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Development of Comprehensive Bridge Specifications and Commentary

This NCHRP digest details the development of the AASHTO LRFD Bridge Design Specifications. The work was performed under NCHRP Project 12-33, "Development of Comprehensive Bridge Specifications and Commentary," and conducted by Modjeski and Masters, Inc., Mr. John M. Kulicki, Principal Investigator.

1.0 HISTORICAL DEVELOPMENT

1.1 PROJECT BACKGROUND

In the late 1970s, the Ontario Ministry of Transportation and Communication, now known as the Ministry of Transportation, decided to develop its own bridge design specification, rather than continue using the AASHTO *Standard Specifications for Highway Bridges*. In considering the basis for these new specifications, a decision was taken to base it on probabilistic limit states. A Code Control Committee, chaired by Mr. Paul F. Csagoly, P. Eng., began to develop background material on the variability of loads and the components that make up resistance, including basic variabilities, such as the dispersion of the values for yield strength of metals, compressive strength of concrete, and the variation of sizes in factory-made and field-made products. A major study to determine the statistical variation in vehicle weights and configurations was also completed. During that time, the basic process for calculating the statistical reliability of a bridge component, based on the mean values of the applied loads and the parameters that went into the determination of resistance, and the standard deviations of these values were also developed. In addition, a process for determining a combination of multipliers on load and resistance to achieve a level of reliability was developed.

In 1979, the first edition of the *Ontario Highway Bridge Design Code* (OHBDC) was released to the design community as North America's first calibrated, reliability-based limit state specification. Since that time, the OHBDC has been updated in 1983 and 1993 and rereleased. Very significantly, the code contained a companion volume of commentary.

As more and more U.S. engineers became familiar with the OHBDC, they recognized a certain logic in the calibrated limit states design and began to question whether the AASHTO Specifications should be based on a comparable philosophy of determining the safety of structures. Many research projects undertaken by the NCHRP, the National Science Foundation (NSF), and various states were bringing new information on bridge design faster than it could be critically reviewed and, where appropriate, adopted into the AASHTO Specifications. It was also becoming clear that the many revisions that had occurred to the AASHTO Specifications had resulted in numerous inconsistencies and the appearance of a patchwork document.

CONTENTS

In spring 1986, a group of state bridge engineers (or their representatives) met in Denver and drafted a letter to the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS) indicating their concern that the AASHTO Specifications were outdated. They also raised the concern that the Technical Committee structure, operating under the HSCOBS, was not able to keep up with emerging technologies. Presentations were made at two regional meetings to mixed reception. Nonetheless, the actions by this group led the way to the development of the Load and Resistance Factor Design (LRFD) Specification. The group members and their respective 1986 affiliations were: James E. Roberts, California DOT; H. Henrie Henson, Colorado Department of Highways; Paul F. Csagoly, Florida DOT; Ho Lum Wong, Michigan DOT; and Charles S. Gloyd, Washington DOT.

In July 1986, a group of state bridge engineers met with the staff of the NCHRP to consider whether a project could be developed to explore the points raised in the Denver letter. This led to NCHRP Project 12-28(7) "Development of Comprehensive Bridge Specifications and Commentary," a pilot study conducted by Modjeski and Masters, Inc. with Dr. John M. Kulicki as Principal Investigator. The following is a list of tasks for this project:

- Task 1 - Review the philosophy of safety and coverage provided by other specifications.
- Task 2 - Review AASHTO documents, other than the Standard Specifications, for their potential for inclusion into a standard specification.
- Task 3 - Access the feasibility of a probability-based specification.
- Task 4 - Prepare an outline for a revised AASHTO Specifications for Highway Bridge Design and commentary, and present a proposed organizational process for completing such a document.

The review of other specifications and the trends in the development of new specifications included work done in Canada (especially the Province of Ontario), Great Britain, the Federal Republic of Germany, and Japan. Personal contacts with practitioners and researchers in other countries provided information on the emerging directions of specification development and insight into what designers were doing to implement specifications, and, in some cases, what designers were choosing not to implement based on a perception of unnecessary complication. Information collected from these various sources indicated that most of these countries appeared to be moving in the direction of a calibrated, reliability-based, limit states specification.

Task 2 can best be summarized as a search for gaps and inconsistencies in the 13th Edition of the AASHTO *Standard Specifications for Highway Bridges*. "Gaps" were areas where coverage was missing; "inconsistencies" were internal conflicts, or contradictions, of wording or philosophy. Many gaps and inconsistencies were found; they are summarized in the list shown in Table 1.1-1.

With respect to Task 3 and the feasibility of using probability-based limit states design, a review of the philosophy used in a variety of specifications resulted in three possibilities, two of which are already included in the current specification. They are as follows:

1. Allowable stress design that treats each load on the structure as equal from the view point of statistical variability. A "common sense" approach may be taken to recognize that some combinations of loading are less likely to occur than others, e.g., a load combination involving a 160 km per hour wind, dead load, full shrinkage and temperature may be thought to be far less likely than a load combination involving the dead load and the full design live load. For example, in the 13th Edition and others, the former load combination was permitted to produce a stress equal to four-thirds of the latter.
2. Load Factor Design, in which a preliminary effort was made to recognize that the live load, in particular, was more highly variable than the dead load. This thought is embodied in the concept of using a different multiplier on dead and live loads, e.g., a load combination involving 130 percent of the dead load combined with the 217 percent of the live load, and requiring that a measure of resistance based primarily on the estimated peak resistance of a cross section exceed the combined load.
3. Reliability-based design, which seeks to take into account directly the statistical mean resistance, the statistical mean loads, the nominal or notional value of resistance, the nominal or notional value of the loads, and the dispersion of resistance and loads as measured by either the standard deviation or the coefficient of variation (i.e., the standard deviation divided by the mean). This process can be used directly to compute probability of failure for a given set of loads, statistical data, and the designer's estimate of the nominal resistance of the component being designed. Thus, it is possible to vary the nominal resistance to achieve a criteria that might be expressed in terms such as the

TABLE 1.1-1 Gaps and Inconsistencies

GENERAL FORMAT

- Division II
- Commentary
- Presentation Format

ANALYSIS AND DESIGN PHILOSOPHY

- Use of more Refined Design Methods for Girder Bridges
- Improved Slab Design
- Effective Flange Width
- Bridge Dynamics
- Foundation Design Methods
- The Current LFD Provisions in the AASHTO Specifications
- Curved Girder Bridges
- The Effect of Skew

ADDITIONAL LOADS

- The Live Load Model
- Thermal and other Environmental Loads
- Ship Collision
- Erection Engineering and Construction Loads
- Combination of Load

TYPES OF CONSTRUCTION NOT COVERED OR PARTIALLY COVERED

- Segmental Concrete Bridges
- Cable-Stayed Bridges
- Multi-Web Box Girder Bridges

PERFORMANCE OF MEMBERS AND SYSTEMS

- Modern Bearing Systems
- Features of Prestressed Concrete Design
  - Fatigue of Prestressed Girders
  - Shielded or Blanketed Strands
  - Design of Compression Members
  - Partial Prestressing
  - Prestress Losses
- Local Stress Requirements
- Time Dependent Concrete Properties
- Foundation Design for Lateral Loads
- Compression Plate Design
- Anchorage Zone Stresses
- Design for Shear and Torsion in Concrete Members by Space Truss Analogy
- United Treatment of Concrete Design
- Continuity Joints for Prestressed I-Beams Made Continuous for Live Load
- Horizontal Shear Requirements and Composite Sections
- Features of Steel Design
  - Carrying Capacity of Distinctly Unsymmetric Plate Girders
  - Splices in Overdesigned Members
  - Net Section Requirements for Built-up Members
  - K-Factors for Compression Members
  - Friction Joints
  - Riveted Construction
  - Sealing Requirements
- Deflection Criteria
- Metal Deck Systems
- Proprietary Wall Systems
- Details which are Sensitive to Distortion-Induced Fatigue
- Connection Design
- The BS5400 Fatigue Detail Catalog

component (or system) must have a probability of failure of less than 0.0001, or whatever variable is acceptable to society. Alternatively, the process can be used to target a quantity known as the "reliability index," which is somewhat, but not directly, relatable to the probability of failure. Based on this "reliability index," it is possible to reverse engineer a combination of load and resistance factors to achieve a specific reliability index.

Although some specifications are being developed in terms of the "probability of failure," it was generally agreed that it would be more appropriate to use the reliability index process to develop load and resistance factors. That way, design could proceed in a process directly analogous to load factor design as it appeared in the 13th Edition of the Standard Specifications.

In May 1987, the findings of NCHRP Project 12-28(7) were presented to the AASHTO HSCOBS outlining the information above and indicating that seven options appeared to be available for consideration:

- Option 1—Keep the Status Quo,
- Option 2—Table Consideration of LRFD for the Short Term,
- Option 3—Immediate Adoption of the OHBDC,
- Option 4—Replace Current Specifications with LRFD Immediately
- Option 5—Replace Current LFD with LRFD in the Near Term,
- Option 6—Develop LRFD for Evaluation Only, or
- Option 7—Develop LRFD as a Guide Specification.

