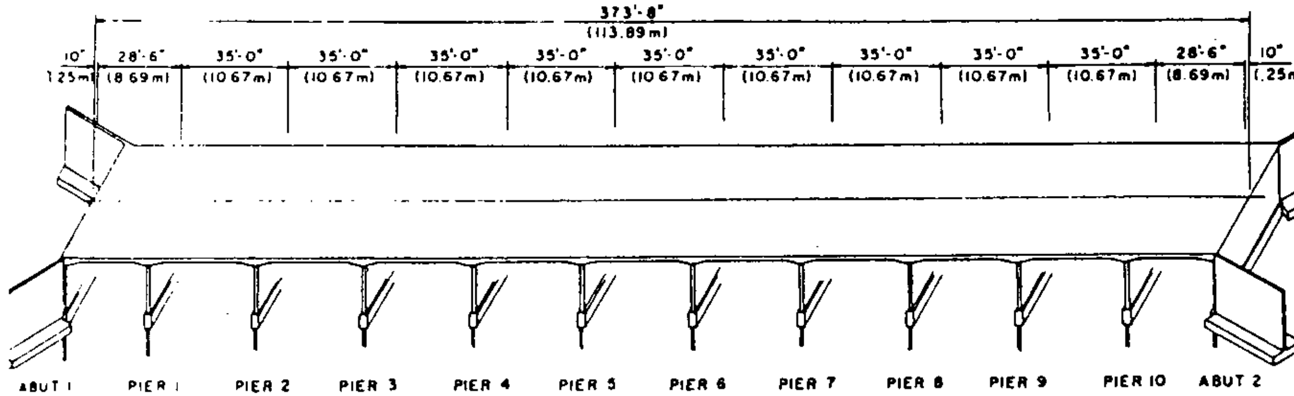
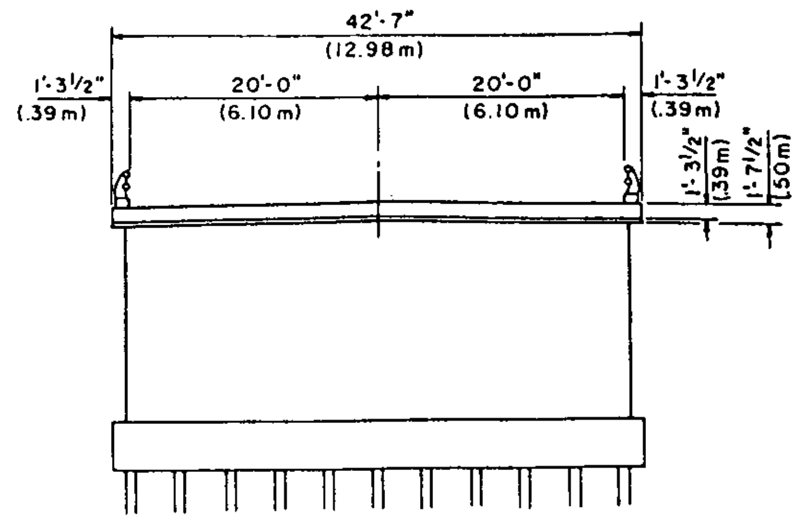


Figure C-11. Example 4—proposed rating factors for span 1, 0.5 point-positive moment.



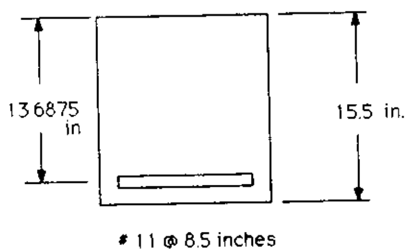
GILA RIVER BRIDGE



TYPICAL SECTION

Figure C-12. Example 5— Gila River bridge near Lake AFB on Route 85, Arizona.

Positive Moment



f'c = 3000 psi

fy = 40,000 psi

As = 5.506 sq. in.

Resistance

$$M_u = A_s f_y \left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(5.506)(40)}{(0.85)(3)(30)} = 2.88$$

$$M_u = 5.506 \left(\frac{40}{12}\right)\left(13.6875 - \frac{2.88}{2}\right) = 224.78 \text{ k-ft}$$

Dead Load

$$25 \frac{\text{k-ft}}{\text{ft}} \text{ (BDS)}$$

Live Load

AASHTO

$$69 \frac{\text{k-ft}}{\text{ft}} \text{ (BDS)}$$

$$E = 4 + 0.06 L = 4 + 0.06(35) = 6.10$$

$$\text{No. Lanes} = \frac{2.5}{(6.10)(2)} = 0.205$$

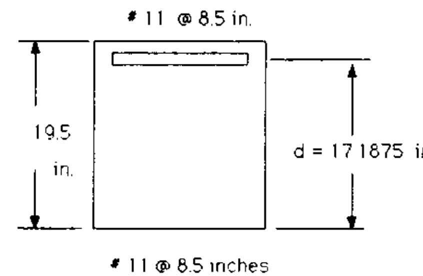
Proposed Rating Procedures

Impact (assume smooth surface)
I+1 = 1.1

M_{LL} = 58.4 (BDS)

Figure C-12. Continued

Negative Moment



f'c = 3000 psi

fy = 40,000 psi

Resistance

$$M_u = A_s f_y \left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(5.506)(40)}{(0.85)(3)(30)} = 2.88$$

$$M_u = 5.506 \left(\frac{40}{12}\right)\left(17.1875 - \frac{2.88}{2}\right) = 289.02 \text{ k-ft}$$

Dead Load

$$48 \frac{\text{k-ft}}{\text{ft}} \text{ (BDS)}$$

Live Load

$$81 \frac{\text{k-ft}}{\text{ft}} \text{ (BDS)}$$

$$E = 4 + 0.06 L = 4 + 0.06(35) = 6.10$$

$$\text{No. Lanes} = \frac{2.5}{(6.10)(2)} = 0.205$$

Proposed Rating Procedures

M_{LL} = 68.5 (BDS)

Impact (assume smooth surface)
I+1 = 1.1

Figure C-12. Continued

Positive Moment

$$RF = \frac{\phi\, M_u - \gamma_D\, M_{DL}}{L\ M_{LL}}$$

$$= \frac{\phi(224.78) - \gamma_D\ (25)}{\gamma_L\ \ (58.4)} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	1.05	1.82	0.54	0.86
	0.60	1.22		0.60	0.99
	0.67	1.39		0.67	1.13
	0.76	1.63		0.76	1.32
	0.84	1.84		0.84	1.49
	0.94	2.10		0.94	1.70
1.60	0.54	0.98	1.84	0.54	0.85
	0.60	1.12		0.60	0.98
	0.67	1.29		0.67	1.12
	0.76	1.51		0.76	1.31
	0.84	1.70		0.84	1.48
	0.94	1.94		0.94	1.69
1.73	0.54	0.91	1.93	0.54	0.82
	0.60	1.04		0.60	0.93
	0.67	1.19		0.67	1.08
	0.76	1.39		0.76	1.25
	0.84	1.57		0.84	1.41
	0.94	1.80		0.94	1.61

Inv. (AASHTO) 1.14

Oper. (AASHTO) 1.89

Figure C-12. Continued

Negative Moment

$$RF = \frac{\phi\, M_u - \gamma_D\, M_{DL}}{\gamma_L\ M_{LL}}$$

$$= \frac{\phi(289) - \gamma_D\ (48)}{\gamma_L\ \ (68.5)} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	0.97	1.82	0.54	0.79
	0.60	1.15		0.60	0.93
	0.67	1.34		0.67	1.09
	0.76	1.60		0.76	1.30
	0.84	1.82		0.84	1.49
	0.94	2.12		0.94	1.71
1.60	0.54	0.90	1.84	0.54	0.78
	0.60	1.05		0.60	0.92
	0.67	1.24		0.67	1.08
	0.76	1.48		0.76	1.29
	0.84	1.69		0.84	1.47
	0.94	1.95		0.94	1.70
1.73	0.54	0.83	1.93	0.54	0.74
	0.60	0.98		0.60	0.88
	0.67	1.15		0.67	1.03
	0.76	1.37		0.76	1.23
	0.84	1.56		0.84	1.40
	0.94	1.81		0.94	1.62

Inv. (AASHTO) 1.13

Oper. (AASHTO) 1.88

Figure C-12. Continued

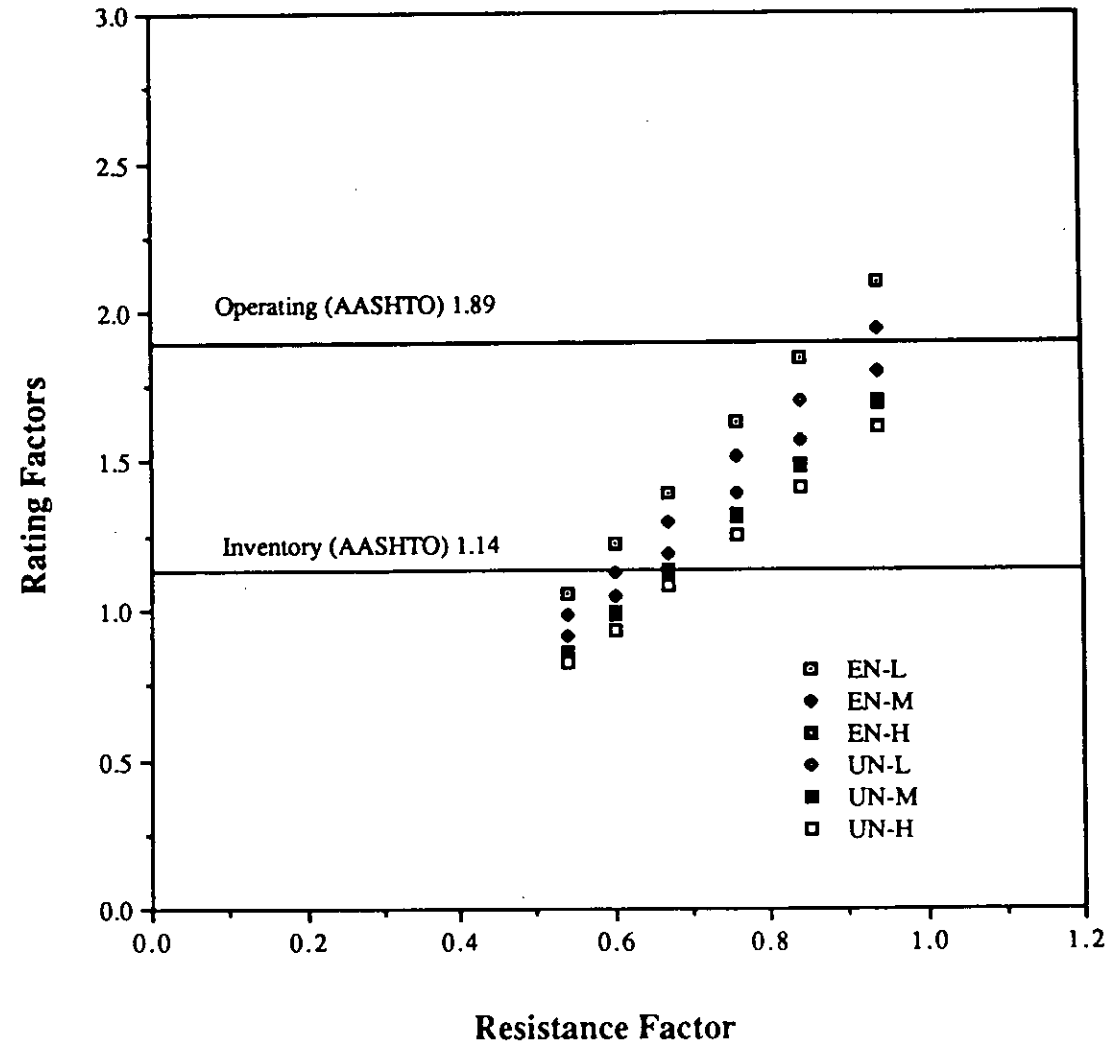


Figure C-13. Example 5—proposed rating factors for positive moment.

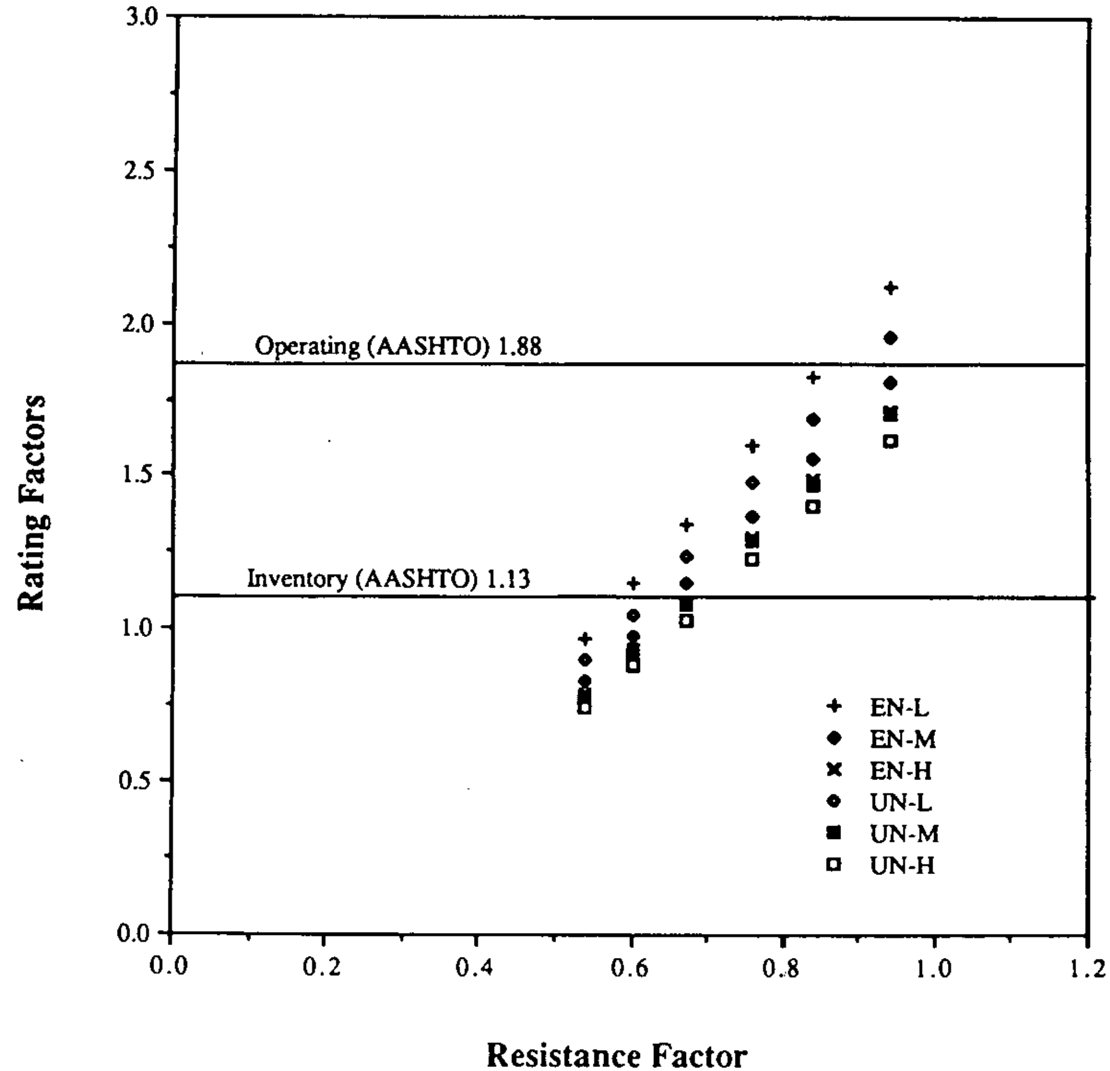


Figure C-14. Example 5—proposed rating factors for negative moment.

