US Army Corps
of Engineers
Waterways Experiment
Station

Contract Report ITL-95-2
July 1995

Event Combination Analysis for Design and Rehabilitation of U.S. Army Corps of Engineers Navigation Structures

by Bruce R. Ellingwood, Johns Hopkins University

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

Contract Report ITL-95-2
July 1995

Event Combination Analysis for Design and Rehabilitation of U.S. Army Corps of Engineers Navigation Structures

by Bruce R. Ellingwood

Department of Civil Engineering
Johns Hopkins University
3400 N. Charles Street
Baltimore, MD 21218

Accesion For		
NTIS CRA&I	X	
DTIC TAB	☐	
Unannounced	☐	
Justification		
By		
Distribution /		
Availability Codes		
Dist	Avail and / or Special	
A-1		

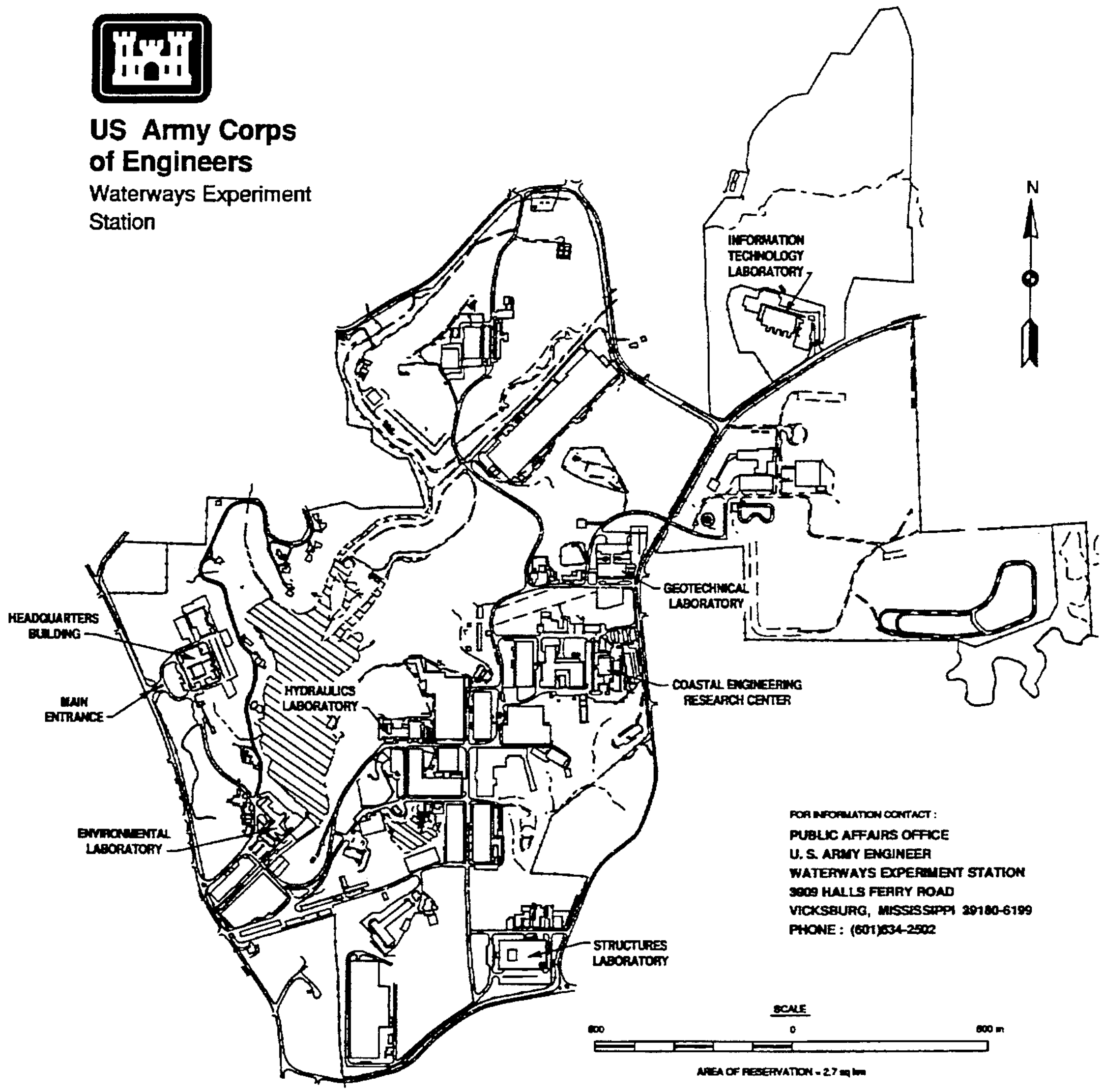
Final report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Monitored by U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

US Army Corps of Engineers
Waterways Experiment Station



Waterways Experiment Station Cataloging-in-Publication Data

Ellingwood, Bruce.
Event combination analysis for design and rehabilitation of U.S. Army Corps of Engineers navigation structures / by Bruce R. Ellingwood ; prepared for U.S. Army Corps of Engineers ; monitored by U.S. Army Engineer Waterways Experiment Station.
48 p. : ill. ; 28 cm. — (Contract report ; ITL-95-2)
Includes bibliographical references.
1. Materials — Dynamic testing. 2. Reliability (Engineering) — Statistical methods. 3. Hydraulic structures. I. United States. Army. Corps of Engineers. II. U.S. Army Engineer Waterways Experiment Station. III. Information Technology Laboratory (U.S. Army Engineer Waterways Experiment Station) IV. Title. V. Series: Contract report (U.S. Army Engineer Waterways Experiment Station) ; ITL-95-2.
TA7 W34c no.ITL-95-2

Table of Contents

	Preface	v
1.	Introduction	1
	1.1 Review of previous work	1
	1.2 Research objectives and scope	5
2.	Probabilistic models of load processes	6
	2.1 Basic load process models	6
	2.2 Permanent loads	11
	2.2.1 Dead loads	11
	2.2.2 Temperature and other self-straining effects	11
	2.2.3 Prestressing	11
	2.3 Environmental loads	11
	2.3.1 Snow loads	11
	2.3.2 Wind loads	12
	2.3.3 Earthquake effects	13
	2.3.4 Flood loads	17
	2.4 Operating loads	18
	2.4.1. Hydrostatic loads	18
	2.4.2 Equipment and hydrodynamic loads	19
	2.4.3 Impact	19
	2.5 Summary	20
3.	Probabilistic basis for event combination analysis	22
	3.1 General considerations	22
	3.2 Fundamental event combination analysis	23
	3.3 Illustration of event combination analysis	31
4.	Recommendations for design	34
5.	Conclusions	37
6.	References	38

List of Tables

Table 2.1 Statistics of structural load events 21

List of Figures

Figure 1.1 Combination of structural loads 2
Figure 2.1 Poisson pulse process models of events and loads 7
Figure 2.2 Upcrossing analysis of time-dependent events 9
Figure 2.3 Typical seismic hazard curves 16
Figure 3.1 Event tree for analysis of event combinations 28
Figure 3.2 Stochastically dependent event occurrences 30

Preface

This report describes a summary of the use of probabilistic event combination techniques in the design and evaluation of navigation structures and presents recommendations for developing methods applicable to U.S. Army Corps of Engineers (USACE) civil works structures. The work was funded under the Civil Works Research and Development Program, Risk Analysis for Water Resource Investments at the U.S. Army Engineer Waterways Experiment Station (WES). Dr. Bruce R. Ellingwood at Johns Hopkins University performed the work under USACE contract number DACW39-92-M-7749 monitored by Dr. Mary Ann Leggett of the Scientific and Engineering Applications Center, Computer-Aided Engineering Division (CAED), Information Technology Laboratory (ITL), WES. The work was coordinated by Messrs. Jerry Foster and Don Dressler of the Engineering Division, Directorate of Civil Works, Headquarters, USACE. The work was performed under the general supervision of Mr. H. Wayne Jones, Chief, CAED, ITL, and Dr. N. Radhakrishnan, Director, ITL.

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

1. INTRODUCTION

1.1 Review of Previous Work

Navigation structures must be designed to withstand the effects of loads and their combinations arising from operating conditions, environmental events and effects, and accidents such as barge impact. The loading requirements in design documents used by the U.S. Army Corps of Engineers (e.g., EM 2703, EM 2101) and nationally recognized specifications and standards (e.g., ASCE Standard 7-88, 1990; AISC, 1986) specify appropriate nominal load values and combinations of loads to be used in design. These loading requirements have evolved gradually and some have been in use for decades. The fact that navigation structures generally have performed well indicates that the current load combination requirements are conservative, although to an unknown degree.

Events giving rise to structural loads are uncertain in their magnitude and structural effect, and the loads vary randomly in space and in time. These uncertainties arise from inherent randomness in naturally occurring or man-made phenomena and from assumptions made in modeling and predicting behavior of complex engineered facilities. Let

$$U(t) = X_1(t) + X_2(t) + \ldots + X_n(t) \tag{1.1}$$

represent a combination of structural load effects (or structural actions due to randomly occurring operational and environmental events), as illustrated in Figure 1.1. The maximum of the combined load effects in time interval $o<t\leq T$ (in which T might be one year, the service life of the facility, or its inspection/maintenance interval, depending on the purpose of the event combination analysis) is denoted,

$$U_{max} = \max_{o<t\leq T} U(t) \tag{1.2}$$

Prudent design would require that the facility be designed for some appropriate and conservative fractile, U_d, of the cumulative probability distribution function (cdf) of U_{max}. In other words, we would like to assert that the probability of exceeding the design envelope furnished by U_d during the service life of the facility is acceptably small.

The extreme values of individual load effects rarely occur simultaneously unless the peak loads are associated with the same event (see Figure 1.1).

