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Computer-Aided Structural Engineering (CASE) Project

Load and Resistance Factor Design for Steel Miter Gates

by Bruce R. Ellingwood
Johns Hopkins University

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Prepared for Headquarters, U.S. Army Corps of Engineers

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by Bruce R. Ellingwood

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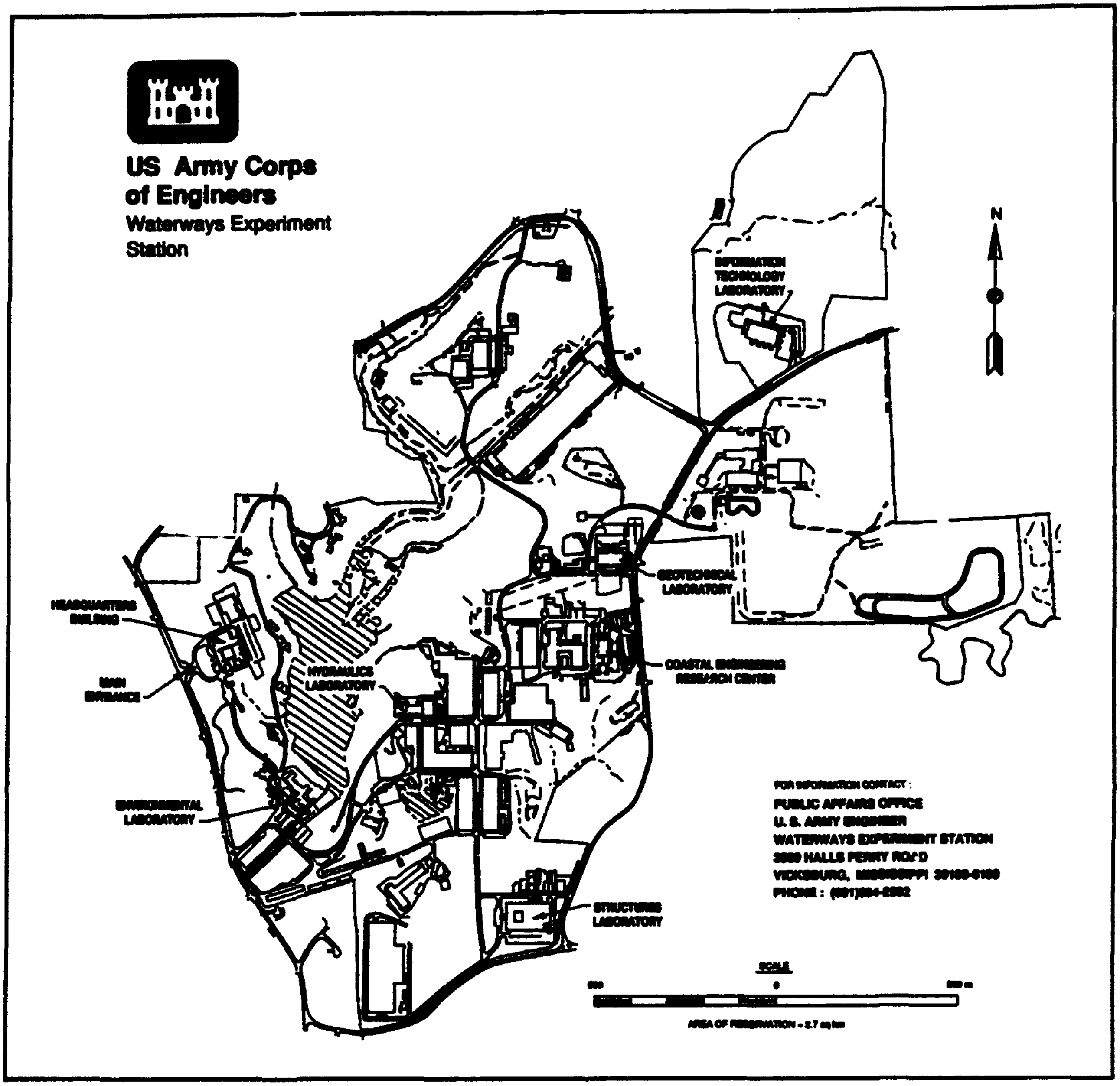

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Preface

The work described in this report was sponsored by Headquarters, US Army Corps of Engineers (HQUSACE), as part of the Civil Works Guidance Update Program (CWGUP). The work was performed under the CWGUP Work Unit S031, "Design of Hydraulic Steel Structures," for which Mr. Cameron P. Chasten, Information Technology Laboratory (ITL), US Army Engineer Waterways Experiment Station (WES), was Principal Investigator. Mr. Donald R. Dressler (CECW-ED) was the CWGUP Technical Monitor for this project, and Mr. Thomas J. Mudd, ITL, WES, is the CWGUP Program Manager. Funding for acquisition of miter gate load data and the publication of this report was provided by the Computer-Aided Structural Engineering (CASE) Project sponsored by the Directorate, HQUSACE. The CASE Project is managed by the Scientific and Engineering Applications Center (S&EAC), Computer-Aided Engineering Division (CAED), ITL, WES.

The work was performed at the Department of Civil Engineering, Johns Hopkins University, by Dr. Bruce R. Ellingwood, under US Army Corps of Engineers (USACE) Contract No. DACW39-92-M-1280. The report was prepared by Dr. Ellingwood, under the general supervision of Mr. Chasten, Mr. H. Wayne Jones, Chief, S&EAC, CAED, and Dr. N. Radhakrishnan, Director, ITL. Some of the results of this work are included in Engineer Manual 1110-2-2105, "Design of Hydraulic Steel Structures."

Acknowledgement is expressed to the Load and Resistance Factor Design Field Review Group (LRFDFRG), assembled to monitor this project, for providing assistance in this work. Members of the LRFDFRG and their affiliations are: Mr. Joseph P. Hartman, HQUSACE; Dr. John J. Jaeger, US Army Engineer District, Jacksonville; Dr. Nathan Kathir, US Army Engineer District, St. Paul; Mr. Chasten, ITL, WES; Mr. Robert J. Smith, Civil Engineering Consultant; Mr. W. Scott Gleason, Civil Engineering Consultant; Dr. Ellingwood, Department of Civil Engineering, Johns Hopkins University; Dr. Theodore V. Galambos, Department of Civil and Mineral Engineering, University of Minnesota; and Dr. Wei-Wen Yu, Department of Civil Engineering, University of Missouri - Rolla. The data on hydrostatic and hydrodynamic loads and tow impact required to utilize the response combination models were obtained with the assistance of staff members at WES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as foll .ws:

Multiply	By	To Obtain
feet	0.3048	meters
inches	0.0254	meters
kips	4.448	kilonewtons
tons	907.185	kilograms
mile per hour	0.447	meters per second
kips per square inch (ksi)	6894.757	kilopascals

1 Background

Miter gates and other hydraulic steel structures (HSS) must be designed to withstand loads due to the weight of the structure and permanent attachments, operating equipment, barge impact, hydrostatic and hydrodynamic effects, and environmental effects. The maximum structural response to appropriate combinations of operating and environmental effects is compared to the design strength of the structure to ensure acceptable performance.

The design of miter gates in the past has been based on allowable stress design (ASD) principles (Headquarters, Department of the Army 1972), the traditional approach to steel design embodied in the American Institute of Steel Construction (AISC) specification (AISC 1978). In ASD, the design safety check is of the form

$$f(\Sigma Q_i) < F_{all} = F_u / FS \tag{1}$$

where

$f(\Sigma Q_i)$ = the elastically computed stress arising from the combined nominal loads, ΣQ

F_{all} = allowable stress

F_u = limiting stress (for yielding, rupture, buckling)

FS = overall factor of safety

The FS represents the traditional way of addressing the problem of design uncertainty and does so in a purely subjective way.

There are a number of shortcomings in ASD (Galambos et al. 1982) which are remedied in the new "Load and Resistance Factor Design Specification for Structural Steel Buildings" (AISC 1986). In contrast to ASD, Load and Resistance Factor Design (LRFD) is a limit states design method which is much better keyed to the behavior of steel structures at design conditions. Design uncertainties are taken into account using modern structural reliability analysis techniques to supplement professional judgement. In LRFD, the basic safety check is of the form

$$\textit{Required strength} < \textit{Design strength} \tag{2a}$$

$$\Sigma\gamma_i\, Q_{ni} < \phi R_n \tag{2b}$$

where

R_n = nominal strength

ϕ = resistance factor that reflects uncertainty in strength

γ_i = load factors to account for uncertainty in the loads

Note that the factors ϕ and γ_i serve the same purpose as the overall *FS* in Equation 1. However, they are reflective of the variability in the individual parameters to which they are assigned.

The LRFD Specification (AISC 1986) provides a comprehensive treatment of design strength for the limit states of yielding, inelastic deformation, fracture, and instability for steel members and connections. Studies have shown that cost savings often can be achieved in design by using LRFD rather than ASD. Efforts are now underway to adapt the LRFD Specification to the design of miter gates and other hydraulic steel structures (Headquarters, Department of the Army 1991 and 1993).

The required strength in Equation 2b is defined for ordinary buildings and other structures by the load requirements in Section 2.4 of American Society of Civil Engineers (ASCE) Standard 7-88 (formerly American National Standard (ANSI) Standard A58.1-1982) on minimum design loads (ASCE 1990). Environmental effects (wind speed, for example) are specified as N-year mean recurrence interval (MRI) values, i.e. those values with a probability of 1/N of being exceeded in a given year. Generally, however, two or more time-varying effects must be considered in combination. As illustrated by the typical load sample functions in Figures 1a through c, the extreme values of the individual effects rarely occur simultaneously, and simply combining the N-year MRI values is unduly conservative. Probabilistic load combination analysis methods (Ellingwood et al. 1982; Galambos et al. 1982) were used to develop the load factors and load combinations used to compute the required strength in Equation 2b for building structures (ASCE 1990). Statistical data are required to define the cumulative probability distribution functions for the environmental loads and to set these loads and load combinations.

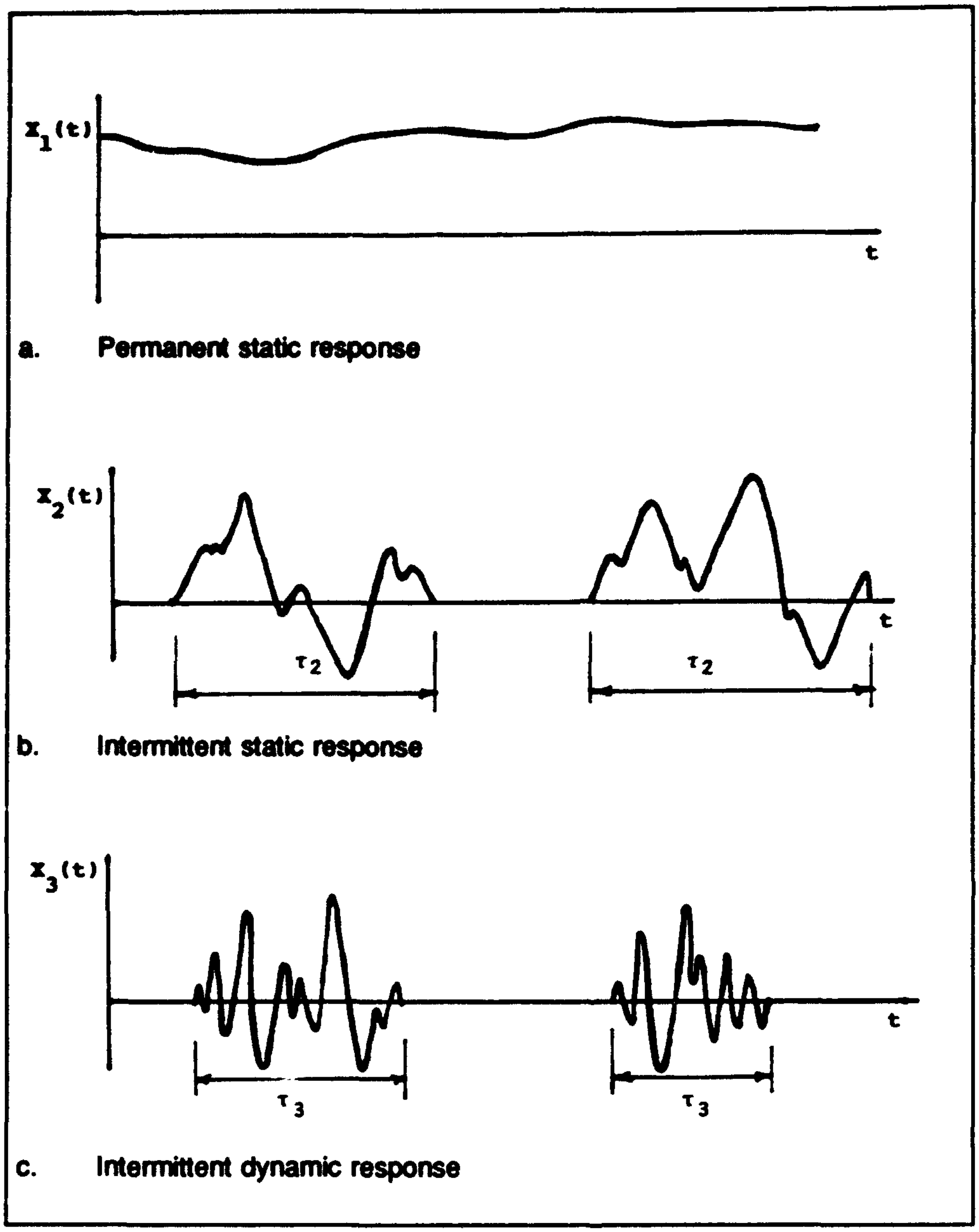

Figure 1. Stochastic responses to environmental events

The application of the load factors in ASCE Standard 7-88 (ASCE 1990) to the loads considered in design of miter gates is questionable, because the load combination analysis alluded only to above utilized data on common structural loads for buildings. One might expect that the load requirements for design of miter gates would have a similar format but that the load factors would be different. Moreover, the load factors in Equation 2b were determined through a calibration process in which common structural members designed by ASD were evaluated using structural reliability techniques. The new load requirements for LRFD were based on the target reliabilities determined through this assessment of existing designs. This calibration process makes sense for ordinary construction, where design criteria are supported by years of experience. There is less experience for miter gates. A review of the past criteria for hydraulic structures (Headquarters, Department of the Army 1972) does not reveal any rational basis for some of the design requirements, e.g. that the AISC-specified allowable stresses be multiplied by a factor of 5/6 in proportioning members for combinations of normal operating or hydrostatic loads.