A recommendation was made to develop a probability-based limit states specification, fill as many of the gaps and inconsistencies as possible, and develop a commentary to the specification. Under the direction of then Chair Robert Cassano of California, the Subcommittee directed the NCHRP to develop a project to complete this task. Thus NCHRP Project 12-33, entitled "Development of Comprehensive Specification and Commentary," was started in July 1988.

1.2 ORGANIZATION OF PROJECT

1.2.1 Research Team

A hierarchial structure was established consisting of a Principal Investigator and Co-Principal Investigator from Modjeski and Masters, Inc., a Code Coordinating Committee and 15 working groups called task groups. Additionally, an Editorial Committee was appointed and charged with the responsibility of assembling the information and keeping it technically consistent.

The original plan was to have the Code Coordinating Committee meet on a regular basis and adjudicate the technical content of the Specifications. While the Code Coordinating Committee met several times in the early part of the development of the specifications, it became apparent that to meet the schedule imposed on the project, the Editorial Committee would have to deal directly with the Task Group Chairs.

1.2.2 Project Schedule

The original plan called for three drafts, which were released and reviewed as follows:

1. The first draft was released in April 1990 and was totally uncalibrated. The primary intent was to show coverage and organization. This draft was released to the AASHTO Bridge Engineers, the FHWA, all members of the NCHRP Panel and Task Group Members, and several private authorities. All told, it was reviewed by about 250 engineers, because many of the departments of transportation circulated it to in-house experts. Approximately 4,000 comments were received concerning the first draft, all were read and reviewed, and many were discussed with Task Group Chairs or sent directly to them. Many of the comments were included in the second draft, but there was no written response to the questions.
2. A second draft was released in late April 1991 to the same group of people. Additionally, it was noted at several regional and national conferences on bridge engineering that all interested parties could obtain a copy of the draft specifications at their cost, and that they would be free to submit review comments. This second draft contained a preliminary set of load and resistance factors, which changed relatively little in subsequent drafts. Approximately 6,000 comments were received for this draft and were processed as outlined previously.
3. The third draft was submitted in April 1992 and was reviewed in the same process that was used for the second draft. For this draft, about 2,000 comments were received and they were processed as described previously.

After reviewing the third draft, the NCHRP Project Panel determined that the specifications were approaching a draft that could be considered for a ballot item, but that additional work would be worthwhile and would reduce modifications needed in the future. Accordingly, the project was extended to include a fourth draft, whose scope included the following items:

- Continue to review the distribution factors developed under NCHRP Project 12-23, which were included in the proposed specification;
- Continue to refine calibration;
- Consider further the need for special short-span live loads;
- Further refine and verify the proposed strip-width method for calculating moments and deck slabs;
- Develop an index to the specification;
- Convert to the SI system of measurement;
- Develop further trial designs; and
- Complete the text for the fourth draft.

This fourth draft was submitted in March 1993 and was accepted as a ballot item at the May 1993 meeting of the HSCOBS.

One of the most valuable features of the process of developing these specifications was two rounds of trial designs. In 1991, and again in 1992, various states and industry groups volunteered to do comparative designs using the 14th Edition of the Standard Specifications and the LRFD Specification. Additionally, interested industry groups also organized their own series of trial designs and contributed information and critiques based on that work. Fourteen states and several industry groups participated in the initial 1991 designs, and 22 states and several industry groups worked on the 1992 set. As would be expected, the 1992 set were more complete and included 9 slab bridges, 20 concrete beam bridges, 9 steel beam and girder bridges, 1 truss, 1 segmental concrete bridge, 2 wood bridges and 5 culverts, and a series of retaining wall designs. The designs included substructure, superstructure, and pile and spread footing foundations. Additionally, a comprehensive set of prestressed beam bridges were evaluated by industry and contributed to the project.

These two series of trial designs achieved several important objectives:

- They exposed areas where further development of load models and resistance formulations were necessary, and where further calibration was advisable.
- They demonstrated that the specification, though considerably longer and more comprehensive than the Standard Specifications, was nonetheless readable and workable.
- They pointed to numerous areas where improvements and clarification in the wording could be made.
- They vastly broadened the base of practicing engineers who were becoming conversant with the LRFD Specification.

All things considered, the two trial design series, which required supplementary meetings of the AASHTO HSCOBS, proved to be one of the most important steps in the development and adoption of the LRFD Specification.

1.3 PROJECT OBJECTIVES

There were several objectives in the development of this new specification. They may be summarized as follows:

- To develop a technically state-of-the-art specification, which would put U.S. practice at or near the leading edge of bridge design.
- To make the specification as comprehensive as possible and include new developments in structural forms, methods of analysis, and models of resistance.
- To the extent consistent with the thoughts described previously, to keep the specification readable and easy to use, bearing in mind that there is a broad spectrum of people and organizations involved in bridge designs.
- To keep specification-type wording and not to develop a textbook.
- To encourage a multidisciplinary approach to bridge design, particularly in the areas of hydraulics and scour, foundation design, and bridge siting.
- To place increasing importance on the redundancy and ductility of structures.

Many changes had to be made in the content and appearance of the Standard Specification to achieve the objectives outlined above. Areas of major changes are identified as follows:

- The introduction of a new philosophy of safety—LRFD.
- The identification of four limit states.
- The development of new load factors.
- The development of new resistance factors.
- The relationship of the chosen reliability level, the load and resistance factors, and load models through the process of calibration.
- The development of improved load models necessary to achieve adequate calibration, including a new live load model.
- Revised techniques for analysis and the calculation of load distribution.
- A combined presentation of plain, reinforced, and prestressed concrete.
- The introduction of limit state-based provisions for foundation design and soil mechanics.
- Expanded coverage on hydraulics and scour.
- Changes to the earthquake provisions to eliminate the seismic performance category concept by making the method of analysis a function of the importance of the structure.
- Inclusion of large portions of the Guide Specification for Segmental Concrete Bridge Design.
- Inclusion of large portions of the FHWA Specification for ship collision.
- Expanded coverage on bridge rails based on crash testing, with the inclusion of methods of analysis for designing the crash specimen.
- The introduction of the isotropic deck design process.
- The development of a parallel commentary.

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It was the underlying principle of NCHRP Project 12-33 to make as much use of existing research findings as possible. The project was not supposed to involve the development of new information, although some limited work was necessary to tie information together to make a comprehensive specification.

2.0 SUMMARY OF RELIABILITY CONSIDERATION

2.1 OVERVIEW OF A PROBABILITY-BASED SPECIFICATION

The investigation of probability-based limit states design started on a note of some skepticism regarding whether this philosophy was mature enough in its development to encompass the combination of art and science involved in bridge engineering. After considering the underlying principles of service load design, load factor design, and limit states design, it became apparent that of these three possibilities, probability-based limit states design was unquestionably the most comprehensive and rational way to proceed. Finally, for clarity, it was decided that the AASHTO version of limit states design would be termed Load and Resistance Factor Design (LRFD), like the AISC Specification.

A consideration of probability-based reliability theory can be simplified considerably by initially considering that natural phenomena can be represented mathematically as normal random variables, as indicated by the well-known bell-shaped curve (see Figure 2.1-1). Use of this assumption leads to closed form solutions for areas under parts of this curve, which can conveniently answer the following questions:

- What percentage of the total number of values fall within a given range $Y \leq X \leq Z$? The answer to this question is given by the area bounded by the two values Y and Z, as shown in Figure 2.1-1.
- What percentage of the total values are such that $X \leq Z$? This is shown by the shaded area in Figure 2.1-2.

The first question and its statistical ramifications are already included in the AASHTO *Standard Specifications for Highway Bridges* in the fatigue design provisions of Article 10.3.1, in which the allowable fatigue stress ranges of the various categories were defined by the so-called "95 percent confidence" limits. To put this in the prospective of Figure 2.1-1, this is equivalent to saying that 95 percent of all the details tested at a given stress range failed at a number of cycles bounded by the values of Y and Z. In this context, the value of Y and the value of Z are each located two standard deviations on either side of the mean or average value.

The question illustrated by Figure 2.1-2 deals very explicitly with the problem of defining loads and resistance of members.

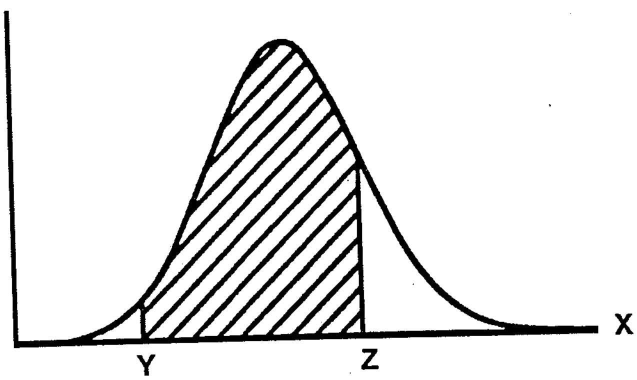

Figure 2.1-1 - Normal Distribution Curve Showing Distribution Bounded by the Values "Y" and "Z."

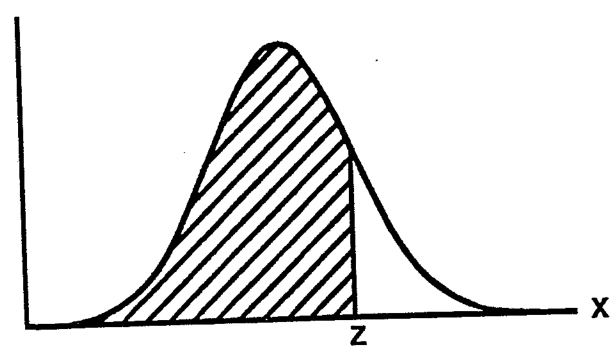

Figure 2.1-2 - Normal Distribution Curve Showing Portion of Distribution Less than or Equal to "Z."

