CHAPTER II

PERFORMANCE-BASED DESIGN AND RELIABILITY OF STRUCTURES

2.1 Performance-based Design Concept

Performance-based design means the methodology which is based on sufficiently realistic environmental and material models so it is possible to make satisfactory prediction of the future behavior of a concrete structure. The predictions are preferably made with a probabilistic approach, for the included parameters are considered. The parameters are described with statistical distribution functions and quantifies from measurement or expert opinions [5].

Performance-based engineering concepts address site selection; conceptual, preliminary and final design; construction; and building maintenance –all of which can have significant impact on achieving owner and developer expectations of how a building may perform over its life. In performance-based design, the owner or developer and design team address the owner's needs and expectations at the initiation of a project. Risks also are taken into account.

Performance-based engineering works best for an owner or designer who recognizes a natural or manmade hazard or risk to overall operations and is willing to invest, upfront, to minimize that risk. Most large corporations and institutions have an in-house risk management group responsible for minimizing the risk of catastrophic losses. Many of those risk managers are recognizing the value of performance-based engineering for several reasons [5].

An investment in performance-based design will pay for itself in case of a catastrophic event, given the potential cost of repair due to structural damage, disruption of operations, value of contents, and perhaps, loss of market share. As risk managers become comfortable with the long-term value of performance-based engineering, it will become a mainstream hazards-risk management tool. Future codes and standards will provide increasingly useful and definitive provisions for performance-based design.

Like any design tool, performance-based design has its limitations. But by tapping the skill of the structural engineering community, owners now can minimize risks from high winds, explosions, earthquakes, fire and. etc.

To describe the distinction between "prescriptive versus performance based engineering" the following examples were offered by Harris (2002): "Performance based: An acceptable level of protection against structural failure under extreme load will be provided" and Prescriptive: 0.5 in. diameter bolts spaced no more than six feet on center shall anchor the wood sill of an exterior wall to the foundation." Hamburger (2002) described performance-based design as: "Design specifically intended to limit the consequences of one or more perils to defined acceptable levels" [11].

The rationale and the heuristic knowledge base that has shaped the "specification-based" prescriptive approach to civil engineering design and evaluation practice has served well during the last Century. A prescriptive approach is easier to implement than a performance-based approach from a design standpoint. Prescriptive design also includes many factors of safety to account for unknowns in both the loading and resistance and to account for simplifications in the analytical techniques. Since their original formulations during the first three decades of the 20^{th} Century, design recommendations, guidelines and model codes covering common structural materials and systems have offered a qualitative promise for performance in their commentaries or related committee reports. For example, the ACI code provisions seek to provide crack-width and deflection control at the serviceability limit states and a ductile failure mode at ultimate limit states. On the other hand, some long-standing prescriptive procedures may be unnecessarily conservative while others may not recognize the "blind-spots" that are created when empirical knowledge is stretched to cover newer and yet unproven materials, systems and processes [11].

The performance based design will incorporate the aspect of strength with safety index, serviceability with serviceability index and durability with durability index. Recently, on the discussion of performance-based design around the world, the safety and serviceability indexes already suggested by standard organizations for that related to probability of failure and probability of risk for each performances. The discussion on determining performance index for durability still not come to an agreement. The distinction of those three performance index will be explained through several steps on next parts of sub-chapters.

2.1.1 Limit-states Design versus Performance-based Design Approaches

Since a quantitative approach to performance-based design is a relatively new concept for most civil engineers, comprehensive basic research in this area is in its infancy. Performance-based design concept depends on many inter-connected issues including classification of constructed systems, definition of performance, tools for measuring performance, quantitative indices that may serve as assurance of performance, and especially, how to describe and measure performance especially under various levels of uncertainty. It is important to note that since all modern building and bridge design codes are now based on the Limit States, or, the Load and Resistance Factor Design (LRFD) concept; the future performance-based design guidelines should reflect the thinking behind this same concept.

The basic of LRFD concept is based on satisfying various limit state functions with predetermined reliability levels. The limit state functions are expected to be different for different types of construction (buildings, bridges, tunnels, dams, nuclear facilities, etc.). They are also expected to be different for different types of loading or displacement actions. If seismic loading needs to be considered, it may have to involve different types of limit states depending on the expected return periods of minor, moderate and major earthquakes.