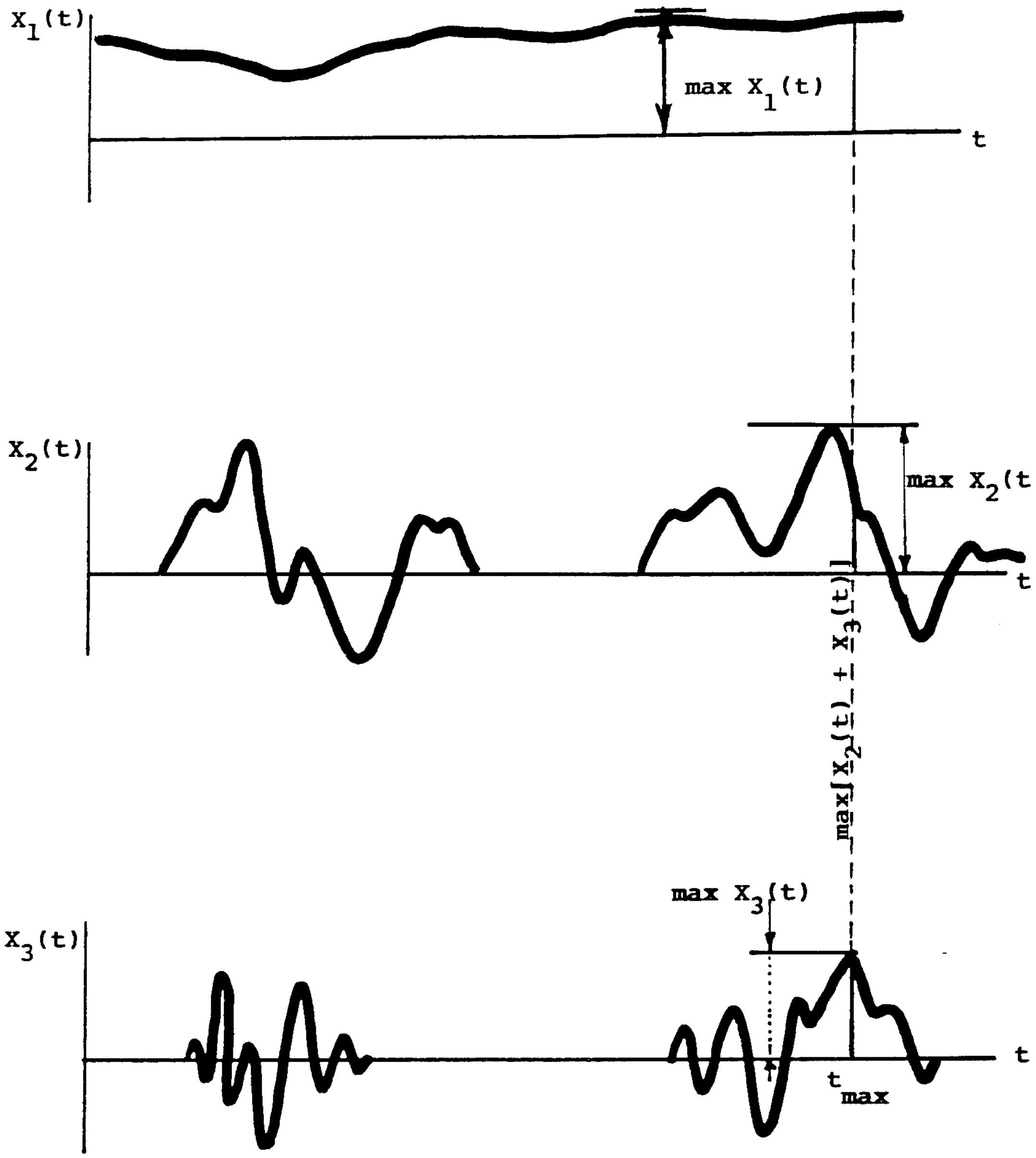


Figure 1.1 - Combination of Structural Loads

Engineers and decision-makers generally have recognized intuitively that combining the peak load effects for design purposes is unduly conservative. This recognition predates any rational consideration of event combination analysis by probabilistic methods. In general,

$$U_{max} < \max X_1(t) + \max X_2(t) + \ldots \quad (1.3)$$

Traditional practice has been to apply "load combination factors" less than unity in estimating the design value of the combined load, U_d. For example, Section 2.3 of ASCE 7-88 (ASCE 1990) for allowable stress design requires that for combinations of live and wind load,

$$U_d = 0.75(D + L + W) \quad (1.4)$$

in which D, L and W = dead, live and wind load from ASCE 7-88. The ASD Specification for steel structures (AISC, 1989) addresses this problem in a slightly different way, increasing the allowable stress by a factor of 1/0.75 or 1.33 for combinations involving wind or earthquake load but leaving the load combination as D + L + W.

There is an unstated small but finite (and presumably acceptable) probability in all the above cases that the load or combined load effect used for design will be exceeded in a given year. The fact that each load or load combination has a small probability of being exceeded and that peak values of loads in combination do not combine concurrently has been recognized by many standard writers, but often is not stated explicitly in the building code or commentaries. Instead, the uncertainties that arise from inherent variability and imperfect modeling are taken into account through the use of judgmental factors of safety. Usually there is no need for the engineer/designer to consider such probabilities explicitly as part of the design process. Indeed, the regulatory climate in the United States would make it difficult to implement a fully probabilistic design procedure in which designers choose the probability level depending on the performance requirements for the facility.

During the past two decades, however, substantial advances have been made in probability-based event and load modeling and in acquiring the necessary supporting data to implement probability-based criteria for many phases of engineered facility design. It now is possible in many instances to determine appropriate design loads and load combinations that are based on a specified probability of being exceeded (Ellingwood, et al, 1982; Galambos, et al, 1982). For example, the nominal loads in ASCE 7 -88 are specified as N-year mean recurrence interval (MRI) values, or the value corresponding to a probability of 1/N of being exceeded in a given year. Parameter N customarily is taken as 50 years for wind or snow loads acting on ordinary buildings; a 50-year MRI windspeed, for example, has a 2% probability of being exceeded annually. The earthquake zone map is based on a seismic hazard analysis which determines contours of ground acceleration with a 10% probability of being exceeded in 50 years; the corresponding MRI is 475 years and the annual probability is 0.21% (FEMA, 1992).

Structural reliability theory and probabilistic load combination analysis methods also lead to better insight and models of how events and structural loads may actually combine. These methods have been applied successfully to building structures, reflected by the proposed design load combinations in ASCE 7-88 for load and resistance factor design for combinations involving live, snow, wind and earthquake effects:

$$U_d = 1.2\,D + 1.6\,L \tag{1.5}$$

$$U_d = 1.2\,D + 1.6\,S + 0.5L + 0.8W \tag{1.6}$$

$$U_d = 1.2\,D + 1.3\,W + 0.5\,L \tag{1.7}$$

$$U_d = 1.2\,D + 1.0\,E + 0.5\,L + 0.2\,S \tag{1.8}$$

$$U_d = 0.9\,D + (1.3W \text{ or } 1.0\,E) \tag{1.9}$$

(The earthquake load combinations in Eqns 1.8 and 1.9 have changed recently as part of the move to implement the NEHRP Recommended Provisions for Seismic Regulations (FEMA 1992). Equation 1.9 is used to check overall stability of the structure or situations where the dead and wind or earthquake effects counteract one another. The load factors 0.5 on L or 0.2 on S in combination with other effects reflects the small probability that extreme values of the two loads occur simultaneously.

On the material side, the AISC LRFD Specification also is based on probabilistic reasoning. For example, the design strength for compact steel members subjected to axial tension (yield), axial compression and bending are given by (AISC 1986),

$$0.90F_yA_g > U_d \tag{1.10}$$

$$0.85F_{cr}A_g > U_d \quad (1.11)$$

$$0.90F_yZ_x > U_d \quad (1.12)$$

in which F_y = yield strength, F_{cr} = critical strength, A_g = gross section area, and Z_x = plastic section modulus, and U_d represents the structural action that is dimensionally consistent with the design strength. In sum, the reliability basis for engineering decision analysis allows consistency in reliability and performance to be built into the design requirements that is not possible when design loads and safety factors are selected judgmentally.

The development of probability-based event combination analysis guidelines for steel or reinforced concrete navigation structures operated and maintained by the U.S. Army Corps of Engineers has only recently been attempted, with an application to steel miter gates for lock and dam structures (Ellingwood 1993). Earthquakes, floods and similar environmental events with the potential to damage a structure severely are rare events in comparison with events giving rise to design-basis operating loads. Since the occurrences of such events generally are statistically independent, the probability of a coincidence of two (or more) of them is very low indeed. In such cases, any requirement that such events be combined in design may contribute little to long-term reliability and performance of navigation and as a result may impose an severe economic burden on the Corps and the public with little benefit to public safety. Progress in probability-based event combination analysis has been made in other contexts, e.g., nuclear power plant systems (Shinozuka, et al, 1981: Schwartz, et al, 1981), offshore platforms, and other critical facilities. The application of this methodology to navigation structures remains to be explored.

1.2 Research Objectives and Scope

This report develops an improved methodology for determining design-basis events, loads and load combinations for use in design of navigation structures operated by the U.S. Army Corps of Engineers. This methodology provides a rationale to support the implementation of design guidelines for event combinations in which the events are taken at less than their full design intensity, e.g., combinations of operating and flood loads. This concept also has been proposed for implementation in the model Eurocodes (1991).

2. PROBABILISTIC MODELS OF LOAD PROCESSES

Probabilistic event or load combination analysis is data-intensive and requires information to describe the rate of occurrence, duration, and intensity of all loads affecting facility performance. This section identifies basic mathematical models of events and structural loads and describes the collection and analysis of available data to identify those temporal characteristics and intensities that are pertinent to the study of their combinations for designing navigation structures.

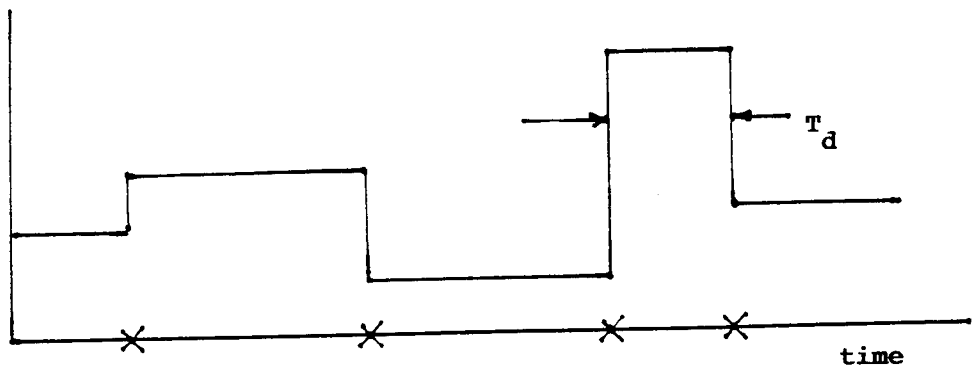
2.1 Basic Load Process Models

Probabilistic descriptions of operating and environmental events and structural actions (forces, deformations) arising from such events must convey, as a minimum, a description of the rate of occurrence and duration of the event and the intensity of the structural action. The occurrence of many events giving rise to time-dependent structural loads can be modeled as a Poisson process (Larrabee and Cornell, 1981; Pearce and Wen, 1984). With this model, the occurrence of loads or change in load magnitudes is assumed to be described by the Poisson probability law (Papoulis, 1965);

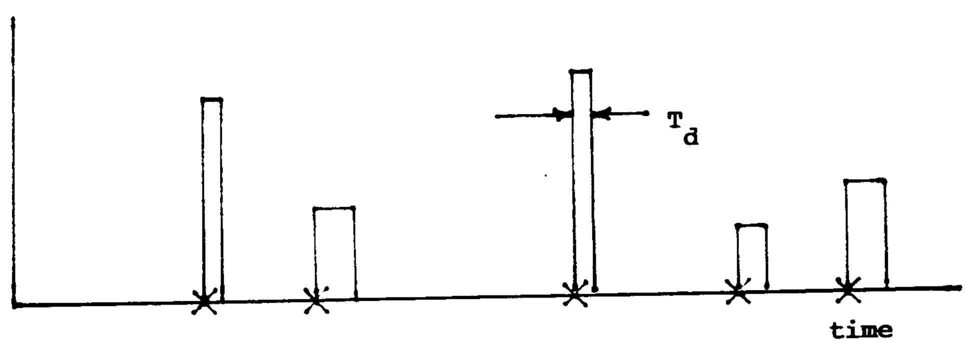
$$P[N(t) = n] = (\lambda t)^n \exp(-\lambda t)/n! \; ; \; n = 0,1,2,... \qquad (2.1)$$

in which N(t) = number of events to occur during interval (0,t). Three sample functions of a Poisson process are illustrated in Figure 2.1. In Figure 2.1(a), the event is more or less continuous and the load is always present (Type I). In Figure 2.1(b), the event and the load are intermittent (Type II). In Figure 2.1(c), the duration of the event is neglected, leading to a sequence of load impulses.Each load event is characterized by a mean rate of occurrence, λ, a mean duration, τ, and a cumulative distribution function (cdf), F(x), modeling pulse intensity. Assuming that the successive pulse intensities are identically distributed and statistically independent random variables, a knowledge of the mean rate of occurrence of the events, their duration, and the cdf of the pulse intensity is sufficient to characterize the stochastic load process.

