STRUCTURAL LOAD MODELS

Nowak, A.S., Collins K.R. Reliability of structures.
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To DESIGN ANY structure, the designer must have an understanding of the types and magnitudes of the loads that are expected to act on the structure during its lifetime.

Many of the types of loads commonly considered in the design of buildings and bridges should be described by probabilistic models of these loads that are used in developing reliability-based design codes.

TYPES OF LOAD

Loads of many types act on structures. These loads can be classified into three categories based on the types of statistical data that are available and the characteristics of the load phenomenon:

Type I. For these loads, data are obtained by load intensity measurements without regard to the frequency of occurrence.
In other words, the time dependence of the loads is not explicitly considered.
Examples of loads in this category are dead and sustained live loads.

Type II. In this category, load data are obtained from measurements at prescribed periodic time intervals.
Thus some time dependence is captured.
Loads in this category include severe winds, snow loads, and transient live load.

Type III. The available data for these loads are obtained from infrequent measurements because the data are typically not obtainable at prescribed time intervals.
These loads occur during extreme events such as earthquakes and tornadoes.

GENERAL LOAD MODELS

Consider a general load Q which is to be modeled for the purposes of conducting a reliability analysis. It is often convenient to express the magnitude of Q_i as

$$Q_i = A_i B_i C_i \tag{6.1}$$

where A_i represents the load itself, B_i represents the variation due to the mode in which the load is assumed to act, and C_i represents the variation due to methods of analysis.

The variable C_i takes into account various approximations and idealizations used in creating the analysis model of the structure.

Examples of such approximations include two-dimensional idealizations of three-dimensional structures and fixed-base versus pinned-base assumptions.

The variable B_i accounts for assumptions about how the loading is applied to the structure.

For example, Figure 6.1 shows two beams.

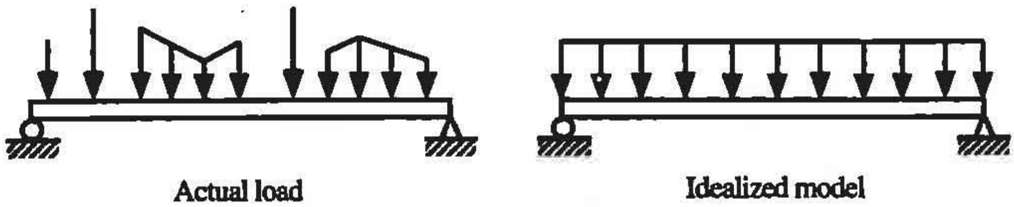


FIGURE 6.1 Idealization of loads on a structure as reflected in the variable B_i in Eq. 6.1.

On the left-band beam, the loading is represented by a combination of concentrated loads and nonuniform distributed loads.
On the right-hand beam the loading is simplified for analysis purposes into a single uniform1y distributed load acting over the entire beam.

To proceed with a reliability analysis, we need at least the mean and variance (or standard deviation or coefficient of variation) for Q. We can linearize the function for Q about the mean values, and then we can calculate the mean, bias factor, variance, and coefficient of variation of a linear function.
The resulting expressions are as follows:

$$\mu_Q \approx \mu_A \mu_B \mu_C$$

$$\lambda_Q \approx \lambda_A \lambda_B \lambda_C$$

$$\sigma_Q^2 \approx (\mu_B \mu_C)^2 \sigma_A^2 + (\mu_A \mu_C)^2 \sigma_B^2 + (\mu_A \mu_B)^2 \sigma_C^2$$

$$V_Q \approx \sqrt{V_A^2 + V_B^2 + V_C^2}$$

where:
μ = mean value,
λ = bias factor,
σ^2 = variance,
V = coefficient of variations,

EXAMPLE 6.1. Estimate the mean, bias factor, and coefficient of variation of Q (given in Eq. 6.1 as a function of the means and coefficients of variation of the variables A_i, B_i, and C_i