The probabilistic basis for LRFD has been described (Ravindra and Galambos, 1978, Ellingwood, MacGregor, Galambos and Cornell, 1982) based on assuming load effects and resistance factors to be statistically independent random variables. A reliability index β is defined in terms of the means and the coefficients of variations for the frequency distributions of the resistance and load effects. This index provides a comparative value of the measure of reliability of a structure or component. More recently, Ang (2004) described the distinctions between *aleatory* and *epistemic* uncertainties, and this implies a need for rethinking the reliability index by recognizing and incorporating the impacts of epistemic uncertainty [11].

2.1.2 Limit States and Limit Events for Performance-based Design

The "limit-states design" or "load-and- resistance factor design" aims to assure that the designed constructed system will have sufficient capacity to satisfy the demands associated with each limit-event with an acceptable probability of failure or with a desired level of structural reliability. In a performance based design, it should not to limit the consideration to only the "probability of failure" but consider the "risk of failure," that would explicitly incorporate the return period of the loading or hazards that prevail at a site and the consequences of failure in addition to the probability of failure. In this context, failure refers to a failure to meet the intended performance objective and not strictly the loss of structural strength or stability leading to a life-safety peril [11].

Table 2.1 lists the limit-states, limit-events, and expected performance goals that are being recommended by the ASCE Committee on Performance Based Design and Evaluation of Constructed Facilities. It is noted that a consensus in the description of limit-states, the corresponding limit events and the corresponding performance goals is a most important step before start standardizing performance-based civil engineering. An issue is whether the same set of limit states and events may govern all types of constructed systems. However, the broader fundamentals of performance-based engineering for either type of construction should not be different [11].

Table 2.1 Limit states, limit events and performance goals [11]

	Utility and functionality	Serviceability and durability	Life safety and Stability of failure	Substantial safety at conditional limit state
Limit states	▪ Environmental impacts ▪ social impacts ▪ Sustainability of functionality throughout lifecycle ▪ Financing; initial cost and lifecycle cost	▪ Excessive: displacements, Deformations, drifts ▪ Deterioration ▪ Local damage ▪ Vibrations	▪ Excessive movements, settlements, geometry changes ▪ Material failure ▪ Fatigue ▪ Local, Member Stability failure	▪ Lack of: multiple escape routes in building ▪ Lack of: post-failure resiliency leading to Progressive collapse of buildings
Limit-events	▪ Operational: Capacity, Safety, efficiency, flexibility, security ▪ Feasibility of: construction, protection, preservation ▪ Aesthetics	**Lack of Durability:** Special limit-state that should govern aspect of global design, detailing, material and construction	**Stability of Failure:** ▪ Incomplete premature collapse mechanism(s) without adequate deformability and hardening ▪ Undesirable sudden-brittle failure mode(s)	Cascading failures of interconnected infrastructure systems Failure of infrastructure elements critical for emergency response: Medical, communication, water, energy, transportation, logistics, command and control
Goals	▪ **Multi-objective performance function for integrated asset-management:** Function relating to Operations and Security	▪ **Multi-objective performance function for integrated asset-management:** Function Relating to Inspection, maintenance and Lifecycle	▪ **Multi-hazard risk management:** Assurance of life safety and quick recovery operations following an event (Days-months)	▪ **Disaster response planning:** Emergency management, protection of escape routes, evacuation, search and rescue needs, minimize casualties ▪ Economic Recovery (within years)

An important characteristic of each limit-event is therefore the return period of the associated demands or loading events within each limit-state. The risk due to failure of a constructed system to perform is defined as the product of three factors: (a) The probability of a demand exceeding an expected value, (b) the probability of the system not performing as desired, and, (c) the consequences of this failure to perform. It follows that the return period (which in turn defines the expected probability of occurrence of a limit-event during the lifecycle of a facility), is a critical factor in defining the risk that should be controlled during design or evaluation. Further, the envelope of actions and resistances to be considered in design or evaluation should be based on an acceptable risk associated with each of the limit-events. For example, the acceptable risk associated with the "incomplete and premature collapse mechanism(s) without adequate deformability and hardening" limit-event within the "safety and stability of failure" for a building system may be as high as 0.0001 and as low as 0.0000001 given the importance and functions, the infrastructure system that is served by the building, location, occupancy, architecture, site and structural attributes of the building. The risk and reliability basis of performance-based design is illustrated in Figure 2.1.

