CHAPTER **1**

INTRODUCTION

This study is concerned with the safety of bridges subjected to the combination of four types of extreme load events: (1) earthquakes, (2) winds, (3) scour, and (4) ship collisions. In addition, these loads will combine with the effects of truck live loads and the effects of dead loads. To ensure the safety of highway bridges under the combined effects of extreme events, this study develops load factors appropriate for inclusion in the AASHTO LRFD design-check equations. Structural reliability methods are used during the load factor calibration process in order to be consistent with the LRFD philosophy and to account for the large uncertainties associated with the occurrence of such extreme events, estimating their intensities, and analyzing their effects on bridge structures.

Analytical models to study the probability of single events and multiple load occurrences are available in the reliability literature and have been used to calibrate a variety of structural design codes ranging from buildings, to offshore platforms, to nuclear power plants, to transmission towers, to ships. This chapter describes the research approach followed during the course of this study, provides an overview of how current codes consider load combinations of extreme events, and describes how reliability analysis methods can be used to calibrate load factors.

1.1 LOAD COMBINATIONS IN CURRENT CODES

Historically, engineers used the one-third stress-reduction rule when combining extreme environmental events, such as wind or earthquake loads, with gravity loads. This rule, which dates to the early 20th century, has been discredited and replaced by load combination factors derived from reliability analyses. The reliability-based effort on load combinations has eventually lead to the development of the American National Standards Institute (ANSI) A58 Standard (ANSI A58 by Ellingwood et. al., 1980). The ANSI document set the stage for the adoption of similar load combination factors in current generation of structural design codes such as the *Manual of Steel Construction* (American Institute of Steel Construction [AISC], 1994), also called the *AISC Manual*; ACI 318-95 (American Concrete Institute [ACI], 1995); *AASHTO LRFD Bridge Design Specifications* (AASHTO, 1998); and many other codes.

The total number of load combinations covered in *AASHTO LRFD Bridge Design Specifications* includes five combinations to study the safety of bridge members for strength limit states, two load combinations for extreme load events, three load combinations for service load conditions, and finally the fatigue loading. The loads in these combinations include the effects of the permanent loads (which cover the weights of the structural components, attachments, and wearing surface) and earth pressure. The transient loads include those caused by the motion of vehicles and pedestrians; environmental effects such as temperature, shrinkage, and creep; water pressure; the effect of settlements and foundations; ice; collision; wind; and earthquakes.

With regard to the extreme loads of interest to this study, the current version of the AASHTO LRFD specifications (AASHTO, 1998) addresses them through the following combinations:

- Strength I Limit State: $1.25\,DC + 1.75\,LL$
- Strength III Limit State: $1.25\,DC + 1.40\,WS$
- Strength V Limit State: $1.25\,DC + 1.35\,LL + 0.4\,WS + 0.4\,WL$ (1.1)
- Extreme Event I: $1.25\,DC + 1.00\,EQ$
- Extreme Event II: $1.25\,DC + 0.50\,LL + 1.00\,CV$

DC is the dead load, *LL* is the live load, *WS* is the wind load on structure, *WL* is the wind load acting on the live load, *EQ* is the earthquake load, and *CV* is the vessel collision load. The dead load factor of 1.25 would be changed to 0.9 if the dead load counteracts the effects of the other loads.

The safety check involving *EQ* is defined in the AASHTO specifications as "Extreme Event I, Limit State." It covers earthquakes in combination with the dead load and a fraction of live load to "be determined on a project-specific basis," although the commentary recommends a load factor equal to 0.5. "Extreme Event II" considers 50% of the design live load in combination with either ice load, vessel collision load, or truck collision load. Wind loads are considered in "Strength Limit III," in which they do not combine with the live load, and in "Strength Limit V," in which they do combine with the live load. It should be noted that the design (nominal) live load for the strength limit states was based on the maximum 75-year truck weight combination and as such

should also be considered an extreme event situation. In most strength limit states, it is the live load factor that changes from 1.75 down to possibly 0.5 when the other extreme loads are associated with a load factor of 1.0. The one exception is the wind load: when used alone with the dead load, it is associated with a 1.4 load factor and a 0.4 factor when combined with the live load. The AASHTO LRFD specifications do not consider the possibility of combining wind, earthquake, and vessel collision, nor do they account for the effect of scour-weakened foundations when studying the safety under any of the transient loads. Also, as mentioned above, the LRFD commentary suggests that 0.50 *LL* may be added to the case when the earthquake load is acting, although the specifications state that "it should be determined on a project-specific basis." The nominal values for the loads correspond to different return periods depending on the extreme event. The return periods vary widely from 50 years for wind, to 75 years for live load, to 2,500 years for earthquakes, while the ship collision design is decided based upon an annual failure rate. Although not specifically addressed in the AASHTO LRFD specifications, the design for scour as currently applied based on FHWA HEC-18 models uses a 100-year return period as the basis for safety evaluation. It is noted that for most load events, the design nominal values and return periods are associated with implicit biases that would in effect produce return periods different than those "specified." For example, the HL-93 design live load is found to be smaller than the 75-year maximum live load calculated from the data collected by Nowak (1999) (see *NCHRP Report 368: Calibration of LRFD Bridge Design Code)* by about a factor of about 1.20. Similarly, the wind maps provided by ASCE 7-95 and adopted by the AASHTO LRFD specifications give a biased "safe" envelope to the projected 50-year wind speeds. These issues are further discussed in Chapter 2.