Solution. First we need to linearize the function for Q. Linearizing about the mean values, we get

$$Q_i = \mu_A \mu_B \mu_C + \left.\frac{\partial Q_i}{\partial A_i}\right|_{\text{mean values}} (A_i - \mu_A) + \left.\frac{\partial Q_i}{\partial B_i}\right|_{\text{mean values}} (B_i - \mu_B) + \left.\frac{\partial Q_i}{\partial C_i}\right|_{\text{mean values}} (C_i - \mu_C)$$

$$= \mu_A \mu_B \mu_C + \mu_B \mu_C (A_i - \mu_A) + \mu_A \mu_C (B_i - \mu_B) + \mu_A \mu_B (C_i - \mu_C)$$

$$= \mu_B \mu_C A_i + \mu_A \mu_C B_i + \mu_A \mu_B C_i - 2\mu_A \mu_B \mu_C$$

The linearized version of Q to get

$$\mu_Q \approx \mu_B\mu_C\mu_A + \mu_A\mu_C\mu_B + \mu_A\mu_B\mu_C - 2\mu_A\mu_B\mu_C$$
$$\approx \mu_A\mu_B\mu_C$$
$$\sigma_Q^2 \approx (\mu_B\mu_C)^2\sigma_A^2 + (\mu_A\mu_C)^2\sigma_B^2 + (\mu_A\mu_B)^2\sigma_C^2$$

Note that in the expression for σ_Q^2 a we assume that the variables are all uncorrelated.

Now we can obtain an expression for the coefficient of variation, V_Q, as follows:

$$V_Q^2 = \frac{\sigma_Q^2}{\mu_Q^2} \approx \frac{(\mu_B\mu_C)^2\sigma_A^2 + (\mu_A\mu_C)^2\sigma_B^2 + (\mu_A\mu_B)^2\sigma_C^2}{(\mu_A\mu_B\mu_C)^2} = \frac{\sigma_A^2}{\mu_A^2} + \frac{\sigma_B^2}{\mu_B^2} + \frac{\sigma_C^2}{\mu_C^2}$$
$$= V_A^2 + V_B^2 + V_C^2$$

$$V_Q \approx \sqrt{V_A^2 + V_B^2 + V_C^2}$$

To relate the bias factors, we first recognize that Q_n, the nominal value of Q, is simply the product of the nominal values of A, B, and C.

Then, starting with the foregoing relationship for the mean values, we can relate the bias factors as follows:

$$\mu_Q \approx \mu_A \mu_B \mu_C$$

$$\lambda_Q Q_n \approx (\lambda_A A_n)(\lambda_B B_n)(\lambda_C C_n)$$

$$\lambda_Q Q_n \approx (\lambda_A \lambda_B \lambda_C) A_n B_n C_n$$

$$\Downarrow$$

$$\lambda_Q \approx \lambda_A \lambda_B \lambda_C$$

In most cases, we need to consider load cases involving several different types of loads.
When several loads are acting together (e.g., $Q_1 + Q_2 + \ldots + Q_n$) the total load can be modeled by using Eq. 6.1 for each load to get

$$Q = c(A_1 B_1 C_1 + A_2 B_2 C_2 + \cdots + A_n B_n C_n) \qquad (6.3)$$

where c is an additional factor which is common for all loads.
It can also be thought of as a load combination factor.

DEAD LOAD

The dead load considered in design is usually the gravity load due to the self-weight of the structural and nonstructural elements permanently connected to the structure.

Because of different degrees of variation in different structural and nonstructural elements, it is convenient to break up the total dead load into two components:

– weight of factory-made elements (steel, precast concrete members) and

– weight of cast-in-place concrete members.

Also, for bridges, a third component of dead load is the weight of the wearing surface (asphalt).

All components of dead load are typically treated as normal random variables.

Usually it is assumed that the total dead load, D, remains constant throughout the life of the structure.

Table 6.1 lists some representative statistical parameters of dead load.

TABLE 6.1 Representative statistical parameters of dead load

	Bias factor, $\lambda_D = \frac{\mu_D}{D_n}$	$\sqrt{V_A^2 + V_B^2}$	Coefficient of variation, V_D
Buildings	1.00	0.06–0.09	0.08–0.10
Bridges	1.03–1.05	0.04–0.08	0.08–0.10

Often there is a tendency on the part of designers to underestimate the total dead load.
Therefore, to partially account for this, use of a bias factor 1.05 rather than the lower values shown in Table 6.1 is recommended.

LIVE LOAD IN BUILDINGS
Design (Nominal) Live Load

Live load represents the weight of people and their possessions, furniture, movable partitions, and other portable fixtures and equipment.
Usually, live load is idealized as a uniformly distributed load.
The design live load is specified in kilonewtons per square meter (kN/m^2).