A design approach that incorporates the risk of a constructed structure not achieving its expected performance at each limit-state and limit-event would offer a far greater flexibility for optimizing how the financial resources available for any given project are allocated to various features of the system. This is illustrated in Figure 2.1 which presents an overview of the performance based design and evaluation approach that incorporates risk as envisioned by the Committee [11].

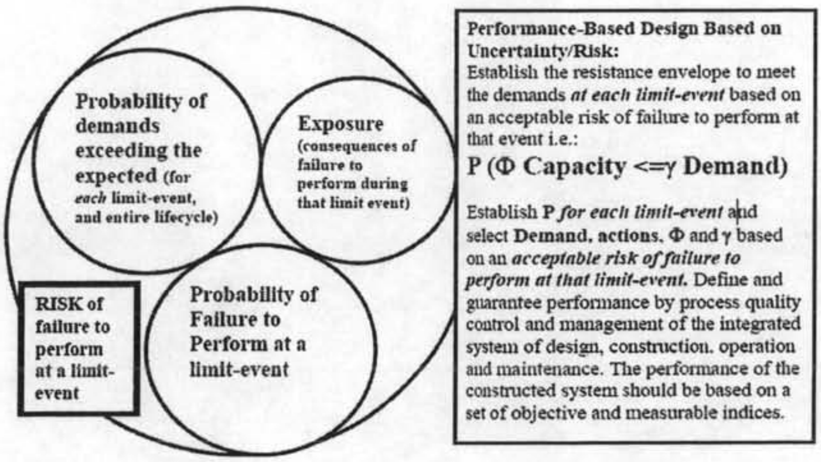


Figure 2.1 Performance-based design and evaluation [11]

Two limit-states justify further discussion. The "durability" limit-state that is included within "serviceability" is only now widely recognized as concern justifying its distinct limit-state that deserves special attention in design and in evaluation. Durability brings a different dimension and may justify a different approach to the selection of materials, proportioning, detailing, construction, maintenance, etc. than a design based only on serviceability and safety. For example, it have to be recognized that special cover and detailing of reinforcement for crack control in a reinforced concrete element may justify more attention to it than the attention spending in detailing for capacity. In many cases durability may be assured only if a designer is in full command of all the mechanisms that influence deterioration. To assure the durability of a design may require extensive "scientific" research in the field on real constructed facilities, integrated with laboratory and analytical studies in order to reveal the actual mechanisms that cause deterioration and how they may be effectively mitigated. This would have to be coupled with an in-depth knowledge of material behavior at the microscopic level [12].

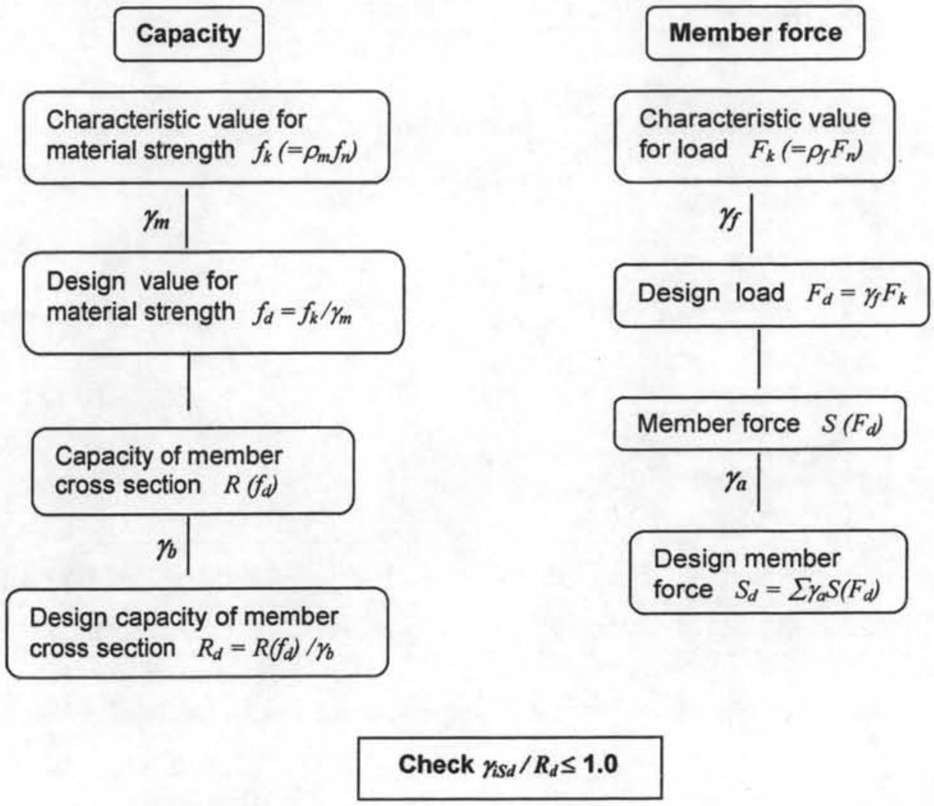
2.1.3 Strength Performance of Concrete Structures