If we now accept the notion that both load and resistance are normal random variables, we can plot the bell-shaped curve corresponding to each of them in a combined presentation dealing with distribution as the vertical axis against the value of load, Q, or resistance, R, as shown in Figure 2.1-3. The mean value of load and the mean value of resistance is also shown, as is a second value somewhat offset from the mean value, which is the "nominal" value, or the number that designers calculate the load or the resistance to be. The ratio of the mean value divided by the nominal value is called the "bias." The objective of a design philosophy based on reliability theory, or probability theory, is to separate the distribution of resistance from the distribution of load, such that the area of overlap, i.e., the area where load is greater than resistance, is tolerably small, say 1 in 10,000. The objective of a LRFD approach to a design specification is to be able to define load factors, shown as γ_{Q_n} in Figure 2.1-3, and resistance factors, shown as ϕ_{R_n} in Figure 2.1-3, in a way that forces the relationship between the resistance and load to be such that the area of overlap is less than or equal to the value that a code-writing body accepts. Note in Figure 2.1-3 that it is the nominal load and the nominal resistance, not the mean values, that are factored.

A conceptual distribution of resistance minus load, combining the individual curves discussed above, is shown in Figure 2.1-4, where the area of overlap from Figure 2.1-3 is shown as negative values, i.e., those values to the left of the origin. It now becomes convenient to define the mean value of resistance minus load as some number of standard deviations, β, from the origin. The variable β is called the "reliability index." The problem with this presentation is that the variation of the quantity resistance-minus-load, is not explicitly known. Much is already known about the variation of loads by themselves or resistances by themselves, but the difference between these has not yet been quantified. However, from probability theory, it is known that if load and resistance are both normal and random variables then the standard deviation of the difference is:

$$\sigma_{(R-Q)} = \sqrt{\sigma_R^2 + \sigma_Q^2} \qquad (2.1\text{-}1)$$

Given the standard deviation, and considering Figure 2.1-4, we can now define the reliability index, β, as:

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad (2.1\text{-}2)$$

Comparable closed-form equations can also be established for other distributions of data, e.g., log-normal distribution. It is very important to realize that a "trial-and-error" process is available for solving for β when the variable in question does not fit one of the already existing closed-form solutions.

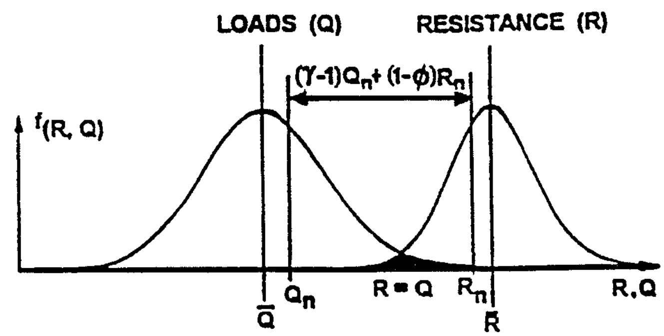

Figure 2.1-3 - Separation of Loads and Resistance.

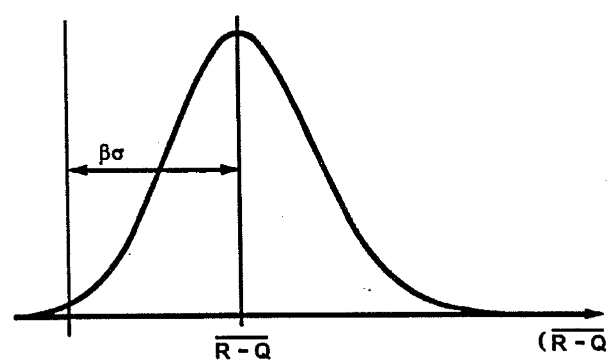

Figure 2.1-4 - Definition of Reliability Index, β.

In a reliability-based code in the purest sense, the designer is asked to calculate the value of β provided by a given design and then compare that to a code-specified tolerable value. A designer would require much knowledge of reliability theory to apply such a pure reliability-based code. Alternatively, through a process of calibrating load and resistance factors by trial designs, it is possible to develop load and resistance factors so that the design process looks very much like the existing Load Factor Design Methodology.

The process of calibrating load and resistance factors starts with Equation 2.1-2 and the basic design relationship; the factored resistance must be greater than or equal to the sum of the factored loads:

$$\phi R = Q = \Sigma \gamma_i x_i \qquad (2.1\text{-}3)$$

Solving for the average value of resistance yields:

$$\overline{R} = \overline{Q} + \beta \sqrt{\sigma_R^2 + \sigma_R^2} = \lambda R = \frac{1}{\phi} \lambda \Sigma \gamma_i x_i \qquad (2.1\text{-}4)$$

Using the definition of bias, indicated by the symbol λ, Equation 2.1-4, leads to the second equality in Equation 2.1-4. A straightforward solution for the resistance factor, ϕ is:

$$\phi = \frac{\lambda \Sigma \gamma_i x_i}{\overline{Q} + \beta \sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad (2.1\text{-}5)$$

Unfortunately, Equation 2.1-5 contains three unknowns, i.e., the resistance factor, ϕ, the reliability index, β, and the load factors, γ.

The acceptable value of the reliability index, β, must be chosen by a code-writing body. While not explicitly correct, we can conceive of β as an indicator of the fraction of times that a design criteria will be met or exceeded during the design life, analogous to using standard deviation as an indication of the total amount of population included or not included by a portion of the normal distribution curve. Using this analogy, a β of 2.0 corresponds to approximately 97.3 percent of the values being included under the bell-shaped curve, or 2.7 of 100 values not included. When β is increased to 3.5, for example, only 2 values in approximately 10,000 are not included.

Assuming that a code-writing body has established a value of β, Equation 2.1-5 still indicates that both the load and resistance factors must be found. One way to deal with this problem is to select the load factors and then calculate the resistance factors. This process has been used by several code-writing authorities.

The steps in the process are as follows:

- Factored loads can be defined as the average value of load, plus some number of standard deviation of the load, as shown as the first part of Equation 2.1-6 below.

$$\gamma_i x_i = \overline{x_i} + n \sigma_i = \overline{x_i} + n V_i \overline{x_i} \qquad (2.1\text{-}6)$$

Defining the "variance," V_i, as being equal to the standard deviation divided by the average value, leads to the second half of Equation 2.1-6. Using the concept of bias one more time, Equation 2.1-6 can now be condensed into Equation 2.1-7.

$$\gamma_1 = \lambda \left(1 + n V_i\right) \qquad (2.1\text{-}7)$$

Thus, it can be seen that load factors can be written in terms of the bias and the variance, as depicted in Figure 2.1-5. This gives rise to the philosophical concept that load factors can be defined so that all loads have the same probability of being exceeded during the design life. This is not to say that the load factors are identical, just that the probability of the loads being exceeded is the same.

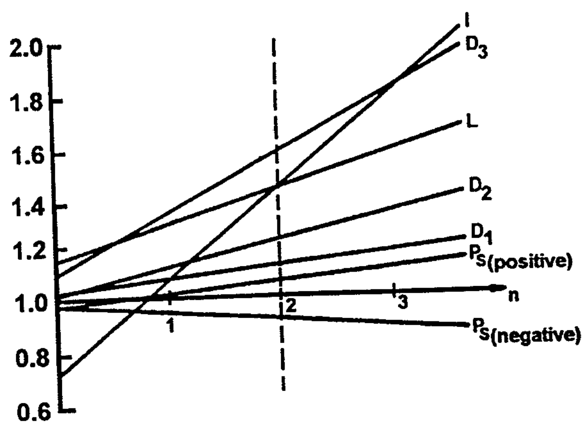

Figure 2.1-5 - Graphical Presentation of Equation 2.1-7 (Nowak and Lind, 1979).

- Using Equation 2.1-5, for a given set of load factors, the value of resistance factor can be calculated for various types of structural members and for various load components, e.g., shear, moment, etc., on the various structural components. Computer simulations of a representative body of structural members can be done, yielding a large number of values for the resistance factor.
- Resistance factors are then grouped by structural member and by load component to determine if they cluster around convenient values. If close clustering results, a suitable combination of load and resistance factors has been obtained.
- If close clustering does not result, a new trial set of load factors can be used and the process repeated until the resistance factors do cluster around a workable number of narrowly defined values.
- The resulting load and resistance factors taken together will yield reliability indices close to the target value selected by the code-writing body as acceptable.

The process above appears to be rather illusive. Fortunately, other jurisdictions had used this calibration process and found it to yield reasonable load and resistance factors, which was also the case in NCHRP Project 12-33. Figure 2.1-6 shows the dispersion of the reliability indices observed for bridges designed to the AASHTO Standard Specifications within the Province of Ontario. After defining load and resistance factors through the process outlined above, analysis of these same sets of bridges produced reliability indices clustered around the target value of 3.5, as shown in Figure 2.1-7. The reason that the values do not plot exactly on the horizontal straightline, indicated by a reliability index of 3.5, is that the resistance factors did not cluster at exactly the same number. Thus, when reasonable and conservative interpretations of the resistance factor values are used, the reliability indices will generally be above the target value and an unavoidable amount of scatter will still result. As can be seen in Figure 2.1-7, a handful of the comparative values were significantly below the target reliability index, and this indicated that additional design provisions were necessary for this particular group of bridges.