The load factors and load combinations specified in the current version of the AASHTO LRFD specifications follow the same trends set by other structural codes that were calibrated using reliability-based methods. For example, AISC's *Manual of Steel Construction* bases its load combinations on the provisions of ANSI A58. These provisions account for the dead load, live load (roof live load is treated separately), wind load, snow load, earthquake load, ice load, and flooding. The primary load combinations are 1.2 dead load plus 1.6 live load and 0.5 of snow or ice or rain load, another combination considers 1.2 dead load plus 1.6 snow or ice plus 0.5 live or 0.8 wind load. The 1.2 dead load is also added to 1.5 times the nominal earthquake load plus 0.5 live load or 0.2 times the snow load.

ANSI A58 has eventually been replaced by the ASCE 7-95 document (American Society of Civil Engineers [ASCE], 1995). The latter gives a long list of load combinations, some of which have been modified from earlier versions. For example, the primary combination for seismic load includes 1.2 times the dead load, 1.0 times the earthquake load, and 0.5 times the live load plus 0.2 times the snow load. This combination uses a seismic load factor of 1.0 based on the National Earthquake Hazards Reduction Program recommendations (NEHRP, 1997) rather than on the 1.5 initially proposed in the ANSI code. The magnitudes of the earthquake forces are based on ground accelerations with 10% probability of being exceeded in 50 years. This is equivalent to a 475-year return period. Recent work by NEHRP has supported increasing the earthquake return period up to 2,500 years, although the latest recommendation allows for a two-thirds reduction in the earthquake magnitudes for certain cases. The return period for the wind loads is set at 50 years.

The ACI Building Code's primary load combination is 1.4 dead load plus 1.7 live load. When wind is considered, the code includes a 1.7 wind load factor but then reduces the total load by 25%. Earth pressure is treated in the same manner as the live load. When fluid pressure is included, it is associated with a load factor of 1.4 and then added to 1.4 dead load and 1.7 live load. Loads due to settlement, creep, temperature, and shrinkage are associated with a load factor of 1.4 and when added to dead and live load, a 25% reduction in the total load is stipulated.

Other agencies that have implemented specifications for load combinations include the U.S. Nuclear Regulatory Commission (US-NRC) and the American Petroleum Institute (API). The US-NRC *Standard Review Plan* (US-NRC, 1989) for nuclear power plants uses load combinations similar to those provided in ASCE 7-95. Extreme event loads are each treated separately with a load factor of 1.0. The only difference between the US-NRC and the ASCE 7-95 provisions are the higher return periods imposed on the extreme event loads, particularly the seismic accelerations.

1.1.1 Summary

All of the codes listed above were developed using reliability-based calibrations of the load factors; yet, it is observed that the types of loads that are considered simultaneously and the corresponding load factors differ considerably from code to code. Some of the differences in the load factors may be justified because of variations in the relative magnitudes of the dead loads and the transient loads for the different types of structures considered (e.g., dead load to live load ratio). Also, in addition to assigning the load factors, an important component of the specifications is the stipulation of the magnitude of the design loads or the return period for each load. For example, if the design wind load corresponds to the 50-year storm with a load factor of 1.4, it may produce a similar safety level as the 75-year storm with a load factor of 1.0. One should also account for the hidden biases and conservativeness built into the wind maps and other load data. Finally, one should note that the safety level is related to the ratio of the load factor to the resistance factor. For example, if the load factor were set at 1.40 with a resistance factor of 1.0, it would produce a similar safety level as a load factor of 1.25 when associated with a resistance factor of 0.90.

Despite the justifications for the differences mentioned above, variations in the load factors may still lead to differences in the respective safety levels implicit in each code. These differences are mainly due to the nature of the currently used reliability-based calibration process, which uses a notional measure of risk rather than an actuarial value. In most instances, the code writers propose (1) load return periods and (2) resistance and load factors based on a "calibration" with past experience to ensure that new designs provide the *same* "safety levels" as existing structures judged to be "acceptably safe." Because the historical evolution of the AASHTO, ACI, AISC, and the other codes may have followed different paths, it is not surprising to see that the calibration process produces different load combinations and different load factors. In addition, most codes use the reliability of individual members as the basis for the calibration process. However, different types of structures have different levels of reserve strength such that for highly redundant structures, the failure of one member will not necessarily lead to the collapse of the whole structure. Therefore, in many cases, the actual reliability of the system is significantly higher than the implied target reliability used during the calibration process. Even within one system, the reliability levels of subsystems may differ—for example, for bridges, the superstructure normally formed by multiple girders in parallel would have higher system reserve than would the substructure, particularly when the substructure is formed by single-column bents.

In particular, the AASHTO LRFD specifications were developed using a reliability-based calibration that covered only dead and live loads (Strength I Limit State) using a target reliability index equal to 3.5 against first member failure. The other load combinations were specified based on AASHTO's *Standard Specifications for Highway Bridges* (AASHTO, 1996); on common practice in bridge engineering; or on the results obtained from other codes. Consequently, the current provisions in the AASHTO LRFD for load combinations may not be consistent with the provisions of the Strength I Limit State and may not produce consistent safety levels, as was the original intent of the specification writers.