The development of probability-based load requirements for use in LRFD requires the following steps:

a. Identification of structures to be covered.

b. Identification of limit states for structural components and systems.

c. Identification of significant loads and load combinations, and the development of a statistical database to support probabilistic load combination analysis.

d. Collection of statistics to describe the structural capacity.

e. Analysis of reliabilities associated with current design procedures (Headquarters, Department of the Army 1972), and selection of target reliability levels.

f. Development of LRFD load requirements, including load combinations and load factors.

g. Comparative design studies to determine cost savings or penalties associated with the proposed criteria.

This report focuses on steps c, e, and f.

2 Objectives and Scope

This report describes an improved basis for determining load requirements for design of hydraulic steel structures using modern structural reliability analysis and probabilistic load combination techniques. This methodology will facilitate the development of design loads and their combinations for use in LRFD.

The methodology is demonstrated for miter gates by considering load combinations involving dead load, hydrostatic load, hydrodynamic load during gate operation, temporal hydraulic head, and barge impact load.

3 Reliability Bases for LRFD

Basic methods of Reliability Analysis

This section presents some of the basic tools for the development of reliability-based load combinations for hydraulic structures such as miter gates.

First-order reliability methods

A limit state represents a condition in which the structural system or one of its components fails to perform its intended functions. For HSS, strength or safety-related limit states include yielding, rupture, instability, or loss of overall equilibrium. Serviceability limit states include excessive deformation or vibration. A limit state is described by a behavioral equation or failure function developed from principles of structural mechanics

$$G(\underline{X}) = 0 \tag{3}$$

where

$\underline{X} = (X_1, X_2, \ldots, X_n)$

= random variables describing loads, material strengths, and dimensions.

Failure is taken, by convention, as the state in which $G(\underline{X}) < 0$.

The limit state probability or probability of failure provides a quantitative measure of reliability and performance. If the joint distribution of the X_i is known, this probability is

$$P_f = \int \ldots \int f\,(x_1, x_2, \ldots)\; dx_1 dx_2 \ldots dx_n \tag{4}$$

where

$f(\underline{x})$ = joint probability density function of $\underline{X}$ and the integration is performed over that portion of the domain of $\underline{x}$ in which $G(\underline{x}) < 0$

The joint density in Equation 4 is rarely available and the integration is difficult to perform. Moreover, P_f is very sensitive to the behavior of the extremes of $f(\underline{x})$, which are difficult to define with the limited data that typically are available in structural reliability analysis. First-order (FO) reliability methods have been developed to address these problems. In FO reliability analysis, P_f as a measure of reliability is replaced by a "reliability index," β, defined as the minimum distance from the mean of $\underline{X}$ to the failure surface, measured (when the random variables are statistically independent) in standard deviation units (Melchers 1987). The coordinates of the point of minimum distance on the failure surface (sometimes termed the checking or design point) can be defined as

$$X^*_i = m_i \pm \alpha_i \, \beta \, \sigma_i, \quad i = 1,2, \ldots, n \tag{5}$$

where

m_i, σ_i = mean and standard deviation in X_i

α_i = sensitivity coefficient describing the relative importance of variable X_i in the reliability analysis

The plus or minus sign on α_i depends on whether X_i is a load or strength variable, respectively. If all variables are described by a normal distribution and function $G(\underline{x})$ is linear in $\underline{x}$, β and P_f are related by

$$P_f = \Phi(-\beta) \tag{6}$$

where

Φ = standard normal probability integral

In this case, FO reliability analysis is a tool for approximately integrating Equation 4. Clearly, estimates of means and standard deviations in loads are essential in developing probability-based design requirements.

Estimation of measures of uncertainty

Let X denote a basic resistance or load variable. The true statistical characteristics of X should be employed when evaluating P_f or β and when deriving appropriate factors of safety for design. However, the random characteristics of X seldom are known precisely in structural engineering owing to insufficient data and imperfect information. This is especially true for miter gates, where a consistent database on operating events does not exist. Estimates of the mean, m_X, and the standard deviation, σ_X, and coefficient of variation, V_X, that describe inherent variability, are obtained from small samples of data or are inferred on the basis of expert opinion. Additional uncertainties arise from the small sample sizes, differences between the laboratory conditions under which some data are gathered and field conditions, and structural modeling errors. As a result of these additional uncertainties, the mean and coefficient of variation used in structural reliability analysis should be determined as

$$m_X = B\, m_X \tag{7}$$

$$V_X = \left[V_X^2 + V_B\right]^{\frac{1}{2}} \tag{8}$$

where

m_X and V_X = data based

B = bias in prediction

V_B = modeling uncertainty, assumed to be vested in the uncertainty in predicting the mean, m_X.

When data are available, m_X and V_X can be estimated from data samples. Frequently, little data are available to supplement engineering judgement. Occasionally, the engineer may be able to estimate from past experience the range over which the data should lie. In this case, the mean and coefficient of variation can be estimated from the range if some assumption is made about the general shape of the probability distribution. If, for example, it is assumed that X has a bell-shaped frequency distribution and that roughly 95 percent of all values lie between x_1 and x_2, the implied mean and coefficient of variation in X are

$$m_X = (x_1 + x_2)/2 \tag{9}$$

$$V_X = 0.5(x_2 - x_1)/(x_2 + x_1) \quad (10)$$

Because of the scarcity of data, it will be necessary to estimate statistics for several of the loads using similar techniques in the sequel.

The Delphi as a basis for uncertainty analysis

A significant amount of load survey and modeling data were available to develop the load combinations in ASCE 7-88 (ASCE 1990) for building structures and in the AISC LRFD specification. The loads used in miter gate design are defined in EM-1110-2-2703 (Headquarters, Department of the Army 1984). These loads apparently are based on experience rather than on in situ load measurements, and their quantitative basis, if any, cannot be determined. Statistical data of the sort available for buildings are not available for loads on miter gates and other hydraulic structures, and it is not feasible to perform the necessary load surveys.

A Delphi, or consensus estimation survey, was conducted to remedy this situation and to provide statistical data on loads necessary to develop probability-based load combinations. A Delphi is an organized method for eliciting expert opinion or intelligent judgement from a group and is used to quantify a phenomenon when empirical data are limited or do not exist. Delphi's have been used in a number of structural engineering applications: in revising the live load tables in ASCE 7-88 (Corotis, Fox, and Harris 1981) and, more recently, in seismic hazard analysis at nuclear plant sites in the Eastern United States where there is little or no historical seismicity (Bernreuter et al. 1989). Of course, the accuracy of the results depends on the knowledge, biases, and judgement of the participants and the extent to which they utilize any available empirical data. In a typical application, a load questionnaire initially is circulated to the group and each participant is asked to provide a best estimate of a parameter and an estimate of its variation. The results of the questionnaire are compiled and recirculated for revisions. Although in the ideal situation there may be several iterations of the questionnaire, constraints prevented this in the current project.

The questionnaire was developed and distributed to each U.S. Army Corps of Engineers (USACE) District and to the lockmaster of each lock and dam that is operated and maintained by the Corps (Chasten 1992b). Each lock operator was asked to complete the survey on the basis of his own experience. A total of 108 responses were received, representing most of the major river systems in the eastern United States.

A typical survey question asked for a best estimate of a parameter, X (say, hydrodynamic head), and a plus-or-minus variation, ΔX. This information is used to estimate the mean and standard deviation (or coefficient of variation) in variable X. The variation given was interpreted as corresponding to ± 2 standard deviations (representing approximately the 95 percent confidence bound for normal random variables).

Suppose that the two estimates provided by the k^{th} participant are x_k and Δx_k, k = 1,2,...N. The mean value of X is estimated from

$$\bar{X} = \frac{1}{N} \sum_k x_k \approx m_X \tag{11}$$

Assuming that the participants are equally experienced, $\bar{X}$ is an unbiased estimate of the mean.

The actual value of X can be expressed as

$$X = m_X + U + V \tag{12}$$

where

m_X = true mean of X

U = zero-mean random variable describing deviation of mean values estimated by participants

V = zero-mean random variable denoting the deviation in the variation estimated by each participant.

Assuming that U and V are statistically independent, the variance in X is estimated by

$$\sigma_X^2 = \sigma_U^2 + \sigma_V^2 \tag{13}$$

where

$$\sigma_U^2 = \frac{1}{N-1} \sum_k (X_k - \bar{X})^2 \tag{14}$$

$$\sigma_V^2 = \frac{1}{N-1}\sum_k\left(\frac{\Delta x_k}{2}\right)^2 = \frac{1}{4N}\sum_k(\Delta x_k)^2 \tag{15}$$

The coefficient of variation in X is estimated as $V_X = \sigma_X/\bar{X}$

The results of the Delphi are summarized later in this report.

Stochastic Modeling of Load Events

Load events occur randomly in time, and the magnitude during any event also is random. If the effect of the load on the structure is static, it can be assumed that the load magnitude remains essentially constant during an event with mean duration, τ. (If the load varies slowly in the interval, the load magnitude can be set equal to the maximum value within the interval). With these assumptions, each load can be modeled as a sequence of random pulses (a so-called pulse process), as illustrated by the sample functions in Figure 2. Permanent loads, such as dead and permanent equipment loads, are essentially constant in time. Sustained loads may change their magnitude in a stepwise fashion from time to time but in between these changes remain relatively constant. Hydrostatic loads fall into this category. Such loads may be intermittent in nature, i.e., there are substantial periods when the load is not acting. Finally, transient loads occur infrequently and usually last for a very short duration; the durations of such loads often are so short in comparison to those of sustained loads that they can be modeled as impulses. Earthquake loads and barge impact loads are examples of short-duration transient loads. The probability densities for these load processes sampled at an arbitrary point in time are illustrated at the left-hand side of each sample function in Figure 2. The discrete probability mass at zero magnitude represents the probability that the load is absent (not acting) at the time of load sampling.

The load processes in Figure 2 must be described statistically to perform the load combination analysis. The probability distributions of the point-in-time loads, Q, and the maximum loads to occur during an interval of time (0,t), Q_{max}, are required for load and load combination analysis (Turkstra and Madsen 1980; Ellingwood et al. 1982). It often is assumed that the occurrence in time of load events is described by a Poisson process. With the Poisson model, the probability of observing n events in interval $(0,t)$ is

$$P[N(t) = n] = \frac{(\lambda t)^n\, e^{-\lambda t}}{n!};\ n = 0,1,2,\ldots \tag{16}$$

where

λ = mean rate of occurrence of events

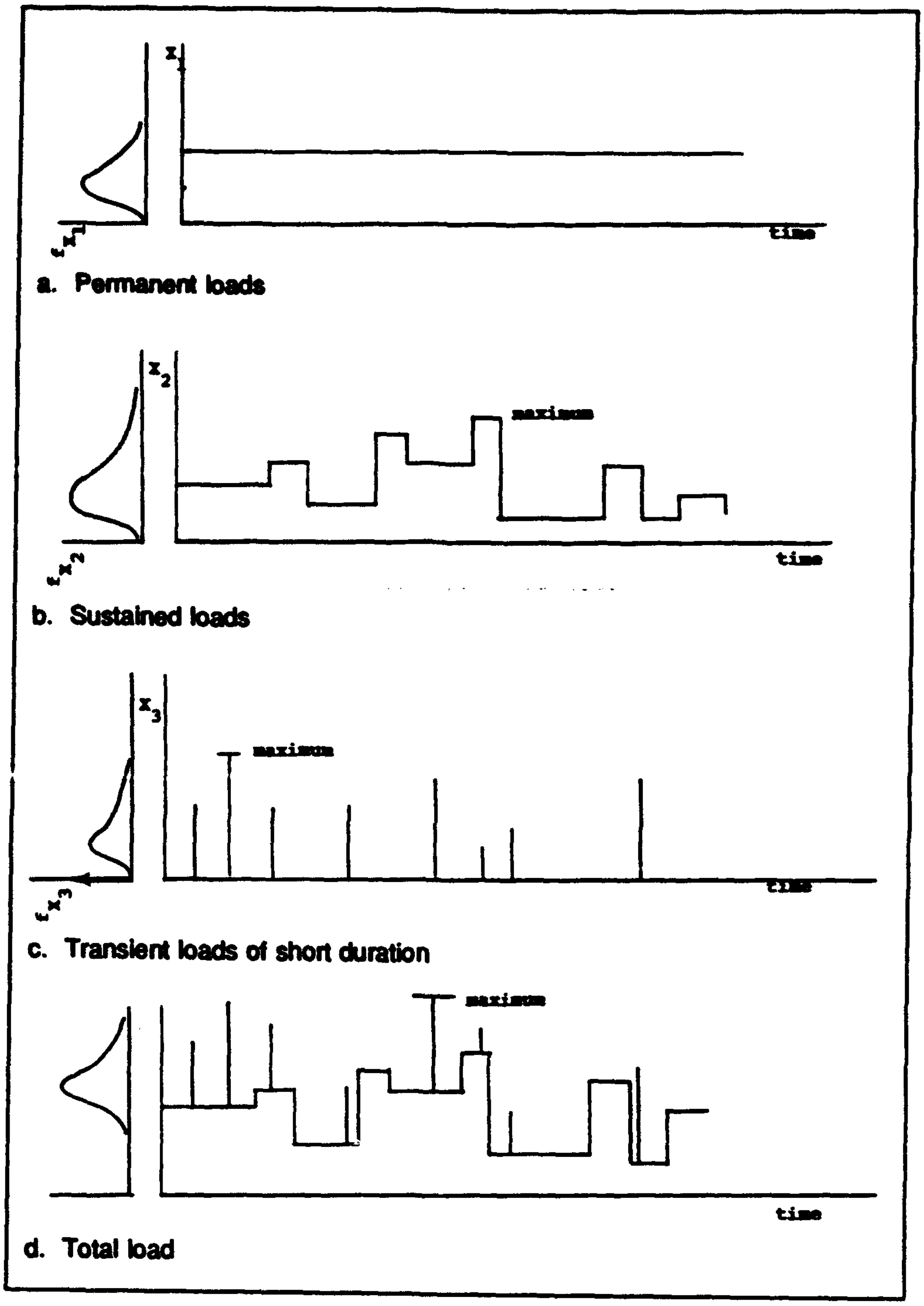

Figure 2. Pulse process models of loads

The probability distribution of the maximum load to occur during $(0,t)$ can be obtained from the theorem of total probability

$$F_{Q_{max}}(x) = \sum_{n} P\left[Q_{max} < x | N = n\right] P[N(t) = n] \tag{17}$$

If the individual loads in the sequence are assumed to be identically distributed and statistically independent random variables, the cumulative distribution function of the maximum load in $(0,t)$ is obtained from Equations 16 and 17 as

$$F_{Q_{max}}(x) = \exp\left[-\lambda t(1 - F_Q(x))\right] \tag{18}$$

This distribution of the maximum load demonstrates the importance of obtaining data on the rate at which accidental events occur (parameter λ) and on their magnitude (described by $F_Q(x)$).