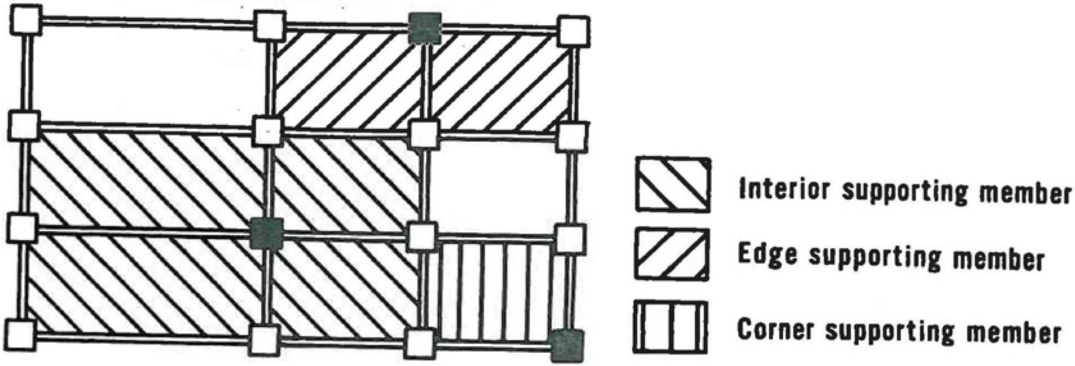
The magnitude of live load depends on the type of occupancy. For example, live loads specified by ASCEI ANSI Standard 7-95 (ASCE, 1996) range from 0.48 kN/m^2 for uninhabited attics not used for storage to 11.97 kN/m^2 for storage areas above ceilings.

The value of live load also depends on the expected number of people using the structure and the effects of possible crowding.

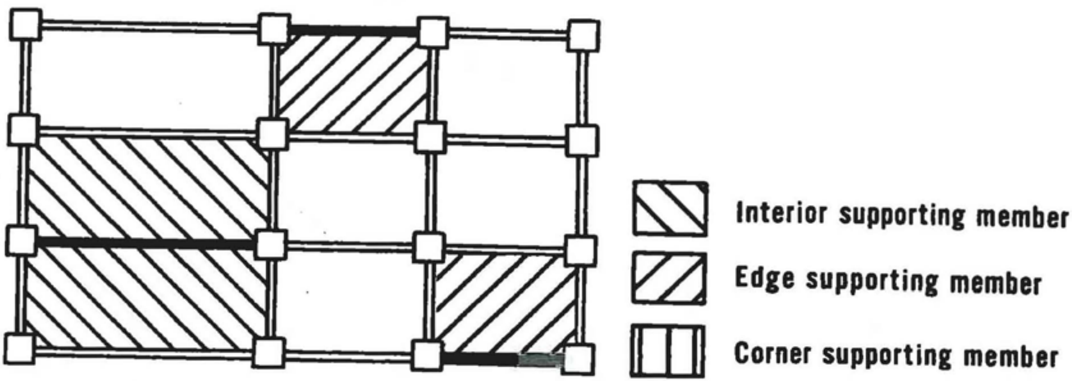
The statistical parameters of live load depend on the area under consideration.
The larger the area which contributes to the live load, the smaller the magnitude of the load intensity.
ASCE 7-95 specifies the reduction factors for live load intensity as a function of the *influence area*.
It is important to distinguish between influence area and tributary area.
The tributary area is used to calculate the live load (or load effect) in beams and columns.
The influence area is used to determine the reduction factors for live load intensity.
Figure 6.3 illustrates this distinction for beams and columns.

FIGURE 6.3 Influence and tributary area for beams and columns.

COLUMNS



BEAMS AND GIRDERS



It is possible to relate influence wand tributary area by the following formulas:

For beams: Influence area = 2 x Tributary area

For columns: Influence area = 4 x Tributary area

When the influence area, A_I, is larger than 37.16 m^2, the design (nominal) live load, L_n, is calculated using

$$L_n = L_0 \left(0.25 + \frac{15}{\sqrt{A_I}}\right) \quad \text{(psf)} \qquad (6.4a)$$

$$L_n = L_0 \left(0.25 + \frac{4.57}{\sqrt{A_I}}\right) \quad (\text{kN/m}^2) \qquad (6.4b)$$

where L_0 is the unreduced design live load obtained from the code.