The chief advantages for a probability-based LRFD specification are as follows: (1) A more uniform level of safety throughout the system will result. (2) The measure of safety will be a function of the variability of loads and resistance. (3) Designers will have an estimate of the probability of meeting or exceeding the design criteria during the design life. (4) The potential exists to place all structural materials and methods of construction on equal footing. (5) A realistic rational framework for future development of the specification will be available. (6) Proponents of future changes in materials and construction techniques can be asked to provide the same measure of reliability that all current materials and construction methods will be asked to meet.

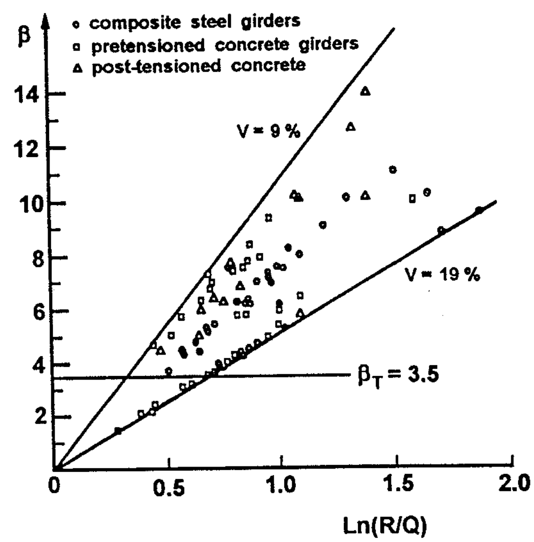

Figure 2.1-6 - Range of Reliability Indices Obtained Using AASHTO Standard Specifications (Nowak and Lind, 1979).

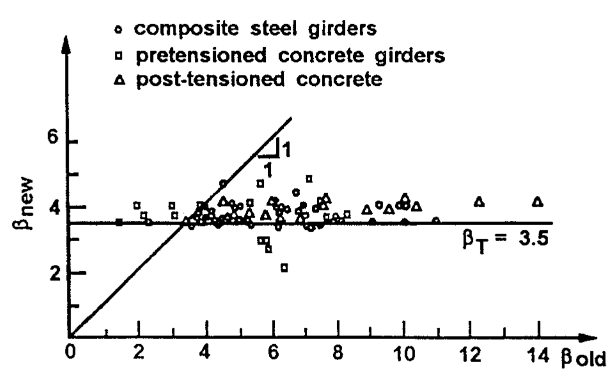

Figure 2.1-7 - Range of Reliability Indices Obtained with Reliability-Based OHBDC (Nowak and Lind, 1979).

There are certainly some disadvantages in basing a specification on this philosophy. Increased design effort will certainly result, as it is realistic to expect that a greater number of load and resistance factors will be available. However, the designer will need little or no knowledge of reliability theory. There will be start-up costs in reeducation and in the upgrade of design aids and design software. In summary, when considering the benefits, the obstacles did not seem to justify staying with the status quo.

The probability-based LRFD for bridge design was seen as a logical extension of the current Load Factor Design procedure. The Service Load Design does not recognize that various loads are more variable than others. The introduction of the Load Factor Design methodology brought with it the major philosophical change of recognizing that some loads are more accurately represented than others. The conversion to probability-based LRFD methodology could be thought of as a mechanism to more systematically and rationally select the load and resistance factors than was done with the information available when Load Factor Design was introduced.

Consideration was given to the impact that probability-based LRFD would have on the design community. It was expressly not intended that a specification be developed that requires all design engineers to be well-grounded in probability or reliability theory. Rather, a specification was developed by this process that does not appear to be a radical departure from the current Load Factor Design provisions. As previously stated, there are more individual load and resistance factors, but the process of computing a combined load group is basically similar to Load Factor Design. Likewise, the process of computing the resistance of a structural member is based on principles not radically different from those in the Standard Specifications, although, where possible, the design requirements are upgraded to the current state of the art.

2.2 OVERVIEW OF THE CALIBRATION PROCESS

2.2.1 Outline of the Calibration Process

The following steps are the major phases of the calibration of the load and resistance factors for the LRFD Specification: (1) develop a database of sample current bridges, (2) extract load effects by percentage of total load, (3) develop a simulation bridge set for calculation purposes, (4) estimate the reliability indices implicit in current designs, (5) revise loads-per-component to be consistent with the LRFD Specification, (6) assume load factors, and (7) vary resistance factors until suitable reliability indices result. This outline assumes that suitable load factors are assumed. If the process of varying the resistance factors and calculating the reliability indices does not converge to a suitable narrowly grouped set of reliability indices, then the load factor assumptions must be revised. In fact, several sets of proposed load factors were investigated to determine their effect on the clustering of reliability indices.

2.2.2 Development of a Sample Bridge Database

Approximately 200 representative bridges were selected from various regions of the United States by requesting sample bridge plans from various states. The selection was based on structural-type material and geographic location to represent a full-range of materials and design practices as they vary around the country. Anticipated future trends were also considered by questionnaires sent to selected states. One hundred seven sets of plans were received from which the 200 representative bridges were selected. Obviously, some plan sets contained more than one bridge or the bridge contained several separable units. The list of structures provided by the state departments of transportation is given in Table 2.2.2-1.

For bridges selected from within this database, moments and shears were calculated for the dead load components, the live load, and the dynamic load allowance. Nominal or design values were calculated using the 1989 edition of the AASHTO *Standard Specifications for Highway Bridges*. The statistically projected live load and the notional values of live load force effects were calculated. Resistance was calculated in terms of moment and shear capacity. For each structure, both the nominal design resistance, indicated by the cross section shown on the plans, and the minimum actual required resistance, according to the 1989 AASHTO Specifications, were developed. Generally speaking, the nominal resistance is larger than the minimum value, because standard available sizes of plates and rebar and other components that comprise resistance do not exactly meet the theoretical requirement. Additionally, a phenomenon known as the "designers bias" is implicit in actual designs. This factor results from the tendency of designers to "bump up" a given component to the next available commercial size.

TABLE 2.2.2-1 Selected Bridges

STRUCTURAL TYPE	REQUESTED SPAN (FT)	PROVIDED SPAN (FT)/STATE
	Steel, Simple Span	
Rolled Beams, Noncomposite	40 to 80	48 - PA 59 - MI 83 - PA
Rolled Beams, Composite	50 to 80	48 - PA 49 - PA 50 - PA 51 - PA 67 - PA 76 - PA 80 - PA 86 - PA
Plate Girder, Noncomposite	100 to 150	78 - PA 100 - PA
Plate Girder, Composite	100 to 180	103 - MI 109 - PA 122 - MI
Box Girder	100 to 180	None
Through-truss	300 to 400	300 - PA 303 - PA 311 - PA 397 - PA
Deck Truss	200 to 400	200 - NY 250 - NY 300 - NY 400 - NY
Pony Truss	150	100 - OK 103 - PA 300 - PA
Arch	300 to 500	360 - NY 436 - NY 630 - NY 730 - NY
Tied Arch	300 to 600	535 - NY
	Steel, Continuous Span	
Rolled Beams	50-65-50 to 80-100-80	74-60 - PA 85-80-85 - MI 76-96-80-60 - PA
Plate Girder	100-120-100	190-180 - MI 120-150-120 - MI 200-200-200 - KY 300-300-300-300 - KY 195-195-195-195 - KY 200-200-200-200-200-200 - KY
Box Girder	100-120-100 to 300-400-300	103-103-103 - MD 123-123-123 - MD 142-150-103 - MD 122-162-122 - IL 116-138-138-138-116 - IL 150-167-175-175-167-150 - IL
Through-truss	400	None
Deck Truss	400	None
Tied Arch	300-500	None

STRUCTURAL TYPE	REQUESTED SPAN (FT)	PROVIDED SPAN (FT)/STATE
Reinforced Concrete, Simple Span		
Slab	20 to 40	30 - OK
T-beam	40 to 80	40 - IL 40 - OK 43 - IL 50-50 - OK 60 - IL
Arch-barrel	40	None
Arch-rib	60	None
Reinforced Concrete, Continuous Span		
Slab, Two-span	30-30 40-40	None
Slab, Three-span	25-25-25	None
Solid Frame	40	40 - CA
T-beam, Frame	55	None
T-beam, Two-span	50-50 0-70	62-62 - CO 71-71 - CO
T-beam, Three-span	40-50-40 to 50-70-50	38-50-38 - TN 40-51-40 - TN 0-51-40 - TN 46-56-39 - TN 47-65-47 - TN 53-73-53 - TN 50-71-42 - TN
Arch		None
Box, Three-span	60-80-60 to 75-90-75	69-119-96 - MD
Prestressed Concrete, Simple Span		
Slab	30 to 40	None
Voided Slab	30 to 50	None
Double T	40 to 60	39 - CO
Closed Box CIP	125	None
AASHTO beam	50 to 100	76 - MI 76 - CO 102 - TX 102 - PA 105 - PA 103 - MI 110 - CO 118 - TX 120 - CO 130 - TX 138 - CO
Bulb	60 to 120	None