The distribution of the maximum of a combination of time-dependent stochastic variables generally cannot be determined exactly, and approximations must be sought. Several methods for evaluating combinations of stochastic variables are available (Larrabee and Cornell 1981; Pearce and Wen 1984; Turkstra and Madsen 1980). All of them require information on the mean rate of occurrence, λ, of each load event, the duration, τ, of the event (see Figure 2), and the probability distribution, $F_Q(x)$, of each load magnitude.

Suppose that a combination of loads is defined as

$$U(t) = X_1(t) + X_2(t) + \ldots \tag{19}$$

$$U_{max} = max U(t); \quad o \leq t \leq T \tag{20}$$

where

$X_i(t)$ = stochastic load event

An upper bound for the probability that U_{max} exceeds the value, x, during period (O,T) is given by

$$G_{U_{max}}(x) < \nu_U(x)T \tag{21}$$

where

$\nu_U(x)$ = mean rate at which the combined process $U(t)$ crosses x with a positive slope

The function $\nu_U(x)$ can be approximated by (Pearce and Wen 1984)

$$\nu_U(x) = \sum_i \nu_i\, G_{U_i}(x) + \sum_i \sum_j \nu_{ij}\, G_{Ui+Uj}(x) + \ldots \tag{22}$$

where

G_{Ui} and ν_i = conditional probability of U_i exceeding x and mean occurrence rate of U_i alone

G_{Ui+Uj} and ν_{ij} = conditional probability of U_i+U_j exceeding x and mean occurrence rate of U_i+U_j occurring simultaneously

If the individual loads are modeled as Poisson pulse processes, the mean rate of occurrence and duration are sufficient to describe the temporal characteristics of the load. The probability, p_i, that the load is nonzero at any time equals $\nu_i \tau_i$. If the load is always nonzero, p_i = 1.0; if the load is intermittent, pi will be less than 1.0. If two intermittent processes combine, the mean rate of occurrence and duration of a coincidence of the two loads i and j are given by (Pearce and Wen 1984)

$$\nu_{ij} = \nu_i\, \nu_j\, (\tau_i + \tau_j) \tag{23}$$

$$\tau_{ij} = \frac{\tau_i \tau_j}{\tau_i + \tau_j} \tag{24}$$

As can be seen from Equations 21 and 22, events (or combinations of events) with very small ν_i (or ν_{ij}) contribute very little to the probability of exceeding x. Load combinations involving loads with very low probability of coincidence do little to enhance structural reliability. This observation will be used subsequently to screen certain load combinations from further consideration.

4 Load and Resistance Data for Lock Structures

Basic Description of Structural Loads

Structural actions on miter gates due to hydrostatic pressure (H_s), temporal head (H_t), hydrodynamic load (H_d), equipment loads (Q), and impact (I_m) arise from the normal operation of the locks. Thus, the statistical analysis of structural loads requires knowledge of the operating characteristics of the lock gates, which depends on the usage of the locks. General operating characteristics of locks on heavily traveled rivers may be quite different from those on secondary rivers, and it might be expected that the design requirements would depend on river use. For this study, the emphasis is on high-use rivers because of the economic impact of lock design and operation.

The "Ohio River Standard" (110 by 600 ft or 34 by 183 m) lock is common on rivers in the North Central Division, USACE. There are 21 of these locks on the Mississippi and 7 on the Illinois River (Reuter 1990). The towboat or tug and the collection of unpowered barges are referred to collectively as the "tow." On the Upper Mississippi River, tows are limited to a maximum of 15 barges which typically are 35 ft wide by 195 ft long (11 m by 59 m). When locking through an Ohio Standard Lock, such tows must be reconfigured. The first lockage would involve nine barges, and the second lockage would involve six, plus the towboat.

The starting point for load event analysis is the rate of occurrence of lockages. The "Lock Accident Study," TR-REMR-HY-7 (Martin and Lipinski 1990), Appendix B, provides data on number of lockages and total tonnage obtained from seven USACE District Offices, comprising up to 11 years of data at 80 locks on 10 rivers. A review of these data revealed that on the Mississipp. River, lockages occurred nearly uniformly during the period April through December of each year, while during the January through March quarter, lockages were approximately 70 to 80 percent of those in the other three quarters. On the Ohio River, on the other hand, lockages were essentially uniform throughout the year. This lack of a strong seasonal effect implies that lockages may be assumed to occur uniformly during the entire year for load combination analysis purposes. A summary of 1986 lockage data for six locks showing the highest average annual damages to miter gates is presented in Table 1.

Table 1
Lockages in 1986 in Locks with High Damage Rates

River	Lock	Lockages	Barges	Tonnage (thousands)	Rate (mo^{-1})
Mississippi	17	4,238	25,616	27,125	353
	21	4,389	25,726	26,038	366
	22	4,443	26,446	26,896	370
	24	4,581	28,084	28,161	383
	25	4,718	28,101	28,159	393
Ohio	Gallipolis	5,972	34,617	34,282	498

Some of the data presented in Appendix B of REMR-HY-7 (Martin and Lipinski 1990) are for 9, 10, or 11 months; all data in Table 1 have been normalized to a 12-month period. No differentiation was made between upstream and downstream lockages in the TR REMR-HY-7 database. Although some of the lockages might have been associated with pleasure craft, the database is not specific on this point.

An idealization of lock operation for purposes of idealizing the temporal characteristics of loads H_s, H_d, H_r, and Q is shown in Figures 3 and 4. Figure 3 shows a downstream lockage and the loads developed. Figure 4 illustrates the temporal characteristics of the resulting loads from two successive lockages, the first from a vessel traveling downstream and the second from a vessel traveling upstream. The upper gate is assumed to be open at the beginning of the cycle. Several distinct stages that are repeated in sequence for each lockage are identified in order to relate temporal characteristics to available data. Appendix D of TR REMR-HY-7 (Martin and Lipinski 1990) indicates that at Mississippi River Locks 24 and 25, the "processing time" is approximately 80 min. In the context of Figures 3 and 4, the processing time is interpreted as the time from when the tow begins maneuvering to enter the lock to the time it exits, including gate operation time. The "chambering" time (time to dewater the lock and open the lower gates) for these two locks averages about 35 min. The time required to open or close the gates is approximately 2 min. (Green, Murphy, and Brown 1964), and the dewatering (watering) period is approximately 30 min. At a typical tow entry velocity of 2 mph (0.90 m/s) (Reuter 1990), the entry or exit period is about 5 to 7 min. Finally, operator experience suggests that tows typically spend 20 to 30 min in the holding area in the handling/maneuvering phase. These times are summarized:

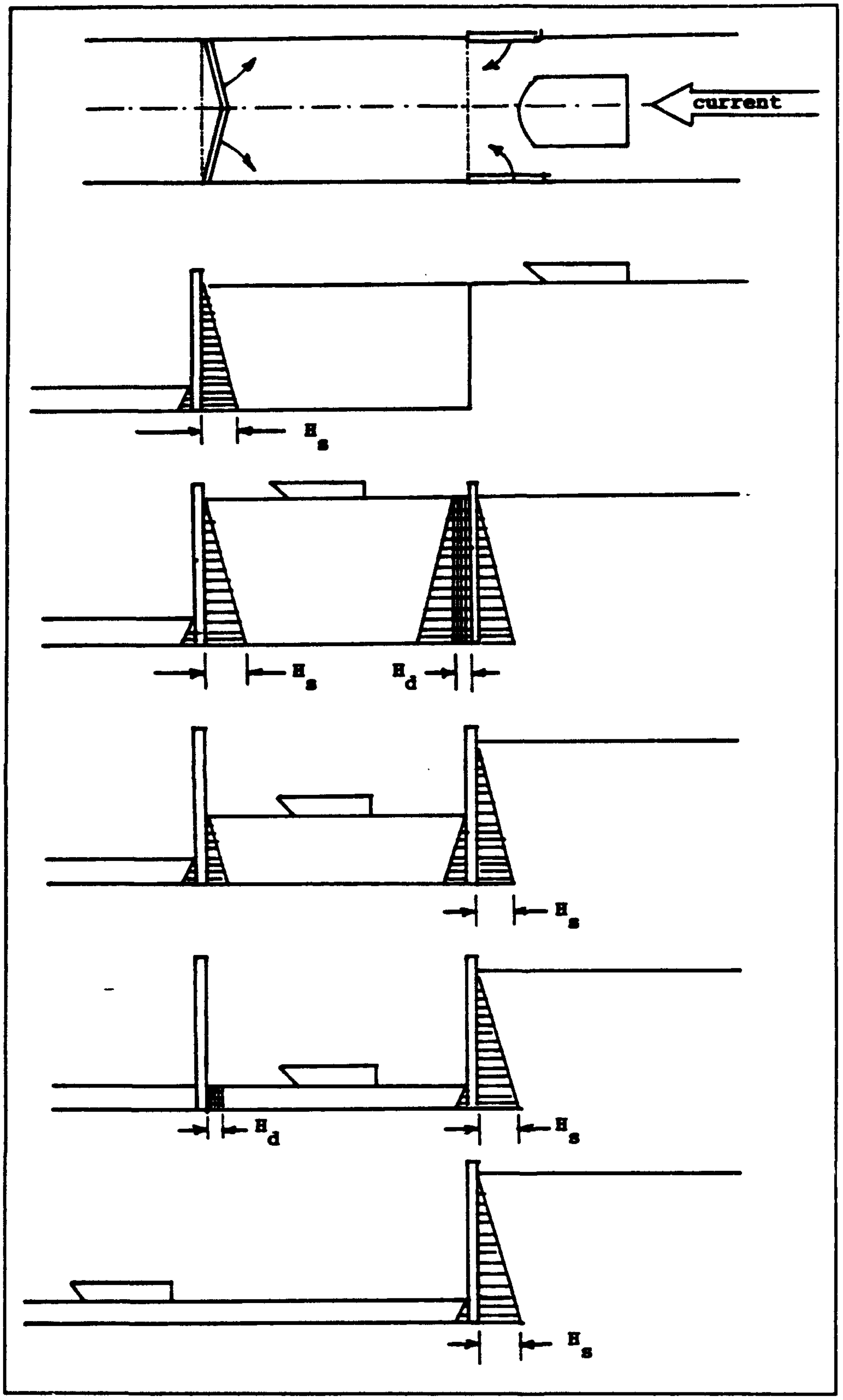

Figure 3. Idealization of loads developed during lockage

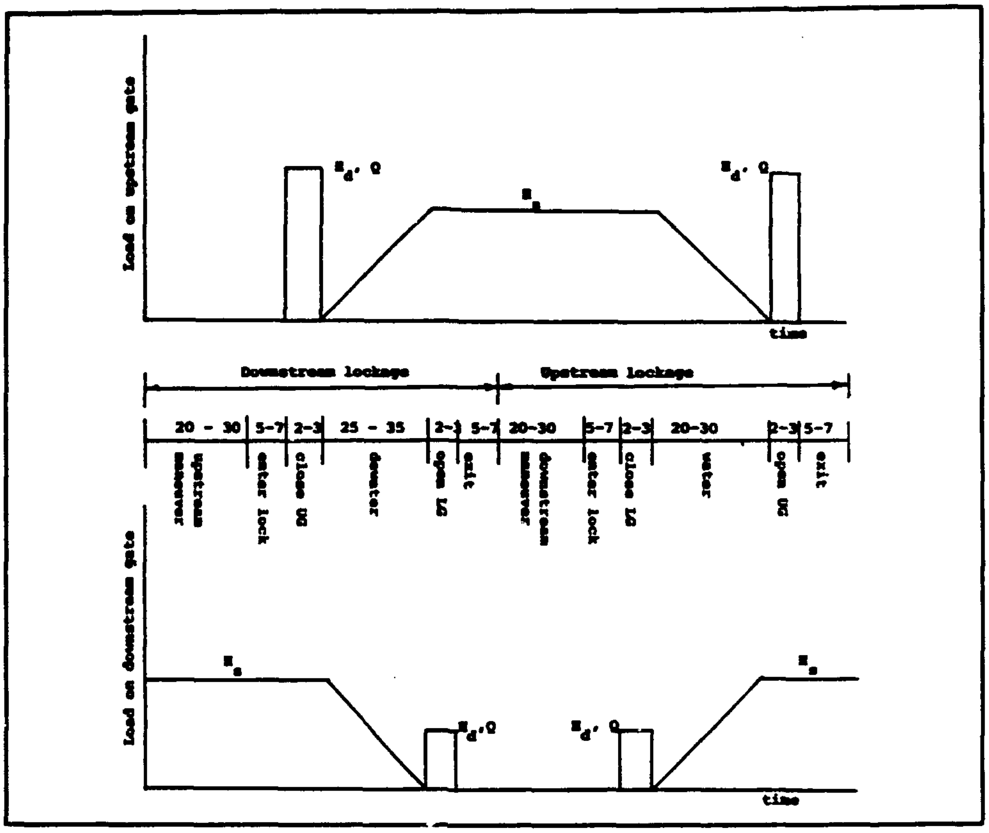

Figure 4. Temporal characteristics of loads during lockage

Handling and maneuvering	20 to 30 min
Tow enters upper lock	5 to 7 min
Close upper gate	2 to 3 min
Dewater lock	20 to 30 min
Open lower gate	2 to 3 min
Tow exits	5 to 7 min
Total	54 to 80 min

Operating Loads

The hydrostatic load, H_S, on the miter gate in the closed position is determined by the upper and lower pool elevations. These pool elevations are site-dependent. Their variation is dependent on weather conditions upstream, primarily rainfall/runoff. Figure 5 shows a vertical section through a typical miter gate, illustrating the resultant hydrostatic pressures and the overall force developed.