Sustained (Arbitrary Point-in-Time) Live Load

Sustained live load is the typical weight of people and their possessions, furniture, movable partitions, and other portable fixtures and equipment.
The term "sustained" is used to indicate that the load can be expected to exist as a usual situation (nothing extraordinary).
Sustained live load, also called an *arbitrary-point-in-time live load*, L_{apt} is the live load that you would most likely find in a typical office, apartment, school, hotel, and the like.
Live load surveys have been performed by many researchers to obtain statistical data on the sustained live load.

Previous investigations (Corotis and Doshi, 1977; Ellingwood, Galambos, MacGregor, and Comell, 1980) have found that the sustained live load can be modeled as a gamma distributed random variable.

Table 6.3 presents some typical values of the bias factors and the coefficients of variation for sustained live load as a function of influence area.

TABLE 6.3 Statistical data for sustained live load as a function of influence area

Influence area, ft^2	Bias factor	Coefficient of variation*
200	0.24	0.59–0.89
1000	0.33	0.26–0.55
5000	0.52	0.20–0.46
10,000	0.60	0.18–0.45

* Coefficients of variation taken from Ellingwood, Galambos, MacGregor, and Cornell, 1980.

Transient Live Load

Transient live load is the weight of people and their possessions that might exist during an unusual event such as an emergency, when everybody gathers in one room, or when all the furniture is stored in one room.

Since the load is infrequent and its occurrence is difficult to predict, it is called a transient load.

Like sustained live load, the transient live load is also a function of the influence area rather than the tributary area.

Maximum Live Load

For design purposes, it is necessary to consider the expected combinations of sustained live load and transient loads that may occur during the building's design lifetime (50-100 years).

The probabilistic characteristics of the maximum live load depend on the temporal variation of the transient load, the duration of the sustained load (which is related to the frequency of tenant changes or changes in use), the design lifetime, and the statistics of the random variables involved (Chalk and Corotis, 1980).

The combined maximum live load can be modeled by an extreme **Type I distribution** (Ellingwood, Galambos, MacGregor, and Comell, 1980) for the range of probability values usually considered in reliability studies.

The mean value of the maximum 50-year live load as a function of the influence area is shown in Figure 6.4. The coefficients of variation are shown in Table 6.4.

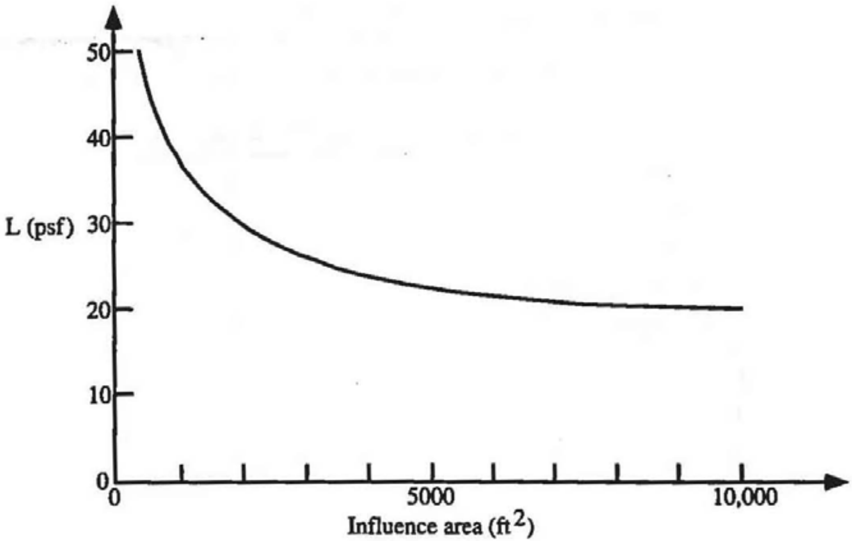


FIGURE 6.4 Mean maximum 50-year live load as a function of influence area.

TABLE 6.4 Coefficients of variation of maximum 50-year live load

Influence area, ft^2	Coefficient of Variation
200	0.14–0.23
1000	0.13–0.18
5000	0.10–0.16
10,000	0.09–0.16

Source: Ellingwood, Galambos, MacGregor, and Cornell, 1980.