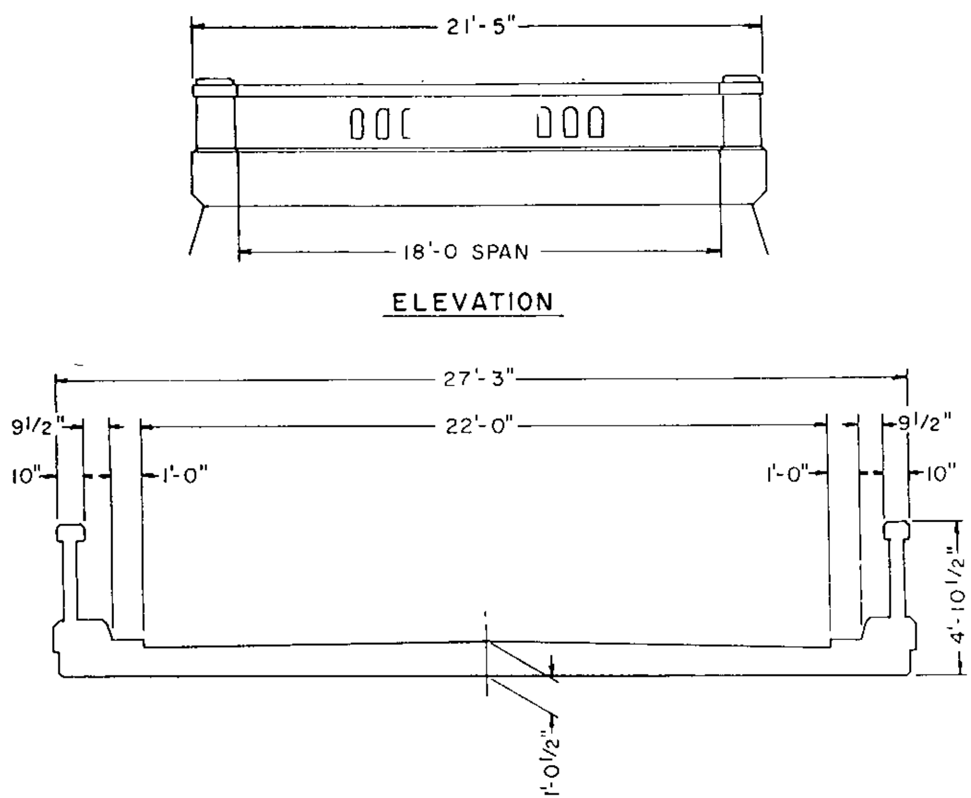
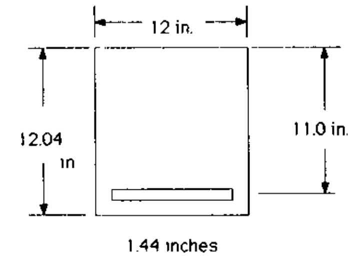


Figure C-15. Example 6—LR 203 (330+18) Armstrong County, Pennsylvania.



Year Built - 1926
$f'_c = 3{,}000$ psi
$f_y = 33{,}000$ psi

<u>Resistance</u>

$C = 0.85\, f'_c\, ba$

$T = A_s\, f_y = (1.44)(33) = 47.52$ kips

$$a = \frac{47.52}{(3)(12)(0.85)} = 1.55 \text{ in}$$

$$M_u = C\left(d - \frac{a}{2}\right) = (47.52)\left[11 - \frac{1.55}{2}\right]$$

$$= 485.89\ \frac{\text{k-in}}{\text{ft}} = 40.49\ \frac{\text{k-ft}}{\text{ft}}$$

<u>Dead Load</u>

		w(k/ft)	M_{DL}(k-ft/ft)
Structural Conc.	$= (0.150)\left(\frac{12.04}{12}\right) =$	0.151	$\frac{(0.151)(19)^2}{8} = 6.79$
Rail	=		
AC Overlay	$= (0.150)\left(\frac{3.75}{12}\right) =$	0.047	$\frac{(0.047)(19)^2}{8} = 2.12$
			8.91

Figure C-15. Continued

Live Load

App A - AASHTO (HS20)

$$M_{LL} = \left(\frac{152}{2}\right)(0.195)(1.30) = 19.27$$

$$E = 4 + 0.06L = 4 + (0.06)(19) = 5.14$$

$$g = \frac{1}{E} = 0.195$$

$$I+1 = 1.30$$

$$RF = \frac{\phi\ M_u - \gamma_D\ M_{DL}}{\gamma_L\ M_{LL}} = \frac{\phi(40.49 - \gamma_D\ (8.91))}{\gamma_L\ \ (16.30)} \qquad \gamma_D = 1.2$$

Proposed Rating Procedure

$I+1 = 1.1$ (Assume smooth surface)

$$M_{LL} = \frac{(152)}{2}(0.195)(1.1) = 16.30$$

Proposed

γ_L	ϕ	RF
1.48	0.54	0.46
	0.60	0.57
	0.67	0.68
	0.76	0.83
	0.84	0.97
	0.94	1.13
1.60	0.54	0.43
	0.60	0.52
	0.67	0.63
	0.76	0.77
	0.84	0.90
	0.94	1.05
1.73	0.54	0.40
	0.60	0.48
	0.67	0.58
	0.76	0.71
	0.84	0.83
	0.94	0.97

γ_L	ϕ	RF
1.82	0.54	0.38
	0.60	0.46
	0.67	0.56
	0.76	0.67
	0.84	0.78
	0.94	0.92
1.84	0.54	0.38
	0.60	0.45
	0.67	0.54
	0.76	0.67
	0.84	0.78
	0.94	0.91
1.93	0.54	0.35
	0.60	0.44
	0.67	0.52
	0.76	0.64
	0.84	0.74
	0.94	0.87

Inv. (AASHTO) 0.60

Oper. (AASHTO) 0.99

Figure C-15. Continued

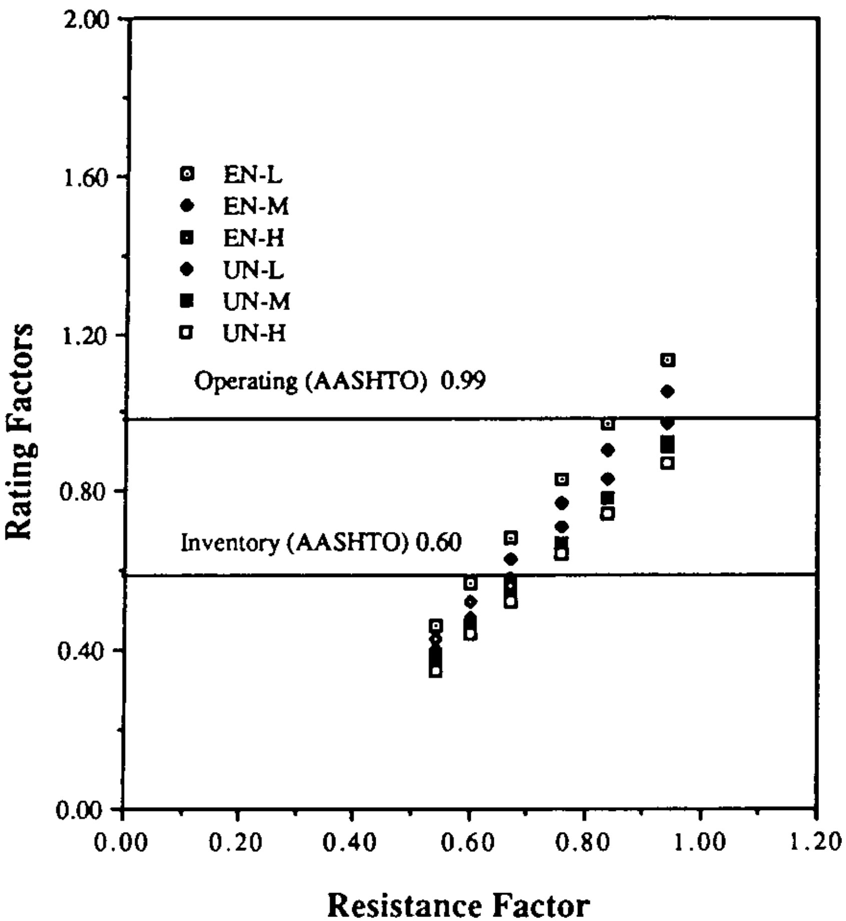
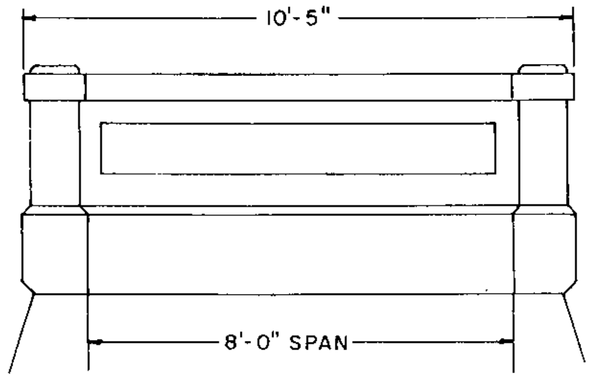
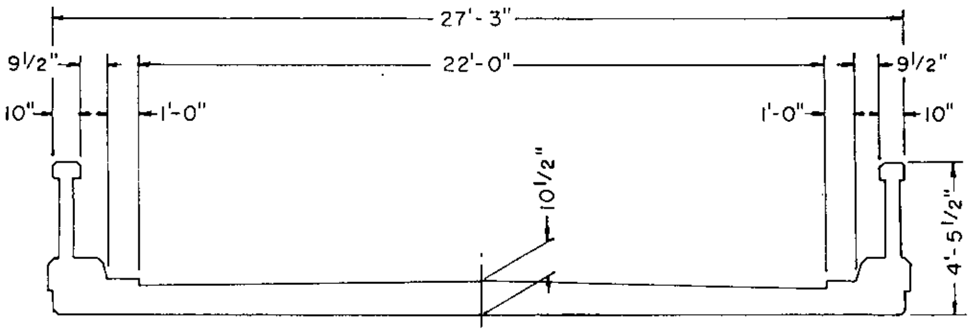


Figure C-16. Example 6—Proposed rating factors span 1, 0.5 point-positive moment.

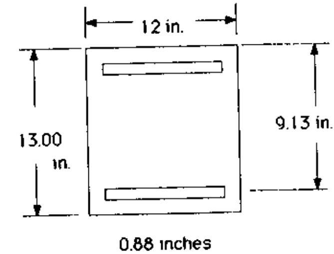


ELEVATION



TYPICAL SECTION

Figure C-17. Example 7—LR 73 (1102+00) Butler County, Pennsylvania.



Year Built - 1925
$f'_c = 2{,}500$ psi
$f_y = 40{,}000$ psi

<u>Resistance</u>

$$C = 0.85\, f'_c\, ba = (0.85)(2.5)(12)a = 25.5a$$

$$T = A_s f_y = (0.88)(40) = 35.2 \text{ kips}$$

$$a = 1.38 \text{ in}$$

$$M_u = C\left(d - \frac{a}{2}\right) = (35.2)\left[9.13 - \frac{1.38}{2}\right] = 297.1 \frac{\text{k-in}}{\text{ft}} = 24.76 \frac{\text{k-ft}}{\text{ft}}$$

<u>Dead Load</u>

		w(k/ft)	M_{DL}(k-ft/ft)
Structural Conc.	$= (0.150)\left(\frac{13.0}{12}\right) =$	0.163	$\frac{(0.163)(8)^2}{8} = 1.30$
Rail	= 0		
AC Overlay	$= (0.150)\left(\frac{2.25}{12}\right) =$	0.0281	$\frac{(0.0281)(8)^2}{8} = 0.22$
			1.52

Figure C-17. Continued

Live Load

AASHTO

$$M_{LL} = \left(\frac{64}{2}\right)(0.223)(1.30)$$

$$= 9.28$$

App A - AASHTO (HS20)

$$E = 4 + 0.06L = 4 + (0.06)(8)$$

$$= 4.48$$

$$g = \frac{1}{E} = 0.223$$

$$I+1 = 1.30$$

Proposed Rating Procedures

$I+1 = 1.1$ (Assume smooth surface)

$$M_{LL} = \frac{(64)}{2}(0.223)(1.1)$$

$$= 7.85$$

$$RF = \frac{\phi M_u - \gamma_D M_{DL}}{L\ M_{LL}}$$

$$= \frac{\phi(24.76 - \gamma_D (1.52)}{\gamma_L\ (7.85)} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	0.99	1.82	0.54	0.80
	0.60	1.12		0.60	0.91
	0.67	1.28		0.67	1.03
	0.76	1.47		0.76	1.19
	0.84	1.63		0.84	1.32
	0.94	1.84		0.94	1.50
1.60	0.54	0.92	1.84	0.54	0.80
	0.60	1.04		0.60	0.90
	0.67	1.17		0.67	1.03
	0.76	1.35		0.76	1.18
	0.84	1.51		0.84	1.31
	0.94	1.70		0.94	1.49
1.73	0.54	0.85	1.93	0.54	0.76
	0.60	0.96		0.60	0.86
	0.67	1.09		0.67	0.97
	0.76	1.25		0.76	1.12
	0.84	1.39		0.84	1.25
	0.94	1.58		0.94	1.42