STRUCTURAL TYPE	REQUESTED SPAN (FT)	PROVIDED SPAN (FT)/STATE
Box Girder	80 to 120	74 - PA 74 - PA 82 - CA 95 - CA 102 - CA 104 - CA 116 - CA 118 - CA 120 - CA 125 - CA 125 - CA
Prestressed Concrete, Continuous Span		
Slab	35-35 to 40-50-40	None
Voided Slab	50-70-50 to 105-105	None
AASHTO Beam	80 to 110	None
Post-tensioned AASHTO Beam	100-100	None
Bulb		None
Box		65-65 - CA 87-85 - CA 93-86 - CA 103-102 - CA 107-102 - CA 110-160 - CA 118-101 - CA 200-200 - CA 60-80-60 - CA 69-82-59 - CA 75-90-75 - CA 69-92-69 - CA 76-90-76 - CA 71-85-71 - CA 66-85-52 - CA
Wood		
Saw Beam		18 - MN
Glulam Beam - Nailed		49-50-49 - MN
Glulam Beam - Doweled		None
Glulam Beam - Composite		None
Truss		50-100-100-49 - MN
Arch		None
Deck - Nailed		32-32-32 - MN
Deck - Composite		None
Deck - Prestressed Transversely		44 - MN
Deck - Prestressed Longitudinally		

2.2.3 Extraction of Load Effects

For each of the bridges in the database, the load indicated by the contract drawings was subdivided by the following characteristic components:

- The dead load due to the weight of factory-made components,
- The dead weight of cast-in-place components,
- The dead weight due to asphaltic wearing surfaces were applicable,

- The dead weight due to miscellaneous items,
- The live load due to the HS20 loading, and
- The dynamic load allowance or impact prescribed in the 1989 AASHTO Specifications.

Full tabulations for all these loads for the full-set of bridges in the database are presented in Nowak, 1993.

2.2.4 Development of New Notional Bridge Live Load Models

Since 1944, the AASHTO Specifications have used the well-known HS truck and lane loading. The largest HS truck specified was the 72-kip HS20-44, although some states had modified it into the 90-kip "HS25." In 1990, the Transportation Research Board released Special Report 225 entitled "Truck Weight Limits—Issues and Options," reviewing configurations of vehicles allowed by various states as exceptions to weight limits. Twenty-two representative vehicles configurations were extracted, the smallest and largest of which are shown in Figure 2.2.4-1.

A typical result of comparing bending moments in simple span and two-span continuous girders ranging from spans of 20 to 150 ft produced by the HS20 loading and the envelope of results produced by the 22 load configurations is shown in Figure 2.2.4-2 for which the following nomenclature applies:

M POS 0.4L	=	Positive Moment at 4/10 Point in Either Span
M NEG 0.4L	=	Negative Moment at 4/10 Point in Either Span
M SUPPORT	=	Moment at Interior Support
Mss	=	Centerline Moment in a Simply Supported Span

Figure 2.2.4-2 clearly indicates that the current design loading is not representative of the wide range of vehicles currently on U.S. highways.

Five candidate notional loads were identified early in the live load development for the AASHTO LRFD Bridge Specification:

1. A single vehicle, called the HTL57, weighing a total of 57 tons and having a fixed-wheel base and a fixed-axle spacing and weights shown in Figure 2.2.4-3. This vehicle is not unlike the design vehicle contained in the 1983 Edition of the OHBDC.
2. A "family" of three loads shown in Figure 2.2.4-4, consisting of a tandem, a four-axle single unit with a tridem rear combination, and a 3-S-3 axle configuration taken together with a uniform load, preceding and following that axle grouping.
3. A design "family," shown in Figure 2.2.4-5, called "HL93," consisting of subsets or combinations of a design tandem, similar to that shown in Figure 2.2.4-4, the HS20 truck given by the three-axle sequence and a uniform load of 640 lb per running ft of lane.
4. A slight variation of the combination of the HS vehicle and the uniform load, which involves an HS25 load, followed and preceded by a uniform load of 480 lb per running ft of lane, with the uniformly distributed load broken for the HS vehicle.
5. An equivalent uniform load in kips per ft of lane required to produce the same force effect as the envelope of the exclusion vehicles for various span lengths, as shown in Figure 2.2.4-6.

Consideration of Figure 2.2.4-6 indicates that each force effect would have to have its own equation for uniform load, and that the equation of the uniform load would be nonlinear in span length. In view of these complexities, an equivalent uniform load without concentrated loads was eliminated.

A comparison of the four remaining possible load configurations was developed for each of the following force effects:

- Centerline moment of a simply supported beam;
- Positive and negative moment at the 0.4L point of a two-span continuous girder, with two equal spans;
- Negative moment at the center pier; and
- End shear and shear at both sides of the interior support of a two-span continuous girder.

The results for centerline moments in a simply supported girder are shown in Figure 2.2.4-7; investigation of other force effects produced similar results.

Data are presented as ratios of a given force effect for each of these live load models divided into the corresponding force effect from the envelope of exclusion loads. Thus, a value of greater than 1.0 on the vertical axis indicates a situation in which the envelope of the exclusion vehicles produce more force effect than the design model under consideration. Ratios for the HS20 vehicle are also included for reference. Figure 2.2.4-7 indicates that the load model involving a combination of either a pair of 25-kip tandem axles and the uniform load, or the HS20 and the uniform load, seems to produce the best fit to the exclusion vehicles.

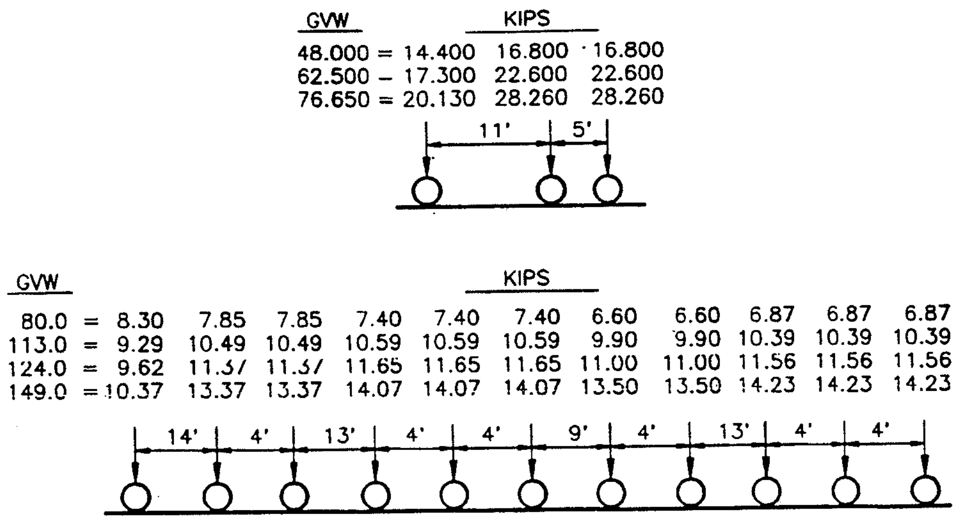

Figure 2.2.4-1 - Range of Exclusion Vehicles.

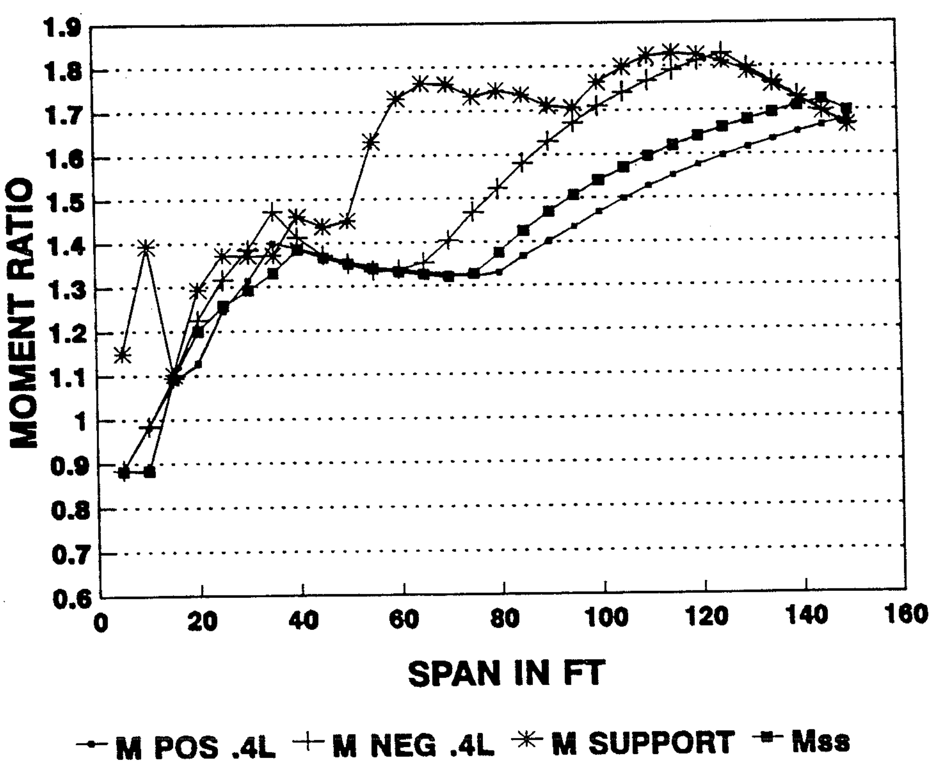

Figure 2.2.4-2 - Ratio of Moments - Exclusion/1989 AASHTO.