Strength or safety performance of concrete structures in term of the performance-based design is intended to avoid the occurrence of limit states of life safety and stability failure include excessive movements, settlements, geometry changes, material failure, fatigue, local and member stability failure. The limit events to have to be prohibited are incomplete premature collapse mechanism(s) without adequate deformability and hardening, and also undesirable sudden-brittle failure mode(s).

Structural engineers have traditionally used various indices for defining the health of a structure depending on purpose, such as safety factor, condition rating, load-capacity rating, sufficiency index, capacity-demand ratio, redundancy, etc. Although it is pragmatic to continue using such deterministic indices that are mainly related to "structural safety" most engineers now recognize the need for a broader definition that relates to performance and health in relation to the entirety of Table 2.1. Such a definition, in fact can be made by generalizing the structural reliability concept [11]:

"We define the health of a constructed system as the probability that it possesses adequate capacity against all probable demands that may be imposed on it in conjunction with the limit-states and limit-events listed in Table 2.1. Here we emphasize that system reliability should cover the entire spectrum of limit states and limit-events in Table 2.1 and not just "structural safety". Further, according to Ellingwood (2004), the distinction between health and reliability is that health is a desirable state and reliability is a measurement of it."

In the performance-based design, each performances of structural members need to be verified. JSCE Guidelines No.3 which is the first performance-based design code stated that for purposes of safety verification, it is simply required to verify that non of the members reaches the ultimate limit state. For design of members, generally, the ultimate limit state for failure of member cross-section is examined for safety verification and other ultimate limit state are not taken into consideration so frequently. For members subjected to one of the member forces among flexural moment, axial load, shear force and torsional moment, verification of safety against failure of member cross section shall be carried out as shown in Figure 2.2 [15].



where; γ_m = material factor
γ_b = member factor
γ_f = load factor
γ_a = structural analysis factor

Figure 2.2 Checking for ultimate limit state for failure of cross-section [15]

When a member is subjected to a combination of loads such as flexural moments and axial load, verification of safety against failure of member cross section shall be carried out by comparing the appropriate design member forces with design capacity of the member cross section, taking into account of the action of combined load [15].

As discussed earlier, incorporating the probability of failure and probability of risk, performance-based design may take advantage of the "Reliability Index: β" as a measure of health or reliability as this relates in concept to the deterministic "Safety Factor" or "Load Rating" most engineers use in practice (Ellingwood, et al, 1982). For example if Capacity and Demand are independent and normal random variables, $\beta = 0$ corresponds to a reliability or $(1-P_f)$ of 0.5, $\beta = 3$ corresponds to a reliability of 0.999, and $\beta = 4.75$ corresponds to a reliability of 0.99999. The latter corresponds to one in a million chance of inadequate capacity to perform [14].

