

Chapter 2

Overview on Performance-Based Engineering Under Multihazard Environments

2.1 Performance-Based Engineering and Design

2.1.1 *Performance-Based Seismic Design (PBSD)*

Historically, building codes have required that buildings be built to meet a minimum level of safety requirement. In current design practice, the code-based design is performed to search for a feasible design solution satisfying all prescriptive mostly empirical code specifications. However, buildings conforming to those codes have been seen a dramatic rise in earthquake-related losses. As estimated by the Federal Emergency Management Agency (FEMA) in 2000a, b and 2008, in the past 10 years total earthquake-related losses were twenty times larger than in the previous 30 years combined (FEMA 349; FEMA 2008) as shown in Fig. 2.1. FEMA's expenditures related to earthquake losses have become an increasing percentage of its disaster assistance budget. Predictions are that future single earthquakes, which will inevitably occur, may result in losses of \$50–100 billion each. Developers, stakeholders, and occupants have become painfully aware of the financial and social consequences of earthquakes and are demanding that practical and cost-effective means be developed to address the issues of damage control and loss reduction.

Over the past several decades, leading structural engineers have promoted the development and application of performance-based seismic design (PBSD) concepts. The basic concept of PBSD is to provide engineers with the capability to design buildings that have a predictable and reliable performance in earthquake. In 1992, FEMA sponsored the development of national consensus guidelines for the seismic retrofit of buildings, the Applied Technology Council (ATC-33) project. This was the first attempt to standardize the performance-based approach. After that, the FEMA-273 (1997), National Earthquake Hazards Reduction Program (NEHRP) Guidelines for the seismic rehabilitation of buildings, provides nationwide acceptable guidelines for the seismic rehabilitation of buildings in the USA.

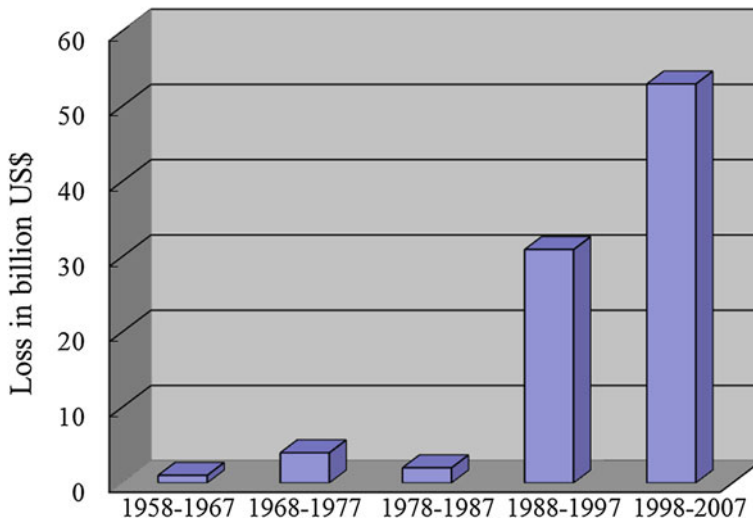


Fig. 2.1 National earthquake-related losses in USA (*Excerpt* FEMA 349; FEMA 2008)

One of significant new features is that the document identifies the methods and design criteria to achieve different levels and ranges of seismic performance. The four building performance levels are collapse prevention, life safety, immediate occupancy, and operational. The similar approach to define building performance levels also can be found in the modern seismic design codes (e.g., the Chinese Code for Seismic Design of Buildings GB50011-2010).

The performance-based seismic design and assessment guidelines for new steel moment frame buildings have been proposed by the SAC FEMA program (FEMA-350 2000a, b). The proposed reliability-based, performance-oriented approach is based on realizing a performance objective expressed as the probability of exceeding a specified performance level (Cornel et al. 2002). Kim and Foutch (2007) made an attempt to extend application of the FEMA approach to RC shear wall buildings. In that study, such as the analytical model, damage measure, and definitions of collapses were examined, and the confidence levels of the buildings were determined utilizing the new model and parameters. It was shown that the FEMA approach can be also useful for RC shear wall buildings.

2.1.2 Performance-Based Wind-Resistant Design (PBWD)

A performance objective is the specification of an acceptable level of damage to a building if it experiences an earthquake of a given severity. This creates a “sliding scale” whereby a building can be designed to perform in a manner that meets the owner’s various economic and safety goals. Further, PBSD permits owners and

other stakeholders to quantify financially or otherwise the expected risks to their buildings and to select a level of performance that meets their needs while maintaining a basic level of safety.

It is obvious that the concept of PBSB is generally applicable to building design against other kinds of nature or human-made hazards. For instance, strong wind is the major factor governing the structural design of most tall buildings in Hong Kong, situated in a typhoon-prone but low seismic region. As modern buildings get taller and increasingly slender, the effects of wind-induced motions become more pronounced and the amount of structural materials required for lateral and torsional resistance and serviceability occupant comfort increases drastically (Smith and Coull 1991). Although wind-induced performance levels (limit states) of buildings have not been well defined compared with seismic engineering, some research work toward wind-induced performance levels in associated with the performance design objectives (design criteria) can be found (Melbourne and Palmer 1992; Chock et al. 1998; Jain et al. 2001).

Wind-induced performance could be related to the dynamic responses of a building under wind actions. Wind-induced responses, in terms of internal force, deflection, drift, and acceleration, are comprehensively used to describe the building system behavior or performance under wind excitations. It is clear that performance levels are closely connected to the concept of limit-state design, which has been widely adopted in most of modern design codes. Actually, the concept of performance levels represent an evolution of prescriptive rules for limit-state designs that have changed during years as more is learned about building behavior. Furthermore, a performance-based design option will facilitate design of buildings to higher standards without explicit specification in the code and will allow rapid implementation of innovative technology. When performance levels are tied to probable losses and risk issues in a reliability-based design framework, the owner's long-term capital planning strategies can be taken into account in the building design process.

Although it has been realized that wind-induced dynamic serviceability is generally the major concerns in the design of tall buildings, research in structural design and optimization for dynamic serviceability has not received as much as attention as that for ultimate strength limit-state design of such structures. In fact, the design of tall buildings in windy climates is generally dominated by serviceability considerations in terms of wind-induced deflections and vibrations, rather than by member strength requirements (Griffis 1993). The deflections or deformations from all load types should not impair the strength or effective functioning of a structure, supporting elements or its components, nor cause damage to the finishes. For typical structures, the deflection limits are recommended by various building design codes. The first step in establishing a serviceability design criterion is to define the load under which it is to be checked. Wind loading criteria for both strength limit states and serviceability limit states in China are normally the same and typically are based on a 50-year return period wind for normal buildings and a 100-year recurrence interval wind for important structures. The design wind

pressures given in Hong Kong wind code (HKCOP 2004a, b) have been determined from the hourly mean wind speed and peak gust wind speed having a return period of 50 years.

Vibration and oscillation of a high-rise structure should be limited to avoid discomfort to users and damage contents. For special structures, including long-span bridges, large stadium roofs and chimneys, and wind-sensitive tall buildings, wind tunnel model tests are recommended for their wind-resistant design to meet various serviceability limits. The serviceability limit states on oscillation, deflection, and acceleration should be checked to ensure serviceable condition for the structure during its service life. The checking procedure for motion perception can be performed by comparing the magnitude of wind-induced vibration and acceptability threshold of motion (or so-called occupant comfort criteria). It has been widely accepted that the perception of wind-induced motion is closely related to the acceleration response of buildings (Kwok et al. 2007). Both peak acceleration and standard deviation acceleration under extreme wind conditions of 10-year or 5-year return period are commonly used to represent building motion (Burton et al. 2007).

Several researchers have suggested introducing acceleration limit states in preventing occupant discomfort as a new design performance level for wind-sensitive buildings (Griffis 1993; Isyumov 1994). Acceleration and occupant comfort criteria for buildings undergoing complex motions were discussed by Melbourne and Palmer (1992). In that study, acceleration criteria to achieve acceptable occupancy comfort in buildings have been developed in terms of peak accelerations as a function of motion frequency and return period. In terms of performance-based wind engineering, Chock et al. (1998) proposed a compatible set of wind design criteria which would be “risk-consistent” with the current framework for performance-based seismic engineering. Performance-based wind engineering design levels recommended by Chock et al. are given in Table 2.1. In the table, the design life of a building, which represents the exposure time for the building, is typically taken as 50 years to evaluate the probability of exceedance from the particular wind design level.

In terms of wind climatology, it is necessary to performing a site-specific design wind speed investigation, in which wind directionality and topographical effects

Table 2.1 Recommended performance-based wind engineering design level

Wind design severity level	Average return period (year)	Probability of exceedance	Performance levels
Very frequent	1	100 % in 50 years	Perception threshold
Frequent	5	99.9 % in 50 years	Occupant comfort
Frequent	10	99.5 % in 50 years	Fear for safety
Occasional	50	64 % in 50 years	Drift/Strength
Rare	475	10 % in 50 years	Safety
Very rare	1000	5 % in 50 years	Collapse prevention

would be considered. Jain et al. (2001) proposed a probability-based methodology used to determine site-specific performance-based design wind speeds for use in wind tunnel measurements and building design. In that paper, it was demonstrated that using such site-specific extreme wind loads can often lead to cost savings for the building owner.

2.1.3 Prediction of Typhoon and Its Risk by Simulation

Among the natural sources of risk, earthquake and typhoon are undoubtedly the most prominent ones. The concept of typhoon risk for tall buildings includes natural factors of atmosphere and water global circulation, local climatological and topographical conditions in the area under consideration, the level of lateral stability of the building structures, constructions, the extent of damage from possible typhoon and their consequences, social and economic factors.

While a typhoon refers to a tropical cyclone occurring in the western Pacific or Indian oceans, a hurricane is a severe tropical cyclone originating in the equatorial regions of the Atlantic Ocean or Caribbean Sea, traveling north, northwest, or northeast from its point of origin, and usually involving heavy rains. Eastern China is suffering by typhoon attacks in an annual basis. For example, according to the report of Hong Kong Observatory, near 4 tropical cyclones with equal or greater than No. 8 Signals (with a sustained wind speed of 63–117 km/h) have attacked the Hong Kong area for each year during the years of 1956–2006. Hurricane is the major nature disaster threatening the east coast of North America. In recent years, before 2006 season, a doubling of hurricane activity for the North Atlantic basin has occurred as well as an increase in the number of storms that are hitting land (Goldenberg et al. 2001).