Inv. (AASHTO) 1.01

Oper. (AASHTO) 1.68

Figure C-17. Continued

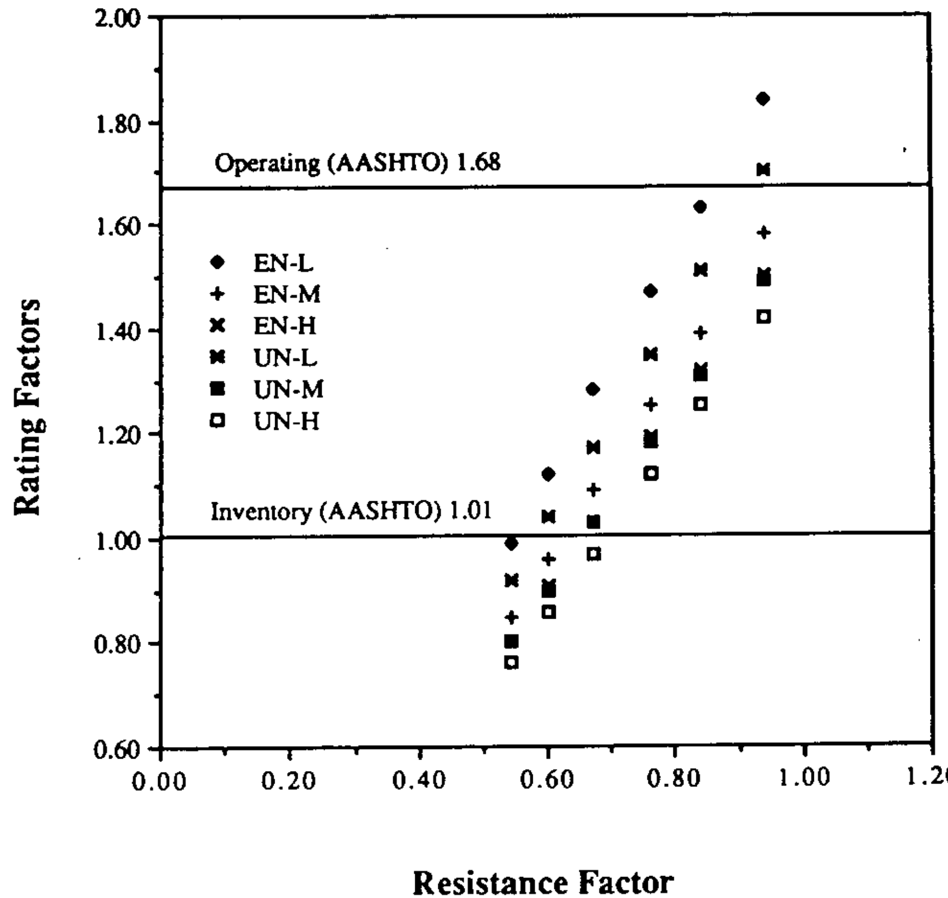
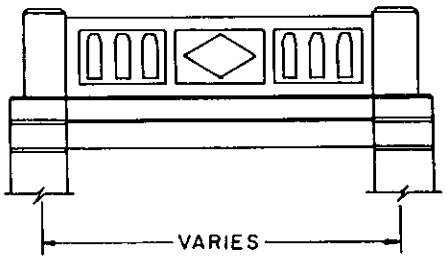
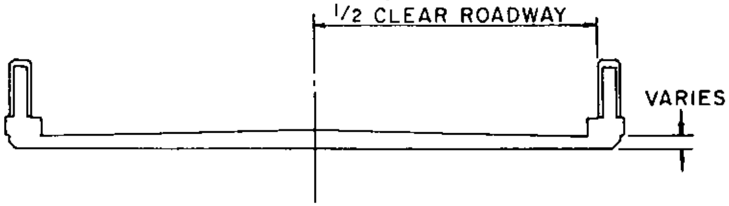


Figure C-18. Example 7—proposed rating factors for span 1, 0.5 point-positive moment.

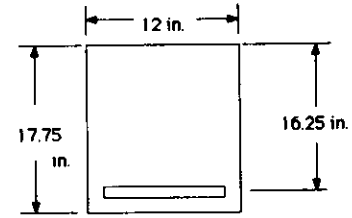


ELEVATION



TYPICAL SECTION

Figure C-19. Example 8—Small Creek, Ohio.



Resistance

Year Built - 1930
$f'_c = 3{,}000$ psi
$f_y = 33$ ksi

$M_u = 84.5$ k-ft/ft (BARS)

Dead Load

		w(k/ft)	M_{DL}(k-ft/ft)
Structural Conc.	$= (0.150)\left(\frac{17.75}{12}\right)$	$= 0.2219$	$\frac{(0.2219)(25)^2}{8} = 17.34$
Rail	$= 0$		
AC Overlay	$= (0.150)\left(\frac{9.917}{12}\right)$	$= 0.124$	$\frac{(0.124)(25)^2}{8} = 9.69$
			27.03

Live Load

AASHTO

$M_{LL} = \left(\frac{207.4}{2}\right)(1.30)(0.182)$ $\qquad \frac{1}{E} = 0.182$ (BARS)

$= 24.5$ $\qquad I+1 = 1.30$

23.6 (BARS)

Proposed Rating Procedures $\qquad I+1 = 1.1$ (Assume smooth surface)

$M_{LL} = 20.0$ (BARS)

Figure C-19. Continued

$$RF = \frac{\phi\, M_u - \gamma_D\, M_{DL}}{\gamma_L\, M_{LL}}$$

$$= \frac{\phi(84.5) - \gamma_D\,(27.03)}{\gamma_L\,(20.0)} \qquad \gamma_D = 1.2$$

Proposed

γ_L	ϕ	RF	γ_L	ϕ	RF
1.48	0.54	0.45	1.82	0.54	0.37
	0.60	0.61		0.60	0.51
	0.67	0.81		0.67	0.66
	0.76	1.07		0.76	0.87
	0.84	1.30		0.84	1.06
	0.94	1.59		0.94	1.29
1.60	0.54	0.41	1.84	0.54	0.35
	0.60	0.57		0.60	0.50
	0.67	0.76		0.67	0.66
	0.76	0.99		0.76	0.86
	0.84	1.20		0.84	1.07
	0.94	1.46		0.94	1.27
1.73	0.54	0.38	1.93	0.54	0.34
	0.60	0.53		0.60	0.47
	0.67	0.70		0.67	0.63
	0.76	0.92		0.76	0.83
	0.84	1.11		0.84	1.00
	0.94	1.36		0.94	1.22

Inv. (AASHTO) 0.80

Oper. (AASHTO) 1.33

Figure C-19. Continued

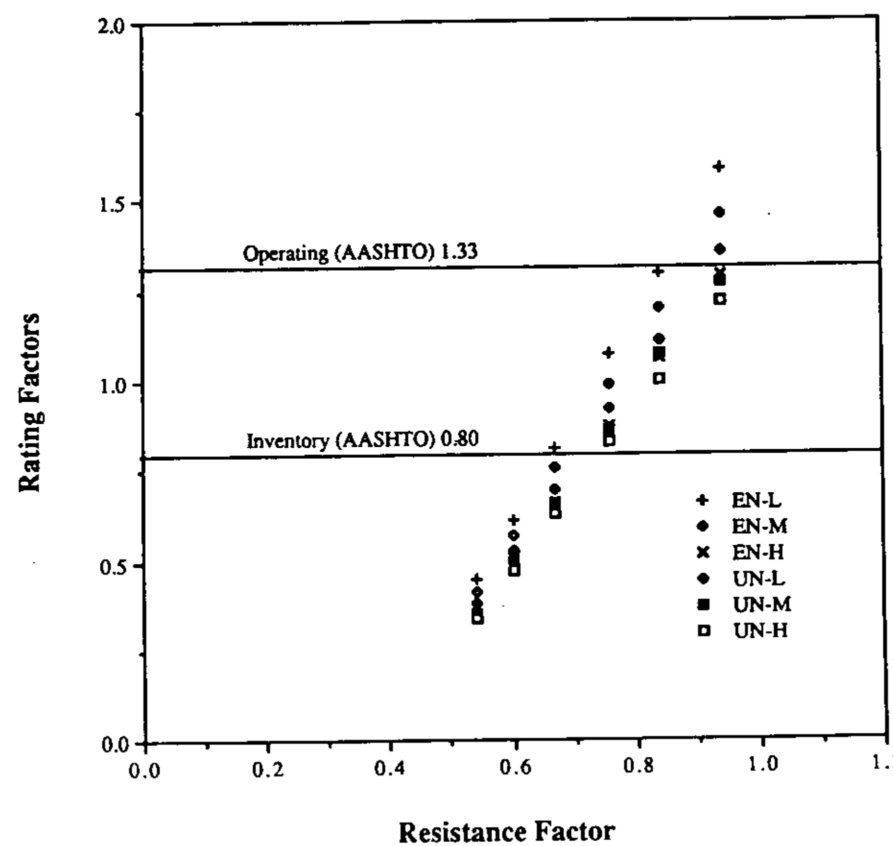


Figure C-20. Example 8—proposed rating factors for span 1, 0.5 point-positive moment.

APPENDIX D

SCHEMATICS OF THE EXISTING T-BEAM AND SLAB BRIDGES

Schematics of the existing T-beam bridges are shown in Figures D-1 through D-16, and those for the existing slab bridges are shown in Figures D-17 through D-28.

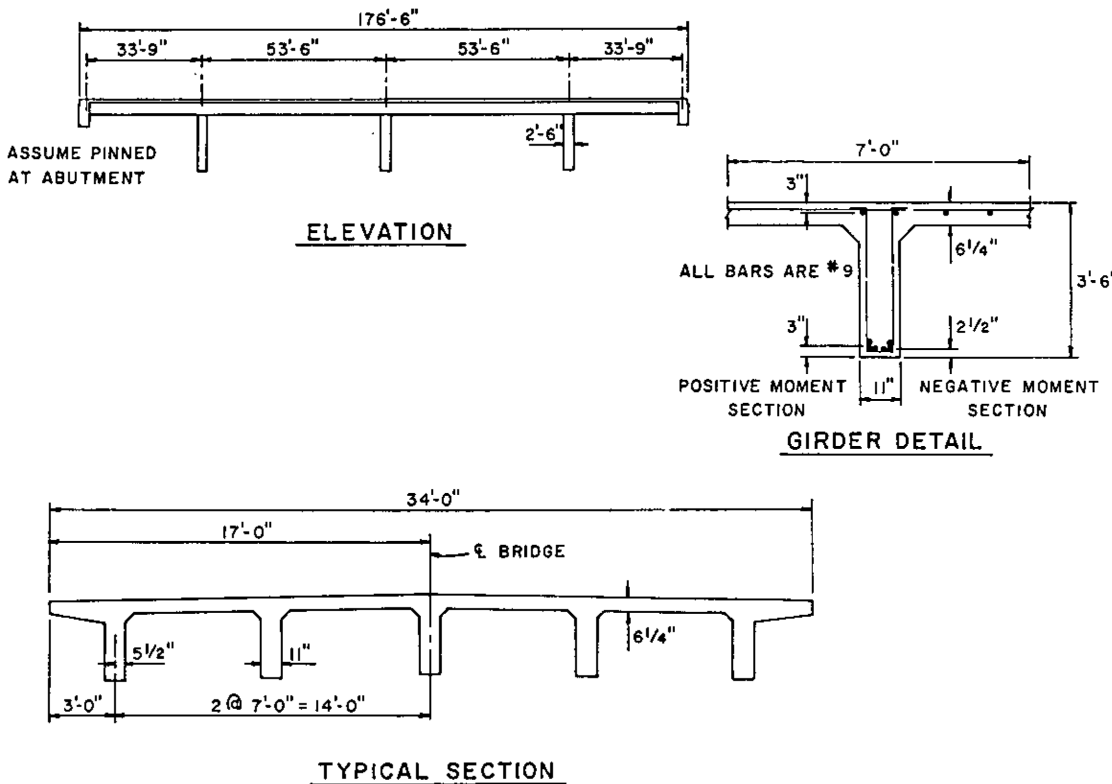


Figure D-1. East Pine Street overcrossing (29-149), California (1963).

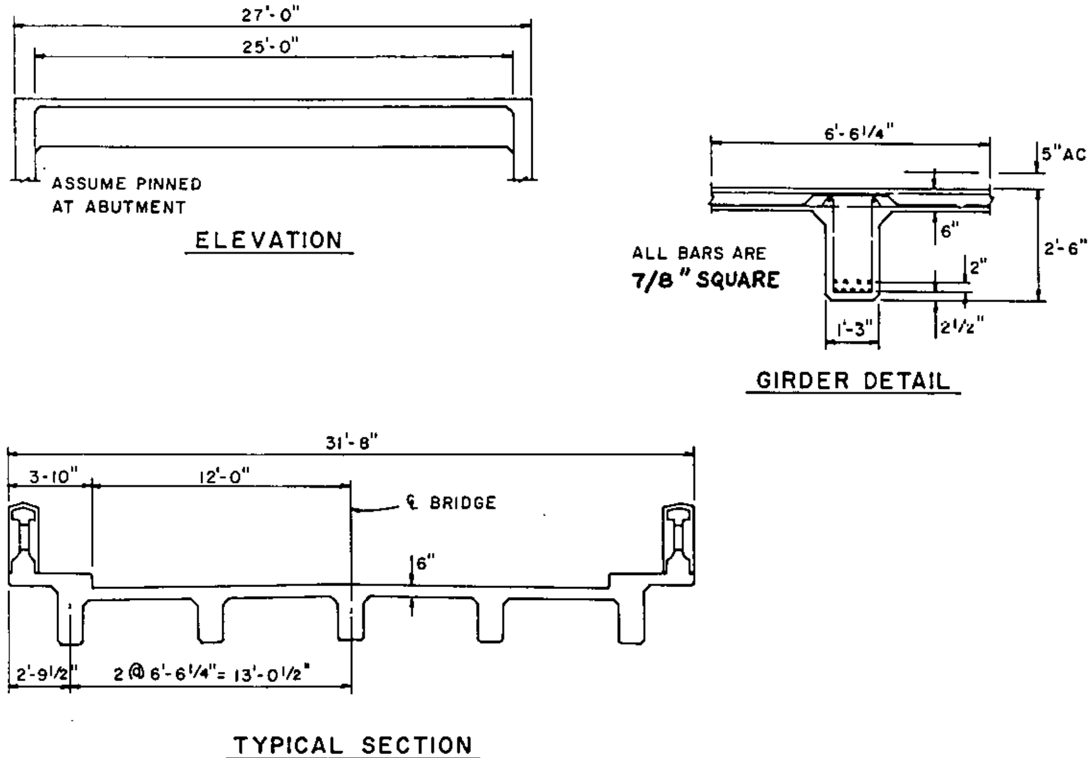


Figure D-2. Woodbridge irrigation canal (29C-190), California (1925).