In theory, when looking at the possibility of load combinations, an infinite number of combinations are possible. For example, the maximum combined live load and earthquake load effect might occur with the largest earthquake, or the second largest earthquake, and so forth, depending on the contribution of the live load in each case to the total effect. The purpose of the calibration process is to provide a set of design loads (or return periods) associated with appropriate load factors to provide an "acceptably safe" envelope to all these possible combinations. The term "acceptably safe" is used because absolute safety is impossible to achieve. Also, there is a trade-off between safety and cost. The safer the structure is designed to be, the more expensive it will be to build. Hence, code writers must determine how much implicit cost they are willing to invest to build structures with extremely high levels of safety.

The next section will review available methods to study the reliability of structures subjected to the combination of load events. This information will be essential to determine how and when extreme load events will combine and what load factors will provide a safe envelope to the risk of bridge failure due to individual load events and the combination of events.

1.2 COMBINATION OF EXTREME EVENTS FOR HIGHWAY BRIDGES

The extreme events of concern to this project are transient loads with relatively low rates of occurrences and uncertain intensity levels. Once an extreme event occurs, its time duration is also a random variable with varying length, depending on the nature of the event. For example, truck loading events are normally of very short duration (on the order of a fraction of a second to 2 to 3 seconds) depending on the length of the bridge, the speed of traffic, and the number of trucks crossing the bridge simultaneously (platoons). Windstorms have varying ranges of time duration and may last for a few hours. Most earthquakes last for 10 to 15 sec while ship collisions are instantaneous events. On the other hand, the effects of scour may last for a few months for live bed scour and for the remainder of the life of a bridge pier for clear water conditions. The transient nature of these loads, their low rate of occurrence, and their varying duration times imply that the probability of the simultaneous occurrence of two events is generally small. The exceptions are when one of the loads occurs frequently (e.g., truckloads); when the two loads are correlated (ship collision and windstorm); or when one of the loads lasts for long time periods (scour or, to a lesser extent, wind).

Even when two load types occur simultaneously, there is little chance that the intensities of both events will be close to their maximum lifetime values. For example, the chances are very low that the trucks crossing a bridge are very heavily loaded at the time of the occurrence of a high-velocity windstorm. On the other hand, because ship collisions are more likely to occur during a windstorm, the effect of high wind velocities may well combine with high-impact loads from ship collisions. Also, once a bridge's pier foundations have been weakened because of the occurrence of scour, the bridge would be exposed to high risks of failure because of the occurrence of any other extreme event. Of the extreme events of interest to this study, only ship collision and wind speeds are correlated events. Although scour occurs because of floods that may follow heavy windstorms, the time lag between the occurrence of a flood after the storm would justify assuming independence between wind and scour events.

For the purposes of this study and following current practice, it will be conservatively assumed that the intensity of any extreme event will remain constant at its peak value for the time duration of the event. The time duration of each event will be assumed to be a pre-set deterministic constant value. The occurrence of extreme load events may be represented

as depicted in Figure 1.1, which shows how the intensities may be modeled as constant in time once the event occurs although the actual intensities generally vary with time.

Methods to study the combinations of the effects of extreme events on structural systems have been developed based on the theory of structural reliability. Specifically, three analytical models for studying the reliability of structures under the effect of combined loads have been used in practical applications. These are (1) Turkstra's rule; (2) the Ferry-Borges (or Ferry Borges–Castanheta) model; and (3) Wen's load coincidence method. In addition, simulation techniques such as Monte Carlo simulations are applicable for any risk analysis study. These methods are intended to calculate the probability of failure of a structure subjected to several transient loads and have been used to calibrate a variety of structural codes ranging from bridges, to buildings, to offshore platforms, to nuclear power plants, to transmission towers, to ships. The next section describes the background and the applicability of these methods.

1.3 RELIABILITY METHODS FOR COMBINATION OF EXTREME LOAD EFFECTS

Early structural design specifications represented the load combination problem in a blanket manner by simply decreasing the combined load effect of extreme events by 25% (e.g., ACI) or by increasing the allowable stress by 33% (e.g., AISC–allowable stress design [ASD]). These approaches do not account for the different levels of uncertainties associated with each of the loads considered, nor do they consider the respective rates of load occurrence and duration. For example, these methods decrease the dead load effect by the same percentage as the transient load effects although the dead load is normally better known (has a low level of uncertainty), is always present, and remains constant with time. The use of different load factors depending on the probability of simultaneous occurrences of the loads is generally accepted as the most appropriate approach that must be adopted by codes in dealing with the combination of loads.

An accurate calibration of the load factors for the combination of extreme loads requires a thorough analysis of the fluctuation of loads and load effects during the service life of the structure. The fluctuations of the load effects in time can be modeled as random processes (as illustrated in Figure 1.1) and the probability of failure of the structure can be analyzed by studying the probability that the process exceeds a threshold value corresponding to the limit state under consideration. Each loading event can be represented by its rate of occurrence in time, its time duration, and the intensity of the load. In addition, for loads that produce dynamic responses, the effects of the dynamic oscillations are needed. Several methods of various degrees of accuracy and simplicity are available to solve this problem. Three particular methods have been used in the past by different code-writing groups. These are (1) Turkstra's rule, (2) the Ferry Borges–Castanheta model, and (3) Wen's load coincidence method (for examples, see Thoft-Christensen and Baker, 1982; Turkstra and Madsen, 1980; or Wen, 1977 and 1981). These load combination models can be included in traditional first order reliability method (FORM) programs. In addition, Monte Carlo simulations can be used either to verify the validity of the models used or to directly perform the reliability analysis. Results of the reliability analysis will be used to (1) select the target reliability levels and (2) verify that the selected load factors would produce designs that uniformly satisfy the target reliability levels. Below is a brief description of the three analytical methods. Chapter 2 will provide a more detailed