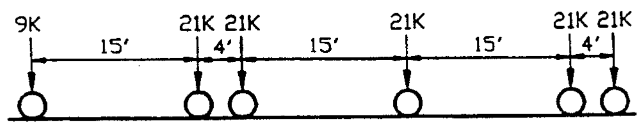

Figure 2.2.4-3 - HTL-57 (57 Tons).

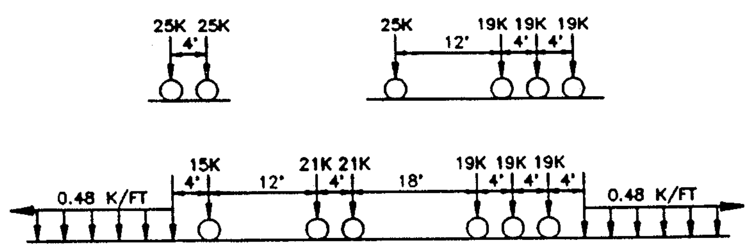

Figure 2.2.4-4 - Family of Three Loads.

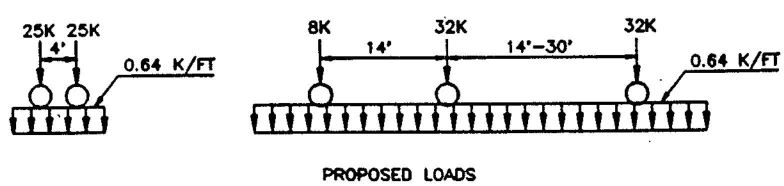

Figure 2.2.4-5 - HL93 Design Load.

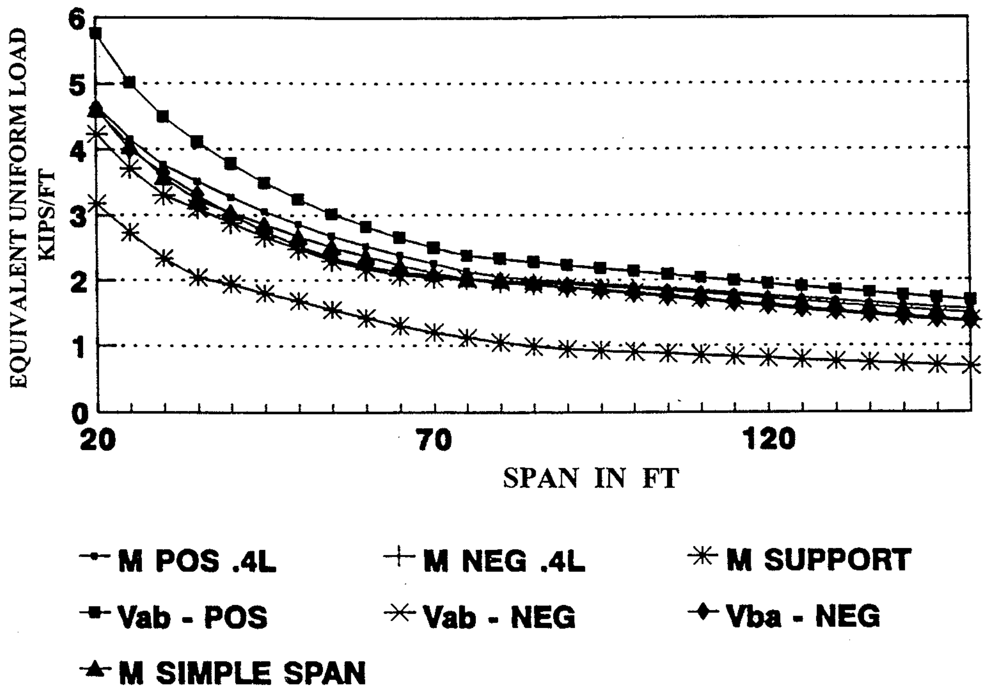

Figure 2.2.4-6 - Equivalent Uniform Load.

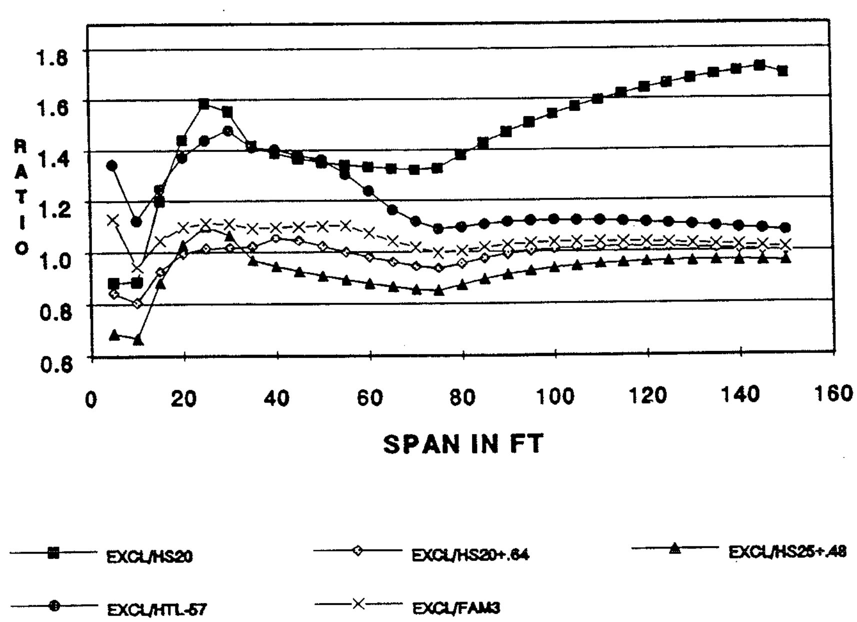

Figure 2.2.4-7 - Results for Simple Span Centerline Moments.

The same conclusion applies to other force effects investigated.

A summary of the moment ratios for the exclusion vehicles divided by either the tandem plus the uniform load or the HS20 truck plus the uniform load is shown in Figure 2.2.4-8. All moment-type force effects indicated previously are accounted for in Figure 2.2.4-8. It can be seen that the results for the force effects under consideration are tightly clustered, very parallel, and form bands of data which are essentially horizontal. The tight clustering of data for the various force effects indicates that one notional model can be developed for all of the force effects under consideration. The fact that the data are essentially horizontal indicates that both the model and the load factor applied to live load can be independent of span length. The tight clustering of all the data for all force effects further indicates that one live load factor will suffice. Almost identical results were obtained for shear.

In summary, the combination of the tandem with the uniform load and the HS20 with the uniform load, were shown to be an adequate basis for a notional design load in the LRFD Specification.

2.2.5 Development of the Simulated Bridge Set

Based on the relative amounts of the loads identified in the preceding article for each of the combination of span and spacing and type of construction indicated by the database, a simulated set of 175 bridges was developed, which comprises:

- Twenty-five noncomposite steel girder bridge simulations for bending moment with spans of 30, 60, 90, 120, and 200 ft, and for each of those spans, spacings of 4, 6, 8, 10, and 12 ft.
- Representative composite steel girder bridges for bending moments having the same parameters as those identified previously.
- Representative reinforced concrete T-beam bridges for bending moments having spans of 30, 60, 90, and 120 ft, with spacings of 4, 6, 8, and 12 ft in each span group.
- Representative prestressed concrete bridges for moments having the same span and spacing parameters as those used for the steel bridges.
- Representative steel girder bridges for shear having the same span and spacing parameters as those identified for bending moment.
- Representative reinforcing concrete T-beams for shear having the same span and spacing parameters indicated previously for bending moment.
- Representative prestressed concrete girder bridges for shear having the same span and spacing parameters as previously indicated for prestressed beams.

Full tabulations of these bridges and their representative amounts of the various loads are presented in Nowak, 1993.

2.2.6 Calculated Reliability Indices and Selection of Target Value

The reliability indices were calculated for each simulated and each actual bridge for both shear and moment. The range of reliability indices which resulted from this phase of the calibration process is presented in Figure 2.2.6-1. It can be seen that a wide range of values were obtained using the current specifications, but this was anticipated based on previous calibration work done for the OHBDC.

The most important parameters which determine the reliability indices for beam and girder bridges are the girder spacing and the span length. In general, reliability indices are higher for larger girder spacing, due to the conservatism of the S/N-type distribution factors used for conventional beam and girder bridges in the 1989 AASHTO Specifications.

These calculated reliability indices, as well as past calibration of other specifications, serve as a basis for the selection of the target reliability index, β_T. A target reliability index of 3.5 was selected for the OHBDC and is under consideration for other reliability-based specifications. A consideration of the data shown in Figure 2.2.6-1 indicates that a β of 3.5 is indicative of past practice. Hence, this value was selected as a target for the new calibration.