LIVE LOAD FOR BRIDGES

For bridge design, the live load covers a range of forces produced by vehicles moving on the bridge.

The effect of live load on the bridge depends on many parameters, such as the span length, truck weight, axle loads, axle configuration, position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders).
Live load on bridges is characterized not only by the load itself, but also by the distribution of this load to the girders.
Therefore, the most important item to be considered is the load spectrum per girder.

The development of a live load model is essential for a rational bridge design and/or evaluation code. Ghosn and Moses (1985) proposed statistical parameters for truck load, including weight, axle configuration, dynamic load, and future growth.

The design live load specified by the AASHTO standard (1996) is shown in Figure 6.5.

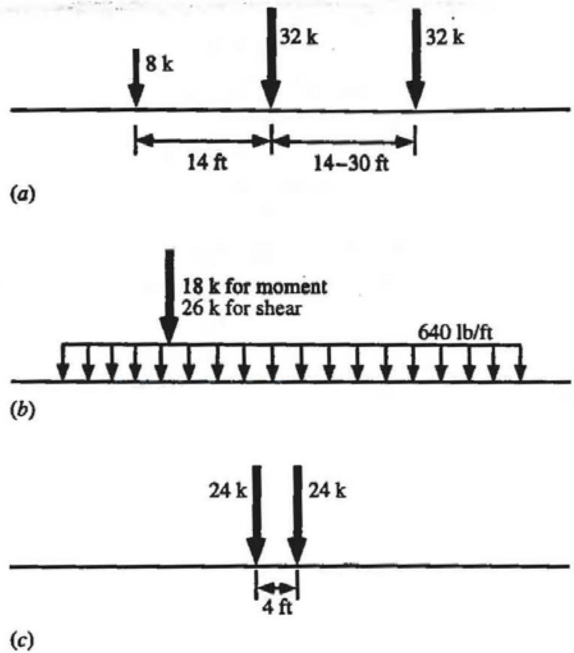


FIGURE 6.5 HS20 loading as defined by AASHTO standard (1996). (*a*) Standard HS20 truck. (*b*) HS20 lane loading. (*c*) Military loading. The code is available only in U.S. units. (1 k = 4.45 kN, 1 lb/ft = 15 N/m, 1 ft = 0.3 m).

For shorter spans, a *military* łoad is specified in the form of a tandem with two 24-k axles spaced at 4 fi. The design load specified by AASHTO LRFD (1998) is shown in Figure 6.6.

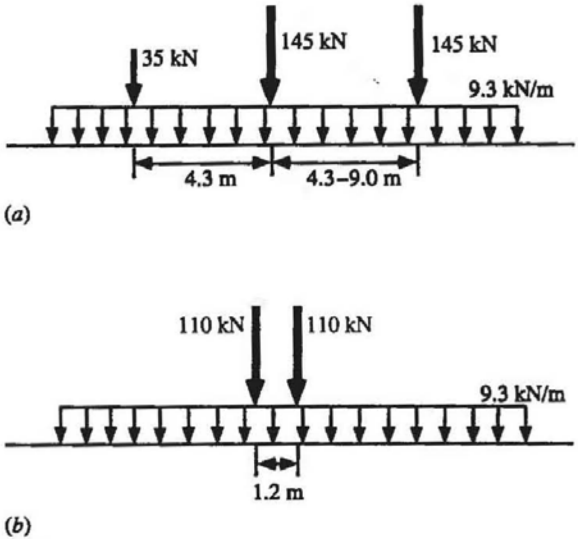


FIGURE 6.6 HL-93 loading specified by AASHTO LRFD (1998). (*a*) Truck and uniform load. (*b*) Tandem and uniform load. The code is also available in U.S. units.

The available statistical parameters of bridge live load have been determined from truck surveys and by simulations.

The measurements show that the design values of bending moments and shears are lower than the actual load effects of today's heavy traffic observed on the highways.

The available truck weight database is limited to selected locations and time periods from a few days to two weeks.

Examples of the cumulative distribution functions (CDFs) of gross vehic1e weight (GVW) for trucks measured on seven bridges in the Greater Detroit area (Michigan) are shown in Figure 6.7 (on normal probability paper).