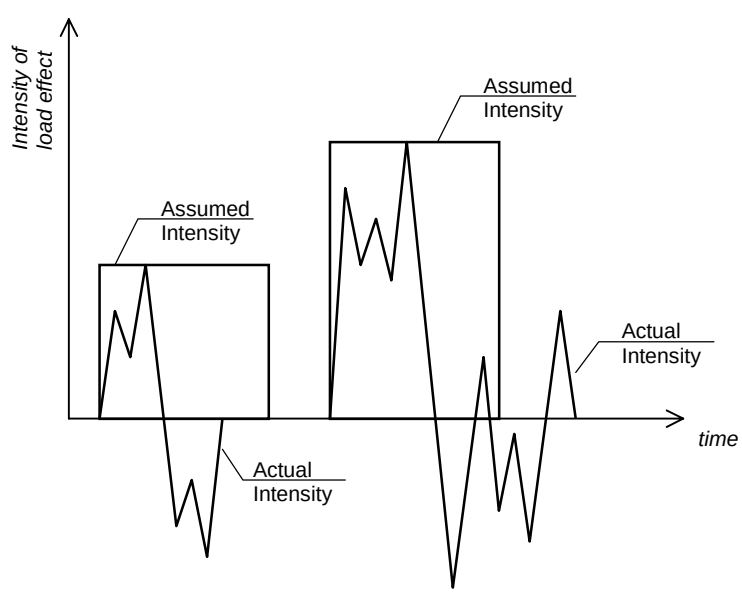

Figure 1.1. Modeling the effect of transient loads.

description of the method used in this study. Chapter 3 and the appendixes provide illustrations on the application of the selected method of analysis and the results obtained.

1.3.1 Turkstra's Rule

Turkstra's rule (Turkstra and Madsen, 1980) is a deterministic (non-random) procedure to formulate a load combination format for the design of structures subjected to the combined effects of several possible loading events. The rule is an over-simplification derived from the more advanced Ferry Borges–Castanheta model. Assuming two load types only (e.g., live load plus wind load), the intensity of the effect of Load 1 is labeled as x_1 and for Load 2, the intensity is defined as x_2. Both x_1 and x_2 are random variables that vary with time. Turkstra's rule may be summarized as follows:

- Design for the largest lifetime maximum value of Load 1 plus the value of Load 2 that will occur when the maximum value of Load 1 is on.
- Also design for the lifetime maximum of Load 2 plus the value of Load 1 that will occur when Load 2 is on.
- Select the larger of these two designs.

In practical situations, the value of the load that is not at its maximum is taken at its mean (or expected) value. Turkstra's rule can thus be expressed as follows:

$$X_{\max,T} = \max\begin{Bmatrix}[\max(x_1) + \bar{x}_2] \\ [\bar{x}_1 + \max(x_2)]\end{Bmatrix} \tag{1.2}$$

where

$X_{\max,T}$ = the maximum value for the combined load effects in a period of time T,
$\max(x_1)$ = the maximum of all possible x_1 values,
$\max(x_2)$ = the maximum of all possible x_2 values,
$\bar{x}_1$ = the mean value of x_1, and
$\bar{x}_2$ = the mean value of x_2.

The rule can be extended for more than two loads following the same logic. Although simple to use, Turkstra's rule is generally found to provide inconsistent results and is often unconservative (Melchers, 1999).

1.3.2 Ferry Borges–Castanheta Model for Load Combination

The Ferry Borges–Castanheta model is herein described for two load processes and illustrated in Figure 1.2 (Turkstra and Madsen, 1980; Thoft-Christensen and Baker, 1982). The model assumes that each load effect is formed by a sequence of independent load events, each with an equal duration. The service life of the structure is then divided into equal intervals of time, each interval being equal to the time duration of Load 1, t_1. The probability of Load 1 occurring in an arbitrary time interval can be calculated from the occurrence rate of the load. Simultaneously, the probability distribution of the intensity of Load 1 given that the load has occurred can be calculated from statistical information on load intensities. The probability of Load 2 occurring in the same time interval as Load 1 is calculated from the rate of occurrence of Load 2 and the time duration of Loads 1 and 2. After calculating the probability density for Load 2 given that it has occurred, the probability of the intensity of the combined loads can be easily calculated.

The load combination problem consists of predicting the maximum value of the combined load effect X, namely $X_{\max,T}$, that is likely to occur in the lifetime of the bridge, T. In the lifetime of the bridge there will be n_1 independent occurrences of the combined load, X. The maximum value of the n_1 possible outcomes is represent by

$$X_{\max,T} = \max_{n_1}[X] \tag{1.3}$$

The maximum value of x_2 that is likely to occur within a time period t_1 (i.e., when Load 1 is on) is defined as $x_{2\max,t1}$. Since Load 2 occurs a total of n_2 times within the time period t_1, $x_{2\max,t1}$ is represented by

$$x_{2\max,t1} = \max_{n_2}[x_2] \tag{1.4}$$

$X_{\max,T}$ can then be expressed as

$$X_{\max,T} = \max_{n_1}[x_1 + x_{2\max,t1}] \tag{1.5}$$

or

$$X_{\max,T} = \max_{n_1}\left[x_1 + \max_{n_2}(x_2)\right]. \tag{1.6}$$

The problem reduces then to finding the maximum of n_2 occurrences of Load 2, adding the effect of this maximum to the effect of Load 1, then taking the maximum of n_1 occurrences of the combined effect of x_1 and the n_2 maximum of Load 2. This approach assumes that x_1 and x_2 have constant intensities during the duration of one of their occurrences. Notice that x_1 or x_2 could possibly have magnitudes equal to zero. If the intensities of x_1 and x_2 are random variables with known probability distribution functions, then the probability distribution functions of the maximum of several events can be calculated using Equation 1.7.