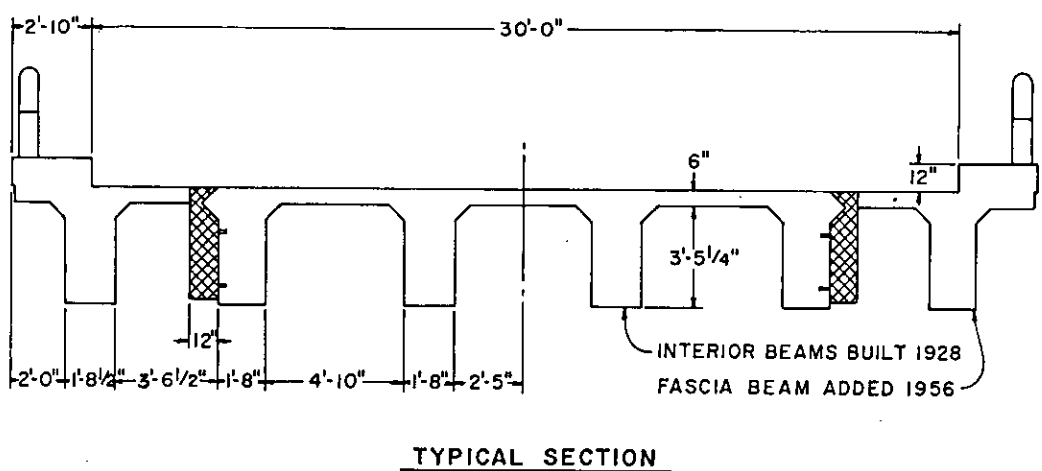


Figure D-3. Illinois structure (058-0037) (1928).

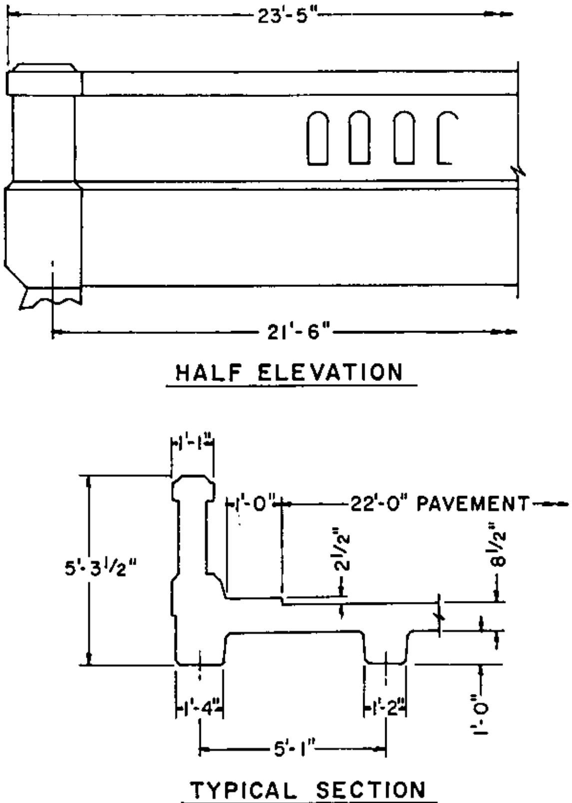


Figure D-4. Armstrong County (LR 69), Pennsylvania (1931).

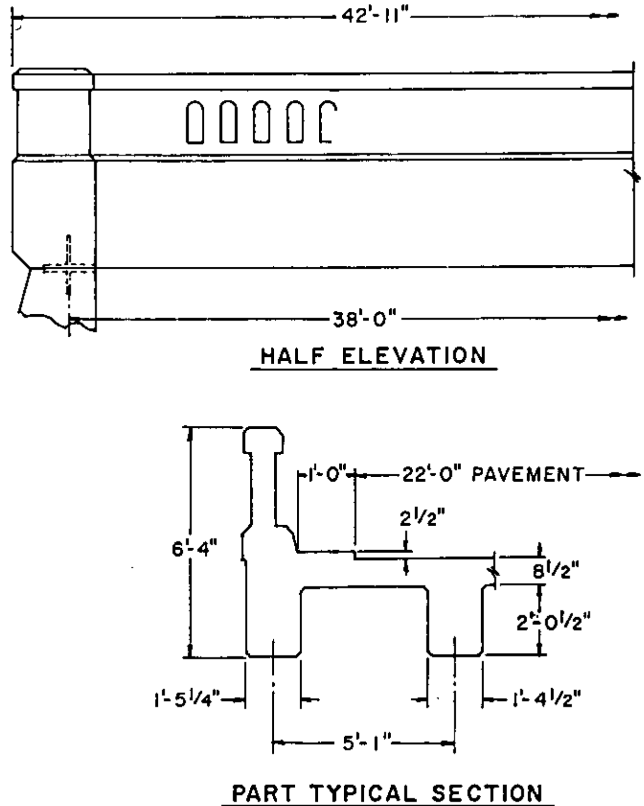


Figure D-5. Butler County (LR 10070), Pennsylvania (1940).

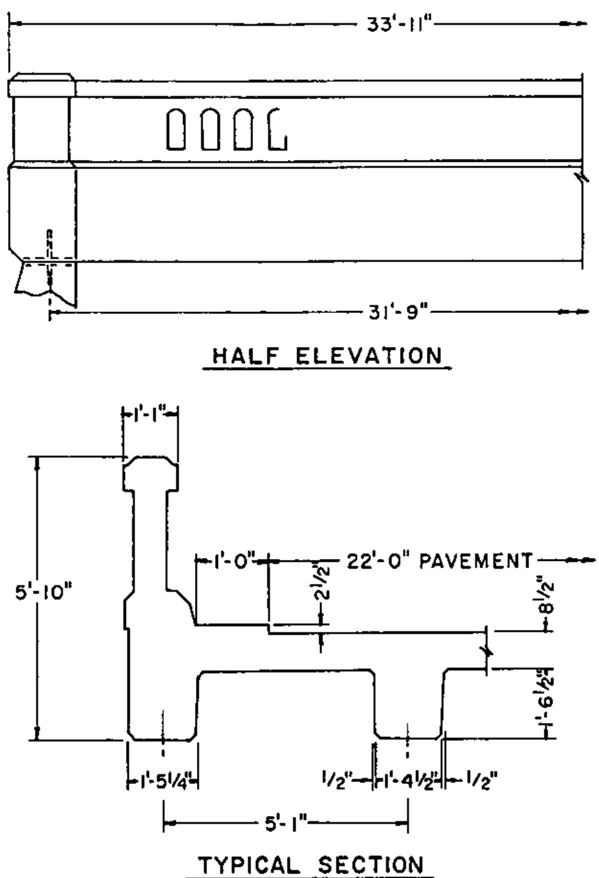


Figure D-6. Jefferson County (LR 436), Pennsylvania (1926).

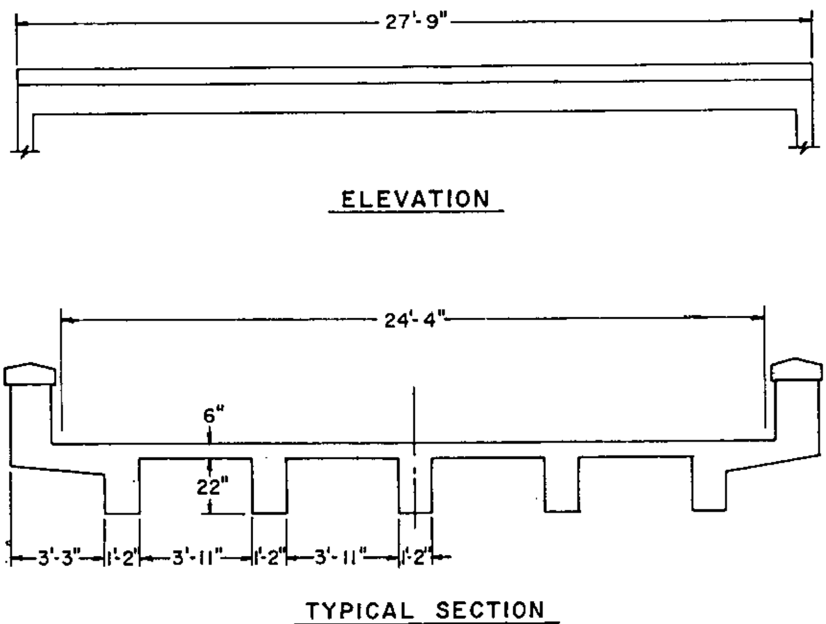


Figure D-7. Branch of Calleguas Creek (52-50), California (1911).

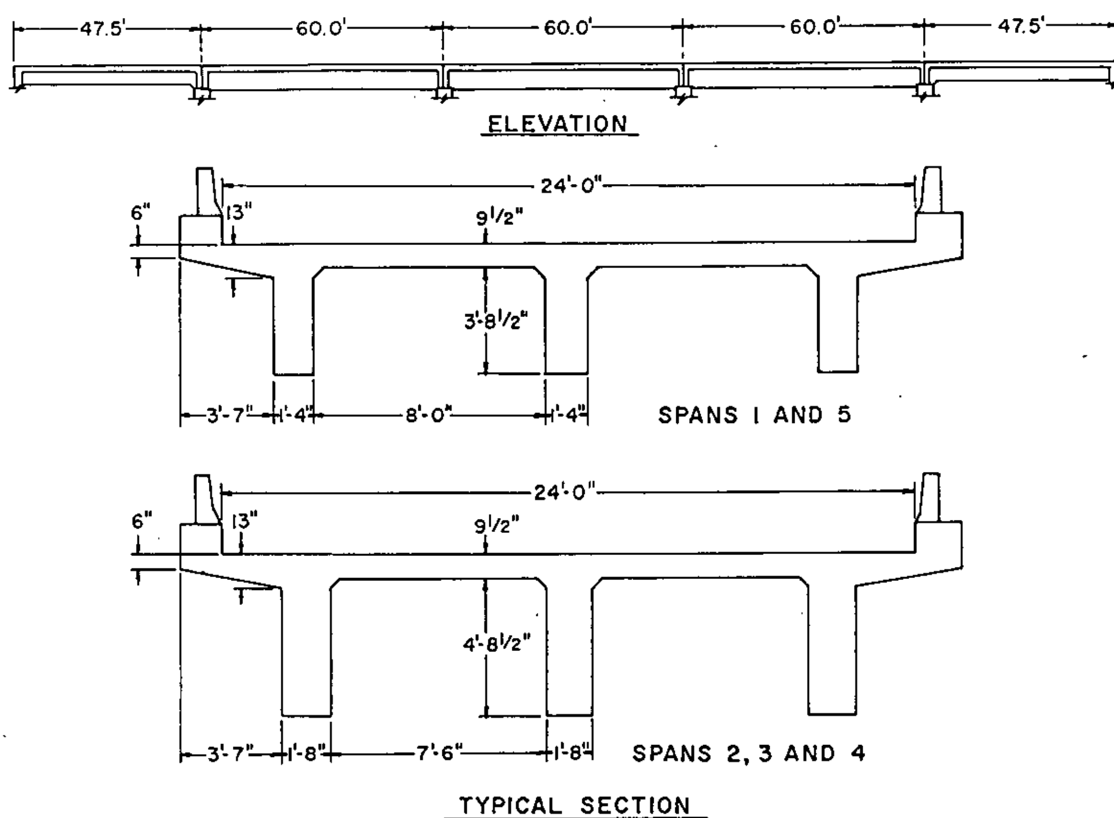


Figure D-8. Shasta River (2-11), California (1928).

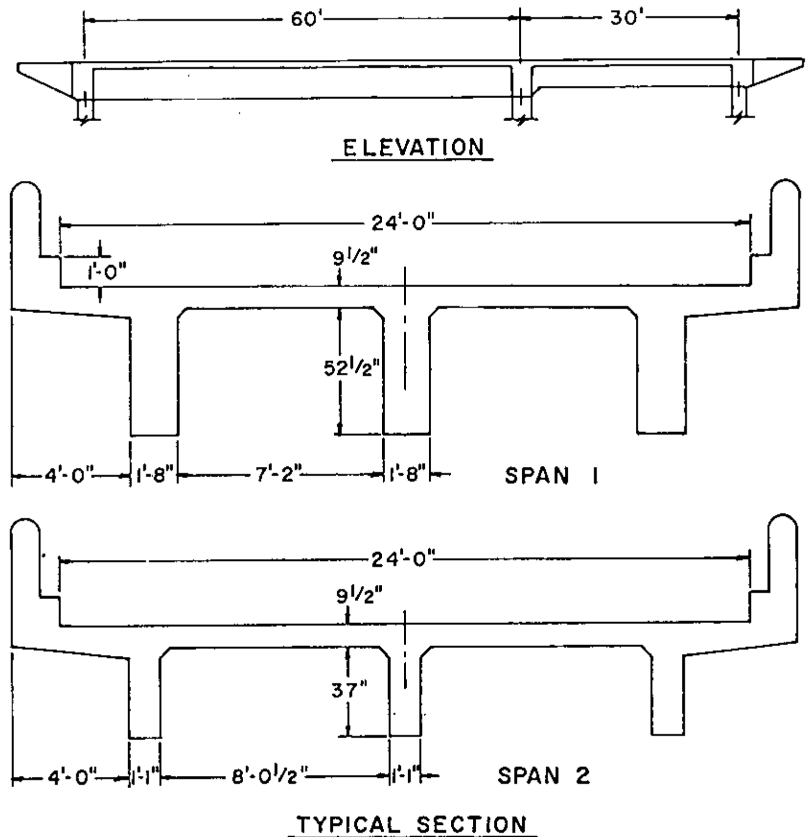


Figure D-9. El Jaro Creek (51-91), California (1929).