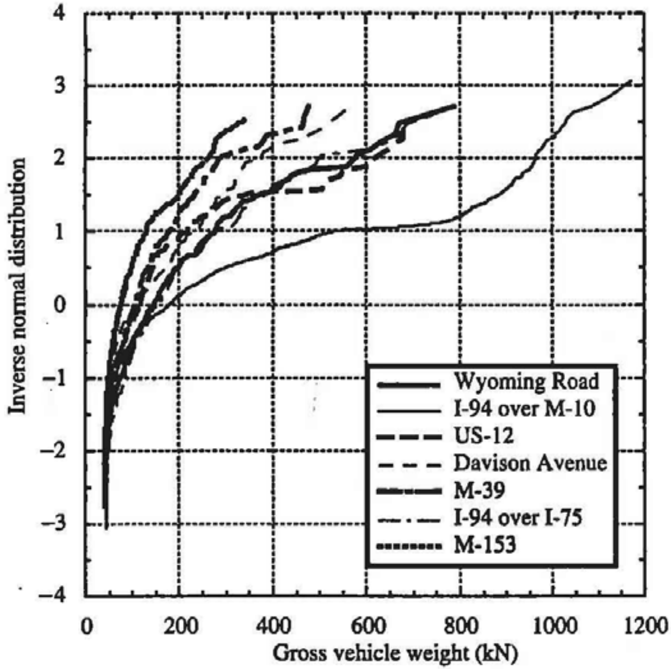


FIGURE 6.7 Gross vehicle weight (GVW) of trucks surveyed on seven bridges in the Greater Detroit area (Michigan).

Michigan legal truck load limits are the most permissive in the United States. The CDFs of axle loads for the same locations are presented in Figure 6.8.

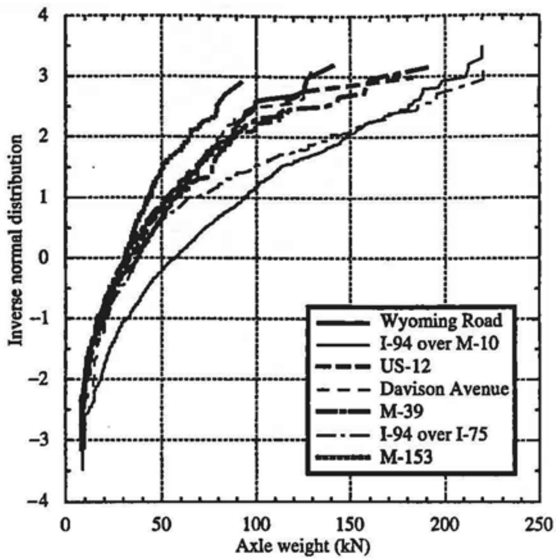


FIGURE 6.8 CDFs of axle loads for trucks surveyed on seven bridges in the Greater Detroit area (Michigan).

The surveyed trucks were used to calculate bending moments.

The CDFs of the resulting moments are plotted on normal probability paper in Figure 6.9.

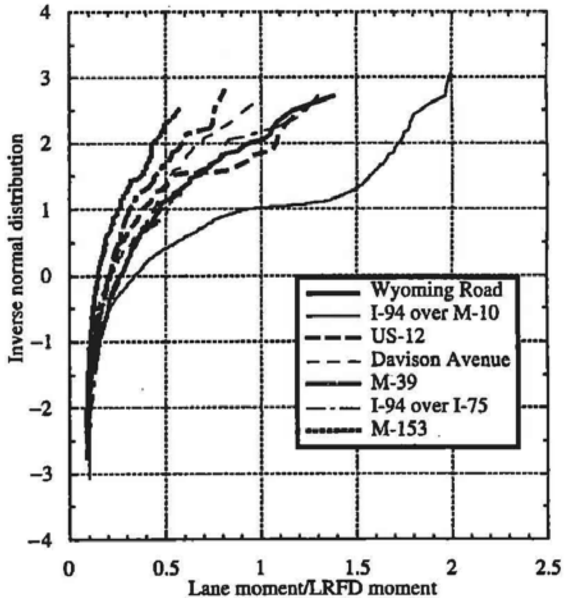


FIGURE 6.9 CDFs of calculated bending moments for trucks surveyed on seven bridges in the Greater Detroit area (Michigan).