The cumulative distribution of a single load event, Y, can be represented as $F_Y(Y^*)$. $F_Y(Y^*)$ gives the probability that the variable Y takes a value less than or equal to Y^*. Most load combination studies assume that the load intensities are independent from one occurrence to the other. In this case, the cumulative distribution of the maximum of m events that

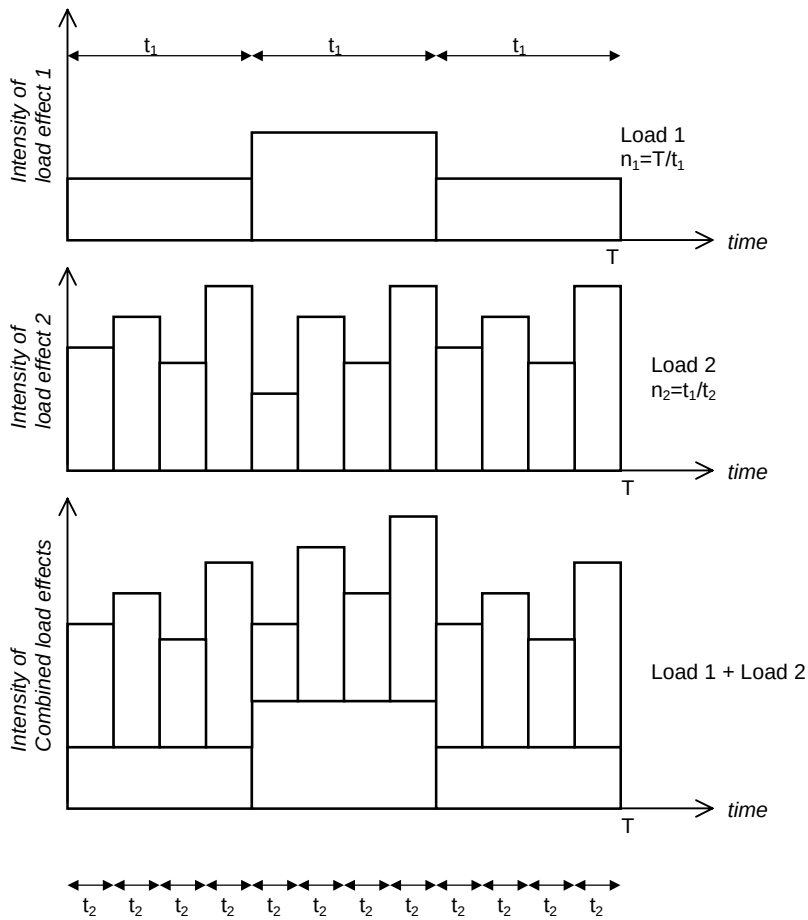

Figure 1.2. Illustration of combination of two load effects according to Ferry Borges–Castanheta model.

occur in a time period T can be calculated from the probability distribution of one event by

$$F_{Y_{\max},m}(Y^*) = F_Y(Y^*)^m \tag{1.7}$$

where m is the number of times the load Y occurs in the time period T.

Equation 1.7 is obtained by realizing that the probability that the maximum value of m occurrences of load Y is less than or equal to Y^* if the first occurrence is less than or equal to Y^*, *and* the second occurrence is less than or equal to Y^*, *and* the third occurrence is less than or equal to Y^*, and so forth. This is repeated m times, which leads to the exponent, m, in the right-hand-side term of Equation 1.7.

This approach, which assumes independence between the different load occurrences, has been widely used in many previous efforts of calibration of load factors for combined load effects. Although the Ferry-Borges model is still a simplified representation of the actual loading phenomenon, this model is more accurate than Turkstra's rule because it takes into consideration the rate of occurrence of the loads and their time duration. The Ferry Borges–Castanheta model assumes that the loads are constant within each time interval and are independent. However, in many practical cases, even when the intensities of the extreme load events are independent, the random effects of these loads on the structure are not independent. For example, although the wind velocities from different windstorms may be considered independent, the maximum moments produced in the piers of bridges as a result of these winds will be functions of modeling variables such as pressure coefficients as well as other statistical uncertainties that are correlated or not independent from storm to storm. In this case, Equation 1.7 has to be modified to account for the correlation between the intensities of all m possible occurrences. This can be achieved by using conditional probability functions; that is, Equation 1.7 can be used with pre-set values of the modeling factor that are assumed to be constant and then by performing a convolution over these correlated variables.

1.3.3 Wen's Load Coincidence Method

Wen's load coincidence method (Wen, 1977, 1981) is another method to calculate the probability of failure of a

structure subjected to combined loads. The load coincidence method is more complicated than the two previously described approaches, but it can be used for both linear and nonlinear combinations of processes, including possible dynamic fluctuations. The load coincidence method was found to give very good estimates of the probability of failure when compared with results of simulations. Unlike the two previously listed methods, which assume independence between two different load types, the load coincidence method accounts for the rate of occurrence of each load event and the rate of simultaneous occurrences of a combination of two or more correlated loads.