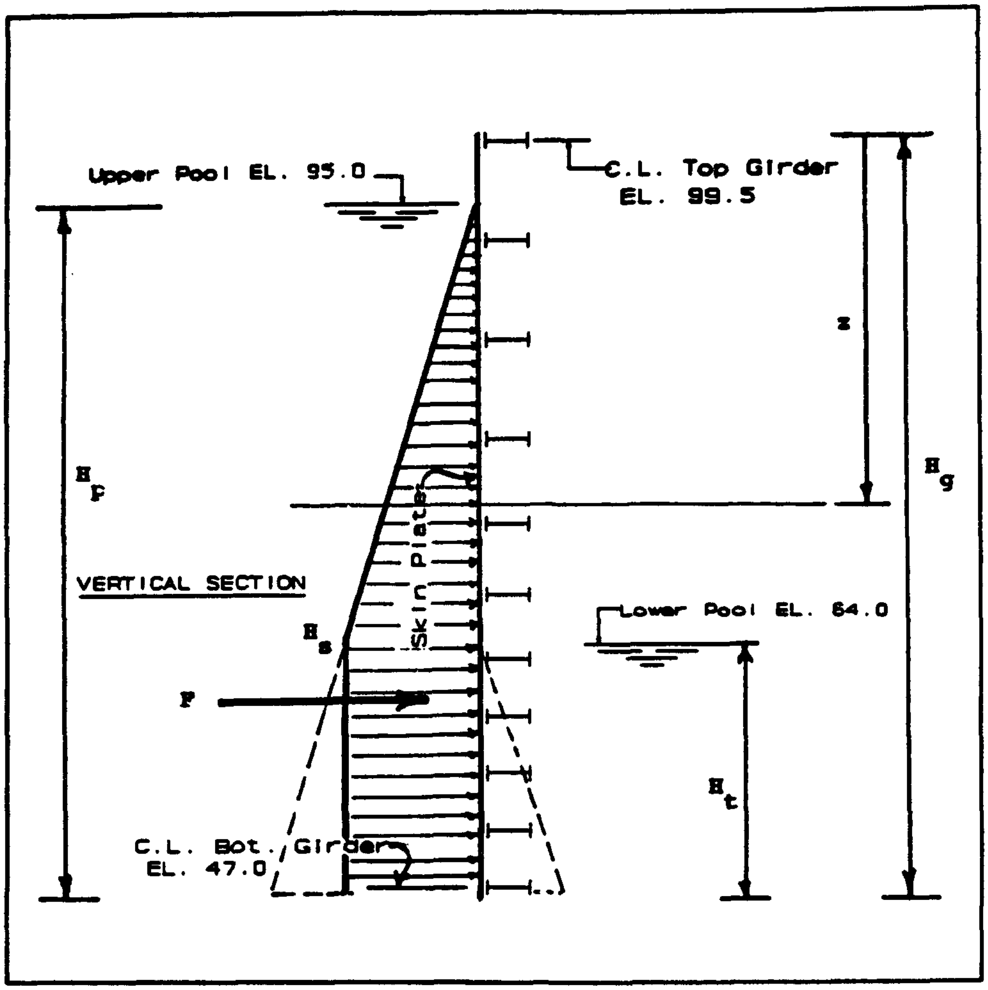

Figure 5. Typical miter gate vertical cross section

The resultant force per linear unit width acting on the miter gate is

$$F = \gamma_w(H_p^2 - H_t^2)/2 \tag{25}$$

where

γ_w = density of water

H_p = depth of upper pool

H_t = depth of lower pool

Since the differential elevation is $\Delta = H_p - H_t$,

$$F = \gamma_w \Delta (H_p + H_t)/2 \tag{26}$$

The maximum annual resultant force occurs when Δ is at its annual maximum value or when H_p is at its annual maximum value. The pressure at any distance, z, below the top of the gate is given as

$$p(z) = \begin{cases} 0\ ;\ z \le H_g - H_p \\ \gamma_w[z - (H_g - H_p)];\ H_g - H_p < z < H_g - H_t \\ \gamma_w(H_p - H_t) = \gamma_w \Delta;\ H_g - H_t < z \end{cases} \tag{27}$$

where

H_g = height of the gate

The load per unit length on a horizontal girder at z is

$$H_s = s\ p(z) \tag{28}$$

where

s = vertical spacing of the girders

The mean and standard deviation of F or H_s can be developed from Equations 26 or 28, as appropriate.

Stage duration curves at Lock and Dams (LD) 24, 25, and 27 on the Upper Mississippi River and at Cheatham LD on the Cumberland River were analyzed to determined statistics of annual extreme pool and tailwater elevations and annual maximum differentials in elevation. The elevations are measured upstream and downstream from the LD's. Statistics on pool elevation and differential are summarized in Table 2. While there is natural variation in pool elevations from year to year, the variability in elevation and in differential is smaller than one might expect because the river level is controlled by dams upstream and downstream for flood control.

The characteristics of river flow are different from lock to lock, and the annual maximum differential does not necessarily coincide with the maximum pool. At LD's 24 and 25, for example, the annual maximum differential often occurs at the same time as the maximum pool, whereas at LD 27, they rarely

Table 2
Statistics for Hydrostatic Loads

LD	Years	Pool			Differential		
		Mean	SD	COV	Mean	SD	COV
24	48	451	2.6	0.01	13.8	0.94	0.07
25	48	437	2.9	0.01	14.3	1.25	0.09
27	37	417	7.5	0.02	15.7	2.65	0.17
Cheatham	15	389	4.0	0.01	unavailable		

occur together. The mean values of pool elevation and mean differentials shown in Table 2 are dependent on the site of the LD. They are of less interest for the probabilistic load modeling herein than the standard deviations (SD) or coefficients of variation (COV), which measure the relative control on water level provided by upstream and downstream dams and directly impact the variability in the load that must be taken into account in the load combination analysis.

The randomness in F and H_s can be inferred from the statistics presented in Table 2. At LD 27, where the maximum overall force is associated with maximum pool, the COV in F from Equation 26 is about 0.30, while the variability in H_s from Equation 28 is 0.17. At LD's 24 and 25, where maximum force and pressure both tend to be associated with the maximum differential, the COV's in F and H_s are on the order of 0.20 and 0.10, respectively. These observations are based on an examination of data at only three sites. Pending acquisition of additional data, the COV in H_s and F is set equal to 0.25; this value contains an allowance for load modeling uncertainty.

The hydrodynamic load H_d and equipment load Q arise during operation of the miter gates from fluid-structure interaction. Miter gates are never operated under static head, and thus loads H_d and Q are mutually exclusive of H_s. Water level on each side of the gate is equalized before the movement of the gate is initiated. H_d and Q include effects of friction, wind loads, surges, and hydraulic drag. Friction and wind effects normally are very small in comparison with the hydrodynamic effects due to fluid-structure interaction (Green, Murphy, and Brown 1964). Under normal operating conditions, the effects of H_d and Q represent a statically equilibrated force system. These loads are not uniform in time. Uncertainty in H_d and Q may arise from variations in pool elevations, which affect the depth of submergence, from surge effects caused by movement of the gates or overtravel of water in the culvert system during filling and from variations in the angular velocity of the gate.

Green, Murphy, and Brown (1964), in Technical Report (TR) 2-651, summarize the results of tests conducted on 1:20-scale models of miter gates. Three different types of operating machinery linkage were considered: modified Ohio River, Panama, and Ohio River. To consider a variety of operating conditions, depth of submergence, operating time, chamber length, bottom clearance under gate, and presence of barges in the chamber were varied independently. These model studies indicate that the hydrodynamic effects depend primarily on the angular velocity of the gate leaf and the depth of submergence, and that the maximum effects usually occur as the gate leaf reaches the mitered position upon closure. Plots of maximum torque versus operating time and depth of submergence (e.g. Plate 47 of TR 2-651) show that the loads are relatively predictable and depend primarily on the velocity of the gate leaf and the depth of submergence, both of which are likely to be known. The Appendix to TR 2-651 summarizes the results of scaled model (1:25) tests conducted in 1942 for the Third Locks of the Panama Canal. Figures 7 and 8 of that Appendix show the torques measured during opening and closing operations at various times with baseline conditions. The maximum deviation from average in four tests was 8.1 percent during the opening operation and 3.8 percent during the closing operation. Using these data with Equation 10, the implied coefficient of variation in Q would be about 0.05.

This variability in Q is associated with the basic hydrodynamic effect, since the tests were conducted under baseline conditions. Additional variability would arise from variations in the pool elevations (submergence) and angular velocity of the gate leaf; these additional uncertainties would increase the overall variability in H_d and Q.

The Delphi participants were requested to provide information on the differential in water level as the gate moves through the water. The current design value is 6 in. (154 mm). On the basis of the Delphi and using Equations 11 through 15, the mean differential is 4.4 in. (112 mm) and the COV is 0.53. This value represents a substantially higher uncertainty in Hd than that indicated from the model tests discussed above.

If there is a submerged obstruction, the bottom of the gate leaf may bind during operation while the operating load is applied at the top of the leaf, causing the gate leaf to twist. In this case, H_d and Q do not form a force couple. The maximum twist in the leaf depends on the maximum operating equipment load, Q. Operating machinery often includes a safety device that limits the maximum force and acts as a structural fuse. Common devices include shear pins that are attached to the operating strut and fail at a certain force, hydraulic relief valves limiting the cylinder pressure supplied to the operating strut, and overload torque devices on electric motors. The maximum operating strut force is supplied to the structural designer by the mechanical engineer.

Many Delphi respondents (59 of 108) indicated that the load-limiting device on their gates had been activated at some time. The frequency of activation varied, but most values cited ranged from 1 in 1,000 lockages to 1 in 100 lockages. Reasons cited included debris or ice caught in the gate bays, high water or opening against a head, and human error. Shear pins usually were reported as mild steel 0.5 to 0.75 in. (13 to 19 mm) in diameter. The behavior of such pins is relatively predictable; the mean and coefficient of variation in ultimate shear strength are about 1.25 times the nominal $R_n = F_v A_{bolt}$ and 0.10 (Galambos et al. 1982). Hydraulic operating systems with relief valves apparently are most common. The behavior of such systems is at least as predictable as the behavior of shear pins. Sources of variability arise from possible malfunctions of the safety relief valve on the cylinder. Few data were reported on torque limits for electric motors.

Temporal hydraulic loads, H_t, occur due to temporary variation in water level and wave action caused by tow movement in the lock or overfilling or underfilling the lock chamber. These loads can occur while the gate is in the mitered or open position. Delphi participants were asked to provide information on both cases. The current design value for any cause of H_t is 15 in. (381 mm).

In the mitered position, H_t can occur due to overfill or underfill of the lock chamber. The momentum of the flowing water determines whether the chamber is filled or emptied to the appropriate level. Using Equations 11 through 15, the mean and coefficient of variation for overfill were 5.4 inches (137 mm) and 0.94; for underfill, 3.8 in. (97 mm) and 1.03. Waves also may be produced by the towboat and barge as the lock is exited. The mean and coefficient of variation were 12.0 in. (305 mm) and 0.76; the latter mean H_t is the largest value of all those reported.

In the open position, H_t may occur from water entrapment as the gate is moved into the lock wall recess. The difference in water elevation behind the gate and in the chamber has a mean value of 3.0 in. (76 mm) and a coefficient of variation of 0.87. Waves also may be pushed outward toward an open and recessed gate as the tow enters or exits the lock. The mean and coefficient of variation for this event are 6.4 in. (163 mm) and 0.88.

The means for temporal head, H_t, for gates in the open position are substantially less than for gates in the mitered position. This suggests that H_t should be treated differently in load combinations intended to be applied to open and mitered gates; in particular, H_{tm} (mitered) and H_{to} (open) should be specified in separate load combinations addressing these conditions.

Ice and mud (denoted C and M) may accumulate on a miter gate and act as a gravity load that adds to the weight of the gate. This accumulation causes twist about the shear center of the gate when the gate is not in the mitered position. A significant number of the Delphi respondents believed that ice accumulation is a significant problem, both on upstream and downstream faces of the gate. Most accumulations reported ranged from 6 to 36 in. (152 to 914 mm); using Equations 9 and 10, the mean and coefficient of variation would be 21 in. (533 mm) and 0.35. Mud and silt do not appear to be as significant a consideration; accumulations reported ranged from 2 to 12 in. (51 to 305 mm), suggesting a mean of 7 in. (178 mm) and a coefficient of variation of 0.36.

In addition to the accumulation of ice and mud on the gate, accumulation of ice sheets in the water on either side of the gate may result in lateral forces on the gate due to lateral expansion of the ice. Many Delphi respondents (59 out of 108) indicated that this is a potential problem. An analysis of their responses suggests that the mean ice thickness is 10.1 in. (257 mm) and the coefficient of variation is 0.77.

Impact loads

Impact loads, Im, arise from accidents in which vessels collide with lock structures. Such accidents may be caused by misalignment during entry or departure, failure of steering or towing accessories, wind and current, turbulence, excessive entry velocity, or pilot error. Each lockage presents an impact hazard. These accidents can result in extensive repairs to both the lock and/or the vessel and in long costly delays. Vessel entry is the most critical part of a lockage. Proper entry velocity is 2-1/2 mph (1.1 m/s) or less. The primary causes of accidents include excessive speed or inability to stop; misalignment of the tow; faulty communications between the lockmaster and tow operator; equipment failure; pilot error; surges in the lock chamber due to filling system; river stage and discharge; current patterns; and visibility (Martin and Lipinski 1990; Reuter 1990). The Delphi reported 71 instances of collision over an unspecified period of time. Most of these collisions involved damage to the girders or the skinplate of the gates.

The incidence of significant impacts, the duration of the impact hazard, and the intensity of the impact are required for event combination and reliability analysis. The basis for the data on impact occurrence is the records kept by each USACE District office on accidents involving damage to U.S. Government property of collisions at locks and dams within the individual jurisdictions. These data are summarized in "Lock accident study," a technical report by Martin and Lipinski (1990). Accident data reflecting the severity and frequency of vessel collision to USACE lock facilities, specifically to miter gates, were collected.

Data on accidents involving lock structures and, in particular, miter gates are summarized for several heavily used rivers in Table 3. These data present a rough overall picture of the incidence of accident loads and, in particular, those affecting miter gates. The Mississippi, Illinois, Ohio, and Tennessee rivers are considered "high-use" rivers, while the Cumberland River (Cheatham Lock) is considered a "low/moderate use" river. The average rate of accidents involving miter gates at locks on the Mississippi, Illinois, and Ohio rivers is approximately 0.06/month. A majority of these accidents involve barge impact. Not all accidents result in significant lock downtime, however.