The increase in hurricane frequency and intensity in the North Atlantic basin may be attributed to global warming (McCullough and Kareem 2007). Climate change will become an important factor to spark extreme weather including fierce hurricanes or typhoons. Climate scientists warned of a future characterized by extreme weather events—long and intense droughts, fierce hurricanes or typhoons, heat waves, and rising sea levels—as a result of rising temperatures in the Intergovernmental Panel on Climate Change (IPCC) report 2001. Scientists now understand much more about how weather systems work, so are able to predict how temperature changes affect rainfall patterns and storms. Jonathan Overpeck, professor of atmospheric sciences at the University of Arizona and a lead author of the IPCC report, pointed out that due to global warming more intense hurricanes will occur accompanied by rising sea levels, which may cause the sea to surge over coastal land. That will make coastal cities like New Orleans much harder to protect.

Wind hazard risk studies could quantify randomness of extreme wind events in terms of statistical distribution and parameters and provide basis for wind-induced performance-based design. Some wind hazard models (e.g., typhoon models) that accurately model the average wind field observations are proposed and available in

the literature (Vickery et al. 1995a, b, 2000). The numerical simulation of wind hazards (e.g., tropical storm, tropical cyclones) originated in the middle of the twentieth century with axisymmetric models, in which flow variations in the azimuthal directions are ignored (Ooyama 1969). Using these typhoon models, it is not difficult to simulate extreme wind events for a local typhoon-prone area, like Hong Kong. Actually, hourly mean wind velocity profile for design purpose according to Code of Practice on Wind Effects in Hong Kong 2004 is obtained based on observed and simulated directionally independent typhoon wind speed data.

Since directional extreme wind speed data are less than non-directional (all directional) wind speed data, it is necessary to develop sophisticated numerical simulation method to produce enough directional wind speed data for statistically estimating two-dimensional (speed and directionality) wind velocity climate. Various sophisticated three-dimensional models were proposed to study the more realistic tropical cyclones (Shapiro 1983). A successful simulation of hurricane Andrew using the fifth-generation PSU/NCAR Mesoscale Model (MM5) with a horizontal resolution of 6 km was reported (Liu et al. 1999). Such models given tropical synoptic conditions could also be applied in practice for short-range weather forecasting (Zhang et al. 2000). The use of these models or in the frame of Monte Carlo simulations as a numerical tool to study statistical characteristics of extreme wind events has been receiving increased attention (Vickery et al. 2000; Matsui et al. 2002; Jain et al. 2001). Using proper probability treatment calibrated by local meteorological data, directional and magnitude characteristic of probability distribution of extreme wind events (two-dimensional wind velocity climate) could be evaluated by typhoon simulation (Simiu and Heckert 1998; Matsui et al. 2002). The extreme value distribution analysis is to be conducted on each set of directional wind speed data to estimate the joint probability density function of wind speed and direction.

Since the past extreme wind events in Hong Kong were mainly tropical cyclones, the focus of the discussion will be placed on the simulation of tropical cyclones. Tropical cyclones can be described using Newton's second law augmented by conservation laws for mass, thermodynamic energy, and water vapor. As mentioned, one of the most rigorously developed wind climate models is MM5 model (Grell et al. 1994), which is open source and publicly available (<http://www.mmm.ucar.edu/mm5/>). After 2008, MM5 model has been upgraded into the Weather Research and Forecasting (WRF) Model, which is a next-generation mesoscale numerical weather prediction system designed for both atmospheric research and operational forecasting needs (<http://www.wrf-model.org/index.php>). It features two dynamical cores, a data assimilation system, and a software architecture facilitating parallel computation and system extensibility. The WRF model is a limited-area, non-hydrostatic, terrain-following sigma-coordinate model designed to simulate or predict mesoscale and regional-scale atmospheric circulation. The similar models have been successfully used for hurricane wind speed prediction and weather forecast. The MM5 or WRF model can be customized using terrain data of Hong Kong to simulate tropical storms which influence Hong Kong

area (Clark et al. 1997). In addition, the model can be simplified for use in Monte Carlo simulations to produce enough hurricane wind speed data (Vickery et al. 2000).

The MM5/WRF model uses compressible Navier–Stokes equations with pressure perturbation and temperature as prognostic conditions to describe an atmospheric motion. The governing partial differential equations (PDEs) of an atmospheric motion can be set up in terms of the terrain coordinates (x , y , σ). The computation domain including local terrain information can be discretized horizontally using B-grid staggering scheme and vertically using non-dimensional pressure sigma (σ) levels, which means that the lower grid levels follow the terrain while the upper surface is flat. Then, the governing equations can be numerically solved with the split-time-step approach where sound waves induced by compressible air flow are treated semi-implicitly on the shorter step (Grell et al. 1994).

Based on the daily recorded hourly mean wind speed data during synoptic wind storms from the local observatory, the parent wind speed, the distribution of the complete population of wind speeds at a particular site, can be modeled with a Weibull distribution (Gomes and Vickery 1977). Such information is important to estimate of fatigue damage for which account must be taken of damage accumulation over a range of wind storms. However, for structural safety design, extreme wind speed rather than daily wind speed is desired for predicting maximum wind effects and then designing the proportional structure to resist extreme wind loading. Using crossing theory of random process, the design wind speed with a particular recurrence interval can be derived from the parent distribution information and the annual mean crossing rate of exceedances of a specified wind speed using Rice's formula (1945), which is discussed in the later section about time-variant reliability. Then, the return period associated with the exceedance of the specified design wind velocity is readily obtained as a reciprocal of the annual mean crossing rate.

Design wind speed could also be conveniently estimated by making use of the annual extreme wind speed values from the local observatory or numerical simulation, directly. In this approach, design wind speed could be estimated by statistical analysis to best-fit observed or simulated typhoon wind speed data into the cumulative distribution function of type I asymptotic extreme value distribution (Simiu and Scanlan 1996; Zuranski and Jaspinska 1996; Minciarelli et al. 2001). Kasperski 2007 systematically discussed the relationship between the design wind speed estimation and the design wind load codification. In that study, it is highlighted that the analysis of the extreme wind speed has to be based on the separate storm mechanisms, e.g., strong frontal depression or other synoptic storms, gust fronts, thunderstorms, and tropical cyclones.

2.1.4 Uncertainty Modeling in Wind Engineering

For wind-excited buildings, major uncertainties rise from either aerodynamic wind loading characteristics or system properties. The uncertainty of wind loading

characteristics is directly affected by statistical properties of design wind speeds, which could be quantified based on the probabilistic wind speed data analysis as reviewed in the previous section. System properties, including mass, stiffness, and damping, are physically determined by the construction material, the structural member, and form. These structural properties further influence the dynamic characteristics of structures, i.e., the natural frequency- and vibration-mode shapes. Various levels of practical randomness due to fabrication and construction process arising from the material, member, or form may cause the structural system exhibiting uncertain behavior. In addition, the employ of computer models to predict system behavior inevitably introduces the model error due to the limits on resolution of the model or lack of sufficient knowledge and data.

A great extent of literature on uncertainties of the structural systems and wind characteristics is available in the context of wind engineering (Kareem 1987; Solari 1997; Hong et al. 2001). Kareem (1987) presented an overall probabilistic viewpoint for wind effects on structures. The discussion encompassed diverse scientific fields such as meteorology, fluid dynamics, statistical theory of turbulence, structural dynamics, and probabilistic methods. Solari (1997) provided closed-form expressions of the first and second statistical moments of the maximum response taking the uncertainties of the parameters and the idealized model error into account for wind-excited structures. The probability distributions of wind-induced maximum responses were also studied in the parameter space by Hong et al. (2001). Reliability analysis results in that paper suggest that the consideration of the uncertainty in structural parameters such as the fundamental frequency of vibration and the damping ratio is very important for the occupant comfort design and that these uncertainties may be ignored for the strength-based or drift design. Solari (2002) discussed the prospects of analytical methods for estimating the wind-induced response of structures and quantifying uncertainties. Closed-form solutions related to the 3D gust effect factor technique were extended to solve the complex probabilistic problems about the propagation of the parametric uncertainties and structural reliability.

The design wind speed is statistically estimated from the limited number of recorded or simulated data sample, which is inherently involved some uncertainties, i.e., sampling error, wind climate model variations. Peterka (1992) has shown that predictions of 50-year return period wind speeds from record lengths of 20–40 years at a single station can have a significant uncertainty associated with sampling error, which is mainly caused by using the limited number of sample data from a short record duration. The uncertainty in the estimation of design wind speeds could be quantified by studying the mean and variance of the estimator for design wind speeds (Grigoriu 1982; Minciarelli et al. 2001). Grigoriu (1982) developed an estimator of the design wind speed from short records by taking into account the correlation between data points. Minciarelli et al. (2001) proposed a probabilistic methodology to reexamining wind load factors adopted in the standard provisions of ASCE 7–98. The study demonstrates that the sampling errors in the estimation of design wind speed and knowledge uncertainties have a significant influence on the determination of wind load factors conforming to a specified safety index.

Structural damping has remained the most uncertain parameter in the dynamic design of civil engineering structures. Structural damping prediction relies upon a high degree of empiricism that is almost entirely based upon full-scale measurements (Glanville et al. 1996). Several damping empirical models were derived from the analysis of damping data obtained from a wide and homogeneous set of full-scale measurements of buildings and other structures (Lagomarsino 1993; Lagomarsino and Pagnini 1995). A log-normal probability distribution function is usually accepted for estimating the statistical properties of the random structural damping (Solari 1996; Pagnini and Solari 1998). Due to lack of precise knowledge on damping mechanism, the prediction and selection of an appropriate damping value is still a subject of discussion and controversy in design practice (Kareem 1987). In general, lower damping in the fundamental mode is assumed since most building structures deform as a rigid body in the elastic range with very little portion of energy being dissipated by deformation of elements and structural connections. Tamura et al. (1994, 1996) and Kitamura et al. (1995) introduced relevant advances in the knowledge of damping dependence on motion amplitude. Some research studies (Kareem and Gurley 1996; Kareem et al. 1998) have also examined the values of damping in higher modes. It is believed that in higher modes a building generally experiences more deformation in elements leading to the result of higher damping.

2.2 Dynamic Response Analysis of High-Rise Buildings

2.2.1 *Analysis Method in Time Domain*

The response of structural systems to external excitations is a general and essential problem in structural dynamics. The procedure for obtaining the response of a structural system to external forces depends to a large extent on the characteristics of the structural system and the type of excitation. The classification of the dynamic analysis problems is given in Fig. 2.2. In terms of deterministic or stochastic systems characterized by linear or nonlinear, the problems are classified into 4 sub-problems denoted by DL (deterministic linear system), DN (deterministic nonlinear system), SL (stochastic linear system), and SN (stochastic nonlinear system). Due to the principle of superposition, which clearly applies to linear systems alone, the responses of linear systems to a given number of distinct excitations can be obtained separately and then combined to obtain the total responses. It is because of this principle that the theory of linear systems is so well developed compared to that of nonlinear systems. Each subproblem (DL, DN, SL, and SN) can be further separated into two by consideration of external deterministic or random excitation. For example, the DL system has DLD (DL system subject to deterministic loading) and DLR (DL system subject to random excitation) problems.