2.2.7 Load and Resistance Factors

2.2.7.1 Load Factors

The load and resistance factors are selected to yield reliability factors close to the target reliability, β_T. For each load component, X_i, load factor, γ_i, can be considered as a function of the bias factor (mean to

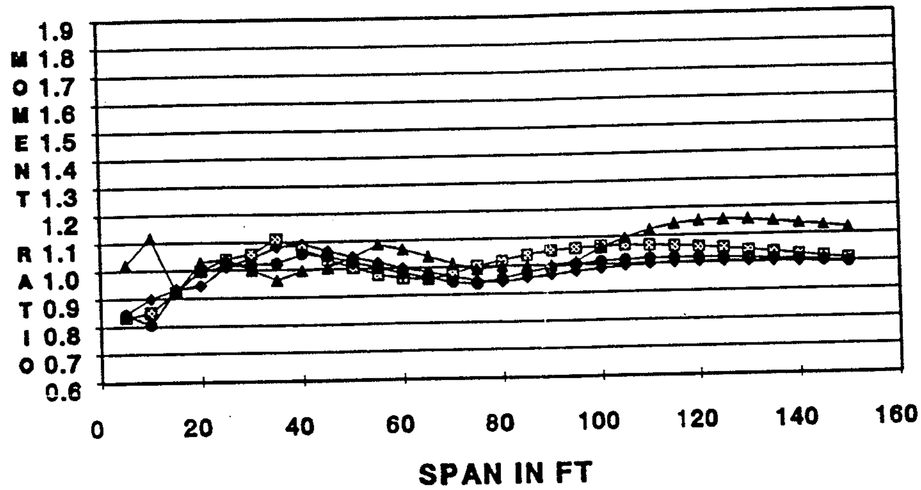

Figure 2.2.4-8 - EXCL/HS20 + 0.64 or Dual 25k Moment Ratio.

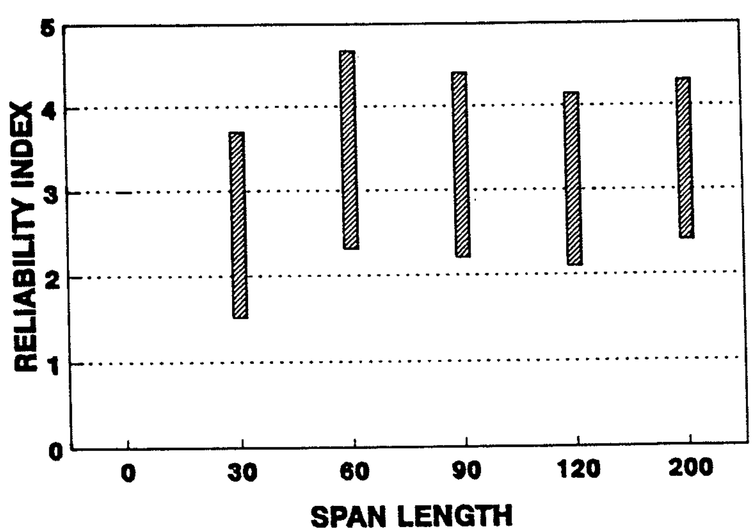

Figure 2.2.6-1 - Reliability Indices Inherent in the 1989 AASHTO Standard Specifications.

nominal ratio), λ_i, and the coefficient of variation, V_i

$$\gamma_i = \lambda_i \left(1 + kV_i\right) \qquad (2.2.7.1\text{-}1)$$

where k is a constant.

The relationship between the nominal (design) load, mean load, and factored load is shown in Figure 2.2.7.1-1. The shaded area in Figure 2.2.7.1-1 is equal to the probability of exceeding the factored load value.

Various sets of load factors, corresponding to different values of k, are presented in Table 2.2.7.1-2. The relationship is also shown in Figure 2.2.7.1-2.

Recommended values of load factors correspond to $k = 2$. For design simplicity, one factor is specified for D_1 and D_2, $\gamma = 1.25$. For D_3, weight of asphalt, $\gamma = 1.50$. For live load and impact, the value of load factor corresponding to $k = 2$ is $\gamma = 1.60$. However, a more conservative value of $\gamma = 1.75$ is used in the LRFD code.

2.2.7.2 Resistance Factors

The relationship between the nominal (design) resistance, mean resistance and factored resistance is shown in Figure 2.2.7.2-1. The shaded area in Figure 2.2.7.2-1 is equal to the probability of exceeding the factored resistance value.

The acceptance criterion in the selection of resistance factors is how close the calculated reliability indices are to the target value of the reliability index, β_T. Various sets of resistance factors, ϕ, are considered. Resistance factors used in the code are rounded off to the nearest 0.05. For each value of ϕ, the minimum required resistance, R_{LRFD}, is determined from the following equation:

$$R_{LRFD} = \left[\frac{1.25D + 1.50\,D_A + 1.75\,(L + I)}{\phi}\right] \qquad (2.2.7.2\text{-}1)$$

where D is dead load, except the weight of the asphalt surface, D_A. The load factors are equal to the recommended values from Section 2.2.7.1.

The calculations were performed using the load components for each of the 175 simulated bridges. For a given resistance factor, material, span and girder spacing, a value of R_{LRFD} is calculated using Equation 2.2.7.2-1. Then, for each value of R_{LRFD} and corresponding loads, the reliability index is computed.

2.2.7.3 Recommended Load and Resistance Factors

The recommended load factors are listed in Table 2.2.7.3-1 and recommended resistance factors are given in Table 2.2.7.3-2.

Reliability indices were recalculated for each of the 175 simulated cases and each of the actual bridges from which the simulated bridges were produced. The range of values obtained using the new load and resistance factors is indicated in Figure 2.2.7.3-1.

Comparing the values of reliability indices obtained with the 1989 AASHTO Specifications and the LRFD Specification indicates that a considerable improvement in the clustering of reliability index values has been obtained. This is a direct result of the integration of the load factor, resistance factor, accurate load models, and suitable resistance models. NCHRP Project 12-33 was not charged to make a wholesale readjustment of the inherent safety in the highway system. Selection of the target reliability index of 3.5 is consistent with that view. However, a fully consistent philosophy has been established for the specification, as indicated by the tightly clustered reliability index value shown in Figure 2.2.7.3-1. At any future time, AASHTO may decide that more or less safety (reliability) is desired. Should such a decision be made, a consistent means is now available to adjust load factors and resistance factors to achieve any increment in reliability.

One of the early concerns about the development of a probability-based LRFD Specification was that it would be used as a basis for reducing the strength of bridges. A comparison was made of the apparent resistance demands required by the 1989 AASHTO Specifications and the LRFD Specification. For purposes of comparison, the "demand" is taken as a sum of the factored loads divided by a resistance factor. Such a comparison is shown in Figure 2.2.7.3-2 for the simulated bridges used in the calibration process. This figure indicates that generally slightly more structure will be required based on the factored loads and the resistance factors alone. A total comparison would also have to include any advances in more realistic analysis methods and resistance formulations, which are included in the LRFD Specification. Some of these features have been, and more may be, adopted into the Standard Specifications.

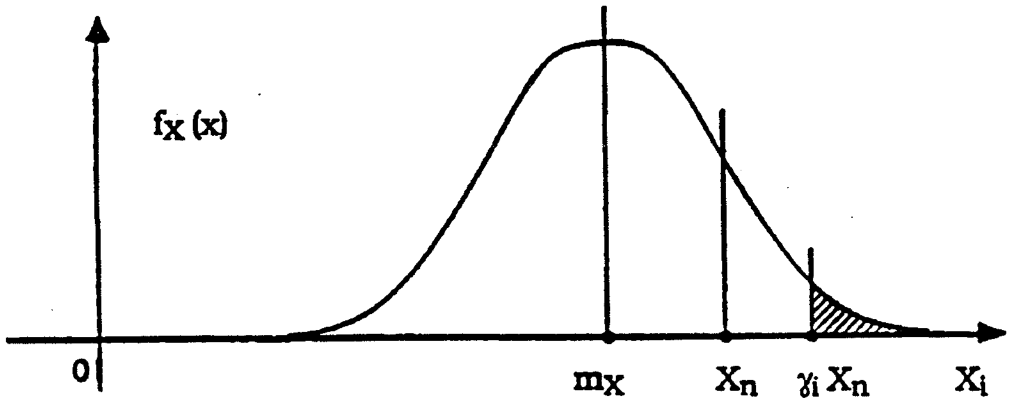

Figure 2.2.7.1-1 Probability Density Function, f_x (x), of Load Component, X_i; Mean Load, m_x, Nominal (design) Load, X_n, and Factored Load, γ_i X_n (Nowak, 1993).

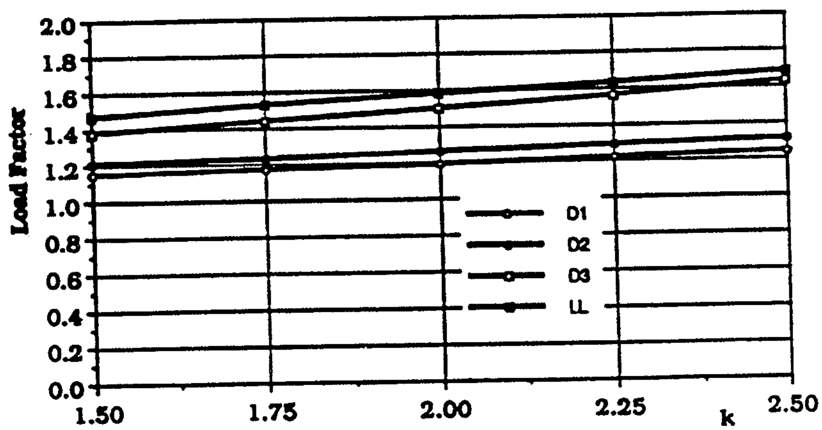

Figure 2.2.7.1-2 - Load Factors vs. k (Nowak, 1993).