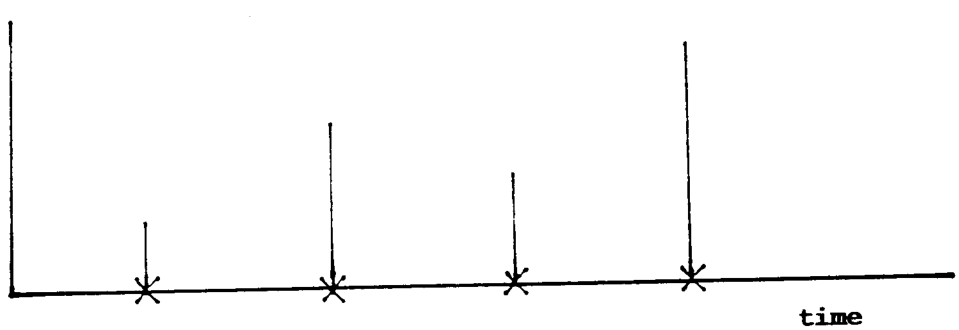
If the pulse process is always nonzero (Type I), the parameters λ and τ are related by $\lambda = 1/\tau$ and the duration, T_d, of each event is an exponential random variable with expected value, $E[T_d] = \tau$. If the pulse process is intermittent (Type II), the probability, p, that the process is nonzero at any time can be obtained from,



(a) Sustained event or load (Type I)



(b) Intermittent event or load (Type II)



(c) Impulse event or load (Type III)

Figure 2.1 Poisson pulse process models of events and loads (X: Poisson occurrences)

$$p = \lim_{T \to \infty} \frac{1}{T} \sum_{k}^{N(T)} T_{dk} = \lim_{T \to \infty} \frac{1}{T} (\lambda T)\, E[T_d] = \lambda \tau \qquad (2.2)$$

The Poisson load process model has been shown in numerous studies to provide an adequate representation of many common structural loads when the structural action during the load event has essentially a constant or static effect. When the structural action is dynamic, the occurrence of the load event still can be described by a Poisson process; however, the structural load may fluctuate during the event occurrence, and other techniques are required to analyze its structural effect. Fluctuating loads due to wind or earthquake sometimes can be approximated conservatively as Poisson pulse processes if one imagines that the intensity of the pulse corresponds to the maximum structural action that occurs during the event. When time-dependence must be considered, (the extremes of the fluctuations can be described by a mean upcrossing analysis (Veneziano, et al, 1977), as described subsequently.

Time-varying responses X(t) arising from randomly occurring events can be characterized in a first-order sense by their pdf $F_X(x)$ or cdf $f_X(x)$ and a mean upcrossing function, $v_x(x)$, defined as the rate at which X(t) crosses level x from below. This is illustrated in Figure 2.2. In the special cases when the changes in the load intensity are described by a Poisson process with rate parameter λ, and the load intensities are described by a sequence of identically distributed and statistically independent random variables with cdf $F_s(x)$ and pdf $f_S(x)$, $v_x(x)$ can be obtained in closed form (Winterstein and Cornell, 1984). For example, when X(t) is always nonzero (Figure 2.1a):

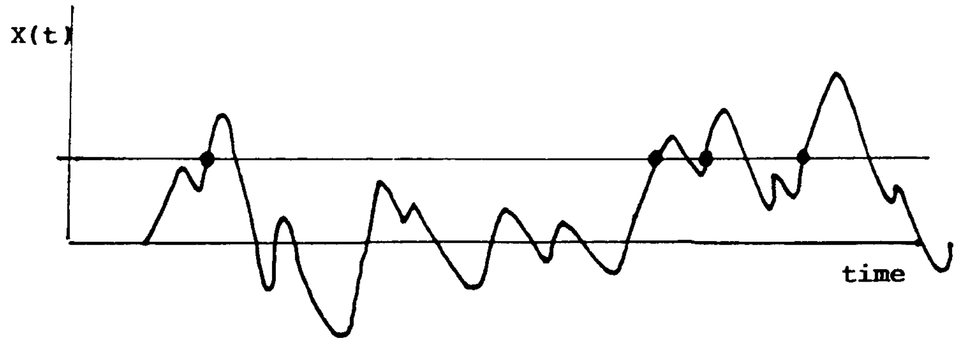
$$f_X(x) = f_S(x) \qquad (2.3a)$$

$$v_X(x) = \lambda\, F_S(x) G_S(x) \qquad (2.3b)$$

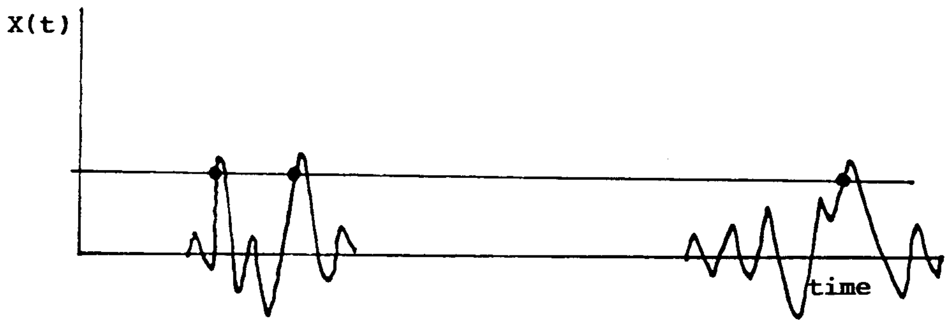
in which $G_S(x) = 1 - F_S(x)$ = complementary cdf. For the intermittent pulse process (Figure 2.1b):

$$f_X(x) = \lambda \tau\, f_S(x) + (1 - \lambda\tau)\, \delta(x) \qquad (2.4a)$$

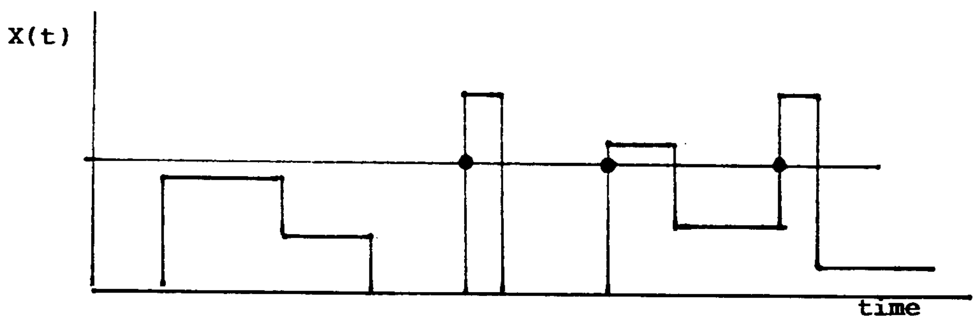
$$v_X(x) = \lambda\, G_S(x)\, [1 - \lambda\tau\, G_S(x)] \qquad (2.4b)$$



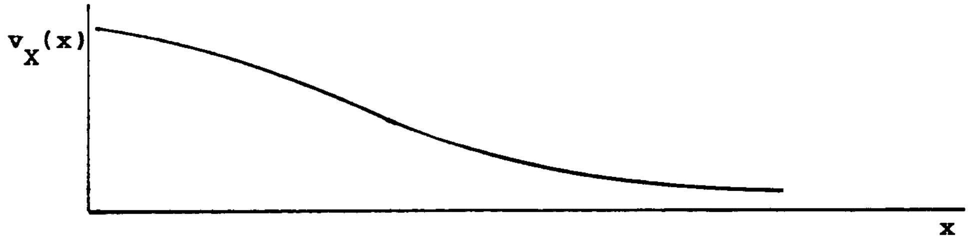
(a) Continuous process



(b) Intermittent process



(c) Intermittent Poisson load process
(Type II in Figure 2.1)



(d) Typical upcrossing function

Figure 2.2 Upcrossing analysis of time-dependent events in which $\delta(x)$ = Dirac delta function. Finally for the impulse process (Figure 2.1c):

$$v_X(x) = \lambda\, G_S(x) \tag{2.5}$$

More generally, the upcrossing rate function can be evaluated from the relation

$$v_u(x) = \lim_{\Delta t \to 0}\left(\frac{P[x(t) \le x \text{ and } x(t+\Delta t) > x]}{\Delta t}\right) \tag{2.6}$$

If the load process is sufficiently well-behaved that $v_X(x)$ can be estimated from one sample of data (mathematically, X(t) is ergodic), and if a reasonably long sample of data is available, then $v_X(x)$ can be estimated directly from this sample of data using Eqn 2.6 directly. If X(t) and its derivative $\dot{X}(t)$ are continuous, the mean upcrossing rate can be expressed from the joint density function of X(t) and $\dot{X}(t)$, defined as $f_{X,\dot{X}}(x,\dot{x})$, as (Rice, 1945),

$$v_X(x) = \int_0^{\infty} \dot{x}\, f_{X\dot{X}}(x,\dot{x})\, d\dot{x} \tag{2.7}$$

provided that the joint density function is available. Unless X(t) is Gaussian, however, this joint density function is unknown or difficult to ascertain.

In addition to its use in describing local fluctuations in load intensity, the mean upcrossing rate function plays a central role in determining the probability that X(t) exceeds a barrier level x during the interval of time (0, T), as will be shown subsequently.

The load statistics in the following sections, particularly those related to environmental load effects, are presented at a level of detail necessary to perform the event combination analysis in subsequent sections. A summary is presented in Table 2.1. Environmental load statistics are strongly site-dependent. Additional published data are available for those wishing to perform a site-specific analysis, as noted in the references.

2.2 Permanent Loads

2.2.1 Dead Loads

Dead loads are random in magnitude but are fixed in position and usually invariant in time if remodeling effects are neglected. They arise from the weight of the structure and permanent equipment and attachments. The dead load can be assumed to be described by a normal cumulative probability distribution function (c.d.f.), with a mean value of 1.0 to 1.05 times the nominal, D, and a coefficient of variation (c.o.v.) equal to 0.10. A major source of this variability arises from the treatment of point loads as area-averaged loads for design purposes.

2.2.2 Temperature and other Self-Straining Effects

In ordinary building structures, temperature influences, shrinkage and creep are generally considered only when they may result in forces or deformations that may be significant to the ultimate limit state considered. In the upper Mississippi region, the maximum average daily temperature variation (during the summer months) is about 25° F. It is believed that forces from this magnitude of variation are insufficient to be a significant design consideration. Shrinkage and creep can be controlled by proper detailing of reinforcement.

2.2.3 Prestressing

Prestressing is a type of imposed deformation. Current codes require that prestressing tendons be stressed to 0.7 f_{pu} immediately following prestress transfer. There are only limited statistical data on prestressing forces. The cdf of prestress effect may be assumed to be normal, with a mean that generally is close to the nominal value, and a c.o.v. of about 0.02 at transfer and about 0.05 after all losses have occurred (Ellingwood, 1983). In view of the magnitude of these statistics in comparison with others, it is reasonable to assume that prestressing effects are deterministic in concrete structures at navigation facilities.

2.3 Environmental Loads

2.3.1 Snow Loads

Snow loads are significant for the design of roofs in the northern tier of states. Stochastic models of snow load on roofs are developed using meteorological data, supplemented by limited snow surveys (Ellingwood and Redfield, 1983; O'Rourke, et al, 1982). Snow loads are dependent on site and roof geometry exposure (ASCE 7 1990) and are seasonal in nature, in contrast with other common environmental loads.

The benchmark statistics needed to construct the snow load pulse process are derived from the annual extreme snow load because this is where the focus has been in the data collection and analysis that supports standard development (ASCE 7 1990). The annual extreme roof snow load can be described by a lognormal distribution, with a mean value that typically is about 0.2S and a c.o.v that typically is about 0.90. The mean rates of occurrence of significant snows during the winter season are site-dependent. In temperate climates, there may be periods of melting, and the load process is intermittent (Type II in Figure 2.1(b)). The mean rate of occurrence, vs, during the snow season would be on the order of 2 - 4 pulses/yr with a mean duration of perhaps 1 week. In northern climates or at high elevations, the snow accumulates during the winter season, and the snow load can be modeled (conservatively) with 1 pulse/yr and a duration which may be as much as 3 months. The cdf of the annual extreme snow load, $F_S(x)$, is related to the cdf of the pulse intensity, $F_i(x)$, by,

$$F_S(x) = \exp [-\lambda_s[1 - F(x)]] \tag{2.8}$$

Thus, the cdf of the pulse intensity can be recovered from a knowledge of the annual extreme and temporal characteristics by,

$$F(x) = 1 + (1/\lambda_s) \ln F_S(x) \tag{2.9}$$

At the upper fractiles of the cdf that are of interest in structural reliability analysis, F_S and F can be modeled by Type I distributions of largest values (Benjamin and Cornell, 1970):

$$F(x) = \exp [- \exp [- \alpha(x - u)]] \tag{2.10}$$

in which α, u = parameters of the distribution.