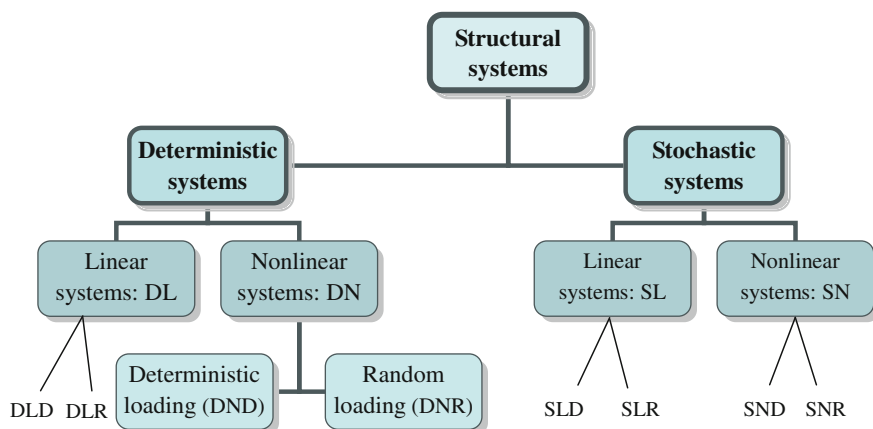


Fig. 2.2 Classification of the dynamic analysis problems of structural systems

The physical sources of wind loads on buildings are complex such that wind forces generated by windstorms can never be described or predicted perfectly (or deterministically). Generally speaking, wind loads on buildings are determined by the pressure distribution around the building structure and, in turn, are controlled by the pattern of air flow around the structure. Since wind is a turbulent flow, the actual pressure distribution around building structures displays a complex pattern of variation in space—in two or three dimensions—as well as variation with time. When the degree of disorder is sufficiently large, there is usually merit and economy in probabilistic rather than deterministic models. In this sense, wind-induced pressure and in turn wind-induced loadings could be treated as random field or random process. The term “field” indicates that the index space for random variables (pressures or loadings) is multidimensional. When a random field depends only on a single parameter, usually time, the random field is reduced into a random process. Random process is a time-indexed family of random variables $X(t)$, which could be observed at discrete points on a time axis.

Although some properties of building systems, e.g., damping, are stochastic in nature, building systems and their structural components are conveniently treated as deterministic in current design practice. Since wind-induced displacements of buildings are relatively small compared with the dimensions of the building structures and their components, it is reasonable to assume the building systems are linear. Therefore, the dynamic response analysis for wind-excited buildings could be reasonably treated as a DLR problem as categorized in Fig. 2.2. The DLR problem is a major subject in random vibration theory (Grandall 1963; Yang 1985).

Before moving to the probabilistic response of a system subject to a random excitation, it is necessary to study the time-domain method to predict the response of a linear system to an arbitrary but deterministic excitation. Due to the principle of superposition, a generic response $y(t)$ to a deterministic excitation $f(t)$ could be analytically obtained using the method of impulse response function (Meirovitch 1986)

$$y(t) = \int_0^t f(\tau)h(t - \tau)d\tau \quad (2.1)$$

which is known as the convolution integral and expressed the response as a superposition of the unit impulse response function, $h(t)$. It is noted that the impulse response function $h(t)$ in Eq. (2.1) is delayed, or shifted, by the time $t = \tau$. The convolution integral could be performed numerically and have been used in the investigation on the geometry of random vibrations (Der Kiureghian 2000) as well as the first excursion probabilities for linear systems by efficient importance sampling (Au and Beck 2001).

However, for a large-scale system, such as building systems, the method of impulse response functions becomes cumbersome. Moreover, the method is not applicable to nonlinear systems. The direct time-stepping method applying to linear or nonlinear equations of motion in three-dimensional (3D) space or in generalized modal space becomes popular with the increase of computer power. The time history analysis method has been implemented in the most of commercial Finite Element Method (FEM) software. The idea of time-stepping methods is to find the responses at next time step given the responses at previous step based upon the finite time differences. The step-by-step Newmark method (Newmark 1959; Clough and Penzien 1993; Chopra 2000) has been widely used in the numerical evaluation of dynamic responses for structural systems. As for any numerical computation procedure, the accuracy of this step-by-step method will depend on the length of the time increment Δt . In general, the ratio of the time increment to the vibration period T satisfying $\Delta t/T \leq 0.1$ will give reliable and stable numerical results (Chopra 2000).

The dynamic response of a structural system to a stochastic excitation is probabilistic in nature. The statistical characteristics of the random response process $Y(t)$ in second-order analysis can be effectively described by the correlation function $R_Y(t_1, t_2)$ at any two different time instant, which can be related to the correlation function of excitation $R_F(\tau_1, \tau_2)$ by

$$R_Y(t_1, t_2) = \int_0^{t_2} \int_0^{t_1} h(t_1 - \tau_1)h(t_2 - \tau_2)R_F(\tau_1, \tau_2)d\tau_1d\tau_2 \quad (2.2)$$

If both the input and the output are stationary stochastic processes, it is expedient to change the variables to $t_2 - t_1 = \tau, t_1 - \tau_1 = \theta_1, t_2 - \tau_2 = \theta_2$; as a result, the above correlation function of response becomes

$$R_Y(\tau) = \int_0^\infty \int_0^\infty h(\theta_1)h(\theta_2)R_F(\tau + \theta_1 - \theta_2)d\theta_1d\theta_2 \quad (2.3)$$

It is noted that the correlation function approach is based on the method of impulse response functions. Such an approach is generally applicable to the response analysis problems for certain or uncertain linear systems (Katafygiotis and Beck 1995), e.g., DLR, SLD, and SLR. The uncertainty characteristic could be reflected in the impulse response function, and the properties of random excitation could be described by $R_F(\tau_1, \tau_2)$. For instance, the integral on the right side of Eq. (2.3) can be easily calculated if the external excitation is a stationary delta-correlated process (white noise), which has the form of the following

$$R_F(\tau) = s\delta(\tau) \quad (2.4)$$

where $\delta(t)$ is a Dirac delta function and s is the intensity of the white noise. Such a correlation structure has been widely used for simplifying the random excitation process, which has a broader spectrum with wider frequency band covering the fundamental vibration frequency value of a structural system.

It is known that Gaussian processes are fully determined by their mathematical expectations and correlation functions and that a Gaussian process remains Gaussian after passing through a linear system. This makes a correlation description a convenient technique for the analysis of random vibrations of a deterministic linear system, e.g., DLR. Furthermore, when the time interval τ separating the two measuring points is zero, then the correlation function gives the mean square value for the random response process as

$$R_Y(\tau = 0) = E[Y(t)^2] \quad (2.5)$$

where $E(\cdot)$ denotes the operator of mathematical expectation.

For instance, a generalized wind force process, practically obtained from linear combination of base moments or torque processes for a tall building excited by atmospheric turbulence, could be reasonably assumed as a Gaussian process due to the central limit theorem, which claims that the probability distribution of the sum of M independent identically distribution random variables tends to become Gaussian when the number M increases. The key requirements for applying the central limit theorem is that there must be an aggregation of many weakly correlated random effects and that no single effect (or small subset of effects) accounts for a dominant fraction of the total variance. This is the typical condition for building aerodynamic that turbulent flows with varying sizes and intensities do not follow the surface of the building body, but detach from it leaving regions of separated flow and a wide trailing wake or vortex, causing very irregular and complicated wind-induced pressure pattern acting on the building. Although the computational modeling of building aerodynamics becomes possible with the remarkable advances in computational fluid dynamics (CFD) (Stathopoulos 1997, 2002; Murakami 2002), physical modeling of building aerodynamics by using scale models in atmospheric boundary layer (ABL) wind tunnels is still a more reliable and practical way to quantify wind effects on building (Cermak 2003). In the wind-tunnel-based

semi-analytical approach, the Gaussian assumption for wind excitations and building responses is widely adopted to predict wind-induced responses of tall buildings.

Problems of nonlinear systems in the theory of random vibrations, e.g., DNR, SND, and SNR in Fig. 2.2, are much more difficult than those of linear systems because the principle of superposition is generally not applicable. Numerical method, such as the time-stepping method, becomes one necessary approach to investigate nonlinear response behavior. As an attempt to develop the theory of nonlinear systems, the method of moment or cumulant functions, based on the solution of differential equations for moment or cumulant functions, is extensively used to analytically investigate the problems of nonlinear statistical dynamics (Di Paola and Muscolino 1990; Di Paola et al. 1992; Papadimitriou and Lutes 1996; Papadimitriou et al. 1999). The method of moment functions is more conveniently described in a state space by introducing state variables or vectors as $\mathbf{Z}$, which could be related to the response processes Y and their derivatives with respect to time as $\mathbf{Z} = \{Y, \dot{Y}\}$. Hence, various moment functions of the order r about state vectors with n components of z_j could be notated as

$$m_{jkl\dots}(t) = E \left[\underbrace{z_j(t)z_k(t)z_l(t)\dots}_r \right] \quad j, k, l = 1, 2, \dots, n \quad (2.6)$$

A moment vector functions consisting of moments of the same order r could be written as

$$\mathbf{m}_r(t) = \left\{ m_{\underbrace{11\dots}_r}(t), m_{\underbrace{11\dots 2}_r}(t), \dots \right\} \quad (2.7)$$

Differential equations for $\mathbf{m}_r(t)$ can be obtained either by directly averaging the system's equations of motion (Papadimitriou and Lutes 1996), or by using Itô stochastic differential equation and by the operation of averaging (Papadimitriou et al. 1999), or by applying the appropriate Fokker–Planck–Kolmogorov (FPK) equation (Guo 1999; Paola and Sofi 2002; Li and Chen 2006). As a result, an equation whose left side there is a time derivative with respect to the moment function is obtained as the following

$$\frac{d\mathbf{m}_r}{dt} = \varphi_r(\mathbf{m}_1, \mathbf{m}_2, \dots, \mathbf{m}_r, \dots) \quad (r = 1, 2, \dots) \quad (2.8)$$

The above hierarchy of equations represents a full description on the original stochastic dynamic problem. However, the system of equations is an infinite system. As a consequence, there arises the problem of the closure of the system in Eq. (2.8), i.e., of its reduction to a closed system of a finite number of equations. Suitable closure schemes have to be adopted. The simplest closure scheme is the

Gaussian closure (GC) (Ibrahim et al. 1985) in which the higher-order moments are expressed in terms of the first two moments, as if the response process were a Gaussian one. In the special case of purely additive white noise excitations, this procedure is analogous to another extensively used approximate method for predicting nonlinear responses, called stochastic linearization (SL) (Roberts and Spanos 1990). Though this procedure is simple and versatile, however, the Gaussian approximation is not satisfactory in the case of highly nonlinear systems and provides inaccurate results in terms of probability density function (PDF), so that the various response statistics needed for reliability analysis are not predicted adequately. Several other approximate techniques are available in the literature, such as the cumulant-neglect closure (Wu and Lin 1984), the stochastic averaging method (Roberts and Spanos 1986; Lin and Cai 2000), the dissipation energy balancing method (Cai and Lin 1988), the multi-Gaussian closure (Er 1998), and the method of weighted residuals (Liu and Davies 1990).