The statistical parameters of bridge live load were derived in conjunction with the development of the LRFD AASHTO code (1998).

An average lifetime for bridges is about 75 years, and this time period was used as the basis for calculation of loads.

A statistical model was developed for the mean maximum 75-year moments and shears by extrapolation of the available truck survey data (Nowak, 1993).

For longer spans, multiple presence of trucks in one lane was simulated by considering three cases: no correlation between trucks, partial correlation, and full correlation.

It turned out that two fully correlated trucks governed.

For a single lane, the bias factor (ratio of the mean maximum 75-year moment/shear to the HS 20 moment/shear, as specified in the AASHTO standard) is plotted versus span length in Figure 6.10, and the bias factor corresponding to HL-93 moment-shear (AASHTO LRFD) is shown in Figure 6.11.
For the maximum 75-year moment, the coefficient of variation of truck load is 0.12. It is larger for shorter periods of time (e.g., 0.20 - 0.25 for the maximum daily truck).

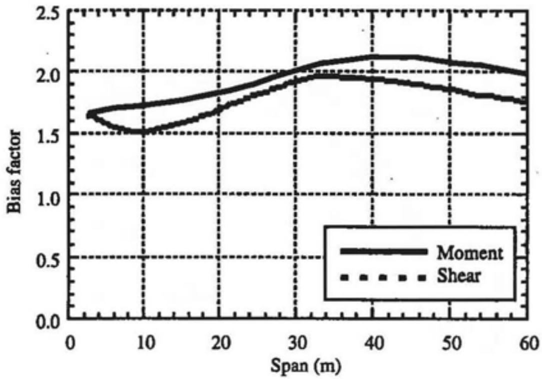


FIGURE 6.10 Bias factors for moments and shears for AASHTO (1992) standard.

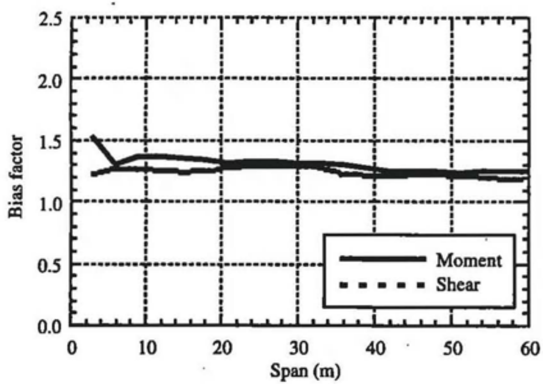


FIGURE 6.11 Bias factors for moments and shears for AASHTO LRFD.

The calculated multilane factors (for ADTI equal to 100, 1000, and 5000 trucks in one direction) are presented in Table 6.5.

TABLE 6.5 Multilane factors

	Number of lanes			
ADTT	1	2	3	4 or more
100	1.15	0.95	0.65	0.55
1000	1.20	1.00	0.85	0.60
5000	1.25	1.05	0.90	0.65

Dynamic load is defined as the ratio of dynamic deflection and static deflection.

The AASHTO standard (1996) specifies impact I, as a function of span length only, by the equation

$$I = \frac{50}{125 + L}$$

where L is the span length in feet (1 ft = 0.305 m).

Actual dynamic load depends on three major factors: road roughness, bridge dynamics (natural period of vibration), and vehicle dynamics (type and condition of suspension system).

The simulations and tests indicate that the dynamic load decreases for heavier trucks (as a percentage of static live load).

The mean dynamic load is less than about 0.15 for a single truck and less than 0.10 for two trucks, for all spans.

The coefficient of variation of dynamic load is 0.80.

The coefficient of variation of a joint effect of live load and dynamic load is 0.18.

ENVIRONMENTAL LOADS
Wind Load

The load (or load effect) on a structure due to wind is a function of many parameters: the wind speed, wind direction, geometry of the structure, local topography, and a variety of other factors.

For design purposes, wind pressures on the outer surfaces of a structure are calculated first, and then these pressures are converted to loads or load effects.