2.3.2 Wind Load

Wind loads depend on the wind speed and building exposure and geometry and are intermittent in nature (Type II in Figure 2.1b). The wind speeds used for design are determined from basic climatological data, while the building aerodynamic effects are taken into account by wind tunnel modeling, supplemented by limited in-situ pressure measurements.

As with the snow load, the benchmark statistics used to model the wind load as a pulse process are derived from the annual extreme wind load because this is where the focus has been in the data collection and analysis that support standard development (ASCE 7 1990). The statistical data are site-dependent. The annual extreme wind load can be modeled with a Type I distribution of largest values, with a mean value typically about 0.30W, in which W = nominal wind load (based on a 50-yr mean recurrence interval wind speed) and a c.o.v typically about 0.60 (Galambos, et al, 1982). The mean rate of occurrence of structurally significant winds is about 2 - 4/yr in extratropical regions, with an average storm duration of approximately 4 hours. The cdf of the pulse intensity (maximum fluctuating wind load during the occurrence of one storm) is defined by Eqn. 2.9, substituting wind load parameters for those of snow load.

2.3.3 Earthquake Effects

There has been substantial research in earthquake-resistant design since the San Fernando earthquake of 1971. Most of this research has shown that the earthquake hazard tends to be underestimated, particularly in the Eastern United States. The recurrence intervals between large events in the Eastern US are large and as a result there is little or no experience with earthquakes and their social impact in that part of the country. Nontheless, their damage potential should be considered in designing navigation structures because of their consequences for the economy and for public safety.

Earthquake loads should be modeled as intermittent (Type II in Figure 2.1b) in nature. Earthquake effects on building structures usually are determined using an equivalent static-for-dynamic analysis in which the base shear is computed from a ground motion parameter (usually acceleration, but sometimes velocity) and general dynamic structural characteristics. The lateral forces that are statically equivalent to this base shear are distributed over the structure in a manner consistent with its mode shape. In the new NEHRP provisions (FEMA 1992), the (modal) base shear is,

$$V = C_s W \tag{2.11}$$

in which C_s = seismic design coefficient and W = represents the effective mass-related weight of the structure. Coefficient C_s is defined as,

$$C_s = 1.2 A_v S/RT^{2/3} \tag{2.12}$$

in which 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, and T = fundamental (modal) period of the structure. The coefficient C_s is tantamount to an inelastic yield spectrum for an oscillator with 5% damping.

For hydraulic structures, the inertial hydrodynamic forces are determined from Westergaard's equation (Westergaard, 1933),

$$p(y) = 0.875 \gamma_w A_v H_y \tag{2.13}$$

in which γ_w = density of water, H_γ = water depth, and y = depth below water surface.

The value of ground acceleration or velocity (or spectral acceleration) used in design is determined from a seismic hazard analysis. Seismic hazard analysis carries with it the largest component of uncertainty in the process leading to earthquake forces on engineered facilities (Reiter, 1990). 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 within these zones of potential future earthquake occurrence is determined and mean rates of occurrence of earthquakes of various magnitudes (or intensities) are identified. Attenuation functions relating effective peak ground acceleration to magnitude and epicentral distance are used to propagate the earthquake ground motion from the source to the building site. Finally, a probability distribution of effective peak ground accelerations 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 cdf or seismic hazard curve, $G_A(a)$, showing the annual probability of exceeding a specified ground acceleration, a.

Typical seismic hazard curves for building sites in the Western US and the Eastern US are compared in Figure 2.3. The seismic hazard curves can be described by a Type II distribution of largest values (Cornell, 1968),

$$G_A(a) = 1 - F_A(a) = 1 - \exp[-(a/u)^{\alpha}] \qquad (2.14)$$

in which a = ground acceleration and u and α are parameters of the distribution. Parameter α determines the slope of the hazard curve. The curve in the Eastern US is much flatter (smaller α) than that in the Western US (see Figure 2.3); 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 US hazard curve saturates at acceleration levels on the order of 1.2 to 1.8 times the design earthquake; in the Eastern US, the saturation level is unknown, but is believed to be on the order of 2 to 3 times the design earthquake.

Parameter α in Eqn 2.14 is related to the c.o.v. in annual maximum effective peak acceleration. Underlying the hazard analysis on which the seismic risk map found in Section 9 of ASCE 7-88 is an overall value α = 2.3 for the continental United States (Galambos, et al, 1982); the associated c.o.v. is 138%. Later hazard analyses (Algermissen, et al, 1982) on which the NEHRP Recommended Provisions for Seismic Regulations (FEMA 1992) are based have not led to significant reductions in this enormous c.o.v., although some variation from site to site is evident in the later study. The factor α tends to be larger for sites in the Western US, decreasing from about 5.5 (c.o.v. = 0.28) at San Francisco, CA to approximately 2.3 (c.o.v. = 1.38) at Boston, MA and Memphis, TN.

The NEHRP Recommended Provisions are being considered, with minor variants, for adoption 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. Earthquakes with magnitudes less than M_L = 4.0 or 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. This probability is much less than the annual probabilities of exceeding the design snow or wind loads in ASCE 7-88, which normally are 0.02. 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 is set equal to 1.0 in the NEHRP Recommended Provisions and in the revision to ASCE Standard 7 (cf Eqns. 1.8 and 1.9) rather than a greater value.

The distributions of the annual extreme effective peak ground acceleration (EPA) and the pulse intensity can be related using Eqn 2.9,

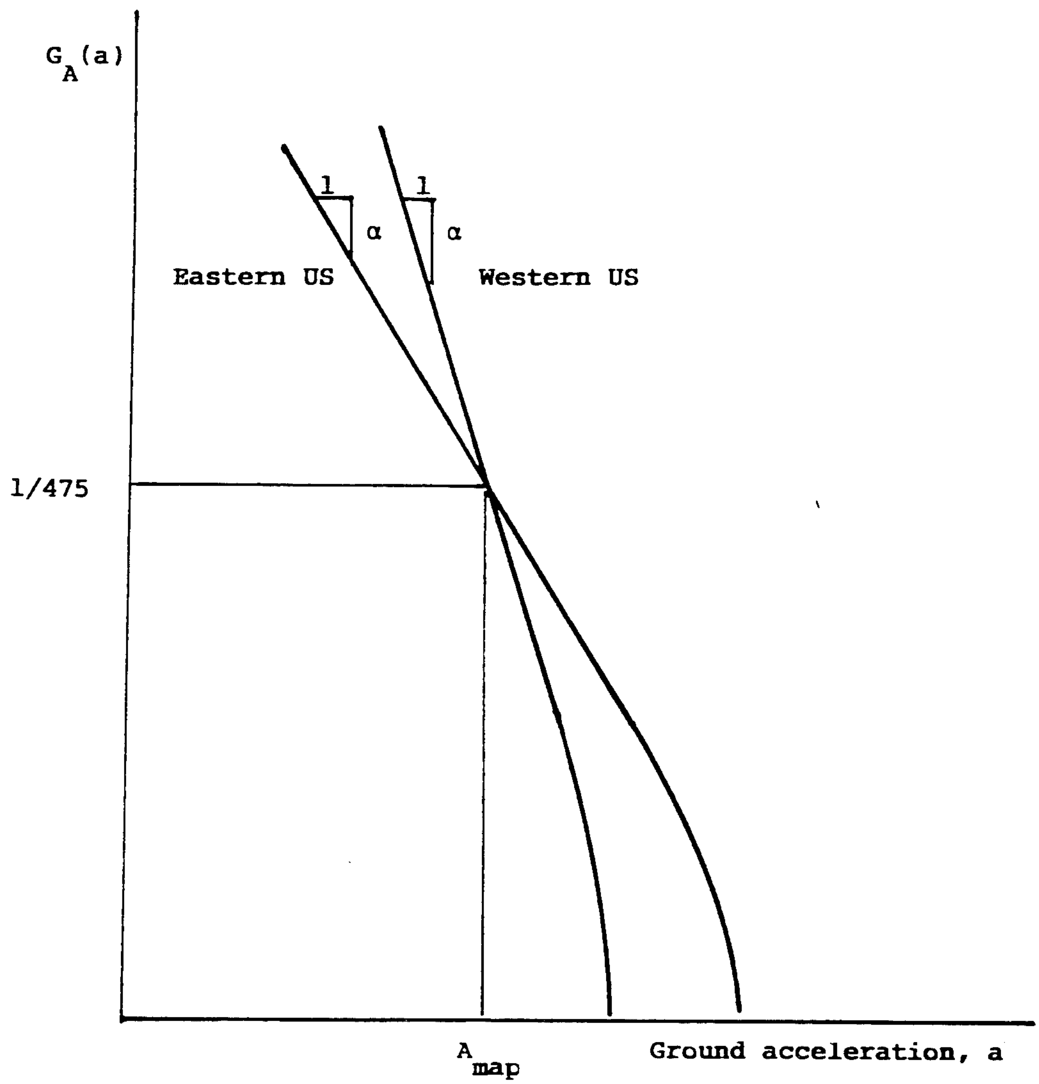


Figure 2.3 Typical seismic hazard curves

substituting the earthquake statistics for those of snow. Thus, the maximum EPA of an individual earthquake can be described by,

$$F(a) = 1 + (1/\lambda_E) \ln F_A(a) \approx 1 - G_A(a)/\lambda_E \tag{2.15}$$

If a_o denotes the minimum level of EPA necessary for a structurally damaging earthquake, $F(a_o) = 0$ and $\lambda_E = -\ln F_A(a_o)$. Parameter a_o typically would be about 0.05g. The parameter λ_E varies from about 0.01/yr for sites in the Eastern US to about 0.10/yr for sites in the Western US, although a few sites in California indicate values of λ_E as high as 0.5. The mean duration of ground shaking would be in the range of 10 to 60 sec, depending on the magnitude of the earthquake.

The annual extreme earthquake structural action can be modeled by a Type II distribution of largest values (Galambos, et al, 1982). The coefficient of variation in the load effect typically is 0.8 - 1.3, practically all of which is attributable to the uncertainty in the basic seismic hazard at a building site.

2.3.4 Flood Loads

A design flood carries with it considerations for both hydrological risk and facility risk. However, facility risk depends on the operation characteristics of the facility, and thus would vary for different navigation structures.

Flood risk estimates are region-dependent. Risk assessment of facilities such as dams has been part of water resources planning and management for many years (Cochrane, et al, 1987). Design flood levels have been most commonly expressed in terms of either annual probability of being exceeded or as a probable maximum flood (PMF). The recommended probability distribution for annual flood flow is the log-Pearson Type III (Interagency, 1982). The annual probability often is selected equal to 0.01, corresponding to a mean recurrence interval of 100 years. Historical records of flood events in most areas of the United States seldom extend back more than 50 years. Thus, extrapolations beyond annual probabilities of 0.01 have limited validity and may be subject to significant sampling error. The PMF is generated from a mathematical model of local climatology and involves the conversion of historical storm and precipitation data, which generally are more complete than flood data, to surface runoff and flood levels. The PMF is derived from the probable maximum precipitation, estimates of which are available from the National Weather Service (Committee, 1983). These mathematical models introduce uncertainties of their own into the determination of flood level. The annual probability associated with a PMF is unknown, but likely is less than 10^{-4}/yr. It should be emphasized that the PMF does not have a probabilistic basis like the 100-year MRI flood; however, studies have indicated that the ratio of 24-hr PMP to 100-year mean recurrence interval precipitation at sites in the upper Mississippi basin is approximately 5 (Safety, 1985).