As presented in above, numerous publications on randomly excited nonlinear dynamic systems have been appeared in the literature and many procedures for obtaining the exact and approximate solutions have been proposed (Zhu and Huang 2001). Other than the approximate method, such as the method of moment functions, the one for obtaining the exact stationary solutions is still attractive. For randomly excited nonlinear dynamic systems, the exact solutions are usually very difficult to obtain. Only when the excitations are Gaussian white noises, the exact solution is possible to obtain by solving the FPK equation governing the transition probability density of a Markov process, together with initial and boundary conditions (Caughey 1971; Caughey and Ma 1982; Dimentberg 1982; Zhu 1990). Recently, Proppe (2003) presented exact stationary probability density functions for nonlinear systems under Poisson white noise excitation. Dimentberg (2005) obtained an explicit expression of a stationary joint probability density of displacements and velocities, which is served as an exact analytical solution to the corresponding FPK equation for random vibrations of a rotating shaft with nonlinear damping. Further developments in random vibration of nonlinear systems can certainly be expected in the future.

2.2.2 Analysis Method in Frequency Domain

The Fourier transform provides the classical method for decomposing a time history into its frequency components. The Fourier transform of a realization of response process $y(t)$ in Eq. (2.1) could be obtained as

$$Y(\omega) = H(\omega)F(\omega) \quad (2.9)$$

where $F(\omega)$ denotes the Fourier transform of the excitation; $H(\omega)$ denotes the frequency response function and is obtained as the Fourier transform of the unit impulse response.

Due to Wiener–Khinchine relationship, the power spectral density (PSD) function of the stationary random response process $Y(t)$ could be related to the correlation function $R_Y(\tau)$ as

$$S_Y(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} R_Y(\tau) e^{-i\omega\tau} d\tau \quad (2.10)$$

On the other hand, the correlation function of the loading process $F(t)$ can be expressed as the inverse Fourier transform as

$$R_F(\tau) = \int_{-\infty}^{\infty} S_F(\omega) e^{i\omega\tau} d\omega \quad (2.11)$$

Substituting Eq. (2.11) into Eq. (2.3), and then successively into (2.10), a simple algebraic expression relating the PSDs of the excitation and response could be obtained as (Meirovitch 1986)

$$S_Y(\omega) = |H(\omega)|^2 S_F(\omega) \quad (2.12)$$

which is the fundamental frequency-domain relationship regarded as the importance result of second-order analysis of the response of a linear dynamic system. It can be viewed as a frequency-domain form of Eq. (2.3), which gives the second moment function of the stationary response. A major difference is that finding the response second moment or autocorrelation function from Eq. (2.3) involves a double integral in the time domain, while Eq. (2.12) simply gives us the response PSD from a multiplication of functions in the frequency domain.

Wind loads on structures consist of the mean component and the fluctuating component. For sufficiently rigid structure, the design wind loading is mainly associated with the pressure on the surface of the structure being, as a first approximation, proportional to the square of the wind speed. Wind speeds in the atmospheric boundary layer have generally been treated as stationary random processes, which can be represented as

$$U(t) = \bar{U} + u(t) \quad (2.13)$$

where $\bar{U}$ = the mean component of the process; $u(t)$ = the fluctuating component with zero mean. The original fluctuating wind speed component $u(t)$ is essentially a summation of harmonic terms as

$$u(t) = \int_{-\infty}^{\infty} S_u(\omega) e^{i\omega t} d\omega \quad (2.14)$$

where $S_u(\omega)$ = the PSD of fluctuating wind speed. Equation (2.14) expresses the wind speed time history as the sum of many uncorrelated component functions of the form $e^{i\omega t} = \cos(\omega t) + i \sin(\omega t)$, each associated with a small interval $d\omega$ on the frequency axis and each multiplied by a spectral amplitude $S_u(\omega)d\omega$. The frequency decomposition representation of the stochastic process provides a foundation for simulation of wind speed time histories or any other stochastic process of interest (Shinozuka and Jan 1972; Deodatis 1996; Shinozuka and Deodatis 1997; Senthoooran et al. 2004; Carassale and Solari 2006).

The wind spectra information (Simiu and Scanlan 1996) is related to meteorological conditions and dependent on the geographical locations, the elevation levels, etc. The van der Hoven spectrum (van Der Hoven 1957) was the first comprehensive spectrum compiled that showed the components and characteristics of the wind in the atmospheric boundary layer in terms of frequency. In many wind engineering applications, it is normal to examine only the micrometeorological conditions (higher frequency region of the van der Hoven spectrum: 10^{-3} –1 Hz). There are many mathematical forms that have been used for wind spectra in meteorology and wind engineering. The most common and mathematically correct of these for the longitudinal velocity component (parallel to the mean wind direction) is the von Karman/Harris form (developed for laboratory turbulence by von Karman (1948) and adapted for wind engineering by Harris (1968)). The von Karman spectrum is commonly used in non-dimensional form as

$$\frac{f \cdot S_u(f)}{u_*^2} = \frac{24 \left(\frac{f L_u}{\bar{U}(10)} \right)}{\left[1 + 70.8 \left(\frac{f L_u}{\bar{U}(10)} \right)^2 \right]^{5/6}} \quad (2.15)$$

where $\bar{U}(10)$ = the mean wind speed at the height of 10 m; f = the frequency; L_u = a turbulence length scale, which is a measure of the average size of the turbulent eddies of the air flow; u_* = the friction velocity, which reflects the roughness of the ground surface and is related to the shear stress constitution in air flow at the surface. Based on the results of a study of about 70 spectra of the horizontal components of gustiness in storing wind, Davenport (1961) suggested the following expression for the spectrum of horizontal gustiness as

$$\frac{f \cdot S_u(f)}{u_*^2} = \frac{4 \left(\frac{f L_u}{\bar{U}(10)} \right)^2}{\left[1 + \left(\frac{f L_u}{\bar{U}(10)} \right)^2 \right]^{4/3}} \quad (2.16)$$

in which $L_u = 1200$ m. It is noted that both the von Karman and Davenport spectra do not reflect the dependence of spectra on height. The height-dependent wind spectrum was proposed by Kaimal (1972) as

$$\frac{f \cdot S_u(z, f)}{u_*^2} = \frac{200 \left(\frac{fz}{U(z)} \right)}{\left[1 + 50 \frac{fz}{U(z)} \right]^{5/3}} \quad (2.17)$$

where $\bar{U}(z)$ = the mean wind speed at the height of z m. The three sets of wind spectra are plotted in Fig. 2.3. The spectrum comparison shows that the Davenport spectrum may overestimate the longitudinal spectra of turbulence in the higher frequency range of tall building serviceability check (e.g., $f > 0.1$ Hz) by as much as 100–400 %, as shown in Fig. 2.3. In the lower frequency range (e.g., $f < 0.01$ Hz), the Davenport spectrum tends to underestimate the spectral value. Since the spectral distribution in the lower frequency range has little influence on tall building response, the Davenport spectrum is still widely used in wind engineering and design wind codes.

Davenport (1961, 1963, 1964) firstly developed a spectral approach to the wind-induced vibration of structures based on the random vibration theory. Other early contributions to the development of this spectral approach were made by Harris (1963) and Vickery (1966). The wind-tunnel-based semi-analytical random vibration approach in the frequency domain has become a common practice to predict wind loads and responses of tall buildings (Davenport and Isyumov 1967; Reinhold 1982; Tschanz and Davenport 1983; Vickery and Daly 1984; Cermak 1977, 2003). By means of either the high-frequency force balance (HFFB) or

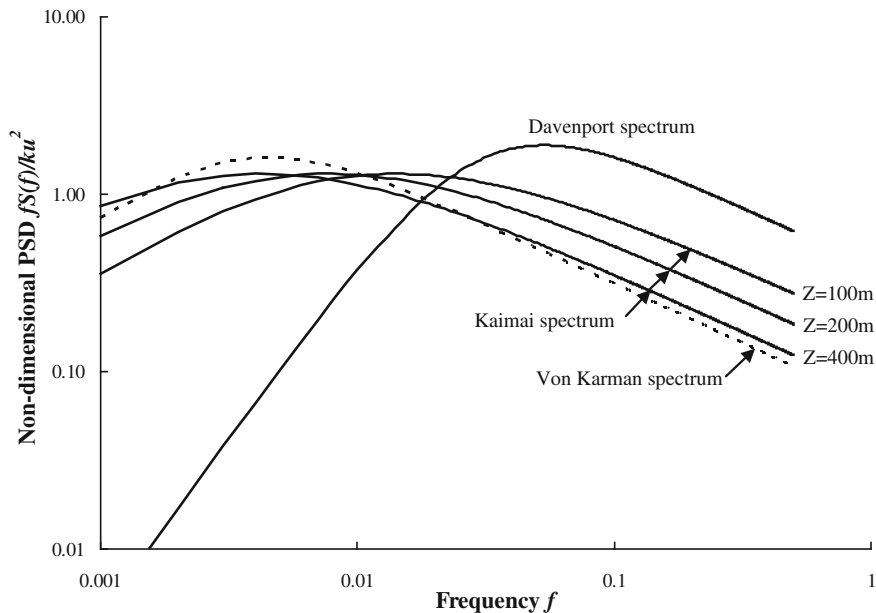


Fig. 2.3 Spectrum of horizontal gustiness

synchronous multipressure sensing system (SMPSS), aerodynamic wind loads can be estimated experimentally on a rigid scale model of the prototype. Based on the measured aerodynamic wind load, the dynamic response of a building system can then be analyzed using spectral approach in the frequency domain. Based on the wind tunnel measurements of wind loads, the dynamic analysis of wind-induced lateral-torsional motion of asymmetric buildings has been studied in the frequency domain by a number of researchers. Tallin and Ellingwood (1985) developed a method to relate alongwind, crosswind, and torsional forces acting on square isolated buildings to building accelerations. The effects on building motion of statistical correlations between components of wind forces and mechanical coupling of the modes due to eccentricities of mass and rigidity were examined and clarified. Based on the equations of motion derived for asymmetric coupled buildings, Kareem (1985) presented a random-vibration-based procedure for estimating the wind-induced lateral-torsional response of tall buildings and examined the effects due to eccentricities in centers of mass and stiffness on the coupled motion of the building. For structurally asymmetric buildings having closely spaced translational and torsional natural frequencies, the statistical correlation between the crosswind and torsional motions has been found to play a significant role in determining the wind-induced response (Islam et al. 1992).