The formulas used to determine wind pressures on structures in the United States are of the general form.

$$p_z = q_z G C \qquad (6.6a)$$

$$q_z = 0.00256 K_z K_{zt} I\, V^2 \quad \text{(U.S. units, psf)} \qquad (6.6b)$$

$$q_z = 0.613 K_z K_{zt} I\, V^2 \quad \text{(SI units, Pa)} \qquad (6.6c)$$

where p_z is the design pressure, q_z is the velocity pressure, K_z is a velocity pressure exposure coefficient, K_{zt} is a topographic factor, Y is the basic wind speed (in miles per hour or meters per second, as appropriate), I is an importance factor, G is a gust factor, and C is a pressure coefficient (ASCE, 1996).

The basic wind speed is defined as the 3-second gust speed at 10 meters above the ground for airport-type terrain with a 50-year mean recurrence interval.

The importance factor used in Eq. 6.6 provides a mechanism to adjust the velocity pressure (which depends on wind speed) to reflect other mean recurrence intervals.

According to ASCE 7-95 (1996), an importance factor of 0.87 corresponds to a mean recurrence interval of 25 years.

This value might be appropriate for the design of a barn for which the threat to human life in the event of failure is minimal.

An importance factor of 1.15 corresponds to a mean recurrence interval of 100 years.

This value is appropriate for a hospital facility since there is a significant threat to human life in the event of a failure.

Analyses of wind speed data suggest that the largest annual wind speed at a particular site tends to follow a Type I extreme value distribution (Simiu, 1979), and this distribution is the most frequently used model for wind speed (Ellingwood and Tekie, 1999).

However, hurricane wind speeds tend to follow a Weibull distribution (ASCE, 1996, Commentary to ASCE 7-95).

Although the probability distributions for V are available, the probability distribution of the wind load (or load effect) is the one of interest for structural reliability calculations.

The distribution of wind load is not necessarily Type I since the wind pressure is proportional to V^2 instead of V.

Furthermore, the other parameters (such K_z, K_{zt}, G, and C) are random in nature as well.

Thus it is difficult to determine the distribution of the wind load (or load effect).

However, studies (Ellingwood, 1981) have indicated that the uncertainty in wind load is dominated by the uncertainty in V^2, and the CDF for wind load can be represented by a Type I distribution for values of the CDF above 0.90 (Ellingwood, 1981).
This region of the CDF is the one of interest in structural reliability analyses.
The distribution parameters depend on the location being considered.

1.1 6.6.2 Snow Load

The weight of snow on roofs can be a significant load to consider for structures in mountainous regions and snow belts.
For design purposes, the snow load on a roof is often calculated on the basis of information on the ground snow cover.
For example, in the United States, the roof snow load for fiat roofs (slope < 5 percent) is calculated using

$$p_f = 0.7 C_e C_t I p_g \qquad (6.7)$$

where C_e is the exposure coefficient, C_t is the thermal factor, I is the importance factor, and Pg is the ground snow load (psf or kN/m^2). For sloping roofs, the snow load is calculated using

$$Ps = C_s P_r \qquad (6.8)$$

where C_s is the roof slope factor and P_r is the flat-roof snow load computed using Eq. 6.7. (Equations 6.7 and 6.8 are from ASCE Standard 7-95.)

As reflected by Eqs. 6.7 and 6.8, the probability distribution for the snow load on a structure will depend on the probability distributions of the ground snow load and the conversion factors C_e, C_s, and C_t. Statistical analyses of meteorological data suggest that the ground snow load can be modeled using either a lognormal or Type I distribution (Boyd, 1961; Ellingwood and Redfield, 1983; Thom, 1966).

However, studies of snow data for the northeast quadrant of the United States indicate that the lognormal distribution is the

preferred distribution for that region (Ellingwood and Redfield, 1983).
The ground-to-roof conversion factor C*s* is often modeled as a lognormal random variable (Ellingwood and O'Rourke, 1985) or normal random variable (Ellingwood, 1981). The distribution of the roof snow load is not clearly based on the distributions of ground snow load and CS ' Statistical studies (Ellingwood, 1981) suggest that a Type II extreme value distribution is appropriate for values of the CDF above 0.90, which is the region of interest in structural reliability studies.
If all the random variables in Eqs. 6.7 and 6.8 are assumed to be independent lognormal variates, then a lognormal distribution for roof snow load is appropriate; this distribution was used by Ellingwood and Rosowsky (1996) in their investigation of combinations of snow and earthquake loads.