The flood statistics of interest for event combination analysis are primarily the mean rate of occurrence and the duration of the flood. It seems unlikely that a navigation facility would be designed to withstand the effects of extreme floods directly. For example, the gates at a lock and dam might simply be opened during a design-basis flood to allow passage of high levels of water without damage to engineered structures.

2.4 Operating Loads

The normal operations of navigation structures such as gates at lock and dam facilities give rise to several types of structural loads. These include hydrostatic pressure, temporal head, hydrodynamic load, equipment loads, and impact. The probabilistic modeling of these loads are dependent on the operating characteristics of the facility. Little statistical data are available to support these probabilistic load models. Statistical analysis of poolwater and tailwater elevations at three lock and dam sites on the Mississippi River has provided some data on hydrostatic forces, while the analysis of a survey of lock operators on several major river systems has provided some preliminary data on the other operating loads. The analysis of these data is described in detail elsewhere (Ellingwood 1993); here, only a summary is provided.

2.4.1 Hydrostatic Loads

The hydrostatic load H_s on a gate in the closed position is determined by the upper and lower pool elevations. These elevations are site-dependent and their variation is determined on weather conditions. While there is natural variation in pool elevation from year to year, the variability in elevation and in differential is relatively small because the river level is controlled by dams for flood control upstream and downstream. The resultant force per unit width of structure is,

$$F = \gamma_w \Delta (H_p + H_t)/2 \qquad (2.16)$$

in which $\Delta = H_p - H_t$ and H_p, H_t = pool and tailwater depths. The hydrostatic pressure at depth z below the upper pool elevation is,

$$
p(z) = \begin{cases} \gamma_w z; & z \leq \Delta \\ \gamma_w \Delta; & z > \Delta \end{cases} \tag{2.17}
$$

The maximum annual differential does not necessarily coincide with the maximum pool.

The randomness in the intensity of F or p(z) is determined primarily by the relative control on water level provided by upstream and downstream dams. For Lock and Dam Nos. 24, 25 and 27 on the Mississippi River, the coefficients of variation in F ranged from less than 0.10 to about 0.30. The mean rate of occurrence and mean durations of these hydrostatic loads is determined by the rate of lockages; for heavily travelled rivers such as the Mississippi or Ohio, typical values might be λ = 200/month and τ = 90 minutes.

2.4.2 Equipment and Hydrodynamic Loads

Hydrodynamic load H_d and equipment load Q arise during the operation of the gates. Under normal operating conditions, Q and H_d represent a statically equilibrated force system. However, 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. Uncertainties in Q and H_d may arise from variations in pool elevations, from surge effects caused by movement of the gates or overtravel of water in the culvert system during filling. Temporal hydraulic loads occur due to temporary variation in water level and wave action caused by tow movement or from overfilling or underfilling the local chamber. The load processes for equipment, hydrodynamic and temporal head were determined from a Delphi of lock and dam operators (Ellingwood 1993).

2.4.3 Impact

Impact loads arise from accidents in which vessels collide with navigation structures. Such accidents may result from failure of steering or towing mechanisms, wind and current, or pilot error. A database on impact occurrence is maintained by each USACE District Office. These data are summarized for lock and dam structures in TR-REMR-HY-7 (Martin and Lipinski, 1990). A review of these data indicated that the average rate of accidents involving miter gates at locks on the Mississippi, Illinois and Ohio Rivers is approximately 0.06/month; however, not all of these accidents required that the lock be shut down for repair. When only those accidents that involved significant damage to the lock (e.g., over $50,000 in damage) are considered, the mean rate of occurrence decreases to approximately 0.016/month, or about once every five years.

The force due to barge impact depends on the mass of the barge and its velocity at impending collision. Forces on navigation structures due to barge impact can be enormous; in a recent impact test at LD 26 on the Mississippi River (Chasten 1992), the force measured as a fully loaded 9-barge tow impacted the gate at 0.94 ft/s (0.64 mph) was over 600 kips. Terminal velocities of this order are not inconsistent with the limited data available from other sources (Ellingwood, 1993).

2.5 Summary

A summary of the event and load process characteristics used in the subsequent event and load combination analysis is presented in Table 2.1. These parameters include the mean rate of occurrence and duration, λ and τ, and the point-in-time probability distributions $F(x)$.

Table 2.1 - Statistics of Structural Load Events

Load	Occurrence Rate(/yr)	Duration	Intensity Mean	c.o.v.	cdf
Dead	const.	50 yr	1.05D	0.10	Normal
Snow	4	1 wk	0.20S	0.90	Type I
Wind	2	4 hr	0.30W	0.60	Type I
Earthq.	0.02	30 s	Site-dep	0.85	Type II
Operations	4,800	2 m	0.7Q	0.50	Type I
Hydrost.	2,400	90 m	$0.9H_s$	0.20	Type I
Temporal	4,800	5 m	$0.5H_t$	0.80	Type I
Impact	0.0013	15 s	1.0Im	n.a.	n.a.

3.0 PROBABILISTIC BASES FOR EVENT COMBINATION ANALYSIS

3.1 General Considerations

In performing a probabilistic event combination analysis with a view toward ensuring structural safety, the primary interest is in structural actions (forces, deformations) and their combinations arising from such events.

Before performing such an analysis, it is important to identify any possible dependencies among individual loads and to screen from consideration other combinations that are not possible. This preliminary evaluation should be done to simplify the event combination analysis, which can be data-intensive and relatively complex mathematically. For example, it would be reasonable to assume in evaluating a lock and dam facility that gate operation is independent of the environment (leaving aside those periods of the year where winter weather may prevent navigation entirely and obviate the need to operate the structure). Thus, one would assume that hydrostatic pressure and earthquake could be treated as statistically independent, and that analysis of combinations of extreme snow (or ice) and operating loads would not be required. Similarly, if one were considering loads acting on a moveable gate, hydrostatic force H_s is exclusive of operating loads H_d or Q because the gate does not operate under differential head; on the other hand, H_d and Q form a statically equivalent couple during normal operation.

Consider now those events or structural actions that may need to be combined according to the above preliminary assessment. Most events giving rise to significant structural loads occur infrequently and usually do not last very long. The appropriate combination of structural actions will depend in a general sense on the rate of occurrence of the events, their duration, the magnitude of the structural action and, in the case of a dynamic force, its scale of fluctuation during the event. Mathematically, such events can be modeled in the macro-timescale by an intermittent pulse process of some kind as described in Section 2.1, one that captures the randomness in occurrence and duration of the events and a general measure of their intensity. If the structural action is essentially static and varies little within a given event occurrence, such a pulse process model may be sufficient since all that is required for safety evaluation is the extreme load to occur during the event. For example, loads due to hydrostatic pressure or snow can be modeled by pulse processes. On the other hand, if the structural action during an event is dynamic, (e.g., earthquake, surge), the macroscale modeling of the load process may not be sufficient, and additional steps are required to model the variations in the micro-timescale.

Methods for modeling such effects for individual loads were presented in Section 2.1.

3.2 Fundamental Event Combination Analysis

The basis for the event combination methodology is the probability distribution of the maximum of a sum of time- dependent structural loads modeled as stochastic processes. Let

$$U(t) = X_1(t) + X_2(t) + \dots \tag{3.1}$$

$$U_{max} = \max_{o<t\leq T} U(t) \tag{3.2}$$

in which $X_i(t)$ = structural action due to a stochastic event (e.g., operating load, temporal hydrostatic load, earthquake).

If two or more loads that are always nonzero are combined, the probability distribution of the combined load involving $X_1(t)$, $X_2(t)$,. . ., at any point in time can be obtained by the convolution theorem of probability theory (Papoulis, 1965). For example, if $U = X_1 + X_2$, the cumulative probability distribution function, $F_U(x)$, is,

$$F_U(x) = \int_o^{\infty} F_{X_1}(x - x_2) \; f_{x_2}(x_2) \; dx_2 \tag{3.3}$$

in which $F_{X_1}(x)$ is the cdf of X_1 and $f_{x_2}(x)$ is the density function for X_2.

Analyzing combinations of intermittent loads which may be zero for significant periods of time is more complex. The possibility must be considered that at any given time, only one of the loads acts or that two (or more) of the loads act simultaneously. Moreover, one generally is more interested in the maximum of the combined effects during a service period, T, rather than the point-in-time combined load effect for design purposes. The analysis of the maximum of a combination of time-dependent stochastic events is related to the difficult first-passage problem of probability theory. Although the distribution of U_{max} generally cannot be determined exactly, conservative approximations are available (Larrabee and Cornell, 1981; Pearce and Wen, 1984; Winterstein and Carroll 1984).

It is assumed in the following that the structural actions $X_i(t)$ represent stresses, strains, deformations or internal forces arising from event i. The limit state equation is written in terms of the processes, $X_i(t)$, rather than in terms of the events. However, the temporal nature of the event and the load generally are directly related. Thus, the processes $X_i(t)$ can be classified into the same basic three categories as shown in Figures 1.1 and 2.1. We seek a conservative approximation to the cdf describing U_{max}.

Upcrossing analysis provides a powerful tool for analyzing extremes of combinations of structural loads(Veneziano, et al, 1977; Rackwitz, 1985). The load process can be nonstationary in microscale without significant error as long as it is stationary in macroscale. It can be shown (e.g., Shinozuka, 1981) that a conservative upper bound to the complementary cdf of U_{max} is given by,

$$P[U_{max} > x] < P[U(O) > x] + \int_o^T v_U(x,t)dt \qquad (3.4)$$

in which $v_U(x,t)$ is the (time-dependent) mean rate at which U(t) upcrosses level x. If the process U(t) is stationary, the mean upcrossing rate function is independent of t, and

$$P[U_{max} > x] < P[U(0) > x] + v_U(x) T \qquad (3.5)$$

If the upcrossings of level x are rare as they usually are in structural reliability analyses, the occurrence of upcrossings can be modeled as a Poisson process with intensity $v_U(x)$. In this case, the complementary cdf is,

$$P(U_{max} > x] = 1 - \exp[-v_U(x)T] \qquad (3.6)$$

Eqn. 3.6 is asymptotically correct as x becomes large. At lower values of x, the upcrossing events are not Poisson; however, It has been found that a good approximation at moderate levels of x is (Ditlevsen, 1981),

$$P[U_{max} > x] \approx 1 - F_U(x) \quad \exp[-v_U(x)T/F_U(x)] \qquad (3.7)$$

Consider for illustration a case where there are two time-dependent structural actions, so that

$$U(t) = X_1(t) + X_2(t) \qquad (3.8)$$

Assume that $X_1(t)$ and $X_2(t)$ are statistically independent. There are four mutually exclusive and collectively exhaustive ways for X_1 and X_2 to occur:

$$E_{00} = \{X_1(t) = 0 \text{ and } X_2(t) = 0\} \tag{3.9a}$$

$$E_{10} = \{X_1(t) > 0 \text{ and } X_2(t) = 0\} \tag{3.9b}$$

$$E_{02} = \{X_1(t) = 0 \text{ and } X_2(t) > 0\} \tag{3.9c}$$

$$E_{12} = \{X_1(t) > 0 \text{ and } X_2(t) > 0\} \tag{3.9d}$$

The mean upcrossing rate $v_U(x)$ can be determined from the theorem of total expectation:

$$v_U(x) = \sum_i v_{U|E_i}(x)\, P[E_i] \tag{3.10}$$

in which i = 10, 02, and 12. This can be expressed as,

$$v_U(x) = v_{10}(x)\, p_{10} + v_{02}(x)\, p_{02} + v_{12}(x)\, p_{12} \tag{3.11}$$

in which v_{10}, $p_{10} = p[E_{10}]$ are the mean upcrossing rate and probability of occurrence of X_1 when $X_2 = 0$; v_{12}, p_{12} are mean upcrossing rate and probability of coincidence of $X_1 + X_2$, etc. Implementation of Eqn 3.11 requires information on the probabilities $P[E_i]$ and mean upcrossing rates v_i.