It has been suggested that downstream tows carry a higher impact hazard than upstream tows. Data obtained independently on operation of the Demopolis Lock, USACE District, Mobile, over the 11-year period 1979 through 1989 (private communication), indicate that the annual number of

Table 3
Accident Incidence Statistics by River System

River	Locks No.	Lock-Months	Accidents: All Accidents Number	Accidents: All Accidents Rate (mo^{-1})	Accidents: Miter Gates Number	Accidents: Miter Gates Rate (mo^{-1})
Upper Miss.						
Rock Island	12	1,248	124	0.099	76	0.061
St. Louis	4	352	97	0.276	58	0.165
St. Paul	12	1,500	173	0.129	134	0.089
Total	28	3,100	394	0.127	268	0.086
Illinois	8	832	21	0.025	9	0.011
Ohio						
Huntington	6	738	84	0.114	55	0.075
Louisville	8	808	52	0.064	25	0.031
Pittsburgh	6	540	32	0.059	na	na
Total	20	2,086	168	0.081	80	0.052
Tennessee	8	576	32	0.056	6	0.010
Cumberland	1	72	1	0.014	0	- -

upstream and downstream tows is roughly equal. However, the annual downstream tonnage exceeded upstream tonnage by factors ranging from 2:1 to more than 10:1 over the 11-year period. Miter gates are not symmetric - the upstream face is smooth while the stiffeners are fabricated on the downstream face. It is assumed that for data analysis and design purposes it is not necessary to distinguish between upstream and downstream tows. Since the downstream and upstream tows occur more or less uniformly over the year, the difference in tonnage downstream and upstream simply would contribute to the variability in the impact load, given that an impact occurs.

Some locks are more hazardous and prone to accidents because of their position with respect to the channel, turns in the river, current outdrafts, wind and turbulence, and other factors noted previously (Reuter 1990). Additional information is provided by considering accident statistics at specific locks. Table 4 presents the same information as Table 3 for the six locks with the highest average annual damages to miter gates (Table 8 (Martin and Lipinski 1990)). The five Mississippi River locks are all single-chamber locks, while the Gallipolis Lock has an auxiliary chamber. These locks also appear in lists of the locks with the most number of accidents, highest average cost per accident, or highest average annual damages, and with the most damages (Martin

Table 4
Accident Incidence Statistics for High-Risk Locks

River	Lock	Lock-Months	All Accidents		Miter Gates		Cat/MG	
			No.	Rate (mo^{-1})	No.	Rate (mo^{-1})	No.	Rate (mo^{-1})
Miss.	17	104	10	0.096	8	0.077	4	0.038
	21	104	8	0.077	5	0.048	4	0.038
	22	104	25	0.240	18	0.173	5	0.048
	24	88	26	0.296	18	0.205	3	0.034
	25	88	29	0.329	15	0.170	1	0.011
Ohio Gallipolis		123	52	0.423	33	0.268	7	0.057

and Lipinski 1990). The "All accidents" column refers to total accidents involving lock structures; "Miter" refers to accidents involving miter gates; and "Cat/MG" refers to those accidents having damages greater than $50,000 in which the miter gate was struck. Approximately 25 percent of the accidents involving miter gates fall in the "Cat/MG" category. Assuming that these percentages apply to locks in general, the mean rate of occurrence of structurally significant accidents to miter gates is approximately 0.016/month, or about 1 every 5 years. Accidents over $50,000 accounted for 77 percent of total damages, and over 72 percent of such accidents involve miter gates.

The force due to tow impact depends on the mass of the tow and its velocity at impending collision. Because of the limited data available to describe these parameters, several questions on the Delphi were designed to elicit this information. Included were queries on the number of barges per lockage, fractions of barges that were full, partially full, or empty of cargo, weight of a fully loaded barge, and initial and terminal velocity of the tow in the lock chamber. A large amount of data were generated in response to these queries, and it was not possible to analyze these data completely in the course of this study. It is clear, however, that impact should be treated as a concentrated force in design rather than as a uniformly distributed load, as is current practice.

The total mass (weight) of a tow in the lock is modeled as

$$W_T = \sum_{i=1}^{N_b} W_{bi} \tag{29}$$

where

W_{bi} = weight of an individual barge

N_b = number of barges comprising the tow

Both W_{bi} and N_b are random variables. The mean and variance of W_T are (Benjamin and Cornell 1970)

$$\overline{W}_T = \overline{N}_b \, \overline{W}_{bi} \tag{30}$$

$$\sigma^2_{W_T} = \overline{N}_b \sigma^2_{W_{bi}} + \overline{W}^2_{bi} \sigma^2_{N_b} \tag{31}$$

in which $\overline{N}_b$, $\overline{W}_{bi}$ are mean values and $\sigma^2_{N_b}$ and $\sigma^2_{W_b}$ are variances of N_b and W_{bi}. Note that randomness in both W_{bi} and N_b contribute to the overall uncertainty in W_T.

A standard barge measures 35 by 195 ft (10.7 by 59.4 m) and, when fully loaded, draws 9 ft (2.74 m); when empty, the barge draws 1.5 ft (0.46 m). Thus, the cargo capacity of a fully loaded barge is approximately 1,500 tons; the empty barge displaces about 290 tons. The randomness in the weight of an individual barge arises from its overall capacity, density of cargo, and the percentage of barge capacity taken up by cargo. The mean and variance in W_b were obtained from the lock operator responses to questions concerning the relative percentage of fully or partially loaded barges passing through the lock and the type of barge. Taking into account the fact that some barges are fully loaded while others are not, the overall mean and standard deviation in individual barge weight for all locks reporting such weights in the Delphi were 870 tons (7.89×10^5 kg) and 303 tons, respectively; the COV is 0.35. Examination of subsets of the data led to similar values. For example, in subset (1), 10 locks measuring 110 by 600 ft on the Mississippi River, St. Paul District, were considered. For this subset, the mean and standard deviation in W_b were 911 and 120 tons, respectively. In subset (2), 10 locks measuring 110 by 600 ft on the Mississippi River, Rock Island District, were considered. For this subset, the mean and standard deviation in W_b were 986 and 298 tons, respectively. Both subsets correspond to high-use rivers.

A brief examination of data reported for other rivers and districts did not indicate any significant variations in the statistics of individual barge unit weights. These barge weights are consistent with those summarized in Table 4.1 (Martin and Lipinski 1990); at LD 25, for example, the unit barge weight reported in 1986 was 1,002 t/barge. Additional data on barge mass were obtained from an independent survey by USACE personnel (Chasten 1992a) of 48 lockages at locks on the Ohio, Kanawha, and Tombigbee Rivers with chamber lengths ranging from 360 ft (110 m) to 1,200 ft (366 m). The mean and standard deviation of barge weight for downstream lockages was 1,361 t/barge and 623 t/barge, respectively. For upstream lockages, the mean and standard deviation were 659 t/barge and 703 t/barge, respectively; the high standard deviation is due to the fact that many of the barges were empty. The overall tonnage in each lockage clearly was dependent on the length of the lock; thus, it would be reasonable to assume that the total mass at impact is linearly proportional to the chamber length for chambers of the same width.

The number of barges in a lockage, N_b, clearly depended on the dimensions of the chamber, and the mean and variance in N_b are similarly dependent. Chamber sizes in the Delphi varied from 56 by 360 ft (17.1 by 109.7 m) to 110 by 1,200 ft (33.5 by 365.8 m). The lockmasters participating in the Delphi provided estimates on the frequency of the number of barges per lockage. These data can be used to determine $\bar{N}_b$ and σ^2_{Nb}; $\bar{W}_T$ and $\sigma^2_{W_T}$ follow directly from Equations 30 and 31. However, the number of barges per lockage reported in the Delphi frequently ranged from 1 to 15 in 110- by 600-ft locks that cannot admit more than 9 barges per lockage. In such cases, the data were renormalized by apportioning the barges to two successive lockages: the first consisting of nine barges and the second consisting of the the remaining barges (plus the towboat). For data subset (1) above, $\bar{W}_T$ = 6,031 tons and σ_{W_T} = 2,343 tons; the coefficient of variation is 0.39. For data subset (2) above, $\bar{W}_T$ = 6,863 tons and σ_{W_T} = 2,242 tons; the coefficient of variation is 0.33. To this, of course, must be added the weight of the empty barges in computing the total mass involved in the impact. These values are higher than those implied from the data in Table 1; at LD 25, e.g., $\bar{W}_T$ averages 5,968 t/lockage. Additional data was provided for the Demopolis Lock during 1979-89. The lock chamber at Demopolis measures 110 by 600 ft, admitting a maximum of nine barges/lockage. The mean and standard deviation of estimated tonnage/tow downstream was 6,615 and 595 tons, respectively. Data reported from the same lock (Chasten 1992a) on 12 downstream lockages showed that the mean and standard deviation were 7,117 and 3,143 tons, respectively. Clearly, more work is required to evaluate statistics of mass at impact.

The lock operators were asked to provide information on the entry and approach velocity of tows in the lock. The entry velocity averaged 2.9 ft/s (0.88 m/s), with a COV of 0.56. The approach velocity averaged 2.2 ft/s (0.66m/s), with a COV of 0.62. The kinetic energy of the tow is proportional to the square of its velocity, and thus the coefficient of variation in kinetic energy at impending impact may be over 100 percent if these estimates are to be believed. Additional confirmation of these values was obtained from an independent survey of barge tow velocity measurements (Chasten 1992a). The mean entry velocity ranged from 1.8 to 3.4 ft/s (1.2 to 2.3 mph), consistent with prudent navigation. The terminal (approach) velocity is defined as the tow velocity in the last 50 to 100 ft (15 to 30 m) of the lock. The weighted mean and COV in terminal velocity are 1.26 ft/s and 0.42, respectively. Interestingly, neither the entry nor the terminal velocity depended on the length of the lock.

A set of barge impact tests were conducted recently at LD 26 on the Mississippi River. The miter gates at LD 26 are vertically framed. Table 5 summarizes the loads measured from a fully loaded nine-barge tow of approximately 15,000 tons (Chasten and Ruf 1991).

Table 5
Barge Impact Test Results

Velocity (ft/s)	Impact Force (kips)
0.36	442
0.59	443
0.73	488
0.94	605

These tests were conducted at very low velocities to avoid permanent damage to the miter gates. Unfortunately, the relation between impact force and velocity in Table 5 is not linear, making it difficult to determine forces due to the terminal velocities reported in the Delphi by extrapolation.

Dead load

The dead load consists of the weight of the miter gate. In contrast to the dead load in building structures, where nonstructural attachments are the main source of variability in dead load, miter gates are well defined as structures and their dead loads are quite predictable. Accordingly, the dead load is assumed to be described by a normal distribution, with a mean value equal to 1.0 D_n, where D_n is the nominal dead load and a coefficient of variation taken as equal to 0.05.

Live loads

Live loads are due to the weight of moveable equipment and attachments and individuals and their possessions. Live loads are not significant in design of miter gates, and thus need not appear in the load combinations. For appurtenant structures, the load combination for live loads appearing in ASCE 7-88 (ASCE 1990) is recommended.

Earthquake loads

Earthquake effects on building structures usually are determined using an equivalent static-for-dynamic analysis in which a base shear is computed from the seismic hazard and general structural characteristics, and from it a distribution of lateral forces is determined in a manner consistent with the fundamental mode shape of the structure. In the new NEHRP provisions (Federal Emergency Management Agency (FEMA) 1992), this base shear is determined as

$$V = C_s W \tag{32}$$

where

C_s = seismic design coefficient

W = total dead load and portions of other loads

Coefficient C_s is defined as

$$C_s = 1.2 A_v S/RT^{2/3} < 2.5 A_a /R \tag{33}$$

where

A_a and A_v = effective peak acceleration and velocity-related accelerations determined from seismic ground acceleration maps or site-specific seismic hazard analysis

S = coefficient for soil profile characteristics

R = structural response modification factor

T = fundamental period of the structure

The coefficient, C_s, is tantamount to an inelastic yield spectrum for an oscillator with 5 percent damping. For hydraulic structures, the seismic hazard analysis is used to determine a value of acceleration; inertial hydrodynamic forces on the gate resulting from the design ground motion are determined from Westergaard's equation (Westergaard 1933; Chiarito and Morgan 1991)

$$p(y) = 0.875 \gamma_w A_v \sqrt{Hy} \tag{34}$$

where

γ_w = density of water

A_v = velocity-related effective peak ground acceleration (expressed in units of gravitational acceleration, g)

H = pool depth

y = depth below pool surface

It is apparent that the basic seismic hazard analysis is a key ingredient in the analysis of earthquake force, regardless of which of these two approaches is taken. Research in earthquake-resistant design during the past 15 years has enabled seismic hazard analysis and design ground motions at a particular site to be placed on a probabilistic basis (Algermissen et al. 1982). Earthquake hazards in the western United States generally can be associated with a series of capable faults. Such faults are not apparent in the eastern United States, and the seismic hazard analysis there begins with an identification of postulated seismic source zones. Seismicity is determined within these zones of potential future earthquake occurrence, and mean rates of occurrence of earthquakes of various magnitudes (or intensities) are identified. Attenuation functions relating peak ground acceleration to magnitude and epicentral distance are used to relate the earthquake ground motion at the building site to the magnitude or intensity at the source. Finally, a probability distribution of effective peak ground acceleration at the site is determined by summing (integrating) over all possible earthquake sources and magnitudes consistent with each underlying source hypothesis. The result is usually presented as a complementary cumulative distribution function, $G_A(a)$, or seismic hazard curve, showing the annual probability of exceeding a specified ground acceleration.

Typical seismic hazard curves for building sites in the western United States and the eastern United States are compared in Figure 6. For moderate accelerations a seismic hazard curve can be described by a Type II distribution of largest values (Cornell 1968).

$$G_A\,(a) = 1 - F_A\,(a) = 1 - exp[-(a/u)^{-\alpha}] \tag{35}$$

where

$F_A(a)$ = cumulative distribution function of effective peak ground acceleration

a = ground acceleration u and α are parameters of the distribution, as described below

Parameter α determines the slope of the hazard curve. The curve in the eastern United States is much flatter (smaller α) than that in the western United States (see Figure 6); this is because of the relatively larger uncertainty associated with eastern seismicity due to the absence of large historical events during the period of modern instrumentation. The western U. S. hazard curve saturates at acceleration levels on the order of 1.2 to 1.8 times the design earthquake; in the eastern United States, the saturation level is unknown but is believed to be on the order of 2.0 to 3.0 times the design earthquake.