Although the HFFB technique has been used widely for quantifying generalized wind forces on buildings, it is primarily suitable for buildings with 1D mode shape in each principal direction. The limitations inherent in the HFFB technique have been discussed and highlighted especially on its applicability to buildings with three-dimensional (3D) mode shapes (Yip and Flay 1995). To overcome these limitations, much effort has been made to refine force balance data analysis techniques to take into account the coupled motion of tall buildings with 3D mode shapes (e.g., Yip and Flay 1995; Holmes et al. 2003; Flay et al. 2003). Chen and Kareem (2005a) presented a new systematic framework for dynamic analyses of 3D coupled wind-induced responses of buildings and the determination of equivalent static wind loads (ESWLs) by representing the building response into the mean, background, and resonant components. Both the cross-correlation of wind loads acting in different directions and the intermodal coupling of modal response components were taken into consideration in the analysis and modeling of ESWLs. Utilizing a representative tall building with 3D mode shapes, Chen and Kareem (2005b) further examined the dynamic wind effects on asymmetric buildings with closely spaced natural frequencies and highlighted the significance of cross-correlation of wind loads and the intermodal coupling of modal responses on the accurate prediction of coupled building responses.

Due to the stationarity of wind speed fluctuation, wind loads and responses are correspondingly stationary and are sufficiently described by PSD functions in the frequency domain. However, the flow field created by some extreme wind events, e.g., thunderstorm, varies significantly from the traditional ABL stationary and homogeneous wind flows (Letchford et al. 2001). Characterization and modeling of transient non-stationary winds and their effects on structures have been received increasing attention in recent years (Holmes and Oliver 2000; Letchford et al. 2001;

Chen and Letchford 2004a, b; Holmes et al. 2005; Huang et al. 2015). Based on the evolutionary PSD (EPSD), Chen (2007) presented a frequency-domain analysis framework for quantifying alongwind tall building response to transient winds. In that study, the transient winds and associated wind loads in buildings are modeled as the sum of deterministic time-varying mean and evolutionary random fluctuating components, as

$$U'(t) = \bar{U}(t) + u'(t) \quad (2.18)$$

The evolutionary PSD of $u'(t)$ could be related to $S_u(\omega)$ as

$$S_{u'}(\omega, t) = |a(t, \omega)|^2 S_u(\omega) \quad (2.19)$$

where $a(t, \omega)$ = a complex-valued deterministic modulation function of both t and ω . The extension of spectral representation to non-stationary random process is due to Priestley (1965). Given the EPSD information, the time history is also possible to be simulated using the computationally efficient cosine series formula as demonstrated by Liang et al. 2007. That paper presents a rigorous derivation of a previously known cosine series formula (Shinozuka and Jan 1972) for simulation of 1D, univariate, non-stationary stochastic processes integrating Priestly's evolutionary spectral representation theory.

2.3 Structural Design Optimization

2.3.1 Classical Optimization Method

Since the 1960s, various methods and formulations for optimization of problems in many diverse fields, such as physics, structural and mechanical engineering, operation research, economics and finances and others, have been developed and discussed in the literature. Although optimization applications in each scientific discipline are quite different, the fundamentally mathematical concept is essentially the same. In mathematics, the term optimization, or mathematical programming (MP), refers to the study of problems in which one seeks to minimize or maximize a real function by systematically choosing the values of real or integer variables from within an allowed set. This problem can be represented as follows:

Given $f(\mathbf{x}) = f(x_1, x_2, \dots, x_n) \quad \mathbf{x} \in \mathbf{R}^n$

Find $\mathbf{x}^*$

Such that $f(\mathbf{x}^*) \leq f(\mathbf{x})$ or $f(\mathbf{x}^*) \geq f(\mathbf{x})$ for all $\mathbf{x}$

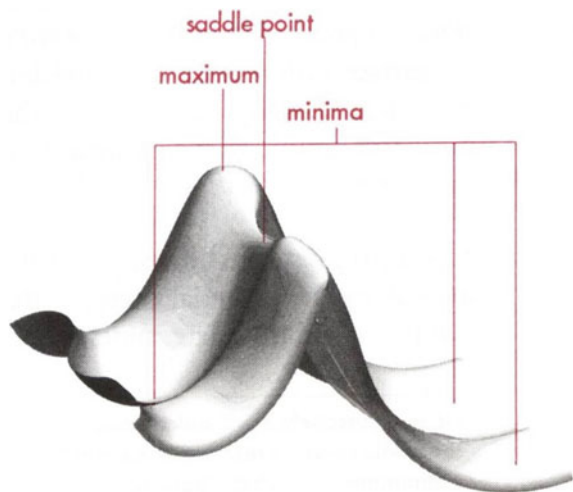
Problems formulated using this technique in the fields of physics may refer to the technique as total-energy minimization, speaking of the value of the function f as representing the energy of the physical system being modeled (Payne et al. 1992).

The iterative minimization technique for total-energy calculation developed by Payne et al. became one of the standard methods in computational molecular dynamics (CMD).

For twice-differentiable functions, unconstrained problems can be solved by finding the points where the gradient of the objective function is zero (i.e., the stationary points) and using the Hessian matrix to classify the type of each stationary point. If the Hessian is positive definite, the function value at the point is a local minimum, if negative definite, a local maximum, and if indefinite, it is some kind of saddle point, as shown in Fig. 2.4. However, existence of derivatives is not always assumed and many methods were devised for specific situations.

The basic classes of methods, based on smoothness of the objective function, are as follows: zero-order (derivative-free) methods; first-order methods; and second-order methods. The golden section and polynomial fitting techniques are commonly used as zero-order methods for one-dimensional minimization. For n -dimensional minimization, the conjugate direction method (Powell 1964) is an efficient method for finding the minimum of a function of several variables without calculating derivatives. As a first-order method, the steepest descent method, which searches for the optimum along the gradient vector, is usually more efficient than zero-order method. Although the steepest descent method is elegant and simple, the convergence rate of the method for functions form an elongated design space is very poor. Fletcher and Reeves (1964) improved the steepest descent method and convert it to the conjugate gradient method, which takes account the trend information of the previous search direction while including the current gradient information. Newton's methods are second-order methods which require second-order derivative information. Newton–Raphson method requires the gradient vector and the Hessian matrix information, which may not necessary, always be available. Quasi-Newton or variable-metric method (Kirsch 1993) uses the

Fig. 2.4 Stationary points of a smooth function



approximate forms of the Hessian matrix and its inverse. Davidson–Fletcher–Powell (DFP) update procedure builds up approximate inverse of the Hessian matrix using only the first derivatives (Fletcher and Powell 1963). Broyden–Fletcher–Goldfarb–Shanno (BFGS) update procedure builds up approximate Hessian matrix rather than its inverse (Kirsch 1993).

Constrained problems can often be transformed into unconstrained problems with the help of Lagrange multipliers λ . For a general minimization problem of $f(\mathbf{x})$ subject to $(j = 1, 2, \dots, m)$ inequality constraints $g_j(\mathbf{x}) \leq 0$ and $(k = 1, 2, \dots, l)$ equality constraints $h_k(\mathbf{x}) = 0$, the unconstrained Lagrangian function could be defined as

$$L(\mathbf{x}, \boldsymbol{\lambda}) = f(\mathbf{x}) + \sum_{j=1}^m \lambda_j g_j(\mathbf{x}) + \sum_{k=1}^l \lambda_{m+k} h_k(\mathbf{x}) \quad (2.20)$$

By differentiating the Lagrangian function with respect to each design variable x_i setting the derivatives to zero, the Karush–Kuhn–Tucker (KKT) necessary conditions are obtained as (Luo et al. 1996)

$$\frac{\partial f(\mathbf{x})}{\partial x_i} + \sum_{j=1}^m \lambda_j \frac{\partial g_j(\mathbf{x})}{\partial x_i} + \sum_{k=1}^l \lambda_{m+k} \frac{\partial h_k(\mathbf{x})}{\partial x_i} = 0 \quad (i = 1, 2, \dots, n) \quad (2.21)$$

The KKT conditions are generally a nonlinear system of equations and may not be easy to solve the system analytically. In practice, the numerical analysis technique (e.g., the Gauss–Seidel iterative technique) is always employed to solve the nonlinear system of equations (Householder 2006).

2.3.2 Formulations for Structural Optimization

2.3.2.1 Mathematical Programming Approach

In the structural optimization literature, two basically different formulation approaches for optimum design have been developed since 1960s, named as mathematical programming (MP) approach and optimality criteria (OC) approach. Schmit (1960) was the first to offer a comprehensive idea of structural synthesis of integrating finite element structural analysis and MP techniques to automate optimal elastic design. MP approach is attractive due to its generality and rigorous theoretical basis. All classical optimization method discussed in previous section could be applied in the MP approach (Kirsch 1993). The OC approach is discussed in the next section.

Traditionally, the finite element structural analysis and design optimization are performed sequentially in the MP approach. That is to say, only the design variables are treated in the formulation of optimization, while the state variables of systems,

such as displacements, accelerations, strains, stresses and internal forces, are evaluated by the finite element structural analysis and treated as implicit functions of the design variables. Therefore, the design sensitivity analysis (Arora 1995; Choi and Kim 2005, Huang and Chan 2007) is needed to perform before optimization for obtaining the sensitivity information of state variables with respect to design variables, which will be demanded by the optimization algorithms. The finite difference methods have been used to calculate the gradients since they are easy to implement using FEM program. However, the finite difference methods have accuracy problems, i.e., the so-called step-size dilemma (Haftka and Gurdal 1992). Given the global stiffness matrix of a structural system, the direct differentiation method or the adjoint variable method can be employed to analytically determine the design sensitivity information (Choi and Kim 2005). Based on the principle of virtual work, Huang and Chan (2007) presented a design sensitivity analysis method for the displacement and eigenvalue sensitivities without the need for the global stiffness matrix information of a building system. It is worth to note that the latest development on efficient structural approximation and reanalysis methods (Kirsch 2000, 2003) could directly be used to quantify various desired sensitivity information within the optimization framework.