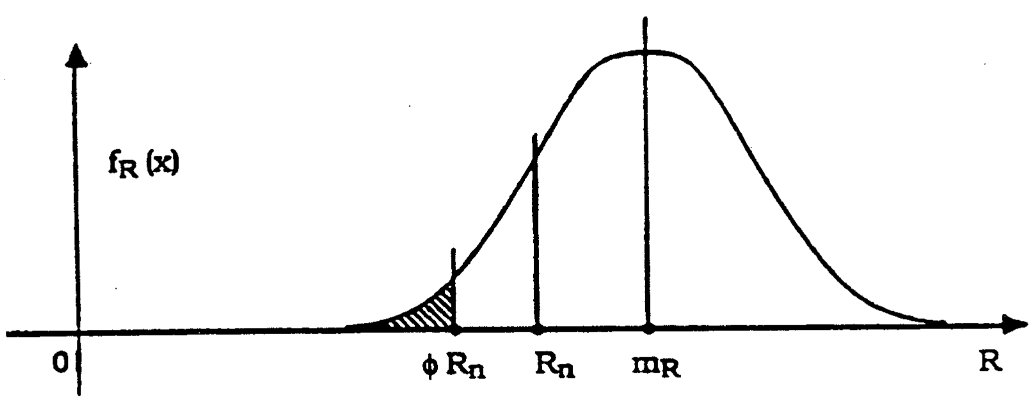

Figure 2.2.7.2-1 - Probability Density Function, $f_R(X)$, of Resistance, R; Mean Resistance, m_R, Nominal (design) Resistance, R_n, and Factored Resistance, ϕ R_n (Nowak, 1993).

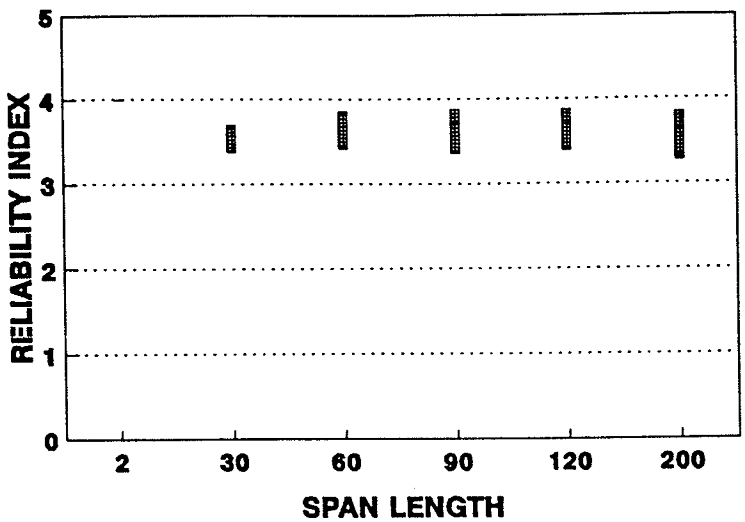

Figure 2.2.7.3-1 - Reliability Indices Inherent in LRFD Specification.

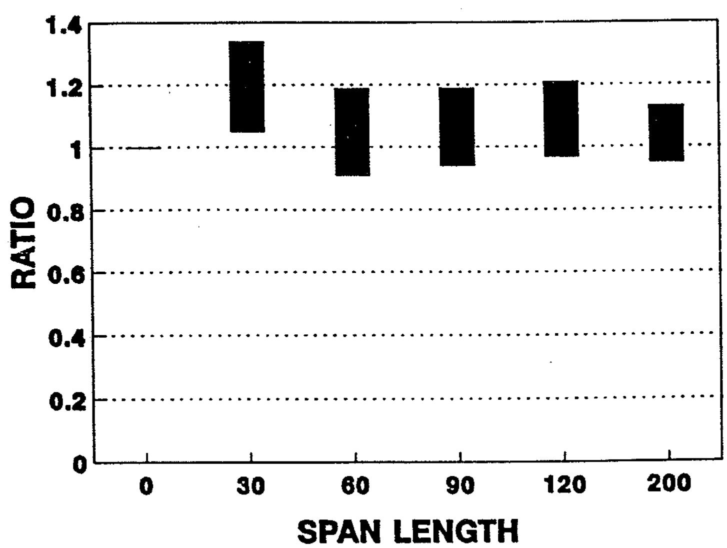

Figure 2.2.7.3-2 - Structural Demand - LRFD vs. Standard Specifications.

TABLE 2.2.7.1-1 **Parameters of Bridge Load Components**

LOAD COMPONENT	BIAS FACTOR	COEFFICIENT OF VARIATION
Dead Load, D_1	1.03	0.08
Dead Load, D_2	1.05	0.10
Dead Load, D_3	1.00	0.25
Live Load (with impact)	1.10-1.20	0.18

TABLE 2.2.7.3-1 **Recommended Load Factors**

LOAD COMPONENT	LOAD FACTOR, γ
Dead Load (except asphalt overlay)	1.25
Dead Load (asphalt overlay)	1.50
Live Load (including impact)	1.75

TABLE 2.2.7.1-2 **Considered Sets of Load Factors**

LOAD COMPONENT	k = 1.5	k = 2.0	k = 2.5
Dead Load, D_1	1.15	1.20	1.24
Dead Load, D_2	1.20	1.25	1.30
Dead Load, D_3	1.375	1.50	1.65
Live Load (with impact)	1.40-1.50	1.50-1.60	1.60-1.70

TABLE 2.2.7.3-2 **Recommended Resistance Factors**

MATERIAL	LIMIT STATE	RESISTANCE FACTOR, ϕ
Noncomposite Steel	Moment	1.00
	Shear	1.00
Composite Steel	Moment	1.00
	Shear	1.00
Reinforced Concrete	Moment	0.90
	Shear	0.90
Prestressed Concrete	Moment	1.00
	Shear	0.90

TABLE 2.2.7.2-1 **Considered Resistance Factors**

MATERIAL	LIMIT STATE	RESISTANCE FACTORS, ϕ	
		LOWER	UPPER
Noncomposite Steel	Moment	0.95	1.00
	Shear	0.95	1.00
Composite Steel	Moment	0.95	1.00
	Shear	0.95	1.00
Reinforced Concrete	Moment	0.85	0.90
	Shear	0.90	0.95
Prestressed Concrete	Moment	0.95	1.00
	Shear	0.90	0.95

3.0 SUGGESTED RESEARCH

Any effort to develop a specification on the scale of the AASHTO LRFD Specification for Highway Bridge Design has to reach a point where upgrading the technical content for new ideas must be stopped in order to finish the text and publish the document. This does not stop the tide of new ideas. Future changes must be expected. Some of the areas where continued development should be expected and encouraged include the following:

- Continue development of a database from which to project bridge loads. This is particularly true of live load for which it was initially thought that much information would be determined from weight-in-motion (WIM) studies. For various reasons, much of the WIM data were not directly usable in the development of the live load model. Coincident with NCHRP 12-33, the FHWA sponsored a project involving the instrumentation of bridges throughout the country. This project has generated large amounts of WIM data, and continuing efforts should be made to extract from that data information for further refinements of load models and analysis techniques.
- There is a continuing need to refine and verify foundation resistance and deformation. More work needs to be done on the large scale testing of foundations, the determination of group action, and the amount of movement which is acceptable. This latter point was a subject of considerable discussion during the development of NCHRP Project 12-33 as structural engineers and foundation engineers are apt to view this issue differently. Clearly, a multidisciplinary consensus is necessary.
- Continuing calibration of the service limit state, the fatigue and fracture limit state, and the extreme event limit states is needed.
- Further simplification of load distribution is warranted. This could take the form of further refinement of orthotropic plate models, which were considered in the development of the LRFD Specification, and suitable PC-type computer program to do a detailed analysis.
- The joint probability of load occurrence remains an issue of much interest. How much live load should be applied with an earthquake loading? Should other loads be applied with that same combination? How much more refinement of wind loadings can be done before site-specific studies are necessary? How should ice, wind, and other loads be combined? Should ship and vehicle collision be applied simultaneously with scour and earthquake and other loadings?
- Continued development of reliability theory should involve more emphasis on system rather than component reliability, the use of second order methods, improved methods of projecting the all important "tails" of measured data, the development of larger and more inclusive databases, and the inclusion of aging and deterioration models.

4.0 ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-33, Development of Comprehensive Bridge Specifications and Commentary, by Modjeski and Masters, Inc. in conjunction with more than 50 subcontractors.

Dr. John M. Kulicki, Senior Vice President and Chief Engineer, Modjeski and Masters, Inc., was the Principal Investigator and author of this report. Professor Dennis R. Mertz, Associate Professor of Civil Engineering, University of Delaware, was the Co-Principal Investigator and co-author of the report. Professor Mertz was previously an Associate at Modjeski and Masters, Inc.

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