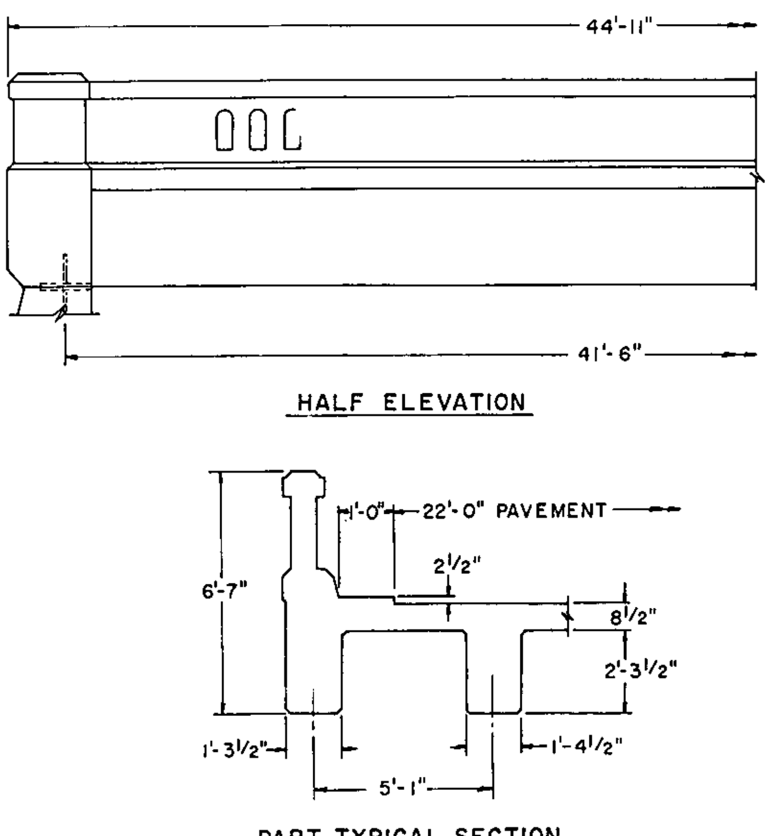


Figure D-10. Clarion County (LR 237), Pennsylvania (1931).

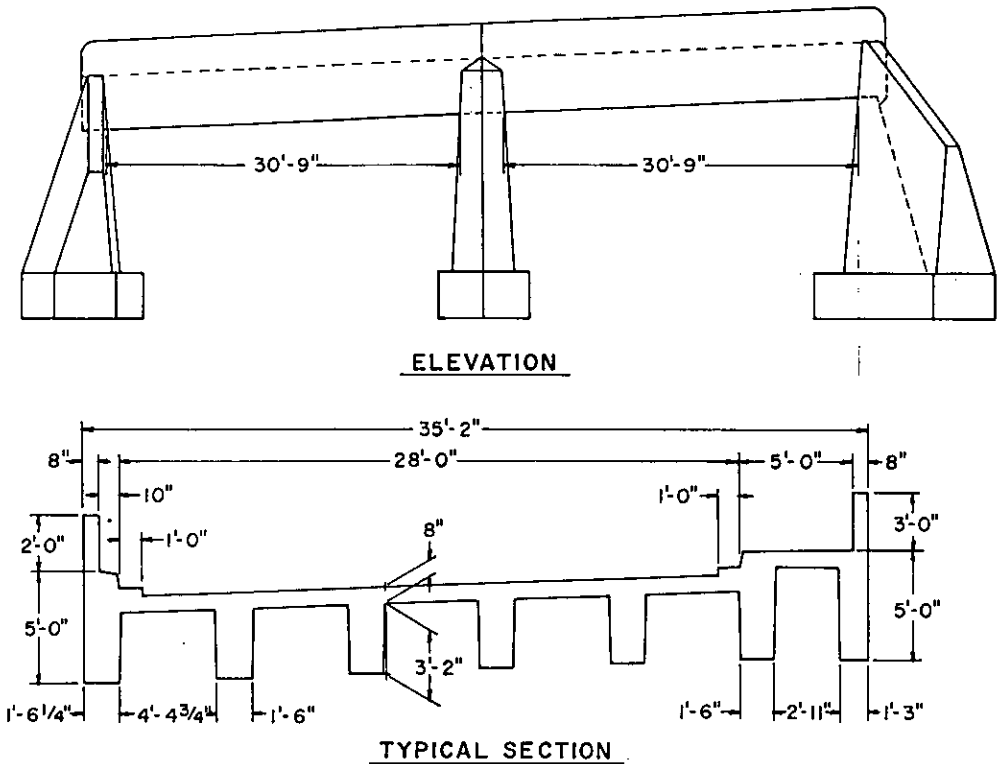


Figure D-11. Cazenovia Creek (332803), New York (1929).

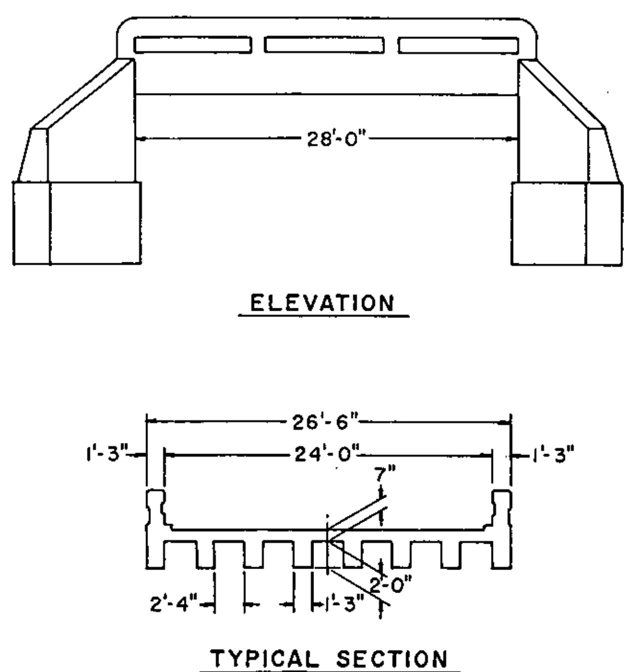


Figure D-12. Butternut Creek (331257), New York (1923).

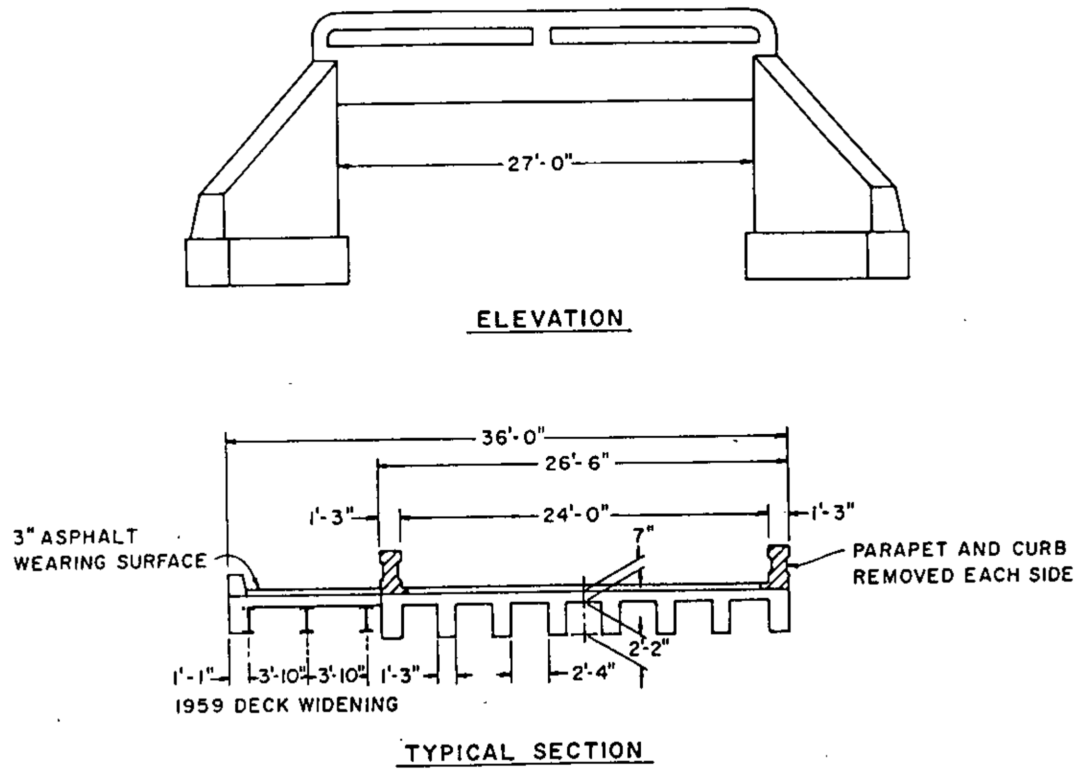


Figure D-13. Skaneatles Creek (331309), New York (1924).

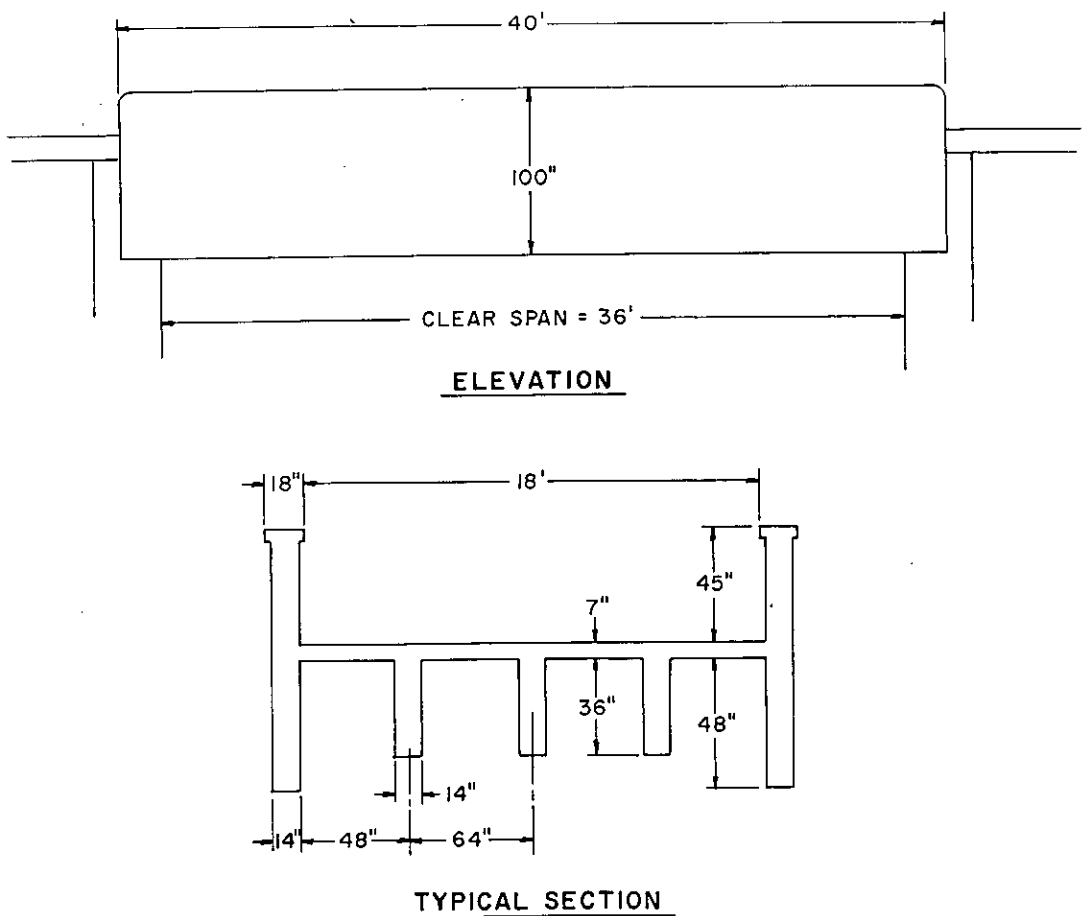


Figure D-14. Lishakill (305051), New York (1911).

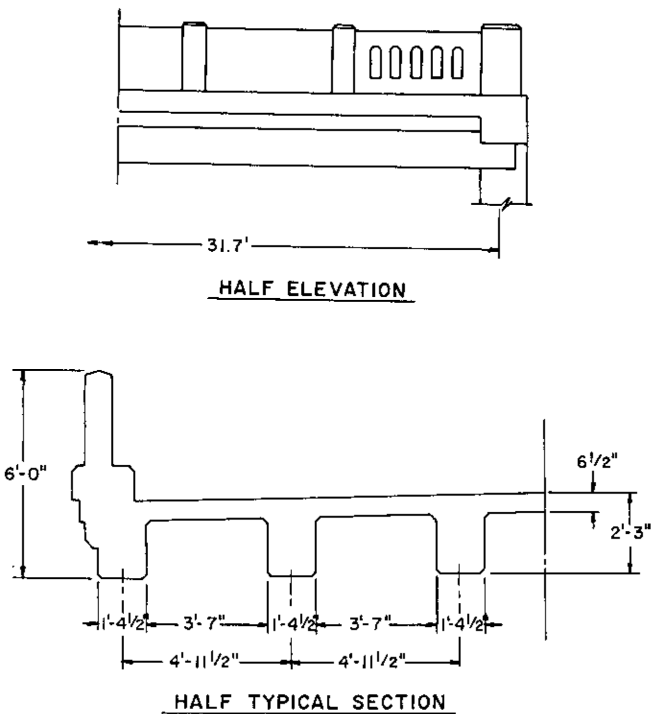


Figure D-15. Sandy and Beaver Creek (30-08760), Ohio (1932).