When the state variables are also treated in the formulation of optimization by formulating the system governing equilibrium equations as equality constraints, the MP approach becomes a general class of formulations known as mathematical programming with equilibrium constraints (MPECs) with simultaneous analysis and design for the optimization problems (Luo et al. 1996; Arora and Wang 2005). The equilibrium constraints could be expressed in terms of functional based on the variational principle in physics to model the equilibrium phenomenon in physics, mechanics, engineering, and other applications. Although the MPECs provide a general approach to formulate various optimization problems in wide disciplines, the MPEC formulation may be posed as a non-convex and non-differentiable optimization problem, which is computationally difficult to solve (Luo et al. 1996; Ferris and Pang 1997). Optimization algorithms have to be explored and devised for different problems posed by MPECs (Hilding et al. 1999; Ferris and Tin-Loi 2001; Evgrafov and Patriksson 2003).

2.3.2.2 Optimality Criteria Approach

As an alternative approach, the optimality criteria (OC) approach was emerged and developed in the late 1960s and early 1970s. Prager et al. (1968) presented the OC method for optimal structure design in analytical forms, while Venkayya et al. (1968) further developed the OC approach using numerical analysis technique for practical large-scale aerospace structures. Based on the way to obtain the optimality criteria, the OC method can be classified into the intuitive OC method and the rigorously derived OC method.

An early example of an intuitive OC approach is the fully stressed design (FSD) method, which is applicable to structures subject only to stress and sizing

constraints. Although the FSD method is effective as a practical design tool, the FSD method only can achieve true optimum for statically determinate structures but not for statically indeterminate structures. For these indeterminate structures, the minimum weight design may not always be fully stressed (Schmit 1960; Razani 1965; Panagiotis et al. 2002). Panagiotis et al. 2002 somewhat overcame the drawback of the FSD method by introducing a strain energy criterion. Pedersen et al. (2005) develop an optimality criterion in eigenvalue problems for shape optimization of plate. The work extends the application area of OC method from the sizing optimization to shape optimization. The rigorously derived OC method was introduced by Cheng and Truman (1983), Chan (1992), Chan et al. (1995) to civil engineering by solving the optimal design problems of building structures. The rigorously derived OC method mainly consists of two complementary phases. The first phase concerns the derivation of a set of necessary conditions (KKT conditions) that the optimal design must satisfy. The second phase involves the application of a numerical recursive algorithm to vary design variables in order to satisfy the KKT conditions for the final optimum design solution. Unlike the MP method, the OC method is more computationally efficient and usually provided a local optimum design with a few analysis and design cycles.

Chan (1992) developed a virtual work formulation to implement the OC method for the design optimization of tall steel buildings. In that study, the OC method has been formulated based on the principle of virtual work, by which the displacement of a tall building could be fairly approximated. Since a tall building, in a global sense, is a statically determinate cantilever structure, internal element force distributions in statically indeterminate tall building structures are relatively quite insensitive to moderate changes in the element sizing variables. Therefore, the virtual work formulation represents a good approximation of the behavior of the drift constraints and the explicitly expressed drift formulation generally leads to smooth and rapid solution convergence. Another advantage of the virtual work formulation OC approach is easy to apply for the optimum design of practical tall building structures, which are generally modeled by the commercial FEM software. The information of internal element forces demanded by the virtual work formulation is readily available from the FEM model of a tall building structure.

Much research effort has been continually devoted to the OC method and its application due to its high efficiency and quick convergence behavior (Chan 1997, 1998, 2001; Chan and Zou 2004; Chan and Chui 2006; Chan et al. 2007). Based on the virtual work formulation (Chan 1992; Huang and Chan 2007), the OC method was employed to solve the optimization problem of large-scale tall buildings subject to multiple equivalent static wind drift and wind-induced vibration constraints (Chan 2001). Latter, the OC method was extended to solve the optimal performance-based seismic design problems of reinforced concrete buildings (Chan and Zou 2004; Zou and Chan 2005). Chan et al. (2007) presented an integration of an aerodynamic wind load analysis and an optimal element resizing technique based on the OC method for lateral drift design of tall steel buildings.

While quick convergence can be normally achieved using the OC technique, it cannot always assure that the global optimum can be found. Advances in recent

research have resulted in a hybrid methodology, namely the OC–GA method, which incorporates genetic algorithms (GAs) into the gradient-based OC technique. The evolutionary GAs are in general more robust and present a better global behavior than the OC. However, GAs alone may suffer from a slow rate of convergence toward the global optimum. In order to benefit from the advantages of both OC and GAs, a hybrid combination of both methodologies has been developed and thus the so-called OCGA method as an attempt to improve the robustness as well as the computational efficiency of the optimization procedure for structural topology and element sizing design of tall buildings (Chan and Wong 2007).

2.3.3 Dynamic Response Optimization

2.3.3.1 Optimization of Structures Under Transient Loads

In this book, the optimization of structures subject to transient loads in a time domain is referred as dynamic response optimization (Kang et al. 2006). Unlike optimizations of structures under static loads, the dynamic response optimization would involve numerous time-dependent constraints. The treatment of time-dependent constraints becomes one of the key topics in the dynamic response optimization. The other two important topics are sensitivity analysis and approximation technique. While the time-dependent constraint treatment is unique for the dynamic response optimization, the sensitivity analysis and approximation are the equally important issue for optimization of structures under static loads, as discussed in previous section.

Much research has been focused on the treatment of numerous time-dependent constraints imposed in the dynamic response optimization problems (Hsieh and Arora 1984, 1985, 1986). Haug and Arora (1979) converted numerous time-dependent constraints into one single equivalent functional constraint by integrating original pointwise constraints over the entire time interval of interest. Hsieh and Arora (1984) presented the worst-case design formulation as the constraint is imposed on the worst response. In this treatment of time-dependent constraints, all pointwise time-dependent constraints are replaced by several constraints that are imposed at the multiple local maximum response points, or just simply by one single constraint that is imposed at the global maximum response point. A hybrid approach, so-called subdomain functional formulation, for treating time-dependent constraints has been discussed and evaluated by Tseng and Arora (1989). The hybrid treatment of pointwise performance constraints for the dynamic response optimization problem is to transform pointwise constraints into several equivalent functional constraints by dividing the entire time domain into several subdomains. Using the worst-case design formulation, Zou and Chan (2005) developed an effective numerical technique for member sizing optimization of concrete building frameworks subject to seismic drift design constraints under response spectrum and time history loadings.

The direct differentiation method and the adjoint variable method as discussed for static structural design optimization are still applicable to the sensitivity analysis for the dynamic response optimization problem. However, in dynamic response optimization, the system behavior is no longer only related to stiffness, mass and damping would also play important roles in the governing equations of motion. The direct differentiation method has been applied to equations of motion and solved by numerical time-stepping methods (Newmark 1959; Greene Haftka 1991; Sousa et al. 1997). As an efficient dynamic analysis method, the mode superposition method has also been employed in sensitivity calculations in linear transient structural analysis (Choi et al. 1983; Wang 1991; Lee 1999). The adjoint variable method is introduced by defining an augmented response function to calculate sensitivity of a dynamic response (Arora and Cardoso 1989; Lee 1999; Kocer and Arora 2002).

Since the dynamic analysis in time domain is expensive in large-scale structural optimization, much effort to approximate the dynamic response of structures has been made. Approximation can be classified into three categories as global approximation, local approximation, and combined approximation (CA). Response surface method (RSM) is the representative one of global approximation (Montgomery 2001; Kim and Choi 2007). RSM builds an approximated response model from data acquired at various design points by using statistical approaches. The built approximation model is usually an explicit function of the design variables and is easy to handle sensitivity analysis (Kurtaran et al. 2002). Unlike the global approximations obtained by analyzing the structure at a number of design points, local approximations are based on the information calculated at a single design point. Jensen and Sepulveda (1998) applied the local approximation by introducing intermediate variables to calculate design sensitivity of transient response of structures.

The combined approximation (CA) method, which attempts to give global qualities to local approximations, has been developed recently and applied for nonlinear and dynamic reanalysis, as well as dynamic sensitivities (Kirsch 2003; Kirsch et al. 2007; Kirsch and Bogomolni 2007). The CA approach is based on the integration of several concepts and methods in numerical analysis, including series expansion, reduced basis, matrix factorization, and Gram–Schmidt orthogonalizations. The advantage is that efficient local approximations and accurate global approximations are combined to achieve an effective solution procedure. Bogomolni et al. (2006) demonstrated that dynamic sensitivities can be efficiently calculated using the CA method and finite difference methods for the discrete linear systems subjected to dynamic loading.

The dynamic response optimization discussed so far solves differential equations to calculate design sensitivity, which is relatively computationally expensive compared to the static response sensitivity calculation. In order to exploit well-established static response optimization techniques in dynamic response, Kang et al. (2001) introduced the equivalent static load (ESL) method in dynamic response optimization by transforming dynamic loads into static loads based on displacement equivalence. Choi and Park (2002) developed a quasi-static

optimization method for the structure under dynamic loads. The sets of ESL are generated at all time intervals and utilized as a multiple loading condition in the optimization process. Actually, the transformation of dynamic loads into equivalent static loads (ESLs) has been well developed in civil engineering for analysis and design of building and bridge structures under earthquake or typhoon loads (Chopra 2000; Davenport 1967, 1995; Holmes 2002; Chen and Kareem 2005a; Chan et al. 2007). Due to its simplicity and efficiency, current design codes adopt the ESLs approach to treat the intrinsic random and dynamic loads, e.g., the response spectrum analysis method for earthquake loads, the gust loading factor method for wind loads. The dynamic optimal seismic or wind-resistant designs using ESL approach or any other are discussed in next two sections, respectively.

2.3.3.2 Seismic Structural Optimization

Much research on the optimal seismic design has been developed even before the establishment of performance-based design concepts, i.e., multiple performance levels and design objectives for buildings under earthquake loading. Bhatti and Pister (1981) considered an optimization problem for the earthquake-resistant design of structural systems that incorporated two levels of design constraints: The first level is for frequent minor earthquake for which the structure is remained in the linear range; the second design level is for less frequent, major earthquakes, for which inelastic deformation and limited damage are allowed. Pantelides (1990) studied the optimum design of active control seismic structures by optimizing the structural members of frames equipped with an active control system. Optimization of reinforcement concrete (RC) buildings against earthquake loads has been presented by Truman and Cheng (1997).