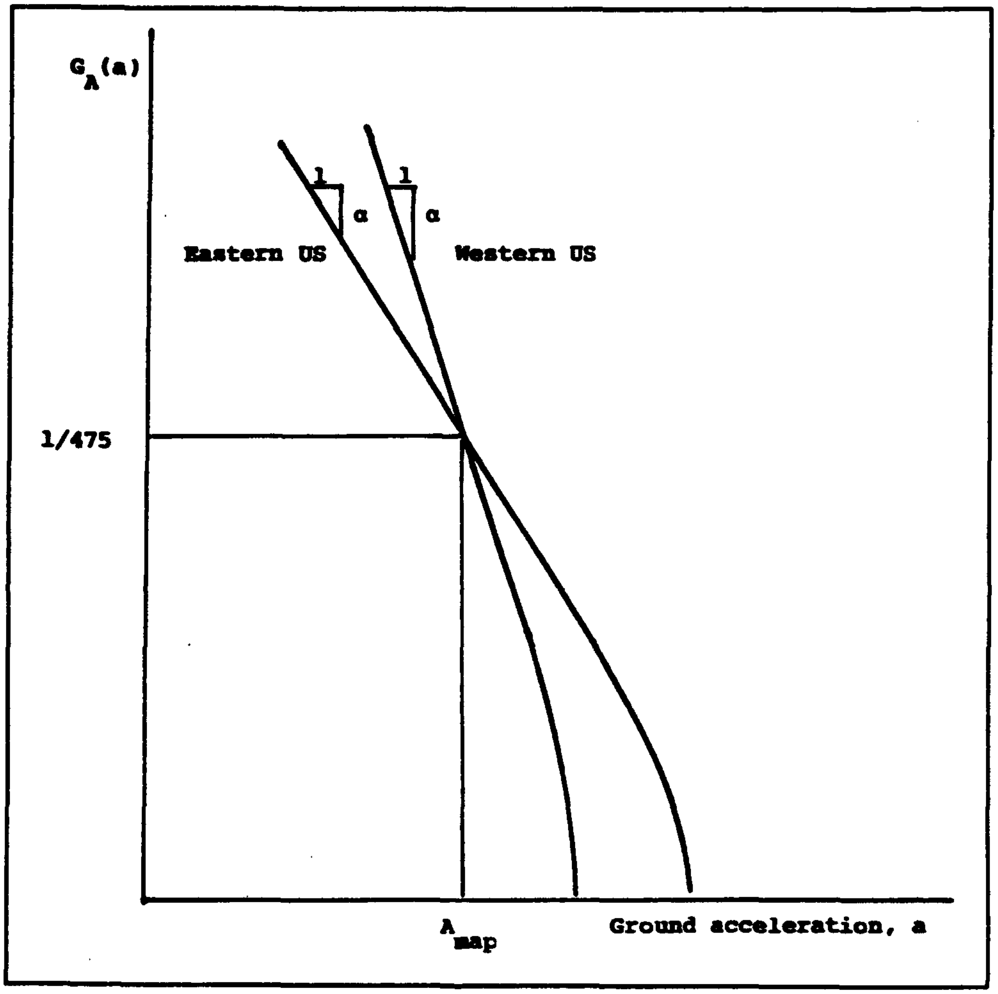

Figure 6. Typical seismic hazard curves

Parameter α is related to the coefficient of variation in annual maximum effective peak acceleration. The seismic hazard analyses (Algermissen et al. 1982) on which the NEHRP Recommended Provisions for Seismic Regulations (FEMA 1992) are based indicate that a tends to be larger for sites in the western United States, decreasing from about 5.5 (COV = 0.28) at San Francisco, CA, to approximately 2.3 (COV = 1.38) at Boston, MA, and Memphis, TN. The latter coefficient of variation is substantially larger than coefficients of variation in other loads considered in the load combinations.

The NEHRP Recommended Provisions are being adopted, with minor variants, by ASCE Standard Committee 7 for its revision to the Standard on Structural Loads and by the Model Codes. The design earthquake envisioned for buildings, as reflected in the design ground motion contour maps, is based on a specified probability of 0.10 or less of being exceeded in 50 years; this corresponds to a mean recurrence interval of about 475 years. This probability is much less than the probabilities of exceeding the design snow or wind loads in ASCE 7-88 (ASCE 1990), which are 0.01 to 0.02 on an annual basis. Earthquakes with magnitudes less than ML = 4.0 or Mercalli intensities less than V were not considered in evaluating seismicity for this map, because it was believed that their capability for causing structural damage was negligible. The uncertainty in the earthquake effect is vested in this conservative specification of the structural action, E, due to the earthquake. As a result, the load factor on E in the NEHRP Recommended Provisions is set equal to 1.0 rather than a greater value.

The design earthquake proposed for USACE hydraulic structures reportedly is based on a probability of 50 percent of being exceeded in 100 years, corresponding to a mean recurrence interval of about 145 years. Under the assumption that the seismic hazard is described by a Type II distribution of largest values, two mean recurrence intervals N_1 and N_2 and corresponding effective peak ground accelerations a_1 and a_2 can be related by

$$N_2/N_1 = (a_2/a_1)^{\alpha} \tag{36}$$

For example, if $\alpha = 2.3$, the NEHRP and USACE design-basis ground motions would be related by

$$a_{NEHRP} = a_{USACE}\,(475/145)^{\frac{1}{\alpha}} = 1.68\ a_{USACE} \tag{37}$$

If $\alpha = 5.5$, the NEHRP and USACE design-basis ground motions would be related by

$$a_{NEHRP} = a_{USACE}\,(475/145)^{\frac{1}{\alpha}} = 1.24\ a_{USACE} \tag{38}$$

In the eastern United States, the selection of a design earthquake ground motion has a more substantial impact on the economics of design because the seismic hazard curve is so flat.

There has been substantial research in earthquake-resistant design since the San Fernando earthquake of 1971 began accelerating developments in this area. Most of this research has shown that the earthquake hazard tends to be underestimated. The recurrence intervals between large events in the eastern United States are large because the hazard curves are flat, and as a result most easterners have little or no experience with earthquakes. Nonetheless, the damage potential is there; the consequences, economic and otherwise, of lock and dam failure should be considered carefully in setting the design criteria.

Other environmental loads

Loads due to snow, wind, temperature, and self-straining actions generally are not significant in the design of miter gates. For appurtenant structures, the load combination for these loads appearing in ASCE 7-88 (ASCE 1990) is recommended.

Summary

Table 6 summarizes the mean occurrence rates, durations and statistics of H_s, H_d and Q, H_d and I_m based on the data summarized. Figure 4 shows the idealizations for the load pulse idealizations used in the subsequent structural reliability analysis. The hydrostatic load, H_s, has been idealized in time by a uniform rather than a trapezoidal pulse for simplicity. Note that the mean occurrence rate of H_s is the number of lockages per year divided by 2 (assuming upstream and downstream lockages alternate), while the mean occurrence rate of H_d and Q is the total number of lockages (each gate must be operated for each lockage).

Table 6
Load Statistics

Load	Rate (mo-1)	Duration	Mean	COV	p.d.f.
D	-	50 yr	1.0 Dn	0.05	Normal
H_s	200	90 min	0.9 H_{sn}	0.25	Type I
H_d,Q(equi)	400	2 min	0.7 H_{dn}	0.53	Type I
Q (obstr.)	0.4	1 min	1.25 Q_n	0.10	Type I
H_{tm}	400	2 min	0.8 H_{tn}	0.76	Type I
H_{to}	400	2 min	0.43 H_{tn}	0.88	Type I
I_m	0.016	15 sec	1.0 I_{mn}	na	Type I

The nominal values, denoted with subscript "n", are assumed to be those defined in EM 1110-2-2703 (Headquarters, Department of the Army 1984). If these were to change at some future time, the mean values in Table 6 also would change. It is assumed that a best estimate of impact force would be used for design.

The statistical data summarized in Table 6 on mean rate and duration of significant load events can be used to screen certain event combinations from subsequent consideration. For example, consider the combination of earthquake with gate operation (loads H_t, H_d, and Q). The mean duration of these events is approximately 2 min (3.803×10^{-6} yr) and they occur at a rate of 4,800/yr. If the mean rate of occurrence and duration of a significant (greater than ML = 4) earthquake is assumed to be 0.10/yr and 30 sec (9.506×10^{-7} yr), Equation 23 can be used to show that the mean rate of a coincidence of earthquake with gate operation is

$$\begin{aligned} \nu_{QE} &= (4{,}800)(0.1)(3.8 \times 10^{-6} + 9.51 \times 10^{-7}) \\ &= 2.29 \times 10^{-3}/yr \end{aligned} \tag{39}$$

Similarly, significant impacts occur at 0.016/mo or 0.19/yr and have a duration of 15 sec (4.75×10^{-7}/yr) or less. The mean rate of coincidence of earthquake and impact is

$$\begin{aligned} \nu_{I_m E} &= (0.19)(0.1)(4.75 + 9.51) \times 10^{-7} \\ &= 2.71 \times 10^{-8}/yr \end{aligned} \tag{40}$$

The mean rates of occurrence of combined loads due to these effects are very small in comparison to those loads associated with normal gate operation, which typically occur on the order of 4,000 to 5,000 times/yr. On the other hand, the mean rate of occurrence and duration of Hs are 2,400/yr and 90 min (1.711×10^{-4}yr). Accordingly, the mean coincidence rate of H_s and E becomes

$$\nu_{H_s E} = 2{,}400(0.10)(1.171 \times 10^{-4} + 9.506 \times 10^{-7}) = 0.0413/yr \tag{41}$$

This mean rate is sufficiently high that H_s and E should be considered in combination.

Structural Resistance of Steel Shapes and Plates

Properties of structural steel required for structural reliability analysis include the yield strength and modulus of elasticity. The existing literature on this subject for common grades of structural steel was reviewed in depth as part of the effort to develop load and resistance factor design for steel structures (Galambos and Ravindra 1978). These data are summarized in

Table 7. Probability distributions of mechanical properties of steel typically are skew-positive, and the lognormal distribution has been a satisfactory model in previous studies.

Table 7
Statistics of Tensile Properties of Steel Plate and Shapes

Element	Property	Mean	COV
Flanges in rolled shapes	F_y	$1.05\ F_y$	0.10
Plates, webs in rolled shapes	F_y	$1.10\ F_y$	0.11
Plates, flanges	F_u	$1.10\ F_u$	0.11
Plates	τ_y	$0.64\ F_y$	0.10
Tension coupon, stub column	E	29 ksi	0.06
Tension, compression coupon	Poisson's ratio	0.3	0.03

Strength properties depend on the rate of load. For mill test conditions, the yield strength is somewhat higher than is obtained under so-called static load conditions. All data presented in Table 7 have been corrected to a static rate of load.

Additional data was located on strength of SA516/Grade 60 carbon steel plate used as a liner in nuclear power plant containments. For 1/4-in.-thick plate, the mean, standard deviation and coefficient of variation in yield strength were 48.5 ksi, 3.3 ksi, and 0.07, respectively (122 samples). For 1/2-in.-thick plate, the mean, standard deviation and coefficient of variation in yield strength were 42.1 ksi, 2.6 ksi, and 0.06, respectively (48 samples). The specified yield, F_y, for this material is 32 ksi. The lognormal distribution provided the best fit to the data of several common distributions tested statistically.

Structural resistance used in LRFD is defined in terms of the structural action or limit state. For example, the resistance for the limit states of tension, flexure, shear, and compression are summarized in the following:

Tension:
- Inelastic deformation: $R = F_y\ A$
- Net section fracture: $R = F_u\ Ae$

Flexure:
- Formation of plastic hinge in compact beam: $R = F_y\ Z_x$

Compression:
- Instability: $R = F_{cr}\ A$
- Inelastic deformation: $R = F_y\ A$

Shear:
- Yielding: $R = F_v\ A_v$

Other limit states are expressed similarly. Statistics to describe strengths of structural shapes and plates for various limit states can be developed from the statistics presented in Table 7. The resistance statistics in Table 8 are typical

Table 8
Resistance Statistics for Steel Members and Components

Member	Mean	C.O.V.
Tension members		
Yield	1.05 F_y A	0.11
Ultimate	1.10 F_u A	0.11
Bending members		
Compact beams	1.07 F_y Z	0.13
Continuous	1.11 F_y Z	0.13
Composite	1.04 M_u	0.14
Lateral-torsional buck	1.06 F_y S	0.14
Beams in shear	1.10 F_v A_w	0.15
Plate Girders		
Yielding	1.08 F_y S	0.14
Lateral-torsional buck	1.20 F_{cr} S	0.17
Shear	1.08 F_v A	0.17
Columns	1.08 F_{cr} A	0.15
Welded connections, shear	0.88 F_v A	0.18
Bolted Connections, shear	0.60 F_u A	0.10

for hot-rolled steel and welded-plate girder structural members. In this table, the strengths F_y, F_u, F_v, and F_{cr} and section properties A, S, and Z are those nominally specified by AISC for design.

5 Development of Probability-Based Design Requirements

Reliability Associated with ASD

Reliability indices β associated with existing design criteria provide benchmarks for the development of new probability-based load combinations. Experience with ordinary building structures has revealed that the β's for existing criteria vary over a considerable range, reflecting the fact that past codes and standards have not dealt with uncertainty in a consistent fashion (Galambos et al. 1982). One simple illustration of this point is the treatment of dead load in combination with other loads in ASD. It might be observed from Equation 1 that the same factor of safety is applied to dead load and to other variable loads in the combination. However, dead loads are relatively predictable, whereas operating and environmental loads can be highly unpredictable (uncertain). Consequently, the likelihood of failure in structures in which overall stress is dead load dominated is much less than in structures where the major portion of the stress arises from operational loads that may vary in time.

To illustrate the calibration process, we consider a compact flexural member in a gate, for which the limit state is the formation of the first plastic hinge. It is assumed in this illustration that the combination of hydrostatic and temporal head governs the design. This component is designed by ASD using the current requirements in EM 1110-1-2101 and EM 1110-2-2703 (Headquarters, Department of the Army 1972 and 1984, respectively)

$$5/6\ F_b\ S_x \geq H_{sn} + H_{tn} \tag{42}$$

where

F_b and S_x = allowable stress and elastic section modulus from the AISC Specification

H_{sn} and H_{tn} = force resultants in the flexural member from nominal (code-specified) hydrostatic and temporal head

(The subscript "*n*" has been appended to the forces to emphasize that these are nominal forces rather than random variables.) The factor 5/6 is required by EM 1110-1-2101; although the basis for this factor is uncertain, it is believed to reflect the severity of the operating environment for lock structures and the possibility of material deterioration over time. A flexural element that satisfies the code requirements is obtained from Equation 42 as

$$S_x \geq (H_{sn} + H_{tn})/(5/6F_b) \tag{43}$$

The limit state for this member is given by (cf Equation 3)

$$G(\ ) = F_y Z_x - H_s - H_t < 0 \tag{44}$$

where

F_y = yield strength

Z_x = plastic section modulus

H_s and H_t = random forces from hydrostatic and temporal head

Observing that Z_x typically is about 1.12 S_x for W-shapes, the limit-state equation for a flexural member designed according to code is defined by substituting S_x from Equation 43 into Equation 44

$$1.344\ (H_{sn} + H_{tn})\ F_y/F_b - H_s - H_t = 0 \tag{45}$$

The reliability index associated with the current code requirement (Equation 43) now can be determined by first-order methods (page 6), using the load and resistance statistics summarized in Tables 6 and 8.