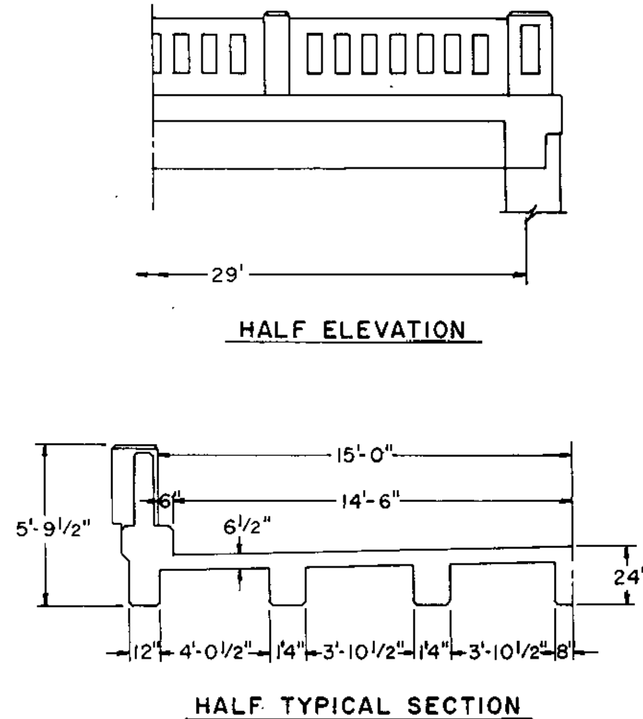


Figure D-16. Ashland County (250-0104), Ohio (1928).

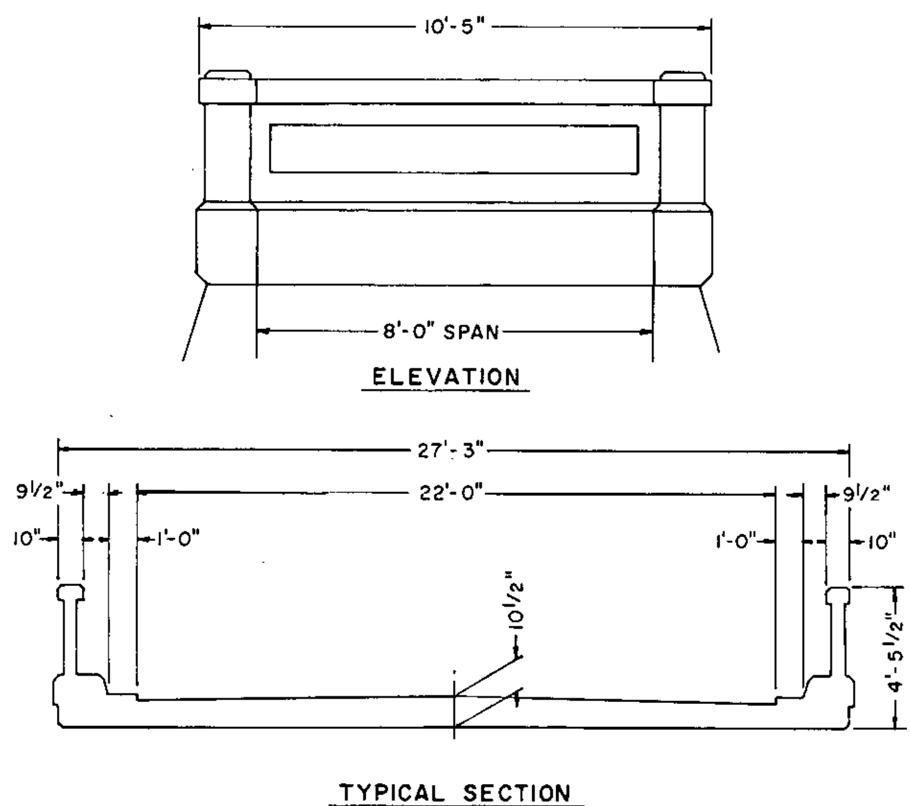


Figure D-17. Butler County (LR 73), Pennsylvania (1941).

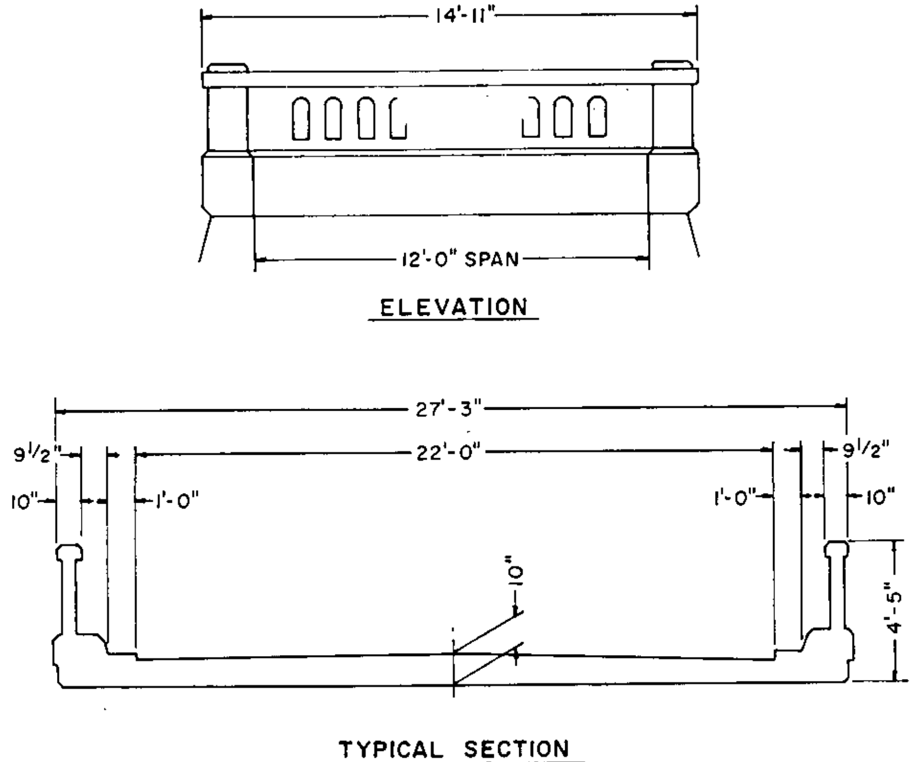


Figure D-18. Armstrong County (LR 66), Pennsylvania (1925).

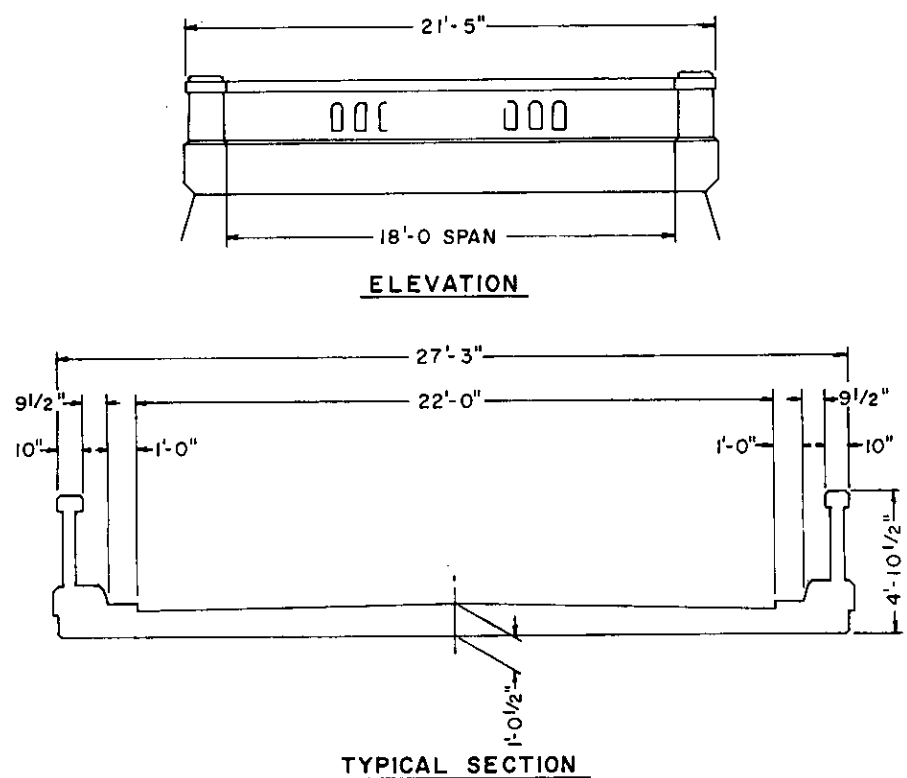
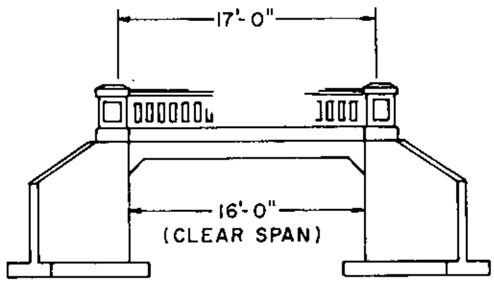
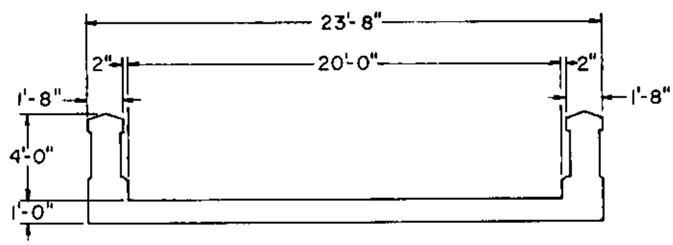


Figure D-19. Armstrong County (LR 203), Pennsylvania (1926).

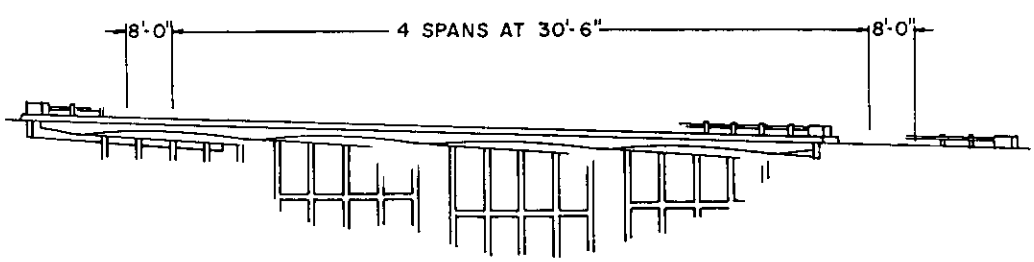


ELEVATION

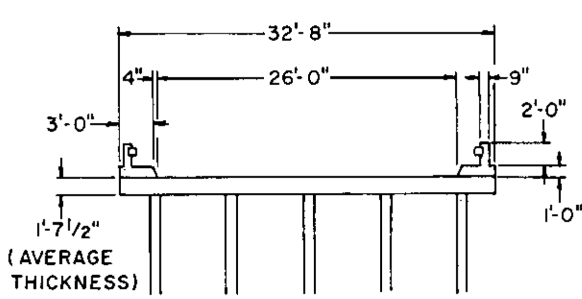


TYPICAL SECTION

Figure D-20. Pocket Creek (20-0149), California (1922).



ELEVATION



TYPICAL SECTION

Figure D-21. Chihuahua Creek (57-0014), California (1941).

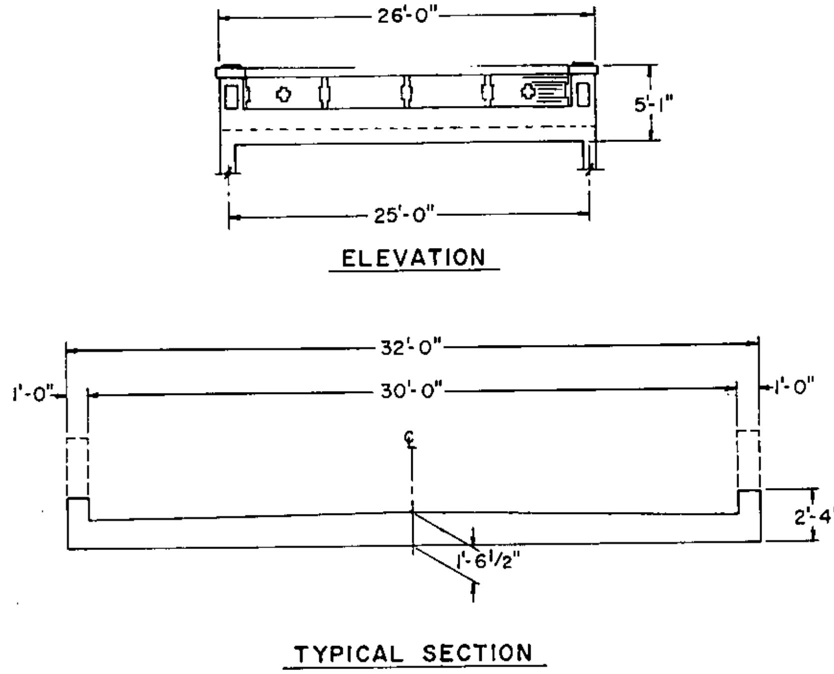


Figure D-22. Adams County Route 31 (001-0039), Illinois (1920).

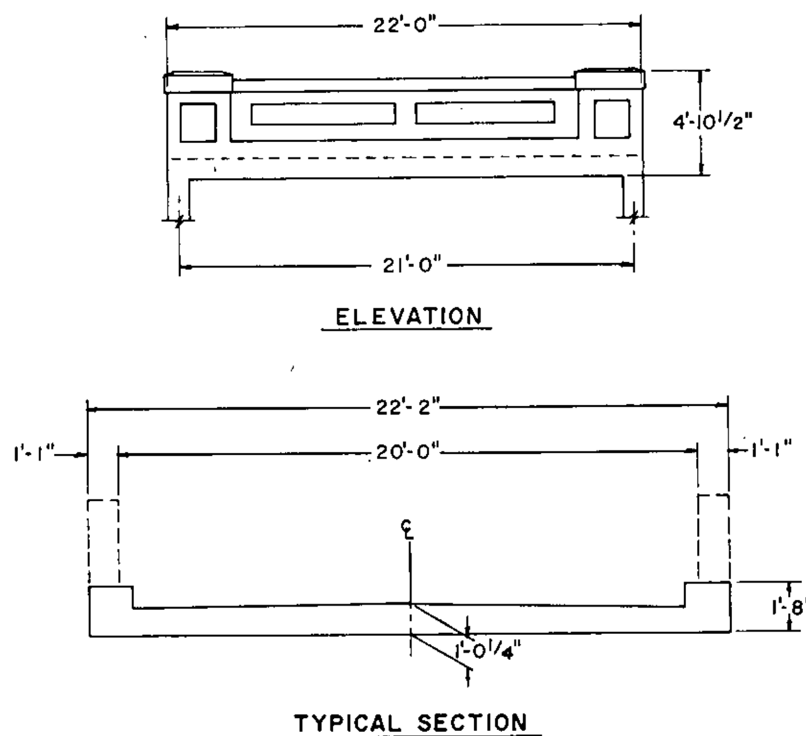


Figure D-23. Adams County Route 102 (001-0041), Illinois (1927).