With increasing popularity of performance-based design approach, many researchers and engineers have proposed various methodologies, which are aimed to incorporate performance-based concepts and criteria into the optimization framework. Foley (2002) presented an overall literature review on optimal performance-based design approach. Ganzerli et al. (2000) addressed the optimal performance-based design of seismic structures. In that study, performance-based design concepts and pushover analysis were incorporated into the optimization method for reinforced concrete structures. Chan CM and Zou XK (2004) presented an effective optimization technique for the elastic and inelastic drift performance design of reinforced concrete buildings under response spectrum loading and pushover loading. The equivalent static load (ESL) method has been used for seismic performance-based design optimization (Gong et al. 2005; Xu et al. 2006). A performance-based design sensitivity analysis procedure for inelastic steel moment frameworks under equivalent static earthquake loading (ESEL) was developed by Gong et al. 2005. Such a sensitivity analysis procedure was later integrated into an optimal performance-based seismic design framework by Xu et al. 2006.

2.3.3.3 Wind-Resistant Structural Optimization

For multistory buildings, the equivalent static wind loads (ESWLs) corresponding to the specific incident wind angle are generally expressed in terms of the along-wind, crosswind, and torsional directions; and each directional ESWL consists of the mean, background, and resonant components. For normal low-rise buildings, i.e., less than 100 m, wind-induced structural loads are dominated by static mean components and quasi-static background components. The wind-induced resonant effects on low-rise buildings are small and negligible such that wind loads can be considered as constant static design loads. The well-developed static response optimization technique can be directly employed in wind-resistant structural optimization (Chan 1992, 1997, 1998). However, for dynamically sensitive tall buildings, wind-induced resonant effects become critical. In order to make an accurate prediction of the wind-induced structural loads on the building, it is necessary that the ESWLs be always updated whenever there exists a significant change in the structural properties of the building.

Recently, a promising and effective integrated wind-induced response analysis and serviceability optimization approach is developed to achieve an optimal design solution of wind-sensitive complex tall buildings (Chan and Chui 2006; Chan et al. 2007). Chan and Chui (2006) presented an occupant comfort wind-induced acceleration design optimization technique for wind-excited symmetric tall steel buildings based on eigenvalue sensitivity analysis using Rayleigh method. Chan et al. (2007) developed an integrated optimal design framework that couples together an aerodynamic wind load updating analysis procedure and a stiffness optimization method for symmetric tall building structures subject to serviceability drift design constraints. When given with the aerodynamic wind load spectra, ESWLs on tall buildings can be updated instantaneously for any change in the member sizes of a structural system during the optimization process (Chan et al. 2007). Although these studies represent a major advance in the use of structural optimization techniques for wind-induced serviceability design of tall buildings, it is necessary to extend the stiffness design optimization technique for general asymmetric tall buildings subject to multiple wind-induced drift and acceleration constraints together.

2.4 Reliability-Based Design Optimization

2.4.1 Reliability Analysis Method

The structural systems consisting of interacting and functionally interconnected elements are designed in order to function or operate in a normal condition without failure or in an extreme condition with a small probability of failure. In reliability theory, failure is any event, which is defined as the violation of a prescribed limit

state by the state of an object (a system or an element). The limit-state concept leads to limit-state design philosophy, which has been adopted in the current machine or building design codes. In reality, the vast uncertainties always exist in systems or elements and their external environment or loadings. The basic uncertain quantities could be described as time-invariant random variables. Consequently, the failure probability is time-independent and the probability of no failure is time-invariant reliability. If the basic uncertain quantities are time-dependent and modeled as random processes, the failure probability is also a function of time. The time-variant probability of failure is defined as the first out-crossing of the state process through the limit state during a given time interval. The probability of failure-free operation of an object during the given time interval could be called as time-variant reliability.

2.4.1.1 Time-Invariant Reliability

The time-invariant probability of failure could be defined as

$$P_f = \int_{g(\mathbf{x}) \leq 0} f(\mathbf{x}) d\mathbf{x} \quad \mathbf{x} \in \mathbf{R}^n \quad (2.22)$$

where $\mathbf{x}$ is a vector of random variables that represents uncertainty quantities of systems and loadings; $f(\mathbf{x})$ is the joint probability density function of the random vector $\mathbf{x}$; $g(\mathbf{x}) \leq 0$ defines the failure domain D_f . A great deal of effort in the past several decades has been devoted to developing efficient algorithms for computing the multidimensional integrals of the form in Eq. (2.22) analytically or numerically (Der Kiureghian 1996; Rackwitz 2001).

The first-order reliability method (FORM) is the earliest and important approximate approach to compute reliability (Shinozuka 1983). In order to take advantage of the rotational symmetry of the standard normal space, the FORM analysis require a transformation of limit-state function $g(\mathbf{x})$ from the original parameter space into the new space of the uncorrelated standard normal variates, as $G(\mathbf{u}) = g[\mathbf{x}(\mathbf{u})]$. The transformation task is usually performed by using the so-called Rosenblatt-transformation (Rosenblatt 1952), which is based on the representation of a multidimensional distribution function by a product of conditional distributions. The simple result of the FORM method then is given by

$$P_f \approx \Phi(-\beta) \quad (2.23)$$

where $\Phi(\cdot)$ = the Gaussian cumulative distribution function; $\beta = \|\mathbf{u}^*\|$, known as the reliability index, is the distance from the origin to the most probable failure point (MPFP) or “design point,” which is a point located on the limit-state surface with minimum distance from the origin, and has the highest probability density among all failure points in the standard normal space. This point $\mathbf{u}^*$ is the solution of the following constrained minimization problem as

Minimize: $\|u\| = \mathbf{u}^T \mathbf{u}$

Subject to: $G(\mathbf{u}) = 0$

Then, the main computational task of the FORM is the location of “design point” by an optimization method (Liu and Der kiureghian 1990). One suitable method is the Lagrange multiplier method by defining an unconstrained Lagrangian function of the form in Eq. (2.20).

In FORM, the limit-state surface is replaced by the tangent hyperplane at $\mathbf{u}^*$. It is natural to attempt a second-order expansion of the limit-state surface based on the assumption that a second-order expansion of the limit-state surface is better than a first-order expansion. Based on the asymptotic analysis, Breitung (1984) developed a second-order reliability method (SORM) to approximate the failure probability in terms of β and the principal curvatures κ_i , $i = 1, \dots, n - 1$ of the parabolic failure surface at $\mathbf{u}^*$ as

$$P_f \approx \Phi(-\beta) \prod_{i=1}^{n-1} (1 - \beta \kappa_i)^{-1/2} \quad (2.24)$$

Der Kiureghian et al. (1987) suggest a more computationally efficient “point-fitted” SORM procedure compared to the original “curvature-fitted” SORM. The exact result for the probability content of the failure domain obtained from the full second-order Taylor expansion of the failure function at the “design point” was presented by Tvedt (1990). Instead of working in the standard normal space, reliability approximation was also achieved by maximizing the log likelihood $l(\mathbf{x}) = \ln f(\mathbf{x})$ in the original variable space (Breitung 1991; Papadimitriou et al. 1996). The log likelihood maximization approach is built on the asymptotic analysis to Laplace-type integrals (Bleistein and Handelsman 1986).

2.4.1.2 Time-Variant Reliability

If the performance state of an object is a random process $\mathbf{Z}(t)$, the time-variant failure event is defined as the first out-crossing of $\mathbf{Z}(t)$ through the limit-state surface $g(\mathbf{x}, \mathbf{Z}) = 0$. Denoting the number of excursions during time interval $[0, t]$ as $N(t)$, the time-variant probability of failure-free operation could be written in terms of the conditioned probability for given $\mathbf{x}$ based on the total probability theorem as

$$P_s(t) = \int P\{N(t) = 0 | \mathbf{x}\} f(\mathbf{x}) d\mathbf{x} \quad \mathbf{x} \in \mathbf{R}^n \quad (2.25)$$

where $P\{N(t) = 0 | \mathbf{x}\}$ is the conditional probability of no out-crossing during the time interval $[0, t]$.

On the other hand, the failure-free operation means that within the time interval the state of the object is always within the safe domain D_s defined by $g(\mathbf{x}, \mathbf{Z}) > 0$. Therefore, the time-variant reliability could be rewritten as

$$P_s(t) = \int P\{\mathbf{Z}(t) \in D_s | \mathbf{x}\} f(\mathbf{x}) d\mathbf{x} \quad \mathbf{x} \in \mathbf{R}^n \quad (2.26)$$

where $P\{\mathbf{Z}(t) \in D_s | \mathbf{x}\}$ is a conditional probability of failure-free operation during the time interval $[0, t]$.

One of most appropriate model for the description of failure events of highly reliable systems is the Poisson distribution, i.e., $P_k(t) = \frac{(vt)^k}{k!} \exp(-vt)$. Poisson distribution is a discrete probability distribution that expresses the probability of k failure events occurring in a fixed period of time t if these events occur with a known average rate v and independently of the time since the last event. Therefore, the conditional probability of failure-free operation is given by letting $k = 0$ in the expression of Poisson distribution as

$$P\{N(t) = 0 | \mathbf{x}\} = \exp(-vt) \quad (2.27)$$

where v = mean failure rate, which means that the expected number of failure events per unit time. The occurrence of failure events means the random state process $\mathbf{Z}(t)$ is crossing the limit state, i.e., simply defined by double barrier of level b in the context of the classical first-passage problem (Vanmarcke 1975). The first result of mean failure (out-crossing) rate for a random scalar process was given by Rice (1945) as

$$v_b = \int_{-\infty}^{\infty} |\dot{z}| f_{Z, \dot{Z}}(b, \dot{z}) d\dot{z} \quad (2.28)$$

where $f_{Z, \dot{Z}}(b, \dot{z})$ is the joint probability density function of the scalar process $Z(t)$ and its derivative process $\dot{Z}(t)$.

For a nonlinear limit-state surface, a generalization of the Rice's formula for a vector stochastic process is available that requires integration over the surface (Belyaev 1968). Based on the generalized Rice's formula, the joint first-passage probability of a vector random process can be estimated (Song and Der Kiureghian 2006). The level crossings of a one-dimensional stochastic process has also been generalized to an n -dimensional field $Z(\mathbf{t})$, $\mathbf{t} \in \mathbf{R}^n$ (Robert and Hasofer 1976; Piterbarg 1996). The high-level crossing problem naturally leads to the asymptotic extremal distributions for random process or field (Piterbarg 1996; Muscolino and Palmeri 2005). The theorem on the asymptotic extremal distributions for homogeneous Gaussian fields has been applied to study the probability distribution of surface gravity waves in the open deep ocean (Socquet-Juglard et al. 2005).