Consider a design situation in which the structural effect of H_{sn} ranges from 1 to 25 times H_{tn}. The latter situation might exist for the submerged lower girders on the miter gate. The results of the FO reliability analysis are presented in Figure 7 as the curve labeled ASD. In the limiting case in which the structural action arises almost entirely from H_{sn}, β = 3.1. The β's are lower for low H_{sn}/H_{tn} because H_{tn} is relatively more important in the load combination and its coefficient of variation is substantially larger than the coefficient of variation in H_s. When $H_{tn} \approx H_{sn}$, relevant for girders in the partially submerged upper portions of a gate, β = 2.7.

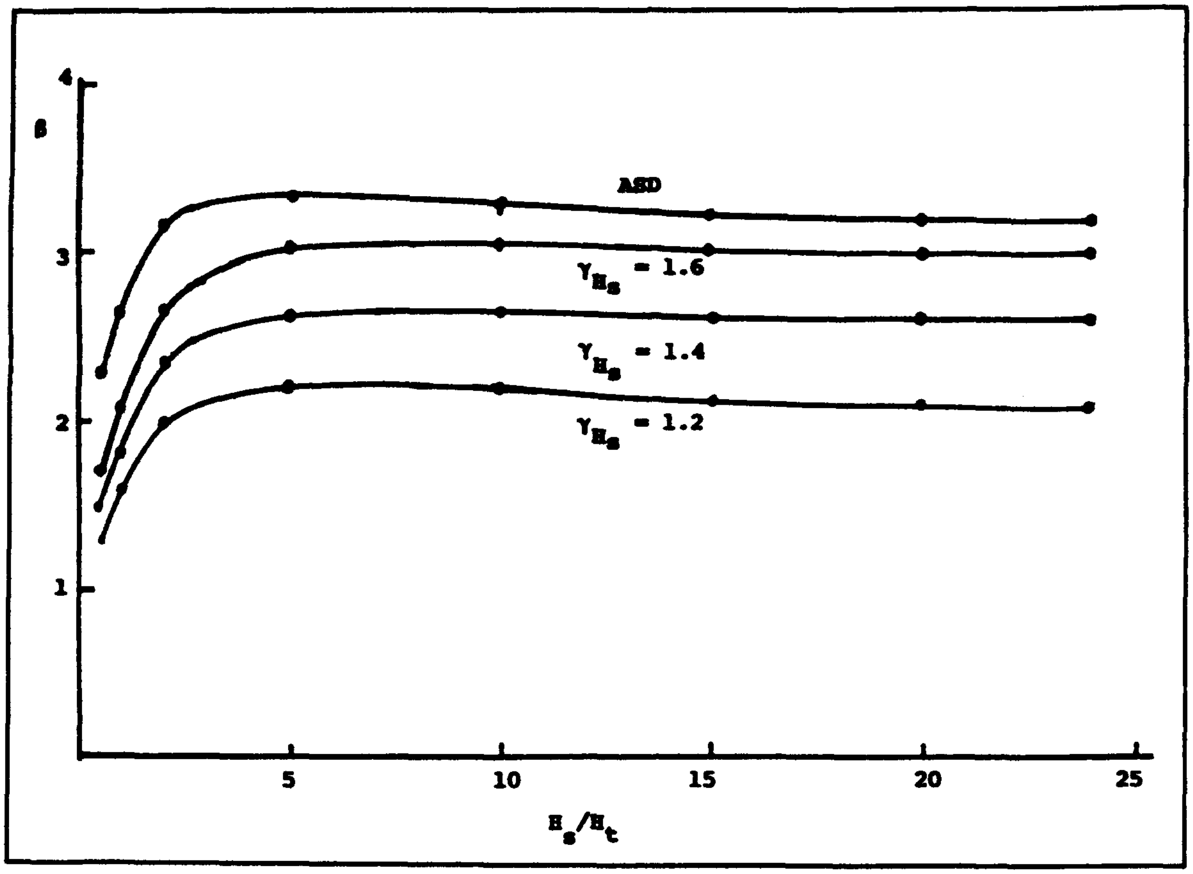

Figure 7. Realiability indices for flexural members designed by ASD and by LRFD

To establish a frame of reference for these reliability indices, an FO reliability analysis of simply supported compact steel beams designed by ASD for the combination of dead plus live load leads to β's that decrease from about 3.0 to about 2.2 (50-year basis) as L_n/D_n increases from about 0.5 to 4 (Galambos et al. 1982). On a comparable 1-year basis, these β's would be approximately 4.1 to 3.4, assuming the loads to be statistically independent events.

Probability-Based Load Criteria

The basis for the load combination analysis is the probability distribution of the maximum, U_{max}, of a sum of time-dependent structural loads. Difficulties in determining the distribution of U_{max} exactly have led to the approximation (Turkstra and Madsen 1980)

$$U_{max} = \max_{i} \left[\max_{o < t < T} X_i(t) + \sum X_j(t) \right] \tag{46}$$

which transforms the load combination analysis into a problem of random variables rather than random processes. Research in probabilistic load combination analysis has shown that failures usually occur when one of the time-varying loads attains its maximum value (Equation 46, term max $X_i(t)$, termed the "principal action") while the other loads equal their point-in-time values (terms $\sum X_j(t)$ in Equation 46, the companion actions). Since it is often not known which of the actions is at its maximum value when the maximum combined effect occurs, each time-varying load must be positioned, in turn, as the principal action to determine the maximum combined effect.

The required strength, U_d, in Equation 2b is determined as an appropriate conservative fractile of U_{max}, i.e., U_d has an acceptably small probability of being exceeded. Consistent with the principal action/companion action format in Equation 46, the required strength is expressed as the set of equations

$$U_d = \gamma_D D + \gamma_Q Q + \sum_i \gamma_i Q_i \tag{47}$$

The factored load, $\gamma_Q Q$ is denoted the principal action, while the terms $\gamma_i Q_i$ are the companion actions. In principle, if there are n time-varying loads, Equation 47 consists of n equations in practice; however, it seldom is necessary to consider all n equations in designing a particular member, since experience rapidly indicates those that control design. Equations 46 and 47 are the basis for the principal action-companion action format used in LRFD and most modern limit state codes (ASCE 7-88; Eurocode No. 1 (ASCE 1990)).

The focus in the following is on the load factors needed to define the required strength, U_d in Equation 47. The first objective in the load combination development process is to minimize the variation in reliability as the loads in a combination vary in proportion to one another. A second objective is to balance the design risk associated with the different load combinations. It is desirable that these two objectives be met so that the load combinations are applicable to the relatively broad scope of structures and components within the purview of the code.

Load factors for designing miter gates are determined using methods similar to those used to develop probability-based limit-state design criteria for building structures (Ellingwood, et al. 1982), utilizing the load and resistance data presented in Section 4 and the results of the calibration studies in Section 5. Noting from Equation 5 that the factored load for a prescribed reliability index is equivalent to the load at the checking point, we have that

$$\gamma_i X_{ni} = m_i (1 + \alpha_i \beta V_i) \tag{48}$$

where

γ_i = load factor

X_{ni} = nominal or characteristic load value specified by the standard

V_i = coefficient of variation in X_i

Solving for γ_i, we obtain

$$\gamma_i = (m_i/X_{ni}) (1 + \alpha_i \beta V_i) \tag{49}$$

Experience has shown that although the product $\alpha_i\beta$ varies in reliability-based design according to the reliability level and the relative importance of load X_i in the combination, a typical value is approximately 2 when X_{ni} is considered as the principal variable load in the combination and when $\beta \simeq$ 3-3.5. Thus, in approximation

$$\gamma_i = (m_i/X_{ni}) (1 + 2 V_i) \tag{50}$$

The ratio (m_i/X_{ni}) reflects the bias in the specified nominal load with respect to its mean value. In current design practice, where the nominal loads tend to be estimated conservatively, this ratio often is less than 1.0. The coefficient of variation V_i is a dimensionless way of representing uncertainty arising from inherent randomness and modeling.

The reliability requirement can be expressed by one equation (Equation 4), while there are many load factors in the design requirements. Thus, the selection of load factors is a trial-and-error process (Ellingwood et al. 1982). Equation 50 provides an initial estimate of the load factor for each load. For example, for the load Hs, we have that

$$\gamma_{H_s} = (0.9) (1 + 2 \times 0.25) = 1.35 \tag{51}$$

Subsequent adjustments to the individual load factors and load combinations are made from FO reliability analyses.

It was determined from the discussion of gate operation characteristics in Section 4 and Equations 23 and 24, that the following coincidences of loads had sufficiently small probability that they could be neglected in the load combinations:

I_m need not be combined with H_d, Q, and H_t;

I_m need not be combined with E;

H_d, Q, and H_t need not be combined with E; and

H_s need not be combined with H_d, Q

The load combinations recommended below correspond to specific load scenarios: (1) gates in the mitered position; (2) gates operating; and (3) environmental effects. Permanent loads appear in all combinations. Time-varying loads appear as principal or companion actions, as appropriate. Postulated loads are assigned a load factor of 1.0, since it is assumed that the conservatism necessary for design is taken into account in the associated hazard scenario and specification of the nominal load. The load specification is made as general as possible; note, however, that the maximum structural action may occur when one (or more) of the loads in a combination is equal to zero. This advisory also is contained in ASCE 7-88 (ASCE 1990) and in the AISC LRFD Specification (AISC 1986). The subscript "n" on load, denoting nominal value, has been omitted in the following for simplicity.

The following loads are considered, individually and in combination:

D = dead load
C = ice load
M = mud load
H_s = hydrostatic head
H_t = temporal hydraulic load (H_{to}: gates open; H_{tm}: gates mitered)
Q = operating equipment load
I_m = impact load
E = earthquake load

The hydrodynamic force, H_d, is not considered since it does not govern in strength design. H_d should be considered in fatigue analysis of the gate; however, fatigue behavior is outside the scope of this study.

Gates in the mitered position

(1) $1.4\,H_s + 1.0\,H_{tm}$

(2) $1.0\,I_m + 1.2\,H_s$

Combination (1) is applicable to the submerged portion of the gate. Combination (2) is applicable to the girders above the waterline. Ice and mud loads are not included in these load combinations because their effects are not significant. They are included in the operating load combinations where their effects may control. Lateral forces due to ice have less of an effect than does the impact force in combination (2). The probability of a coincidence of impact and lateral ice is negligible. Thus, this effect is not considered. It is assumed that there is a potential for impact any time from the beginning of handling and maneuvering in the holding area to the time when the tow completes its exit from the lock.

Gates operating

(3) $1.2\,D \;+\; 1.6\,(C+M) \;+\; 1.0\,H_{to}$

(4) $1.2\,D \;+\; 1.2\,Q \;+\; 1.6\,(C+M)$

During normal gate operation, there are no differential hydrostatic loads. The temporal hydraulic load acts on the submerged portion of the gate as it moves through the water and is statically equilibrated by the force on the operating strut. This may cause twist of the gate leaf, the effects of which are checked by combination (3). However, if there is a submerged obstruction, the gate may bind during operation, causing a force on the operating strut not statically equilibrated by the hydrodynamic effects; combination (4) addresses this situation. The load factor 1.6 on $(C + M)$ is consistent with a mean load of 0.95 times the nominal specified load and a coefficient of variation of 0.35.

Environmental effects

(5) $1.2\,D \;+\; 1.0\,E \;+\; 1.2\,H_s$

The presumption in combination (5) is that the gate is in the mitered position. The probability of a coincidence of earthquake forces with forces from gate operation or impact is negligible (cf Equation 23 and related discussion in Section 4). The earthquake force, E, on a structural component is determined from p(y) defined in Equation 34. The load factor of 1.0 on E is based on the assumption of a NEHRP-magnitude design earthquake. For an earthquake with an MRI of 145 rather than 475 years, this load factor should be increased to 1.4.

The design equation is (cf Eqn 2b)

$$\alpha\,\phi\,R_n > U_d \tag{52}$$

A reliability factor, α, has been introduced to the design equation to account for the severity of the operating environment for hydraulic structures and the difficulties in their inspection and maintenance. Factor α normally is 0.9; if the structure cannot be inspected regularly or if it is in brackish water or seawater, $\alpha = 0.85$. Each adjustment increases the reliability index by about 0.5. No adjustments are made for the nominal resistance, R_n, or resistance factors, ϕ, defined in the current LRFD Specification (AISC 1986).

The results of reliability analyses of flexural components designed by the proposed LRFD load combinations are presented in Figure 7 where they are compared to similar analyses for flexural components designed by ASD. Three curves are presented, corresponding to load factors on H_s (in combination 1) of 1.2, 1.4, and 1.6. The load factor 1.4 leads to designs that are somewhat less conservative than those obtained using ASD and EM1110-2-2101 (Headquarters, Department of the Army 1972). However, the decrease in β is comparable, in an average sense, to the decrease that occurred in the changeover from ASD to LRFD for steel buildings. Moreover, the reliability of the gate leaf as a structural system is substantially higher than indicated in Figure 7 due to its highly redundant nature.

6 Recommendations

The development of load combinations for designing miter gates was handicapped by the lack of statistical data on which to base the reliability analyses and load factors. A Delphi was designed to provide the necessary data on an interim basis. A large amount of data was developed by the Delphi, and it was not possible to analyze these data completely within the scope of this work. Additional analyses of these data should be performed, focusing in particular on the data needed to specify barge impact load I_m. Attempts also should be made to design a data acquisition program to improve the database on operating loads.

The current specification for loads due to temporal head, H_t, also should be revised. The Delphi indicated that the characteristics of temporal head for gates in the mitered and open positions were substantially different. Under the circumstances, consideration should be given to specifying two different values to H_t rather than one value, such as the 15 in. (381 mm) found in EM 1110-2-2703 (Headquarters, Department of the Army 1984). Suggested values might be H_{tm} = 15 in. (382 mm) and H_{to} = 9 in. (229 mm); however, additional measurements should be taken at selected locks to confirm and possibly modify the statistics reported in the Delphi.