The Wen load coincidence method can be represented by the following equation (Wen, 1981):

$$P(E, T) \approx 1 - \left\{ \exp\left\{ -\left[\sum_{i=1}^{n} \lambda_i p_i + \sum_{i-1}^{n-1} \sum_{j=i+1}^{n} \lambda_{ij} p_{ij} + \cdots \right] T \right\} \right\} \quad (1.8)$$

where

$P(E,T)$ = the probability of reaching limit state E (e.g., probability of failure or the probability of exceeding a response level denoted by E) in a time period T;

n = the total number of load types each designated by the subscripts i and j;

λ_i = the rate of occurrence of load type i;

p_i = the probability of failure given the occurrence of load type i only;

λ_{ij} = the rate of occurrence of load types i and j simultaneously; and

p_{ij} = the probability of failure given the occurrence of load types i and j simultaneously.

The process can be extended for three or more loads. For example, with the combination of two load types such as the combination of wind load and live load, n would be 2 and load type i may represent the live load, and load type j may represent the wind load. The rate of simultaneous occurrences of live load and wind load, $\lambda_{ij,}$, would be calculated from the rate of occurrence of the wind, the rate of occurrence of the live load, and the time duration of each of the loads. The probability of failure of the bridge given that a wind and a live load occurred simultaneously is p_{ij}.

Wen's method is valid when the load intensities are pulse-like functions of time that last for very short duration. Wen's method is an extension of the one-load approach that assumes that failure events are independent and occur following a Poisson process. The loads are assumed to have low rates of occurrences, and failure events are statistically independent. Correlation between the arrival of two different load types is considered through the λ_{ij} term, while correlation between load intensities is considered through the proper calculation of the p_{ij} term. As with the case of the Ferry-Borges model, Wen's method does not account for the fact that there is correlation among the probability of failure of different events that may exist because of the presence of common modeling variables. Adjustments to consider this effect are more difficult to incorporate because of the Poissonian assumptions.

This study uses the Ferry-Borges model because it provides a more intuitive approach to the load combination problem than does the mathematical formulation of Wen's load coincidence method. The Ferry-Borges model is directly implementable in Level II reliability programs as demonstrated by Turkstra and Madsen (1980) and can be modified to account for the correlation from the modeling uncertainties using conditional probability distribution functions. Similarly, Monte Carlo simulations can be easily applied to use the Ferry-Borges model, including the consideration of correlation of load effects from different time intervals.

1.4 RELIABILITY-BASED CALIBRATION OF LOAD FACTORS

The calibration process performed for the Strength I Limit State of the AASHTO LRFD (see Equation 1.1) followed traditional methods available in the reliability literature. These methods are similar to those used during the development of AISC's *Manual of Steel Construction* (1994), ACI's *Building Code Requirements for Structural Concrete ACI 318-95* (1995), and many other recently developed structural design and evaluation codes. The purpose of the theory of structural reliability is to provide a rational method to account for statistical uncertainties in estimating the capacity of structural members, the effects of the applied loads on a structural system and the random nature of the applied loads. Since absolute safety is impossible to achieve, the objective of a reliability-based calibration is to develop criteria for designing buildings and bridges that provide acceptable levels of safety.

The theory of structural reliability is based on a mathematical formulation of the probability of failure. On the other hand, the absence in the reliability formulation of many potential risks such as human errors, major defects, deliberate overloads, and so forth implies that the calculated values of risk are only notional measures rather than actuarial values. In addition, the calibration process often uses incomplete statistical information on the loads and resistance of structural systems. This is due to the limited samples of data normally available for structural applications and because each particular structure will be subjected during its service life to a unique and evolving set of environmental and loading conditions, which are difficult to estimate a priori. As an example, for bridges subjected to vehicular loads, such unique conditions may include the effect of the environment and maintenance schedules on the degradation of the structural materials affecting the strength and the particular site-dependent truck weights and traffic conditions that affect the maximum live load. The truck weights and traffic conditions are related to the economic function of the roadway, present and future weight limits imposed in the jurisdiction, the level of

enforcement of such limits, the truck traffic pattern, and the geometric conditions including the grade of the highway at the bridge site as well as seasonal variations related to economic activity and weather patterns. Such parameters are clearly very difficult to evaluate, indicating that the probability of failure estimates obtained from traditional reliability analyses are only conditional on many of these parameters that are difficult to quantify even in a statistical sense.

Because the reliability-based calibration gives only a notional measure of risk, new codes are normally calibrated to provide overall levels of safety similar to those of "satisfactory" existing structures. For example, during the development of bridge codes, the specification writers would assemble a set of typical member designs that, according to bridge engineering experts, provide an acceptable level of safety. Then, using available statistical data on member strengths and loads, a measure of the reliability of these typical bridges is obtained. In general, the reliability index, β, is the most commonly used measure of structural safety. The reliability index, β, is related to the probability of failure, P_f, as shown in the following equation:

$$P_f = \Phi(-\beta) \tag{1.9}$$

where Φ is the cumulative standard normal distribution function.

During the calibration of a new design code, the average reliability index from typical "safe" designs is used as the target reliability value for the new code. That is, a set of load and resistance factors as well as the nominal loads (or return periods for the design loads) are chosen for the new code such that bridge members designed with these factors will provide reliability index values equal to the target value as closely as possible.