The first step in reliability analysis is the formulation of the limit state function for safety is to prohibit incomplete premature collapse mechanism(s) without adequate deformability and hardening, and also undesirable sudden-brittle failure mode(s) during design service life. For a structural member having capacity, R, and subjected to dead load, D, live load, L, and wind effect, W, a failure function that represents the safety margin, g(R, D, L, W), can be formulated as

$$g(R, D, L, W) = R-D-L-W \tag{2. 1}$$

The limit state function is obtained by equating g(R, D, L, W) to zero. The reliability index, β, is usually used to measure structural safety. It is defined as the ratio of the mean, μ_g, to the standard deviation, σ_g, of g

$$\beta = \mu_g / \sigma_g \tag{2. 2}$$

For the simple case of all variables in the limit state function being normally distributed, β can be computed from

$$\beta = \frac{\mu_R - \mu_D - \mu_L - \mu_W}{\sqrt{\sigma_R^2 + \sigma_D^2 + \sigma_L^2 + \sigma_W^2}} \tag{2. 3}$$

where μ_R, μ_D, μ_L, μ_W = mean values of R, D, L, and W, respectively, and σ_R, σ_D, σ_L, and σ_W = standard deviations of R, D, L, and W, respectively.

The relationship between the reliability index and probability of failure, P_f, for the case of a normally distributed safety margin is defined as

$$p_f(t) = \Phi(-\beta) \tag{2. 4}$$

where Φ = cumulative standard normal distribution function.

A structure may be considered to be safe, i.e., its safety is maintained, as long as it or its members do not fail. In the case of a statically highly indeterminate structure, its safety may not be immediately lost even when some of the members reach the ultimate limit state and become incapable of carrying load. In cases where partial failure of members is permitted but the overall safety of the structure still needs to be maintained even after partial failure of some members, the nonlinear and the past-failure behavior of members should be appropriately taken into consideration at the time of safety verification as in seismic performance verification [15].

2.1.4 Serviceability Performance of Concrete Structures

The performance-based design take into consideration the aspect of serviceability of structure as the key performance aspect. Serviceability performance itself include some other aspect which related to the comfortable use and convenience. CEB-FIP Model Code 1990 [12] includes four aspects in serviceability limit state i.e. stress of material, crack widths, deformations and vibrations. Exceeding the limit state of stress or limit state of cracking may lead to limited local structural damage mainly affecting the durability of the structure. Excessive deformations may produce damage in non-structural elements or load bearing walls and affect the efficient use or appearance of structural or non-structural elements. Vibration may cause discomfort, alarm or loss of ability to use.

For stress level, under service load conditions, CEB-FIP Model Code 1990 stated the limitation of stresses for tensile stresses in concrete, compressive stresses in concrete and tensile stresses in steel. The limitation of tensile stresses is an adequate measure to reduce the probability of cracking. The limitation of compressive stresses in concrete should avoid excessive compression, producing irreversible strain and longitudinal cracks. Tensile stresses in reinforcement should be limited with an appropriate safety margin below the yielding strength, preventing uncontrolled cracking.

It should be ensured that, with an adequate probability, cracks will not impair serviceability and durability of the structure. Cracks do not, per se, indicate a lack of serviceability or durability; in reinforced concrete structures, cracking might inevitable due to tension, bending, and shear, torsion, without necessarily impairing serviceability or durability [12].

Deformations -include deflections and rotations- during service period may be harmful to the appearance of the structure, the integrity or non-structural parts, and the proper function of the structure or its equipment. On the practical verification of deflection, for simple building elements under specified circumstances, it may not be necessary to calculate deflections explicitly if certain limitations of the span-depth ratio are respected [12].

Vibrations of structures may affect the serviceability of a structure as functional effects (discomfort to occupants, affecting operation of machines, etc.) and structural effects (mostly on non-structural elements. as cracks in partition, loss of cladding, etc.). Vibrations can be caused by several variable actions, e.g.

- rhythmic movements made by people such as walking, running, jumping and dancing
- machines
- waves due to wind and water
- rail and road traffic
- construction work such as driving or placing by vibration of sheet piles, compressing soil by means of vibrations as well as blasting work.

To secure satisfactory behavior of a structure subject to vibrations, the natural frequency of vibration of the relevant structure should be kept sufficiently apart from critical values which depend on the function of the corresponding building.

Asian Concrete Model Code 2006 [13] stated that performance indices for serviceability should include performance of user comfort and performance of functionability. It also provide example of performance indices for comfortable use and performance indices for functionability related to serviceability of concrete structures shown in Table 2.2 and 2.3.