The previous REMR study and the Delphi in this study provided information on the incidence of barge impact, tonnage, and velocity at the time of impending impact. There appear to be inconsistencies in the data reported in these two studies. Additional work is needed to develop improved barge impact loads to be used in design. This research should take into account the significant nonlinearities in material and structural behavior that are likely to occur during significant impacts. Reportedly, a test program is in progress aimed at shedding light on miter gate behavior during impact. It makes sense to consider design procedures that are based on the concept of energy dissipation rather than withstanding the force by traditional elastic design methods, since the latter are likely to lead to prohibitive weight requirements. In the load combinations, a load factor of 1.0 was assigned to impact; it is customary to deal with rare events in this way, and the load combinations will not need to be redone when additional data on impact forces becomes available. In any event, impact should be treated as a concentrated force, not as a uniformly distributed load.

The LRFD seismic requirements should be made as consistent as possible with the new NEHRP provisions nearing implementation (NEHRP 1992). Notwithstanding current practice, the possibility of earthquake damage to lock and dam structures and the economic impacts of such damage should be considered.

A set of tentative designs of miter gate components should be performed using the proposed design requirements and a comparison should be made with gates designed by ASD and EM 1110-1-2101 (Headquarters, Department of the Army 1972) before the LRFD criteria supplant ASD.

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13. ABSTRACT *(Maximum 200 words)*

The design of hydraulic steel structures (HSS) in the past has been based on allowable stress design (ASD) principles that address design uncertainty in a subjective manner. This technical report describes the methodology for an improved basis for determining load requirements for the design of HSS using modern structural reliability analysis and probabilistic load combination techniques. This methodology will facilitate the development of design loads and their combinations for use in load and resistance factor design (LRFD). This report demonstrates the methodology for miter gates by considering load combinations involving dead load, hydrostatic load, hydrodynamic load during gate operation, temporal hydraulic load, earthquake load, and barge impact load.

This report describes the reliability bases for LRFD, load and resistance data for lock structures, and development of probability based design requirements. Significant loads and load combinations for miter gates are identified, a statistical database to support probabilistic load combination analysis is developed, and analysis of reliabilities associated with current ASD procedures and selection of target reliability levels is presented. Based on these studies, LRFD load requirements for miter gates including load combinations and load factors are developed.

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Load combination
Miter gate
Probability
Random variable
Reliability
Resistance
Statistics
Strength

WATERWAYS EXPERIMENT STATION REPORTS PUBLISHED UNDER THE COMPUTER-AIDED STRUCTURAL ENGINEERING (CASE) PROJECT

	Title	Date
Technical Report K-78-1	List of Computer Programs for Computer-Aided Structural Engineering	Feb 1978
Instruction Report O-79-2	User's Guide: Computer Program with Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)	Mar 1979
Technical Report K-80-1	Survey of Bridge-Oriented Design Software	Jan 1980
Technical Report K-80-2	Evaluation of Computer Programs for the Design/Analysis of Highway and Railway Bridges	Jan 1980
Instruction Report K-80-1	User's Guide: Computer Program for Design/Review of Curvilinear Conduits/Culverts (CURCON)	Feb 1980
Instruction Report K-80-3	A Three-Dimensional Finite Element Data Edit Program	Mar 1980
Instruction Report K-80-4	A Three-Dimensional Stability Analysis/Design Program (3DSAD)	
	Report 1: General Geometry Module	Jun 1980
	Report 3: General Analysis Module (CGAM)	Jun 1982
	Report 4: Special-Purpose Modules for Dams (CDAMS)	Aug 1983
Instruction Report K-80-6	Basic User's Guide: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Instruction Report K-80-7	User's Reference Manual: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Technical Report K-80-4	Documentation of Finite Element Analyses	
	Report 1: Longview Outlet Works Conduit	Dec 1980
	Report 2: Anchored Wall Monolith, Bay Springs Lock	Dec 1980
Technical Report K-80-5	Basic Pile Group Behavior	Dec 1980
Instruction Report K-81-2	User's Guide: Computer Program for Design and Analysis of Sheet Pile Walls by Classical Methods (CSHTWAL)	
	Report 1: Computational Processes	Feb 1981
	Report 2: Interactive Graphics Options	Mar 1981
Instruction Report K-81-3	Validation Report: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Feb 1981
Instruction Report K-81-4	User's Guide: Computer Program for Design and Analysis of Cast-in-Place Tunnel Linings (NEWTUN)	Mar 1981
Instruction Report K-81-6	User's Guide: Computer Program for Optimum Nonlinear Dynamic Design of Reinforced Concrete Slabs Under Blast Loading (CBARCS)	Mar 1981
Instruction Report K-81-7	User's Guide: Computer Program for Design or Investigation of Orthogonal Culverts (CORTCUL)	Mar 1981
Instruction Report K-81-9	User's Guide: Computer Program for Three-Dimensional Analysis of Building Systems (CTABS80)	Aug 1981
Technical Report K-81-2	Theoretical Basis for CTABS80: A Computer Program for Three-Dimensional Analysis of Building Systems	Sep 1981
Instruction Report K-82-6	User's Guide: Computer Program for Analysis of Beam-Column Structures with Nonlinear Supports (CBEAMC)	Jun 1982

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	Title	Date
Instruction Report K-82-7	User's Guide: Computer Program for Bearing Capacity Analysis of Shallow Foundations (CBEAR)	Jun 1982
Instruction Report K-83-1	User's Guide: Computer Program with Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)	Jan 1983
Instruction Report K-83-2	User's Guide: Computer Program for Generation of Engineering Geometry (SKETCH)	Jun 1983
Instruction Report K-83-5	User's Guide: Computer Program to Calculate Shear, Moment, and Thrust (CSMT) from Stress Results of a Two-Dimensional Finite Element Analysis	Jul 1983
Technical Report K-83-1	Basic Pile Group Behavior	Sep 1983
Technical Report K-83-3	Reference Manual: Computer Graphics Program for Generation of Engineering Geometry (SKETCH)	Sep 1983
Technical Report K-83-4	Case Study of Six Major General-Purpose Finite Element Programs	Oct 1983
Instruction Report K-84-2	User's Guide: Computer Program for Optimum Dynamic Design of Nonlinear Metal Plates Under Blast Loading (CSDOOR)	Jan 1984
Instruction Report K-84-7	User's Guide: Computer Program for Determining Induced Stresses and Consolidation Settlements (CSETT)	Aug 1984
Instruction Report K-84-8	Seepage Analysis of Confined Flow Problems by the Method of Fragments (CFRAG)	Sep 1984
Instruction Report K-84-11	User's Guide for Computer Program CGFAG, Concrete General Flexure Analysis with Graphics	Sep 1984
Technical Report K-84-3	Computer-Aided Drafting and Design for Corps Structural Engineers	Oct 1984
Technical Report ATC-86-5	Decision Logic Table Formulation of ACI 318-77, Building Code Requirements for Reinforced Concrete for Automated Constraint Processing, Volumes I and II	Jun 1986
Technical Report ITL-87-2	A Case Committee Study of Finite Element Analysis of Concrete Flat Slabs	Jan 1987
Instruction Report ITL-87-1	User's Guide: Computer Program for Two-Dimensional Analysis of U-Frame Structures (CUFRAM)	Apr 1987
Instruction Report ITL-87-2	User's Guide: For Concrete Strength Investigation and Design (CASTR) in Accordance with ACI 318-83	May 1987
Technical Report ITL-87-6	Finite-Element Method Package for Solving Steady-State Seepage Problems	May 1987
Instruction Report ITL-87-3	User's Guide: A Three Dimensional Stability Analysis/Design Program (3DSAD) Module	Jun 1987
	Report 1: Revision 1: General Geometry	Jun 1987
	Report 2: General Loads Module	Sep 1989
	Report 6: Free-Body Module	Sep 1989

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	Title	Date
Instruction Report ITL-87-4	User's Guide: 2-D Frame Analysis Link Program (LINK2D)	Jun 1987
Technical Report ITL-87-4	Finite Element Studies of a Horizontally Framed Miter Gate Report 1: Initial and Refined Finite Element Models (Phases A, B, and C), Volumes I and II Report 2: Simplified Frame Model (Phase D) Report 3: Alternate Configuration Miter Gate Finite Element Studies–Open Section Report 4: Alternate Configuration Miter Gate Finite Element Studies–Closed Sections Report 5: Alternate Configuration Miter Gate Finite Element Studies–Additional Closed Sections Report 6: Elastic Buckling of Girders in Horizontally Framed Miter Gates Report 7: Application and Summary	Aug 1987
Instruction Report GL-87-1	User's Guide: UTEXAS2 Slope-Stability Package; Volume I, User's Manual	Aug 1987
Instruction Report ITL-87-5	Sliding Stability of Concrete Structures (CSLIDE)	Oct 1987
Instruction Report ITL-87-6	Criteria Specifications for and Validation of a Computer Program for the Design or Investigation of Horizontally Framed Miter Gates (CMITER)	Dec 1987
Technical Report ITL-87-8	Procedure for Static Analysis of Gravity Dams Using the Finite Element Method – Phase 1a	Jan 1988
Instruction Report ITL-88-1	User's Guide: Computer Program for Analysis of Planar Grid Structures (CGRID)	Feb 1988
Technical Report ITL-88-1	Development of Design Formulas for Ribbed Mat Foundations on Expansive Soils	Apr 1988
Technical Report ITL-88-2	User's Guide: Pile Group Graphics Display (CPGG) Post-processor to CPGA Program	Apr 1988
Instruction Report ITL-88-2	User's Guide for Design and Investigation of Horizontally Framed Miter Gates (CMITER)	Jun 1988
Instruction Report ITL-88-4	User's Guide for Revised Computer Program to Calculate Shear, Moment, and Thrust (CSMT)	Sep 1988
Instruction Report GL-87-1	User's Guide: UTEXAS2 Slope-Stability Package; Volume II, Theory	Feb 1989
Technical Report ITL-89-3	User's Guide: Pile Group Analysis (CPGA) Computer Group	Jul 1989
Technical Report ITL-89-4	CBASIN–Structural Design of Saint Anthony Falls Stilling Basins According to Corps of Engineers Criteria for Hydraulic Structures; Computer Program X0098	Aug 1989

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	Title	Date
Technical Report ITL-89-5	CCHAN–Structural Design of Rectangular Channels According to Corps of Engineers Criteria for Hydraulic Structures; Computer Program X0097	Aug 1989
Technical Report ITL-89-6	The Response-Spectrum Dynamic Analysis of Gravity Dams Using the Finite Element Method; Phase II	Aug 1989
Contract Report ITL-89-1	State of the Art on Expert Systems Applications in Design, Construction, and Maintenance of Structures	Sep 1989
Instruction Report ITL-90-1	User's Guide: Computer Program for Design and Analysis of Sheet Pile Walls by Classical Methods (CWALSHT)	Feb 1990
Technical Report ITL-90-3	Investigation and Design of U-Frame Structures Using Program CUFRBC Volume A: Program Criteria and Documentation Volume B: User's Guide for Basins Volume C: User's Guide for Channels	May 1990
Instruction Report ITL-90-6	User's Guide: Computer Program for Two-Dimensional Analysis of U-Frame or W-Frame Structures (CWFRAM)	Sep 1990
Instruction Report ITL-90-2	User's Guide: Pile Group–Concrete Pile Analysis Program (CPGC) Preprocessor to CPGA Program	Jun 1990
Technical Report ITL-91-3	Application of Finite Element, Grid Generation, and Scientific Visualization Techniques to 2-D and 3-D Seepage and Groundwater Modeling	Sep 1990
Instruction Report ITL-91-1	User's Guide: Computer Program for Design and Analysis of Sheet-Pile Walls by Classical Methods (CWALSHT) Including Rowe's Moment Reduction	Oct 1991
Instruction Report ITL-87-2 (Revised)	User's Guide for Concrete Strength Investigation and Design (CASTR) in Accordance with ACI 318-89	Mar 1992
Technical Report ITL-92-2	Fiinite Element Modeling of Welded Thick Plates for Bonneville Navigation Lock	May 1992
Technical Report ITL-92-4	Introduction to the Computation of Response Spectrum for Earthquake Loading	Jun 1992
Instruction Report ITL-92-3	Concept Design Example, Computer Aided Structural Modeling (CASM) Report 1: Scheme A Report 2: Scheme B Report 3: Scheme C	 Jun 1992 Jun 1992 Jun 1992
Instruction Report ITL-92-4	User's Guide: Computer-Aided Structural Modeling (CASM) - Version 3.00	Apr 1992
Instruction Report ITL-92-5	Tutorial Guide: Computer-Aided Structural Modeling (CASM) - Version 3.00	Apr 1992

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	Title	Date
Contract Report ITL-92-1	Optimization of Steel Pile Foundations Using Optimality Criteria	Jun 1992
Technical Report ITL-92-7	Refined Stress Analysis of Melvin Price Locks and Dam	Sep 1992
Contract Report ITL-92-2	Knowledge-Based Expert System for Selection and Design of Retaining Structures	Sep 1992
Contract Report ITL-92-3	Evaluation of Thermal and Incremental Construction Effects for Monoliths AL-3 and AL-5 of the Melvin Price Locks and Dam	Sep 1992
Instruction Report GL-87-1	User's Guide: UTEXAS3 Slope-Stability Package; Volume IV, User's Manual	Nov 1992
Technical Report ITL-92-11	The Seismic Design of Waterfront Retaining Structures	Nov 1992
Technical Report ITL-92-12	Computer-Aided, Field-Verified Structural Evaluation	
	Report 1: Development of Computer Modeling Techniques for Miter Lock Gates	Nov 1992
	Report 2: Field Test and Analysis Correlation at John Hollis Bankhead Lock and Dam	Dec 1992
Instruction Report GL-87-1	User's Guide: UTEXAS3 Slope-Stability Package; Volume III, Example Problems	Dec 1992
Technical Report ITL-93-1	Theoretical Manual for Analysis of Arch Dams	Jul 1993
Technical Report ITL-93-2	Steel Structures for Civil Works, General Considerations for Design and Rehabilitation	Aug 1993
Technical Report ITL-93-3	Soil-Structure Interaction Study of Red River Lock and Dam No. 1 Subjected to Sediment Loading	Sep 1993
Instruction Report ITL-93-4	Load and Resistance Factor Design for Steel Miter Gates	Oct 1993