Moses and Ghosn (1985) found that the load and resistance factors obtained following a calibration based on "safe designs" are insensitive to errors in the statistical database as long as the same statistical data and criteria are used to find the target reliability index and to calculate the load and resistance factors for the new code. Thus, a change in the load and resistance statistical properties (e.g., in the coefficients of variation) would affect the computed β values. However, the change will also affect the β values for all the bridges in the sample population of "typical safe designs" and, consequently, the average β (which is also the target β). Assuming that the performance history of these bridges is satisfactory, then the target reliability index would be changed to the new "average," and the final calibrated load and resistance factors would remain relatively the same.

The calibration process described above does not contain any preassigned numerical values for the target reliability index. This approach, which has traditionally been used in the calibration of LRFD criteria (e.g., AISC, AASHTO), has led code writers to choose different target reliabilities for different types of structural elements or for different types of loading conditions. For example, in the AISC LRFD, a target β equal to 3.5 may have been chosen for the reliability of beams in bending under the effect of dead and live loads. On the other hand, a target β equal to 4.0 may be chosen for the connections of steel frames under dead and live loads, and a target β equal to 2.5 may be chosen for beams under earthquake loads. Such differences in the target reliability index clearly reflect the economic costs associated with the selection of βs for different elements and for different load conditions, as well as different interpretations of modeling variables.

As mentioned earlier, the failure of one structural component will not necessarily lead to the collapse of the structural system. Therefore, in recent years there has been increased interest in taking into consideration the safety of the system while designing new bridges or evaluating the safety of existing ones. The same approach followed during the calibration of new codes to satisfy reliability criteria for individual structural members can also be used for the development of codes that take into consideration the system effects. For example, Ghosn and Moses (1998) and Liu et al. (2001) proposed a set of system factors that account for the system safety and redundancy of typical configurations of bridge superstructures and bridge substructures. These system factors were calibrated to satisfy the same "system" reliability levels as those of existing "satisfactory" designs.

However, whether using system reliability or member reliability indexes for criteria, the discrepancies between the observed βs for different load types raise the following question: if the reliability index β for live load is 3.5 and for earthquake loads β is 2.5, what should be the target when combining live loads and earthquake loads? On the other hand, the discrepancies in the target reliabilities can be justified using a risk-benefit argument. For example, codes should tolerate a higher risk for the design of bridges (or structures) against a particular event if the costs associated with reducing this risk are prohibitive. This risk-benefit argument can be formalized using the expected cost of a bridge or any structure defined as

$$C_T = C_I + C_F P_f \tag{1.10}$$

where

C_T = the expected total cost,
C_I = the initial cost for building the structure,
C_F = the cost of failure, and
P_f = the probability of failure.

The initial cost of a structure increases as the safety level is increased. On the other hand, the probability of failure decreases as the safety level increases. Thus, to provide an optimum balance between risk and benefit, the target reliability index that should be used is the one that minimizes the expected total cost, C_T.

Although conceptually valid, the use of Equation 1.10 in an explicit form has not been common in practice because of the difficulties associated with estimating the cost of failure, C_F. Instead, code writers have resorted to using different target reliabilities for different types of members and loads based on calibration with previous acceptable designs. A risk-benefit approach is possible if the implicit costs of a structural failure can be extracted based on current designs. Aktas, Moses, and Ghosn (2001) have demonstrated the possibility of using the risk-benefit analysis described in Equation 1.10; however, more work is still needed in evaluating the relationship between C_I and C_F before the actual implementation of this approach during the development of design codes.

Because more research is needed before the implementation of Equation 1.10 becomes possible, this report will use a traditional method of calibrating the load factors to provide a target reliability index that will be extracted based on satisfying the same safety levels of existing "satisfactory designs." For example, because the reliability index obtained for bridges that satisfy current scour design procedures is relatively small compared with that obtained for bridges designed to satisfy the wind load requirements, the target reliability level that will be used for the combination of scour and wind will be chosen to be equal to the reliability level for bridges subjected to wind alone. This will ensure that bridges subjected to combinations of wind loads and scour will have safety levels as high as those that may be subjected to high winds. A similar approach will be used when combining scour with other extreme load events. For the load combinations involving earthquakes, the reliability index obtained from earthquakes alone will be used for target. This is because the engineering community has determined that current earthquake design procedures provide sufficient levels of safety in view of the enormous costs that would be implied with any increases in the current design procedures. By using the logic described in this paragraph, a risk-benefit analysis is implicitly used in a relatively subjective manner.

1.5 RESEARCH APPROACH

The objective of this study is to develop design procedures for the application of extreme load events and the combination of their load effects in *AASHTO LRFD Bridge Design Specifications.* The load events considered in this study will include live loads, seismic loads, wind loads, ship collision loads, and scour. The design procedures will consist of a set of load factors calibrated using reliability-based methods that are consistent with the reliability methodology of the AASHTO LRFD specifications. The purpose of the AASHTO specifications is to provide procedures to proportion bridge members such that they will provide sufficient levels of safety in their service lives for any possible load type or combination of loads. This means that the bridges are expected to satisfy an acceptable level of reliability that strikes a balance between safety and cost.