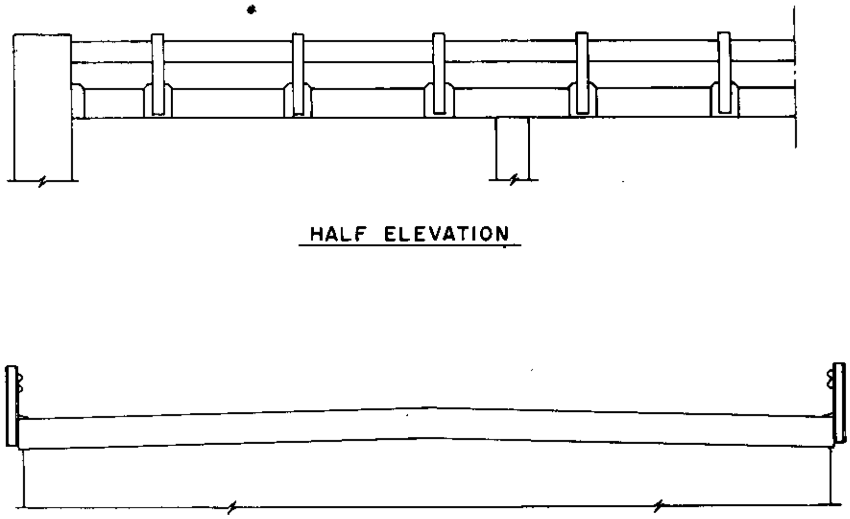


Figure D-24. Standard continuous slab bridge for Ohio.

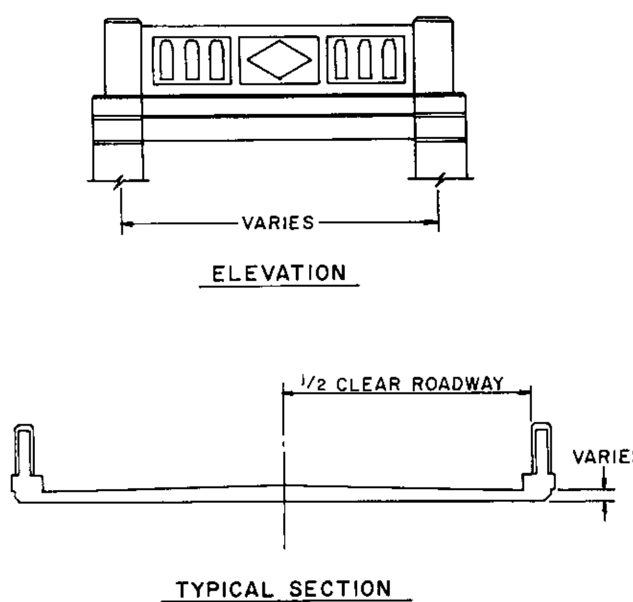


Figure D-25. Standard slab bridge for Ohio.

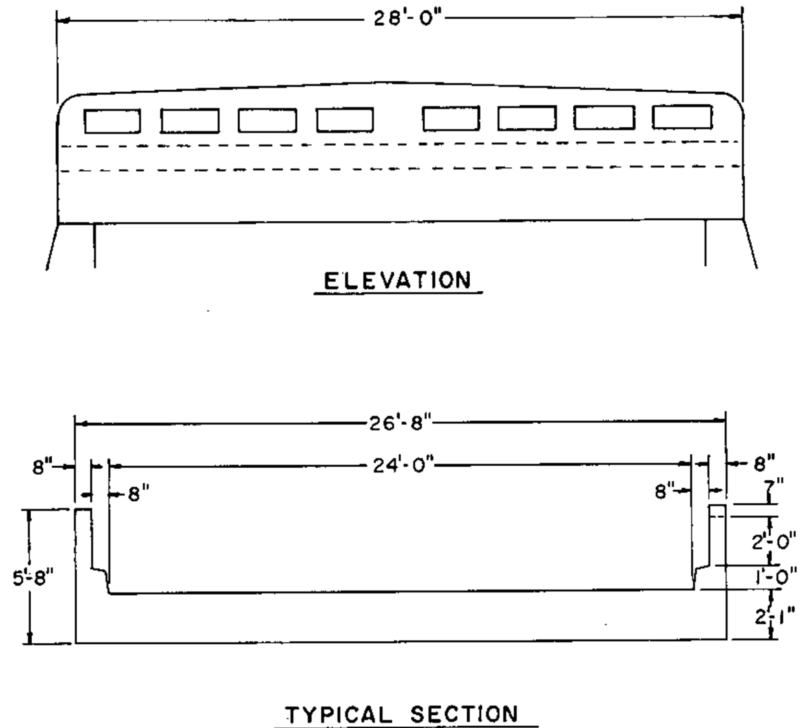


Figure D-26. Basic reservoir (330158), New York (1935).

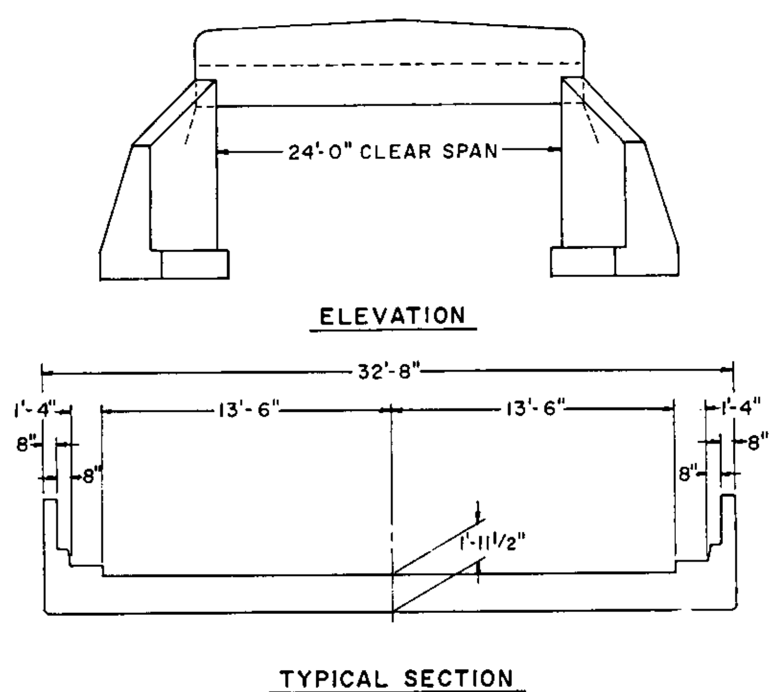
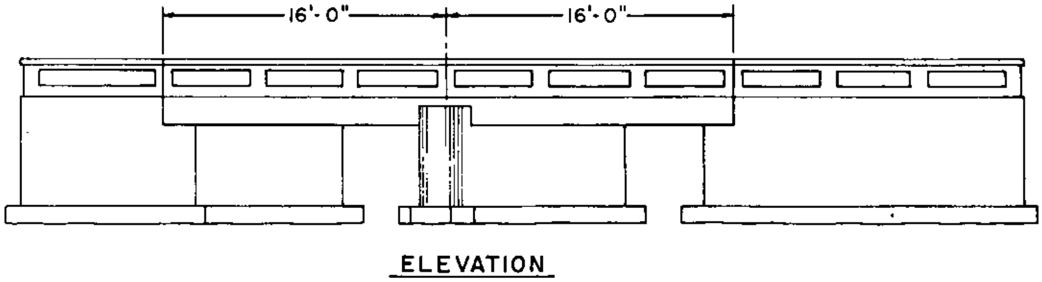


Figure D-27. Got Creek (332643), New York (1928).



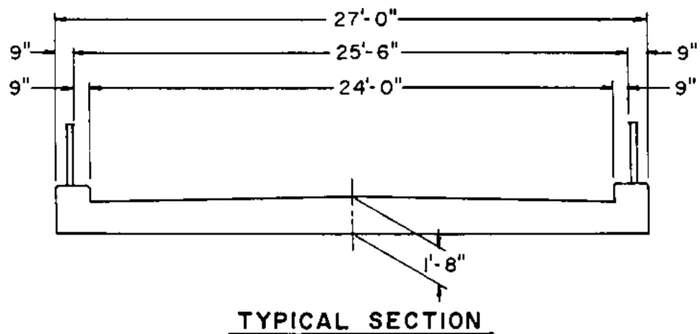


Figure D-28. Spring Brook (332844), New York (1929).

APPENDIX E

FIELD INVESTIGATION PROCEDURES

SECTION 1

GUIDELINES

1.1 GENERAL

Rating a structure for its safe load carrying capacity begins with a thorough field investigation. All physical features of the bridge that affect its structural integrity shall be examined. Any damaged or deteriorated areas should be noted, and adequate data on the deteriorated section should be obtained so that their effect can be considered by the rater. Cursory field checks should be conducted annually on each deteriorated structure after it is rated. If major changes in deterioration seem to have occurred, another thorough field investigation should be made. In any event, thorough field investigations should be made at a minimum of once every 5 years, with the rating adjusted as appropriate.

Section 1.2 identifies the minimum information that shall be included in the field inspection report for various types of reinforced concrete bridges that have been damaged or show signs of deterioration. A cross section of a typical T-beam bridge, shown in Figure E-1, is included as an example to illustrate some of the more common types of deterioration. It should be noted that reduction in the amount of reinforcing steel shown in the figure should not be made if the as-built plans are not available. This refinement is not warranted in this case because the uncertainty is significantly larger without the plans. Section 1.3 covers various in-situ testing methods for determining concrete strength.

1.2 FIELD INSPECTION

The following outline identifies information that the field reviewer needs to collect for a structure to be rated.

1.2.1 Simple-Span Bridges

Deck Surface

1. Plot the location of spalls and undersurface delamination (incipient spalls) on a scaled drawing that shows the location of girder lines. (a) Locate undersurface delamination through an acceptable means (chain dragging, Delamtec and more sophisticated systems are available). (b) Outline areas of spalls and undersurface delaminations on the deck. (c) Plot the spalled and delaminated areas on the drawing.

2. Plot areas of heavy scaling and record estimated thickness of lost deck surface.

3. Plot exposed steel information including approximate values for the following: (a) number of bars exposed; (b) length of exposure; (c) evidence of pitting, including amount and depth of pits, and estimate approximate remaining cross-sectional area

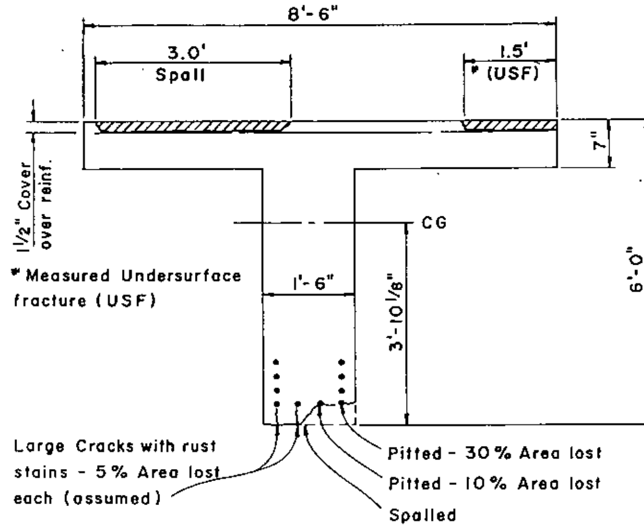


Figure E-1. Deteriorated T-beam.

(will probably require some concrete removal with a hand type chipping hammer); (d) depth of top of bar below the concrete surface.

4. Plot areas of rust stains.

Deck Soffit

1. Plot and describe areas of unusual cracking and discoloration.

2. Plot spalling and scaling.

3. Plot exposed steel information.

4. Record approximate remaining cross-sectional area of exposed transverse bars.

5. Plot all rust stains.

6. "Sound" with a hammer any areas suspected of undersurface delaminations. If any are found, plot their locations.

7. Describe and plot the location of any unusual deflections, especially those caused by live loads.

Girders

1. Collect the same information as collected for the deck soffit.

2. Record the approximate remaining cross-sectional area of any longitudinal bars exposed in the bottom of a girder within the middle ¾ of the span length.

3. Describe apparent "bond" conditions of each bar for its entire length.

4. Describe any web cracking patterns, including any crack widths occurring within ⅛ of the span length from the girder supports.

5. Plot all rust stains.

1.2.2 Continuous Span Bridges

Deck Surface

1. Collect the same information as collected for the simple span.
2. Record approximate remaining cross-sectional area of longitudinal bars exposed within ⅕ of the span length on either side of the bent.

Deck Soffit

1. Collect the same information as collected for the simple span.

Girders

1. Collect the same information as collected for the simple span.

1.3 IN-SITU TESTING

One factor affecting the load-resisting capacity of a reinforced concrete bridge is strength. It is a difficult task to determine the concrete strength of an existing structure, especially for an older structure for which construction records are not available.

At this time, only core samples taken from the concrete in question give a close indication of its strength. Core samples are not, however, always readily obtainable, and the rating engineer needs to be assisted by other methods. To assist him in selecting the one that best fits his needs, various methods will be discussed.

Some of the nondestructive testing methods available at this time include: rebound hammer, ultrasonic pulse velocity, Windsor probe and pull-out. These methods are discussed in more detail in the commentary.

SECTION 2

COMMENTARY

2.1 GENERAL

Before the safety and serviceability of a structure suffering from premature deterioration can be determined, a field inspection is needed to identify factors affecting resistance, such as concrete strength and the magnitude and location of the deterioration.