The probabilities $P[E_i]$ depend on the distribution functions of the inter-arrival times of X_1 and X_2 and durations of these events. Under the assumption that the occurrence of individual load events can be described by a Poisson process, with rate parameter λ the inter-arrival times,T_a, for the events are independent random variables described by an exponential distribution, with parameter λ;

$$F_{T_a}(x) = 1 - \exp[-\lambda_i x];\ x \geq 0 \tag{3.12}$$

The expected value of T_a, $E[T_a]$, is $1/\lambda$. We now suppose that the durations, T_d, of the event also are exponential with parameter, μ, but impose the additional condition that T_d cannot be longer than T_a. The duration of the event thus is described by the following conditional cdf:

$$F_{T_d}(x) = P[T_d < x | T_d < T_a] \quad (3.13a)$$

which can be expressed as,

$$F_{T_d}(x) = 1 - \exp[-(\lambda + \mu)x]; \quad x \geq 0 \quad (3.13b)$$

The expected value of T_d is $E[T_d] = 1/(\lambda + \mu)$. The probability that the event is acting, or "on", is $p = E[T_d]/E[T_a] = r/(1 + r)$, in which $r = \lambda/\mu$, while the probability that the event is not acting, or is "off," is $1 - p = 1/(1+r)$. When $\lambda/\mu \rightarrow 1.0$, the process is almost always "on"; as $1/\mu \rightarrow 0$, one obtains a process consisting of impulses with (essentially) no duration, as in Figure 2.1c.

When two events with the above temporal characteristics are possible, there will be times at which there are none, one, or two events acting at one time. A standard result from renewal theory states that the probabilities of having neither X_1 nor X_2, X_1 without X_2, X_2 without X_1, and X_1 and X_2 together (cf Eqns 3.9) are,

$$\begin{aligned} p_{00} &= 1/\prod_{i=1}^{2}(1 + r_i) \\ p_{10} &= r_1 \, p_{00} \\ p_{02} &= r_2 \, p_{00} \\ p_{12} &= r_1 r_2 \, p_{00} \end{aligned} \quad (3.14)$$

Extensions to more than two events are straightforward (Shinozuka, 1981). The parameters r_i generally will be much less than unity for transient events arising from infrequent environmental events such as floods or earthquakes.

The mean upcrossing rates of X_1 and X_2 in Eqn 3.11 can be computed analytically (Eqns 2.3 - 2.5) or from a time history of the load for a well-behaved process (Eqn 2.6). For the combined event $X_1 + X_2$, the point crossing formula (Larrabee and Cornell, 1981)

$$v_{12}(x) = \int_0^\infty v_1(x-y)\, f_2(y)\, dy + \int_0^\infty v_2(x-y)\, f_1(y)\, dy \quad (3.15)$$

expressing the combined upcrossing rate in terms of the marginal pdfs and upcrossing rates can be used.

An alternate procedure for event combination analysis is denoted the load coincidence method (Wen, 1977). Using the load coincidence method, the complementary cdf is given by,

$$P[U_{max} > x] = \sum_i \lambda_i TG_i + \sum_i \sum_j \lambda_{ij} TG_{ij} + \ldots \tag{3.16}$$

in which λ_i = mean rate of occurrence of the i_{th} load acting alone, G_i = probability that load i acting alone exceeds x, λ_{ij} = mean rate of occurrence of $x_1 + x_j$, G_{ij} = probability that $X_1 + X_j$ exceeds x, etc. Eqn 3.16 implies that the mean upcrossing rate is equal to the occurrence rate of the combinations multiplied by the probability that the combination exceeds x, given that it occurs.

The similarities of Eqns 3.5 and 3.11 to 3.16 should be noted. The equations are identical if

$$v_i(x)T = G_i(x) \tag{3.17a}$$

$$p_i = \lambda_i \tau_i \tag{3.17b}$$

These identities hold if the processes are "sparse," i.e., $\lambda_i \tau_i < 1$. This is characteristic of processes describing significant operating, environmental and accidental loads, which occur infrequently and have short duration. The accuracy of both approaches has been established by Monte Carlo simulation (Wen, 1977; Wen, 1981; Winterstein and Cornell, 1984). Both approaches show good agreement at the levels of x significant for structural safety studies.

Figure 3.1 illustrates the analysis of two intermittent events with an event tree (Winterstein, 1980) to facilitate better visualization of the event combination methodology. The occurrence of the extreme load events is assumed to be modeled by Poisson processes, and thus the probability of load 1 (or load 2) occurring in interval of time (t,t+dt] is $\lambda_1 dt$ (or λ_2 dt), i.e.,

$$P[\text{load 1 occurs in } (t,t+dt]] = \lambda_1 dt$$

This probability, in itself, does not guarantee that only load 1 acts, since it is possible that load 2 is also acting at the time load 1 acts. To ensure that the load events are mutually exclusive, the possibility of a joint occurrence of loads 1 and 2 must be taken into account. The probability that load 1 occurs

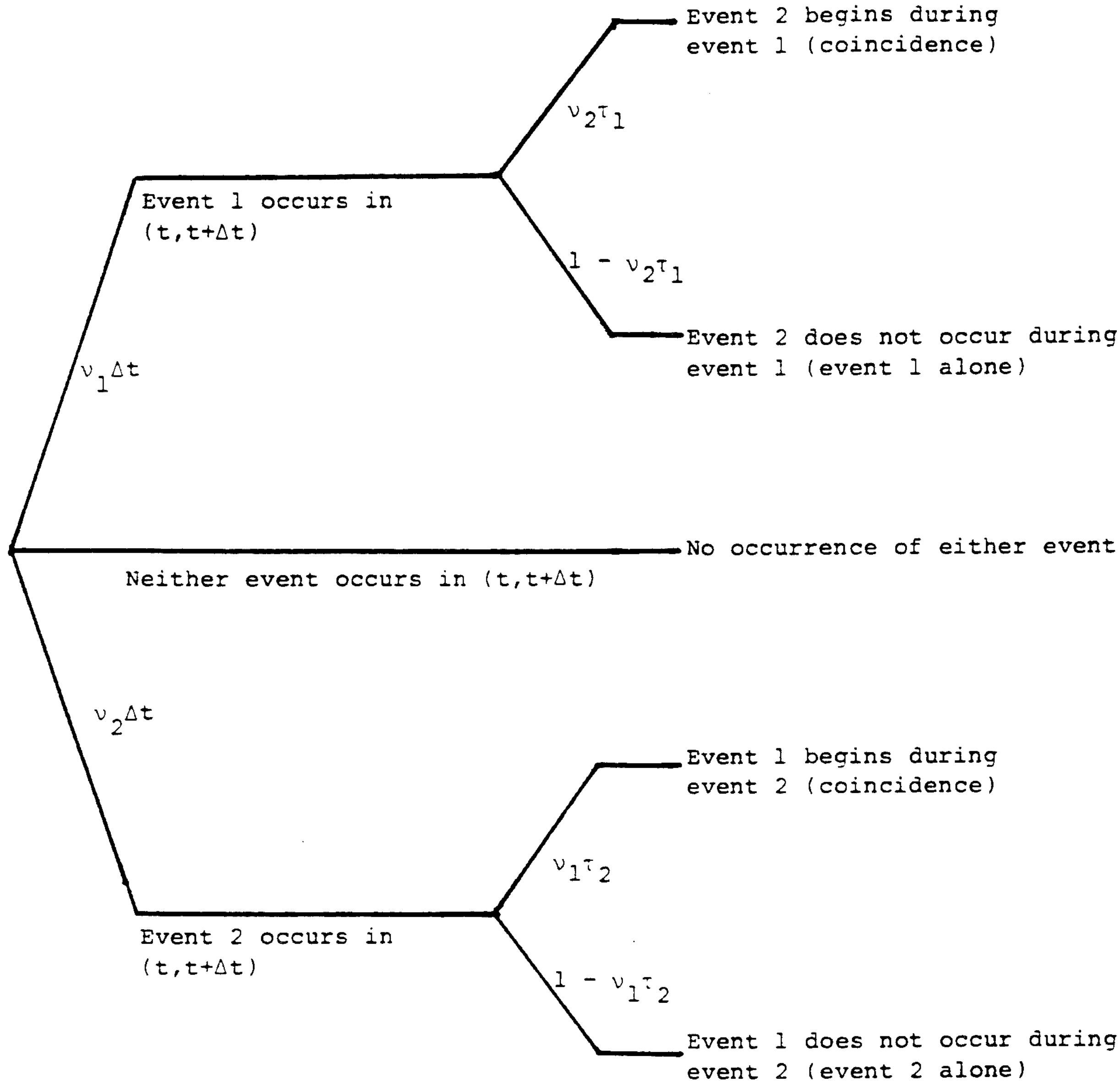


Figure 3.1 Event tree for analysis of event combinations (after Winterstein, 1980)

and load 2 does not occur during the interval in which X_1 acts is $\lambda_1 dt(1- \lambda_2 \tau_1)$. A similar analysis must be done for load X_2 in combination with load X_1. The probability of a coincident event occurring in (t,t+dt] is $\lambda_1\lambda_2 dt(\tau_1 + \tau_2)$. The mean rates of occurrence of the mutually exclusive events are then:

$$\lambda_{10} = \lambda_1 - \lambda_{12} = \lambda_1 - \lambda_{12} (\tau_1 + \tau_2) \quad (3.18a)$$
$$\lambda_{02} = \lambda_2 - \lambda_{12} = \lambda_2 - \lambda_{12} (\tau_1 + \tau_2) \quad (3.18b)$$
$$\lambda_{12} = \lambda_1\lambda_2 (\tau_1 + \tau_2) \quad (3.18c)$$

The mean durations of the mutually exclusive events are τ_1 and τ_2 and, for the coincident event,

$$\tau_{12} = \tau_1\tau_2/(\tau_1 + \tau_2) \quad (3.19)$$

If the occurrence of one event influences the occurrence of a second event, the two load processes are no longer statistically independent and Eqns 3.18 and 3.19 are not applicable. The Poisson occurrence model on which the event combination analysis is based can be modified to take this dependence in occurrence into account.

The situation can be visualized by the load process sample functions illustrated in Figure 3.2. Suppose that event 1 occurs according to a Poisson point process, with mean rate λ_1. Event 2 also occurs according to a Poisson point process with rate λ_2; however, some of these latter events are initiated by event 1 while others occur independently. Suppose that event 2 is initiated by event 1 with a probability, p, and at a random time delay θ. Event 2 can also occur at random, with mean rate of occurrence λ_2 as before. Then, the occurrence of event 2 is a Poisson process (Wen, 1981), with mean occurrence rate $\lambda_2 = \lambda_2 + p_{v1}$. Although events 1 and 2 individually are Poisson point processes, collectively, the events may cluster because of the common term λ_1.

The mean rate of coincidences of such clustered events (cf Eqns 3.18) is dependent on the distributions of the durations of each event and on the delay time,θ. If the event durations and θ are modeled by exponential distributions, the mean coincidence rate is, approximately (Wen, 1981),

$$\lambda_{12} = \lambda_2\lambda_1(\tau_2 + \tau_1)[1 + p_{v1}/(\lambda_2\lambda_1 \mu_\theta] \quad (3.20)$$

in which μ_θ is the mean delay time. Note that the first part of Eqn 3.20 is the same as Eqn 3.18c; the portion in brackets represents the contribution of stochastic dependence in event occurrences.