To achieve the objectives of the study, this project will first review the basic reliability methodology used during previous code calibration efforts. Models to study the reliability of bridges subjected to the effects of each load taken individually will be adopted from previous bridge code calibration efforts (for the live loads and ship collisions) and from models developed during the calibration of structural codes for buildings (for wind loads and earthquake loads). Because existing specifications for scour are not based on reliability methods, a scour reliability model will be developed for the purposes of this study.

Basic bridge configurations designed to satisfy the current AASHTO specifications will be analyzed to find the implicit reliability index values for bridge design lives of 75 years and for different limit states when these bridges are subjected to live loads, wind loads, earthquakes, vessel collisions, or scour. The limit states that will be considered include column bending, shearing failure, and axial failure of bridge columns, bearing failure of column foundations, and overtipping of single-column bents.

The reliability calculations will be based on assumed probability distributions for the random variables describing the effects of extreme load events on bridge structures. These assumed distributions will be based, where possible, on available statistical data pertaining to these random variables. To study the probability of load combinations, data on the rate of occurrence of each load event, the rate of occurrence of simultaneous load events, the magnitude of each load event, and the time duration of each load event will be required. This information will be assembled from the available reliability literature, as will be described in the next chapter. Also, available statistical data on the capacity of the bridge systems to resist the applied loads will be needed. This information is available from reports describing the calibration efforts of the AASHTO LRFD specifications and other available information on the behavior of structures and foundation systems (e.g., Nowak, 1999; Poulos and Davis, 1980).

In addition to studying the reliability of the typical bridge configurations under the effects of individual extreme events, the reliability analysis will be performed for these same bridge configurations when they are subjected to the combined effects of the extreme events under consideration. Reliability methods for combining the effects of several loads have been developed by researchers in the field of reliability, as described in Section 1.3. As explained earlier, the Ferry-Borges model for load combination will be used because it provides a simple and reasonable model that is easy to implement. Modifications on the classical Ferry-Borges model will be made to account for statistical and modeling uncertainties of time-dependent and time-independent random variables.

The reliability analysis will be performed for a number of bridge configurations and for different modes of failure.

Because the extreme load events being considered are mostly horizontal loads that primarily affect the substructure of bridges, the analysis will be performed for typical bridge bents subjected to lateral loading in the transverse direction. Emphasis will be placed on failure of the columns in bending and of the foundation system subjected to lateral loads, although other failure modes such as shearing failures or axial compression of column or foundation will be considered depending on the applied load (e.g., shearing failures are important modes for barge collisions and axial compression is important for the failure of multicolumn bents). When applicable, such as in the case of bending of multicolumn bents, system effects will be taken into consideration.

The results of the reliability analyses will subsequently be used to calibrate load factors and load combination factors that will provide bridge designs with adequate levels of safety when subjected to extreme load events. To minimize the changes to the current AASHTO specifications, the load factors will be applicable to the effects of the nominal loads corresponding to the same return periods as those currently in use. The load factors will be calibrated such that bridges subjected to a combination of events provide reliability levels similar to those of bridges with the same configurations but situated in sites where one threat is dominant. Thus, the proposed load factors will be based on previous experiences with "safe bridge structures" and will provide balanced levels of safety for each load combination. As mentioned above, the target reliability indexes for the combination of events will be selected in most cases to provide the same reliability level associated with the occurrence of the individual threat with the highest reliability index. Lower reliability index target values may be justified in the cases (such as earthquake loads) when increased reliability levels would result in unacceptable economic costs. The analysis will consider structural safety as well as foundation safety and system safety will be compared with member safety. The goal is to recommend a rational and consistent set of load factors that can be used during the routine design of highway bridges. These load factors will be presented in a specifications format that can be implemented in future versions of the AASHTO LRFD specifications.

1.6 REPORT OUTLINE

This report is divided into four chapters and nine appendixes. Chapter 1 gave a review of the problem statement and an overview of the proposed research approach. Chapter 2 describes the reliability models used in this study for the different load applications and combinations of loads. Chapter 3 provides the results of the reliability analysis of typical bridge configurations subjected to the extreme events of interest and combinations of these events. Chapter 3 also provides the results of the calibration process and determines the load factors for the combinations of extreme events. Chapter 4 gives the conclusion of this study and outlines future research needs.

Appendix A summarizes the results of this study in an AASHTO specifications format that provides the load factors for the combination of extreme events. The format is suitable for implementation in future versions of *AASHTO LRFD Bridge Design Specifications.* Appendix B details the reliability model for the analysis of scour as developed by Professor Peggy Johnson for the purposes of this study. The reliability models used for the other extreme events were extracted from the literature and are similar to those used by other researchers during the calibration of various structural design codes. Appendix C provides details of the reliability calculations for a three-span bridge used as the basis for the calibration. Appendix D describes the model used for the analysis of a long-span arch bridge subjected to scour and earthquakes. Appendix E describes the analysis of a long-span bridge over the Ohio River in Maysville for vessel collisions. Appendix F describes the earthquake analysis of the Maysville Bridge. Appendix G describes the wind analysis model for both the Interstate 40 and the Maysville Bridges. Appendix H describes the reliability analysis model used for a multispan bridge subjected to earthquakes. Appendix I performs a statistical analysis of available scour data and proposes an alternative model for the reliability analysis of bridge piers under scour. The examples solved in the appendixes serve to provide details about the models used in the body of the report and also to illustrate how big projects can be specifically addressed in detail.

Appendixes A, B, C, H, and I are published herein. All appendixes (Appendixes A through I) are contained on *CRP-CD-30*, which is included with this report.