A Markov process is a stochastic process in which the probability distribution of the current state is conditionally independent of the path of past states. Let the evolution of the state process $\mathbf{Z}(t)$ be a Markov process. Then, the conditional probability of failure-free operation could be rewritten in terms of transition probability density $f_{\mathbf{Z}}(\mathbf{z}, t | \mathbf{Z}(t_0) = \mathbf{z}_0, \mathbf{x})$ for the realization of process $\mathbf{Z}(t)$ encompassed by the limit-state surface as

$$P\{\mathbf{Z}(t) \in D_s | \mathbf{x}\} = \int_{D_s} f_{\mathbf{Z}}(\mathbf{z}, t | \mathbf{Z}(t_0) = \mathbf{z}_0, \mathbf{x}) d\mathbf{z} \quad (2.29)$$

The transition probability density $f_{\mathbf{Z}}$ is governed by the FPK equation as (Lin and Cai 1995)

$$\frac{\partial f_{\mathbf{Z}}}{\partial t} = - \sum_{j=1}^n \frac{\partial}{\partial z_j} (a_j f_{\mathbf{Z}}) + \frac{1}{2} \sum_{j=1}^n \sum_{k=1}^n \frac{\partial^2}{\partial z_j \partial z_k} (b_{jk} f_{\mathbf{Z}}) \quad (2.30)$$

where a_j = the drift coefficient; b_{jk} = the diffusion coefficient. These coefficients are determined from the equations of motion for a structural dynamic system. Very recently, the FPK equation method was successfully applied to obtain the probability density function of the nonlinear response of an articulated leg platform by Kumar and Datta (2008).

2.4.1.3 Simulation Method

The Monte Carlo simulation method (Fishman 1996) provides approximate solution to a variety of mathematically formulated problem by performing statistical sampling experiments on a computer. The method, remarkably, applies to problems with absolutely no probabilistic content and to those with inherent probabilistic structure, e.g., reliability analysis. Although the standard Monte Carlo simulation (MCS) method offers a very robust solution to reliability problem, the computational requirement by MCS is quite prohibitively high when evaluating small failure probabilities (Proppe et al. 2003). Therefore, importance sampling method was developed as an efficient simulation technique by sampling more frequently from inside the failure set (Harbitz 1986). The first-passage failure probability (time-variant reliability problem) of linear systems subjected to Gaussian white noise excitation was effectively estimated using importance sampling method (Au and Beck 2001). In that paper, a suitable importance sampling density was developed by studying the elementary failure region and elementary failure probability based on the impulse response function.

Other class of efficient simulation methods is based on the geometry properties of limit-state functions and failure domains in the standard normal space. Der Kiureghian (2000) offered a new outlook to solve reliability problem for linear systems subject to Gaussian excitation by investigating the geometry of random

vibrations. Katafygiotis and Cheung (2004) developed a wedge simulation method to calculating the reliability of linear dynamic systems through studying the interaction between the different linear elementary failure domains. Similarly, the domain decomposition method based on exploiting the special structure of the failure domain was proposed as a more efficient approach for estimating the first-passage probabilities of linear systems by Katafygiotis and Cheung (2006).

2.4.2 Reliability Index Optimization Approach

In engineering practice, reliability analysis of mechanical and building systems is served as a key procedure in the design optimization of those systems with reliability constraints. Today, it is widely recognized that the design optimization method as a more rational and effective design tool should also account for the stochastic nature of engineering systems (Frangopol and Maute 2003; Schueller 2007). Frangopol and Maute (2003) presented a literature review for reliability-based design optimization (RBDO) of civil and aerospace structures. In that paper, reliability index approach is formulated as

Minimize: $f(\mathbf{d})$

Subject to: $\beta_j(\mathbf{x}, \mathbf{d}) - \beta_j^L \geq 0 \quad j = 1, \dots, n_\beta$

where $\mathbf{x}$ = random variables, which represent various uncertain quantities rising from systems and loadings; $\mathbf{d}$ = design variables, which may be deterministic design parameters or statistical property values (e.g., mean and standard deviation) of random variables $\mathbf{x}$; β_j^L = the lower limits of reliability index for the associated limit states prescribed. The relationship between the limiting value of reliability index and the target failure probability could be given by FORM as $P_f^T \approx \Phi(-\beta_j^L)$.

One of the first attempts for the development of RBDO was made by Moses in 1969. It was stated in his study which focused on the relationship between reliability and optimization that “an optimization procedure which uses overall structural failure probability as the behavior constraint should produce more balanced designs, consistent with the development of rational safety.” Since 1960s, much work has been done toward the goal stated by Moses (Surahman and Rojiani 1983; Frangopol 1985a, b; Kim and Wen 1990; Enevoldsen et al. 1994; Chang et al. 1994; Chandu and Grandhi 1995; Zou and Mahadevan 2006). Some of those literature, the reliability index approach was employed. Given the target failure probability, a reliability-based optimization procedure for reinforced concrete structures was developed by Surahman and Rojiani (1983). The component or system reliability-index-based optimization problem is formulated by Enevoldsen et al. 1994. Then, the problem is solved using the FORM in estimating the reliability and sequential quadratic programming techniques. Chang et al. (1994) used

the reliability index approach for optimum seismic design of steel structures subjected to seismic loadings of the Uniform Building Code. Zou and Mahadevan (2006) developed an efficient decoupling reliability-based design optimization method to solve a practical vehicle side-impact design problem subject to multiple reliability index constraints.

2.4.3 Performance Measure Optimization Approach

The reliability index constraint can be also formulated in terms of probabilistic performance measure constraint (Tu et al. 1999). The time-invariant failure probability given in Eq. (2.22) would be constrained during RBDO so as to be within the limit of target failure probability as

$$P_f = \int_{g(\mathbf{x}, \mathbf{d}) \leq 0} f(\mathbf{x}) d\mathbf{x} \leq P_f^T \approx \Phi(-\beta_j^L) \quad (2.31)$$

Since the statistical description of limit-state function $g_j(\mathbf{x}, \mathbf{d})$ is characterized by its cumulative distribution function (CDF) $F_{g_j}(t) = P(g_j \leq t)$, the probabilistic constraint of Eq. (2.31) can be rewritten as

$$0 \leq F_{g_j}^{-1} \left[\Phi(-\beta_j^L) \right] = t^* \quad (2.32)$$

where t^* = the performance measure. Therefore, Eq. (2.32) represents a probabilistic performance measure constraint. On the other hand, the performance measure constraint can be easily transformed back into the reliability index constraint as

$$\beta_j^* = -\Phi^{-1} [F_{g_j}(0)] \geq \beta_j^L \quad (2.33)$$

Lee et al. (2002) compared the target-performance-based approach with the conventional reliability-index-based approach for solving the structural design optimization problem. The illustrative examples in that paper demonstrated that the performance measure approach is superior to the reliability index approach in view of both computational efficiency and numerical stability. Youn et al. (2003) developed a hybrid analysis method to take advantages of the performance measure approach for effective evaluation of probabilistic constraints in the RBDO process. The performance measure concept is found to be effective to establish the CDF of output performance function for practical engineering applications, i.e., side-impact crashworthiness, involved numerous input uncertainties (Youn et al. 2005). It is worth to note that in the RBDO framework, depending on the characteristic of

design problems, the trade-off between the accuracy and efficiency must be considered in selecting the appropriate reliability methods among the various approximation or simulation approaches discussed in the previous sections (Jensen 2005).

2.5 Summary

Based on the literature review, a number of conclusions were drawn in the following:

1. The performance-based design approach seems to be a promising direction for designing economical and reliable building structures satisfying intended performance design objectives under high-risk hazard-induced actions, e.g., wind and seismic loads. The source of typhoon risk and the method to predict it have been reviewed. Major uncertainties involved in wind-sensitive tall building design are identified, and the methods to model and quantify uncertainty are discussed.
2. Literature review revealed that the dynamic theory of linear systems is so well developed compared to that of nonlinear systems. Although many literature are available addressing the nonlinear dynamic analysis problems, there is still lacking a general and efficient approach to predict nonlinear response of a randomly excited stochastic system. Some approximation methods with additional assumptions have to be adopted to solve practical engineering analysis and design problems. Wind-tunnel-based semi-analytical method in frequency domain becomes a practical and effective way to predict wind-induced dynamic responses of tall buildings for the design purpose.
3. The OC approach has gained tremendous popularity for optimal building structural design due to higher efficiency than other classical optimization method or mathematical programming approach. The OC algorithm based on the virtual work formulation has been employed in this study as a key optimization engine in the computer-based framework of reliability performance-based design optimization of wind-sensitive tall buildings.
4. Some methods for dynamic response optimization have so far been developed and studied. However, these methods are only feasible for mechanical structures subjected to deterministic transient time history loads. Dynamic response optimization concepts have to be extended to deal with situations of random vibration in the optimal design of tall buildings under multihazard environment.
5. Reliability theory and reliability analysis methods have been extensively reviewed in the context of performance-based engineering. Due to the complexity of multidimensional integral, most of reliability analysis problems have to be approximately solved using numerical analysis technique, including optimization algorithms and simulation. It is worth to note that the Markov model (consequently, FPK equations) is equally important to study nonlinear

responses of randomly excited systems and time-variant reliability problems. For wind-induced time-variant reliability problem, the classical first-passage problem solution could offer a convenient way to quantify the serviceability failure probability. The latest development of efficient simulation methods for randomly excited linear system could also help to solve this reliability problem.

Throughout the process of literature review, it is believed that the further development is needed for performance-based engineering of high-rise buildings under multihazard environment by investigating some of the key research components regarding the effective simulation and analysis methods for hazard load effects, dynamic response optimization, structural assessment and identification, wind-induced time-variant reliability analysis and the performance-based assessment and optimization framework. This comprehensive study would have provided practical and useful analysis, design, and assessment guidelines for structural and wind engineers to understand and improve and fulfill performance objectives of high-rise structures, which are safe over their design life and serviceable for intended functions under multihazard environment.

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High-Rise Buildings under Multi-Hazard Environment
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Huang, M.

2017, XXVII, 232 p. 100 illus., 66 illus. in color.,

Hardcover

ISBN: 978-981-10-1743-8