2.2 FIELD INSPECTION

2.2.1 Deterioration Location

The effect deterioration has on a structure is dependent on the location and the amount of the deterioration. Any level of deterioration located in a high stress area would have a significantly higher influence than would the same level in a low stress area. Consequently, it is imperative that a field reviewer report not only the kind and magnitude of deterioration, but also its location with respect to girder line, portion of span, girder depth, etc. The collected field data will serve two purposes: (1) it will provide data needed to rate the structure for its load-carrying capacity, and (2) it will provide a basis for the comparison of data collected in the future. The former is important for immediate rating purposes, while the latter is important for future determinations of the rate of decline in the structure's capacity, and is therefore useful for planning either rehabilitation or replacement.

Most engineers intuitively feel that any amount of premature deterioration at a critical location reduces the load-carrying capacity of a highway structure to some degree. However, there is considerable disagreement as to the extent of the capacity reduction.

Two reasons for the difficulty in quantifying the reduced load-carrying capacity of a structure suffering from deterioration are a lack of knowledge about the actual load-carrying capacity of the structure before the premature deterioration occurred (which is dictated by such factors as construction materials, construction methods) and a lack of knowledge about how any imposed loads will be distributed around the deteriorated areas. Because this knowledge is not available, making a field evaluation of the factors influencing the load-carrying capacity of a structure is more of an art than a science.

Like most materials, reinforced concrete deteriorates more rapidly in some environments than in others. The environmental effects on deterioration are covered in many publications. The purpose of the guidelines is not to review the causes of deterioration, but rather to examine the effects of deterioration on the strength and serviceability of a highway structure.

The guidelines will also identify methods that can be used to improve the evaluation of the strength and serviceability of highway structures showing signs of premature deterioration. The following outline covers the types of deterioration that have been most responsible for reducing the strength and serviceability of reinforced concrete. For each type of deterioration, there are circumstances that contribute to the development of these factors. Evidence of these circumstances may increase the probability that a loss in resistance is present or pending, but it does not prove it is so. There is also probably a limit to which the deterioration can proceed without manifesting itself with hard evidence. Therefore, in cases where certain types of circumstantial evidence are present, there is justification to make a small reduction in the anticipated resistance due to the increased probability of harmful deterioration.

In the presence of hard evidence, the problem becomes one of measuring or estimating the probable extent of the deterioration. It may be possible to establish a value from measurements that will quantify deterioration. There are inaccuracies in the current methods of in-situ strength measurement, and it must be remembered that all measurements currently available can only sample concrete strength. Therefore, both the probability of inaccurate measurements and the extent to which the sampling of data is representative of the whole must be considered in establishing nominal resistance.

2.2.2 Detecting and Measuring Deterioration

The following outline covers both circumstantial and hard evidence for each type of deterioration. In addition, questions are formulated which must be answered if this evidence is to be used more effectively for strength evaluation.

Results of Deterioration that Can Affect Resistance of Concrete Sections

I. Loss of cross-sectional area of steel reinforcement.
   A. Mechanism: corrosion of steel.
   B. Detectable circumstantial evidence.
      1. Excess chlorides in the concrete as indicated in tests.
      2. Presence of water as indicated by leaching.
      3. Reduction in alkalinity of cement as indicated by leaching.
      4. Exposure of steel due to concrete spalling, insufficient cover, or excessive cracking.
   C. Detectable hard evidence.
      1. Concrete spalling or excessive cracking of concrete along the line of a reinforcing bar due to the expansive action of corrosion.
      2. Rust stains.
      3. Visible scaling of an exposed reinforcing bar.
   D. Questions relating to strength evaluation.
      1. Can the increased risk of steel corrosion be determined for each of the items in "C"?
      2. In the absence of items listed in C, what is the maximum corrosion that can exist in the presence of items listed in B?
      3. What is the maximum extent of corrosion that can take place without exposing reinforcing steel?
      4. Can the items listed in C be related quantitatively to the percentage of steel area lost due to corrosion?

II. Loss of anchorage of the reinforcing steel.
   A. Mechanism: corrosion of steel at the steel to concrete interface—loss of contact area due to spalling.
   B. Detectable circumstantial evidence.
      1. All items previously mentioned for loss of rebar area due to corrosion.
   C. Detectable hard evidence.
      1. Wide longitudinal cracks along rebar line.
      2. Spalled concrete which exposes rebar.
   D. Questions relating to strength evaluation.
      1. How severe must bond loss be before it is more critical than steel area loss? Will it ever be a problem in a completely covered bar?
      2. How is corrosion of the rebar perimeter related to bond loss? Can this be quantified?
      3. How effective is a partially exposed bar in resisting bond stresses?

III. Stress concentration due to pitting of reinforcing steel.
   A. Mechanism: corrosion of Steel.
   B. Detectable circumstantial evidence.
      1. All items previously mentioned for loss of reinforcing steel area due to corrosion.
   C. Detectable hard evidence.
      1. All items previously mentioned for loss of reinforcing steel area due to corrosion.
   D. Questions relating to strength evaluation.
      1. Same questions as for loss of cross-sectional area of steel reinforcement.

IV. Loss of effective concrete cross-sectional area.
   A. Mechanism: fracturing of concrete.
   B. Detectable circumstantial evidence.
      1. Surface cracking of concrete.
      2. Undersurface delamination of concrete.
   C. Detectable hard evidence.
      1. Loss of concrete volume (spalling, scaling).
      2. Observation of action under load.
      3. Large cracks perpendicular to the direction of stress application that do not or cannot close.
   D. Questions relating to strength evaluation.
      1. When is concrete cracking sufficient to decrease the effectiveness of a cross section of a concrete member? Can the decrease be quantitatively related to the observed cracking?
      2. What are the effects of a spall or large cracks on the flow of stress within the concrete? Is stress concentration a problem?
      3. What are the effects of spalling or loss of section effectiveness on the response of the structure?

V. Reduction in concrete strength.
   A. Mechanism: poor quality of construction, inferior materials, chemical and physical degradation of the concrete matrix, and undesirable additives.
   B. Detectable circumstantial evidence.
      1. Evidence of poor construction practice (e.g., excessive shrinkage cracks, poor distribution of materials, excessive number of large air voids, etc.).
      2. Excessive cracking and/or popouts due to aggregate reaction or expansion.
      3. Leaching of cementitious materials.
      4. Discoloration.
      5. Weak concrete in similar structures of the same vintage in the area.
   C. Detectable hard evidence.
      1. Test results (core samples, Schmidt hammer, pullout, ultrasonic pulse velocity, etc.).
   D. Questions relating to strength evaluation.
      1. What is the reliability of existing test methods? Will the reliability increase when there are corroborating results from more than one test method?
      2. How important is concrete strength to service-ability?

2.3 IN-SITU TESTING

One factor affecting the load-resisting capability of a reinforced concrete bridge is concrete strength. However, it is generally difficult to determine the concrete strength of an existing structure, especially if it is older and its construction records are not available.

Most nondestructive, in-situ testing requires knowledge of either the original cylinder strength of the concrete or information on the mix, especially on the aggregate characteristics. As a further complication, none of the existing nondestructive methods have been developed to the point of providing a very high degree of accuracy. As stated by Chabowski and Bryden-Smith (*E.1*): "There is at present no completely satisfactory method of determining the in situ strength of concrete. The existing methods of establishing the strength directly by nondestructive testing methods gives only a broad indication of the quality of the concrete."

Application techniques also greatly affect the results. Also, significant variations in results can occur when they are obtained by different operators.

At this time, only core samples taken from the concrete in question give a close indication of its strength. Details on procedure for concrete core testing are contained in the ASTM C823 "Standard Practice for Examination and Sampling of Hardened Concrete in Construction." Core samples are not, however, always readily obtainable, and the rating engineer needs to be assisted by other methods. To assist him in selecting the one that best fits his needs, various methods will be discussed.

It should be realized at the beginning that most nondestructive test methods require knowledge of the concrete mix proportions, type of aggregate, age, curing, or other types of similar information. Because most experienced engineers can evaluate the concrete's quality fairly accurately without testing when given these types of information, most nondestructive test methods offer little to the solution of the problem of evaluating concrete strength.

If there is no construction data available (plans, concrete mix, aggregate source, strength), some insight into the structure's strength can be obtained by reviewing available information on similar structures constructed in the area at about the same time as the structure being evaluated. This is especially true for gathering information on the type of reinforcing steel that would have been used in the structure, its concrete design strength, and various codes under which it would have been designed and built.

Some of the nondestructive testing methods available at this time include the rebound hammer, ultrasonic pulse velocity, Windsor probe, and pull-out.

The most commonly used rebound hammer is the Schmidt N2. The hammer is essentially a hardness tester. An important fact to remember when using a rebound hammer, though, is that the type of aggregate used in the concrete, the roughness of the concrete's surface, and the age of the concrete are all significant variables affecting the accuracy of the results.

Samarin and Meynink describe the theory and pitfalls of using the ultrasonic pulse method on deteriorated concrete (*E.2*): "It must be remembered however that the theory is developed for a homogeneous, elastic and semi-infinite solid, whereas concrete is heterogeneous and isotropic, rheologically complex and always finite."

The pulse velocity method for strength determination is hampered by surface finish, internal cracking, and buried reinforcing steel. Also, for closer approximations of test and actual strength values, a statistically significant correlation has to have already been developed for the type of concrete being tested.

The Windsor probe (penetration probe gage testing—ASTM C803) is a proprietary, in situ, semi-nondestructive tool for testing the strength of concrete. A probe gage is driven by a power-activated device into the concrete, and its penetration is determined. The compressive strength is determined by correlating the kinetic energy with the depth of the probe gage's penetration. A severe limitation to using the Windsor probe on existing structures is the requirement that the Moh hardness rating (scratch test) of the course aggregate be known.

A method called "pull-out" appears to offer the most accurate measurement of concrete strength aside from core samples. The device to be pulled-out can either be cast in the concrete or implanted into mature concrete. It is the latter placement method that is applicable when determining the strength of older structures. This method is not totally nondestructive because it does result in a small, spalled concrete cone. Its value lies in the fact that, as with the extracted core sample method, it measures more directly the actual strength of the concrete. All of the other methods currently available for testing older structures rely on correlating indirect measurements with concrete strength.

The pull-out method has not been totally developed, but interest in it is high and its availability in the near future appears promising.

From the discussions on the various methods of measuring in-situ strength, it is quite clear that the first thing a person must do to make in-situ strength measurements is to thoroughly research the available methods. To insure usable results, one must become thoroughly acquainted with each method's application requirements, strong points, and limitations.

SECTION **3**

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APPENDIX F

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APPENDIX G

GLOSSARY AND NOMENCLATURE

GLOSSARY OF TERMS

allowable stress design or working stress design: A method of proportioning structures such that the computed elastic stress does not exceed a specified limiting stress.

calibration: A process of adjusting the parameters in a new standard to achieve approximately the same reliability as exists in a current standard or specification.

coefficient of variation: The ratio of the standard deviation to the mean of a random variable.

factor of safety: A factor by which a designated limit state force or stress is divided to obtain a specified allowable value.

format of design checking procedure: An ordered sequence of products of load factors and load effects that must be checked in the design process.

first-order second-moment (FOSM) reliability methods: Methods which involve (1) linearizing the limit state function through a Taylor series expansion at some point (first order); and (2) computing a notional reliability measure, the safety index, which is a function only of the means and variances (first and second moments) of the random variables rather than their probability distributions.

failure: A condition where a limit state is reached. This may or may not involve collapse or other catastrophic occurrences.

limit states: Criteria beyond which a structure or structural element is judged to be no longer useful for its intended function (serviceability limit state) or beyond which it is judged to be unsafe (ultimate limit state).

limit states design: A design method that aims at providing safety against a structure or structural element being rendered unfit for use.

load effect: The force in a member or an element (axial force, shear force, bending moment, torque) due to the loading.

load factors: A factor by which a nominal load effect is multiplied to account for the uncertainties inherent in the determination of the load effect.

load and resistance factor design: A design method that uses factors and resistance factors in the design format.

nominal load effect: Calculated using a nominal load; the nominal load frequently is defined with reference to a probability level, e.g., 50-year mean recurrence interval wind speed used in calculating the wind load.

nominal resistance: Calculated using nominal material and cross-sectional properties and a rationally developed formula based on an analytical and/or experimental model of limit state behavior.

probability distribution: A mathematical law that describes the probability that a random variable will assume certain values; either a cumulative distribution function (cdf) or a probability density function is used.

probabilistic design: A design method that explicitly utilizes probability theory in the safety checking process.

probability of failure: The probability that the limit state is exceeded or violated.

reliability of survival (reliability): The probability that the limit state is not attained.

reliability index: A computed quantity defining the relative reliability of a structure or structural element.

resistance: The maximum load-carrying capacity as defined by a limit state.

resistance factor (or capacity reduction factor): A factor by which the nominal resistance is multiplied to account for the uncertainties inherent in its determination.

target reliability: A desired level of reliability in a proposed design method.

NOMENCLATURE

A	Cross-sectional area
A_i	Analysis random variable for dead load category i
A_s	Area of steel reinforcement
c	Depth of Whitney's compressive stress block
d	Distance from extreme fiber in compression to the centroid of steel reinforcement
D_i	Dead load intensity of dead load category i
DL	Dead load effect
E	Wheel line distribution width of slab bridges
$F_x(x)$	Cumulative probability distribution function of random variable x
$f_x(x)$	Probability density function of random variable x
$f_{RQ}(r,q)$	Joint probability density function of random variables R and Q
f_y	Yield stress level of steel reinforcement
$\mathbf{G}$	Gradient vector of the limit state function g
g	Limit state function, or safety margin, or distribution random variable
$\bar{g}$	Mean value of g
H	Multiple presence random variable
$h_T(t)$	Hazard function during $[o,t]$
I	Impact factor
L (or SL)	Span length
LL	Live load effect
$L_T(t)$	Reliability during $[o,t]$
N_R	A correction random variable
P_F	Probability of failure
Q	Load effects
R	Resistance
RF	Rating factor

r	Superstructural condition rating
r_M	Longitudinal moment ratio
S	Girder spacing
T	Random variable indicating the time to the first failure event
V_R	Coefficient of variation of R
W	Truck weight
β	Safety (reliability) index
β_o	Target safety index
γ_D	Dead load factor
γ_L	Live load factor
λ	Wavelength of the roadway roughness
δ_R	Bias coefficeint of R
ϕ	Capacity reduction factor (or resistance factor)
σ_g	Standard deviation of g

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