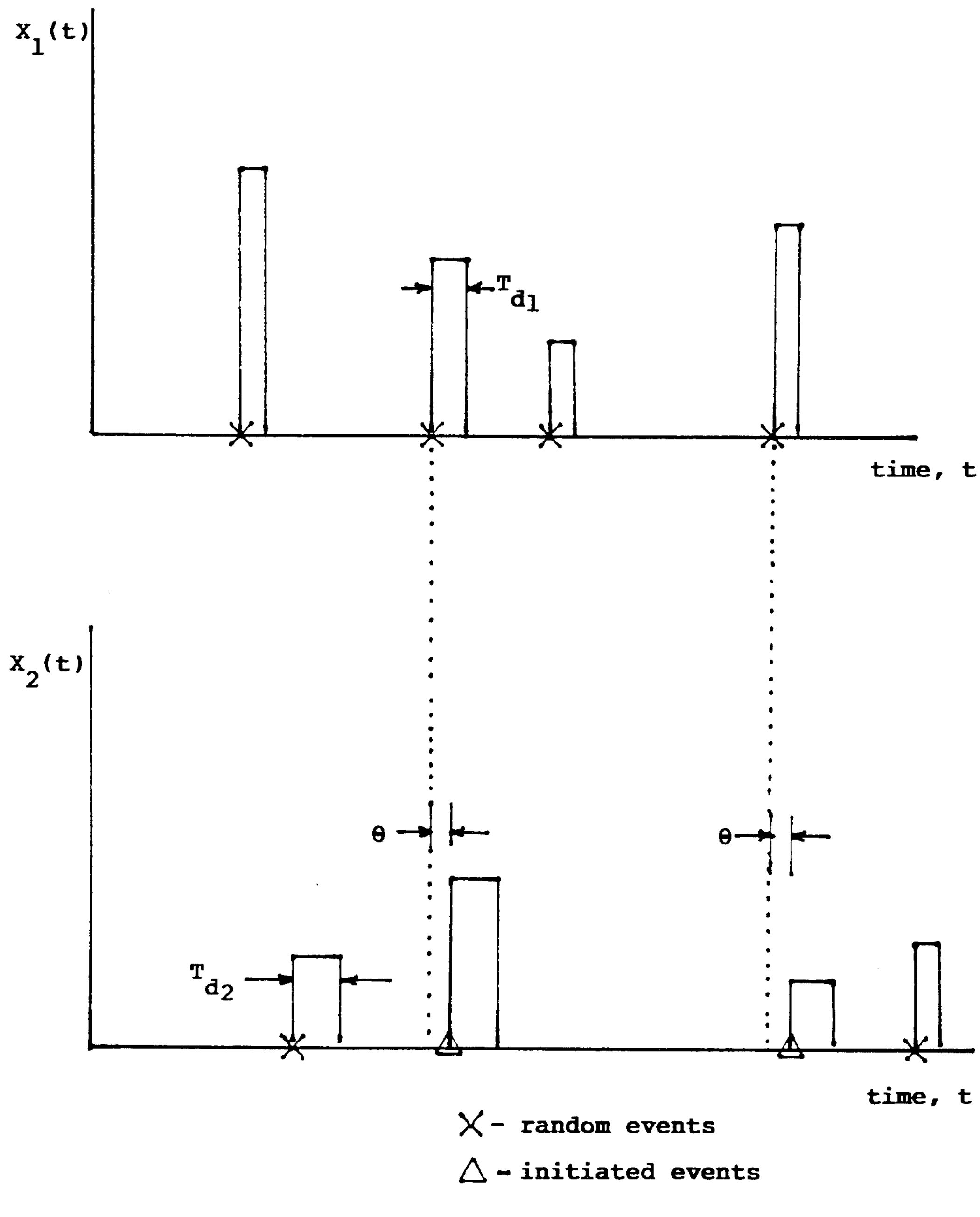


Figure 3.2 Stochastically dependent event occurrences

3.3 Illustration of Event Combination Analysis

Statistical characteristics of the structural loads are summarized in Table 2.1. Type II loads (Figure 2.1) occur intermittently and may equal zero for significant periods of time. If the pulse process is intermittent, the probability that the process is nonzero at any time is $p=\lambda\tau$. For a combination of two intermittent processes, the mean rate of occurrence and the duration of a coincidence of two Type II loads i and j are given in Eqns 3.18 and 3.19. The mean rates of occurrence and durations of loads acting individually and in combination can be used to identify those combinations of events that lead to load combinations requiring codification for design. The principal action-companion action format used in modern limit states design (cf Eqns. 1.5 - 1.9) requires that each of the time-varying loads assume, in turn, the position of the principal load. If the probability of a joint occurrence of a companion action with the principal action is very small, then the companion action may be set equal to zero in the load combination without much affecting the probability that the design load combination will be exceeded. Deterministic load combinations involving loads with essentially zero probability of load coincidence do not enhance structural reliability. For practical purposes, if a combined load event has a coincidence probability of less than 10^{-5}/yr, its contribution to risk is negligible and there is no need to consider it in a load combination analysis.

In the following paragraphs, the occurrence rates of several possible combined events for navigation structures are illustrated. Such an analysis for a particular facility may enable a number of event combinations to be screened from further design consideration.

Operating loads and wind. The mean rate of occurrence of operating loads H_d and Q is 400/month (4800/yr) and the mean duration is 2 minutes (3.8×10^{-6} yr). The mean rate of occurrence of significant extratropical winds is assumed to be 2/yr with a typical duration of 4 hr (4.56×10^{-4} yr). The coincidence of operating load and wind has a mean rate of occurrence,

$$\lambda_{OW} = (4800 \times 2)(3.8 \times 10^{-6} + 4.56 \times 10^{-4}) = 4.41/\text{yr}$$

$$\tau_{OW} = (3.8 \times 10^{-6})(4.56 \times 10^{-4})/(3.8 \times 10^{-6} + 4.56 \times 10^{-4}) = 3.77 \times 10^{-6} \text{ yr}$$

$$p_{OW} = (4.41/\text{yr})(3.77 \times 10^{-6} \text{ yr}) = 1.66 \times 10^{-5}$$

Operating loads and earthquake. The mean rate of occurrence of potentially significant earthquakes (magnitude 5 or greater) is assumed to be 0.02/yr or less at sites in the upper Mississippi, with a mean duration of 30 seconds (9.51×10^{-7} yr). The coincidence of operating load and earthquake has a mean occurrence rate

$$\lambda_{OE} = (4800 \times 0.02)\,(3.8 \times 10^{-6} + 9.51 \times 10^{-7}) = 4.56 \times 10^{-4}/yr$$

$$\tau_{OE} = 7.61 \times 10^{-7} yr$$

$$p_{OE} = 8.7 \times 10^{-10}$$

Operating loads and impact. The mean rate of occurrence of significant impacts at miter gates is about 0.19/yr, with a duration less than 15 seconds (4.76×10^{-7} yr). The mean occurrence rate of the combined event is,

$$\lambda_{OI} = (4800 \times 0.19)\,(3.8 \times 10^{-6} + 4.76 \times 10^{-7}) = 0.0039/yr$$

$$\tau_{OI} = 4.23 \times 10^{-7} yr$$

$$p_{OI} = 1.7 \times 10^{-9}$$

This analysis assumes, however, that an impact can occur when the gates are in motion; if lock operation precludes such an event, the mean rate of occurrence of a coincidence would be zero.

Earthquake plus impact. Using the occurrence statistics from previous examples, the mean rate of occurrence of a coincidence of these events is,

$$\lambda_{EI} = (0.02 \times 0.19)\,(9.51 \times 10^{-7} + 4.76 \times 10^{-7}) = 5.42 \times 10^{-9}/yr$$

$$\tau_{EI} = 3.17 \times 10^{-7} yr$$

$$p_{EI} = 4.3 \times 10^{-5}$$

Earthquake plus flood. Assume that the mean rate of occurrence of significant flooding is 0.10/yr and with a mean duration of 2 days ((0.0055 yr). The mean rate of occurrence of a coincidence of extreme flood and earthquake is,

$$\lambda_{EF} = (0.02 \times 0.10)\,(9.51 \times 10^{-7} + 0.0055) = 1.10 \times 10^{-5}/yr$$

$$\tau_{EF} = 9.51 \times 10^{-7} yr$$

$$p_{EF} = 2.6 \times 10^{-11}$$

On the basis of the above illustrations, only the combination of operating load and wind would appear to be worthy of additional consideration in developing design load combinations. The results are site-specific, however, and additional studies should be performed before drawing any definitive conclusions.

4. RECOMMENDATIONS FOR DESIGN

While the event combination methodology described in Section 3.2 might be applied in specific instances for evaluating an existing structure, for general design purposes the probabilistic statement must be transformed to a set of deterministic criteria. With the probability distribution of U_{max} determined, a design value, U_d, is selected with an acceptably small probability of being exceeded. There are three barriers to this approach: (1) A target probability for the event that U_{max} exceeds U_d must be established; (2) The determination of fractiles of a distribution is difficult and incompatible with routine design computations; and (3) There is a need to codify the design process to apply to a wide variety of design situations falling within the scope of the building code.

A target reliability measure (limit state probability or reliability index) must be established as a basis for design and evaluation considering the events that have not been screened out by the process in Section 3.2.

The limit state probability due to a particular event for a component or system can be written as

$$P(F) = P(F|E)\,P(E) \qquad (4.1)$$

in which P(E) = probability of the event and P(F|E) = limit state probability, given the event. This breakdown of the limit state probability into its constituent parts is useful in order to focus attention on appropriate strategies for damage prevention, mitigation and control. Reductions in failure probability and consequences of severe events can be accomplished by reducing either of the two probabilities in Eqn 4.1. A similar approach was taken in developing design criteria to reduce the risk of progressive collapse in buildings (Ellingwood and Leyendecker, 1978) or to reduce risks from fires (CIB W14,1983).

The event probability, P(E), can be reduced by reducing or eliminating the possibility of hazard in the operation of the facility. This can be done by controlling the use of the facility or by training and education. Generally, it is not feasible to eliminate all sources of hazard. Such measures to reduce P(E) generally are nonstructural and, indeed, nontechnical in nature.

The probability of failure given a structurally significant accident, P(F|E) depends primarily on the nature of the construction of the facility. Steps taken by the structural engineer to design damage tolerance into the structure through enhanced strength or protective systems mainly affect the probability, P(F|E).

The development of appropriate structural design criteria, therefore, begins with this quantity.

Event probabilities depend on the nature of the event. For normal operating events, $P(E) = 1.0$ and $P(F) = P(F|E)$. On the other hand, rare or accidental events such as significant barge impact occur infrequently (mean rate of occurrence on the order of 10^{-3} or less, cf Table 2.1). For rare or accidental events, special measures beyond the usual ASD or LRFD methods (e.g., protective or sacrificial elements) may be appropriate. For example the limit state probability (on an annual basis) for significant barge impact would be,

$$P(F) = 10^{-3} P(F|E) \tag{4.2}$$

To maintain economy in the allocation of design and construction resources, special design measures should be selected so that the resulting $P(F|E)$ is such that the risk of failure due to the accidental event is balanced with competing risks from other operational and environmental events.

To establish a frame of reference for these risks, the limit state probabilities of steel or reinforced concrete flexural members designed to withstand gravity loads are on the order of 0.0005 to 0.005 on a 50-year basis, (Ellingwood, et al, 1982; Galambos, et al, 1982). On an annual basis, then, these limit state probabilities are of the order 10^{-5} to 10^{-4}. However, the flexural limit state usually is benign and not life-threatening. Because of the severe economic consequences of failure of a navigation facility, it is reasonable to require $P(F)$ to be less than 10^{-5}. If one were to set $P(F|E) = 0.01$ in the above example, the annual probability of structural failure would be on the order of 10^{-5}, placing this risk in the low-probability background with other involuntary risks (Wilson and Crouch, 1987).

Criteria for risk-consistent design of a navigation facility should be developed to be consistent with a set of performance objectives expressed in probabilistic terms. A conditional limit state function, g( ), must be postulated using principles of mechanics and experimental data to relate the load and structural response variables of interest. The probability distributions of each variable, X, must be determined. The (conditional) limit state probability is obtained by integrating the joint probability density function of X over that region where $g( ) < 0$. In the example above, this probability would be compared with the target limit state probability $P[F|E] = 0.01$; if the computed probability is greater than the target, the strength of the structure must be enhanced in some way. Similarly, design requirements corresponding to a target limit state probability of 0.01 can be developed (Galambos, et al, 1982; Ellingwood, et al, 1982). One can consider only the load combination aspect of the safety check, leaving material behavior and limit state considerations to responsible material specification-writing groups.

Research in probabilistic load combination analysis has shown that failures that do not arise from negligence usually occur when one of the time-varying loads attains its maximum value while the other loads equal their point-in-time values (Turkstra and Madsen, 1980). This suggests that Eqn 1.2 can be replaced for practical design purposes by the set of equations,

$$U_d = \gamma_D D + \gamma_Q Q_n + \Sigma \gamma_i Q_i \tag{4.3}$$

The load combination format in Eqn 4.3 is referred to as a "companion action" format. The factored load γ_Q Q is denoted the principal action, while the terms γ_i Q_i are the companion actions. In principle, each variable load must be positioned, in turn, in the position of the principal variable load in order to determine the maximum combined effect; in practice, this seldom is necessary. Most modern load combination schemes invoke a companion action format of some kind (ASCE 7-88, 1990; Eurocode No. 1, 1991). The development of specific load combinations is outside the scope of this report.

5. CONCLUSIONS

Event combinations for design or evaluation of navigation structures should be evaluated probabilistically, and design criteria can be developed to be consistent with a level of performance expressed in probabilistic terms.

1. The probability of a joint occurrence of common loads at their peak values is very small. Thus, when considering what loads should be combined, a significant reduction in the combined load from the sum of nominal design values is appropriate. A principal action-companion action format for combining structural actions is recommended for maintaining uniform reliability.

2. The illustration of the event combination methodology in section 3.3 indicated the following:

   The probability of a coincidence of operating loads and earthquake is on the order of 10^{-10}.

   The probability of a coincidence of earthquake and impact is on the order of 10^{-15}.

   The probability of a coincidence of earthquake and flood is on the order of 10^{-11}.

It appears as if combinations of these events are very rare and would not need to be considered in design or evaluation. However, further study of these recommendations would be desirable.

3. Load combinations for dealing with accidental events should be based on a conditional probability of about 0.01-0.1 that the combined load effect is exceeded, given that an accident occurs. The overall unconditional probability of structural failure using this design approach is estimated to be on the order of 10^{-5}/yr or less.

4. Research should be undertaken to develop a protocol for combining extreme events in design of navigation facilities.

6. REFERENCES

ACI (1983). "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit,MI 111p.

AISC (1986). "Load and resistance factor design specification for structural steel buildings." American Institute of Steel Construction, Chicago, IL.

AISC (1989). "Specification for structural steel buildings: Allowable stress design and plastic design." American Institute of Steel Construction, Chicago, IL.

Algermissen, S.T., et al (1982). "Probabilistic estimates of maximum acceleration and velocity in rock in the contiguous United States." Open File Report 82-1033, U.S. Geological Survey, Denver, CO.

ASCE (1990). "Standard 7-88 Minimum Design Loads for Buildings and Other Structures." American Society of Civil Engineers, New York.

Benjamin, J.R. and Cornell, C.A. (1970). "Probability, statistics and decision in civil engineering." McGraw-Hill, New York, NY.

Breitung, K. and Rackwitz, R. (1982). "Nonlinear combination of poisson renewal pulse processes." J. Str. Mech., 10(2): 145-166.

Chasten, C. and Ruf, T. (1991). "Miter gate barge impact testing, lock and dam 26, Mississippi River." Proc. 1991 USA Corps of Engineers Str. Engrg. Conf.

Chen, H.-C. and Wen, Y.K. (1989). "Reliability of redundant structures under time-varying loads." in Proc. 5th Int. Conf. on Structural Safety and Reliability, Vol. II, ASCE, pp. 935-942.

CIB W14 (1983). "A Conceptual Approach Towards a Probability Based Design Guide on Structural Fire Safety." Fire Safety Journal, 6(1):1 - 79.

Cochrane, H. C., Ferrell-Dillard, R. and Baumann, D.D. (1987). "Socioeconomic considerations in dam safety risk assessment." IWR Report 87-R-7, US Army Corps of Engineers Institute for Water Resources, 1987, 163 pp.

Committee on Safety for Dams (1985). "Safety of dams: flood and earthquake criteria." National Academy Press, Washington, DC, 321 pp.

Cornell, C.A. (1968). "Engineering seismic risk analysis." Bull. Seismological Soc. Am. 58(5):1583-1606.

Cramer, H. and Leadbetter (1967). "Stationary and related stochastic processes." John Wiley, New York.

Ditlevsen, O. (1981). "Probabilistic modeling of man-made load processes and their individual and combined effects." in Proc. 3rd Int. Conf. on Structural Safety and Reliability, Elsevier, Amsterdam, pp. 103 - 134.

Ellingwood, B.R. and Leyendecker, E.V. (1978)."Approaches for Design Against Progressive Collapse." J. Str. Div., ASCE, 104(3):413-423.

Ellingwood, B., et al, (1982). "Probability Based Load Criteria: Load Factors and Load Combinations." J. Str. Div., ASCE, 108(5):978-997.

Ellingwood, B. (1983). "Probability-based safety checking of nuclear plant structures." NUREG/CR-3828, US Nuclear Regulatory Commission, Washington, DC, 73 pp.

Ellingwood. B. and Redfield, R. (1983). "Ground snow loads for structural design." J. Str. Engr. ASCE109(4):950-964.

Ellingwood, B. and Corotis, R.B. (1991). "Load combinations for buildings exposed to fires." Engr. J. AISC 28(1):37-44.

Ellingwood, B. (1993). "Load and resistance factor design for steel miter gates." U.S. Army Engineers Waterways Experiment Station Research Report, Vicksburg, MS.

EM-1110-1-2101 (1972). "Working stresses for structural design." US Army Corps of Engineers, Washington, DC, Nov. 1963 (rev. Jan. 1972).

EM 1110-1-2703 (1984). "Engineering and design:lock gates and operating equipment." US Army Corps of Engineers, Washington, DC.

ETL 1110-2-532 (1992). "Reliability assessment of navigation structures." Engineer Technical Letter (CECW-ED), US Army Corps of Engineers, Washington, DC

Eurocode No. 1 (ENV-91-1, 1991). "Common unified rules for different types of construction and material." European Committee for Standardization, Brussels.

FEMA (1992). "NEHRP recommended provisions for the development of seismic regulations for new buildings (2 Vol.)." Federal Emergency Management Agency Report 223, Washington, DC.

Galambos, T.V., et al, (1982). "Probability Based Load Criteria: Assessment of Current Design Practice." J. Str. Div., ASCE, 108(5):959 - 977. Interagency Advisory Committee on Water Data (1982). "Guidelines for determining flood flow frequency." Washington, DC.

Larrabee, R. and C. A. Cornell (1981). "Combination of Various Load Processes." J. Str. Div., ASCE, 107(1):223 - 239.

Martin, S.K. and Lipinski, M.E.(1990). "Lock accident study." Technical report REMR-HY-7, US Army Corps of Engineers, Washington, DC.

Melchers, R.E. (1987). "Structural reliability: analysis and prediction." Ellis Harwood Publishers, Ltd., Chichester, UK.

O'Rourke, M., Redfield, R. and Von Bradsky, P. (1982). "Uniform snow loads on structures." J. Str. Div. ASCE 108(12):2781-2798.

Papoulis, A. (1965). Probability, random variables, and stochastic processes. McGraw-Hill, New York.

Pearce, T.H. and Wen, Y.K. (1984). "Stochastic Combinations of Load Effects." J. Str. Engr., ASCE, 110(7):1613 - 1629.

Rackwitz, R. (1985). "Reliability of system under renewal pulse loading." J. Engrg. Mech. ASCE 111(9):1175-1184.

Reiter, L. (1990). "Earthquake hazard analysis - issues and insights." Columbia University Press, New York, NY.

Rice, S.O. (1945). "Mathematical analysis of random noise." Bell System Tech. Journal. Vols. 22 and 24; Reprinted in N. Wax (ed.), "Selected papers on noise and stochastic processes." Dover, New York, 1954.

Schwartz, M.W., et al (1981). "Load combination methodology development: Load combination program project final report." Report NUREG/CR-2087, US Nuclear Regulatory Commission, Washington, DC, 128 pp.

Shinozuka, M., et al (1981). "Probability based load criteria for the design of nuclear structures: a critical review of the state-of-the-art." Report NUREG/CR-1979, US Nuclear Regulatory Commission, Washington, DC.

Shinozuka, M. (1981). Stochastic characterization of loads and load combinations." in Proc. 3rd Int. Conf. on Structural Safety and Reliability, Elsevier, Amsterdam, pp. 57 - 76.

Turkstra, C. and Madsen, H. (1980). "Load combinations for codified structural design." J. Str. Div., ASCE, 106(12):2527-2543.

Veneziano, D. Grigoriu, M. and Cornell, C.A. (1977). "Vector-process models for system reliability." J. Engrg. Mech. ASCE 103(3):441-460.

Wen, K.Y. (1977). "Statistical combination of extreme loads." J. Str., Div. ASCE. 103(5):1079-1093.

Wen, Y.K. (1981). "A clustering model for correlated load processes." J. Str. Div. ASCE 107(5):965-983.

Westergaard, H. (1933). "Water pressures on dams during earthquakes." Transactions ASCE 98:418-471.

Wilson, R. and E.A.C.Crouch (1987). "Risk Assessment and Comparisons: An Introduction." Science 236(4799):267-270.

Winterstein, S. (1980). "Combined dynamic response: extremes and fatigue damage combinations." Report R80-46, Dept. of Civil Engrg., Massachusetts Institute of Technology, Cambridge, MA.

Winterstein, S. and Cornell, C.A. (1984). "Load combinations and clustering effects." J. Str. Engrg. ASCE 110(11):2690-2708.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
July 1995

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Event Combination Analysis for Design and Rehabilitation of U.S. Army Corps of Engineers Navigation Structures

5. FUNDING NUMBERS

6. AUTHOR(S)
Bruce R. Ellingwood

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
Department of Civil Engineering
Johns Hopkins University
3400 N. Charles Street, Baltimore, MD 21218

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers, Washington, DC 20314-1000
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

10. SPONSORING/MONITORING AGENCY REPORT NUMBER
Contract Report ITL-95-2

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

Event combinations for design or evaluation of navigation structures should be evaluated probabilistically, and design criteria can be developed to be consistent with a level of performance expressed in probabilistic terms.

a. The probability of a joint occurrence of common loads at their peak values is very small. Thus, when considering what loads should be combined, a significant reduction in the combined load from the sum of nominal design values is appropriate. A principal action-companion action format for combining structural actions is recommended for maintaining uniform reliability.

b. The illustration of the event combination methodology indicated the following:

(1) The probability of a coincidence of operating loads and earthquake is on the order of 10^{-10}.

(2) The probability of a coincidence of earthquake and impact is on the order of 10^{-15}.

(3) The probability of a coincidence of earthquake and flood is on the order of 10^{-11}.

It appears as if combinations of these events are very rare and wouid not need to be considered in design or evaluation. However, further study of these recommendations would be desirable.

(Continued)

14. SUBJECT TERMS

Evaluation techniques	Navigation structures	Reliability assessment
Event combinations	Probabilistic loads	Risk analysis
Joint probabilities	Rare events	Structural assessment

15. NUMBER OF PAGES
48

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

13. (Concluded).

c. Load combinations for dealing with accidental events should be based on a conditional probability of about 0.01-0.1 that the combined load effect is exceeded, given that an accident occurs. The overall unconditional probability of structural failure using this design approach is estimated to be on the order of 10^{-5}/yr or less.

d. Research should be undertaken to develop a protocol for combining extreme events in design of navigation facilities.