Table 2.2 Examples of performance indices for comfortable use [13]

Performance items	Performance Indices
Comfortable ride/walk	Acceleration, natural period of structure/component, gap/step, or type of pavement
Comfortable stay	Deformation (slope angle, etc)
Vibration	Vibration level around structure, on natural period of structure/component
Noise	Noise level around structure, or type/shape/height of soundproof wall
Odor	Density of substance with odor around structure, or amount of substance with odor in/inside structure.
Humidity	Humidity around structure, or water contents in/inside structure
Aesthetics	Crack density, crack widths, or amount of dirt on surface of structure
Visual safety	Deformation, crack density or crack width.

Table 2.3 Examples of performance indices for functionability [13]

Performance items	Performance Indices
Shielding	Amount of substance/energy to penetrate structure/component, penetration rate, or crack width/density
Permeability	Amount of substance/energy to penetrate structure/component, penetration rate, or porosity of structure/component

Serviceability limit state are required to be determined to suite the purpose of use of structures and shall be checked or verified by appropriate methods of which accuracy and applicable range are clarified. As far as the serviceability limit states are concerned, various limit state may be considered. In general, however, only the serviceability limit states for cracking, displacement, deformation, and vibration, may be examined [15]. The verification for each aspects are presented as follows.

Limiting Value of Stresses

Compressive stress in concrete and tensile stress in reinforcement due to flexural moment(s) and axial force(s) shall not exceeding the limiting value. Maximum limit on the compressive stress of concrete is introduced in order to avoid excessive creep strain and cracks in the longitudinal direction due to compressive forces. These limiting values have been determined considering the modulus of elasticity of concrete and the conditions for the creep coefficient of concrete [15].

Cracking

Cracks occurring in the concrete structures become a cause for reduction in durability due to reinforcement corrosion, deterioration in functions such as water tightness and air tightness, large deformations, impairment of appearance, etc. Therefore, it shall be examined by an appropriate method that such functions, durability and appearances of structures are not impaired due to cracking in concrete. Performance of concrete in the concrete cover to protect reinforcement from corrosion due to chloride ingress is achieved by not only controlling crack width but also providing good quality of concrete. Based in this fact, examinations of serviceability limit cracks for durability should be made, in principle, by conforming both crack width is less than the permissible width and that chloride concentration at reinforcement in concrete predicted by the transport analysis considering the effect of crack, does not exceed the threshold value for initiating corrosion during the design life [15].

When water tightness is important, examination of cracking shall, in principle, be carried out, by confirming that either the crack does not occur or the width of the crack is not greater than the permissible value. When appearance of structure is particularly important, examination for crack width may be carried out by a method similar to that for durability, and appropriately setting a permissible crack width. On the basis of past test data, it is reasonable to consider that crack width at the surface of members, which has a great influence on corrosion of reinforcement, depends on concrete cover. Thus, assuming that the permissible crack width may be increased with increasing concrete cover, the permissible crack width have been determined depending on the environmental conditions and the type of reinforcement [15]. On the same way of thinking, the examination of crack width due to the structural action should be carried out using the most accurate and appropriate method.

Displacements and Deformations

Displacements and deformations, in general, are related to maintaining functions and serviceability for safety and comfort with moving traffic, preventing damages due to excessive displacements and deformations, and maintaining esthetics of structures. Considering the purpose of use of a structure, enough stiffness and appropriate camber should be provided, and support need to be selected adequately. It is advisable to examine the influences of gasp kinks between members and expansion/shortening of member if necessary [15].

Examinations verifications for displacement and deformation of structures or member shall be carried out using appropriate methods to ensure that the functions, serviceability, durability and appearance of the structure or member are not impaired. Short-term displacement and deformation, and long-term displacement and deformation shall be considered separately. Short-term displacement and deformation refer to instantaneous displacement and deformation of the structure or member upon application of load(s). Long-term displacement and deformation include short-term displacement and deformation and additional displacement and deformation under sustaining loads. Short-term displacement and deformation, and long-term displacement and deformation of the structure or member shall be smaller than the permissible displacement and deformation [15].

Vibration

Vibration are rarely cause of problems in concrete structures. However,, in case when the time period of variable loading is close to the natural period of members, resonance may result. This may bring lead to an uncomfortable environment during use and cause cracks in the structure. In this case, it is advisable to take some countermeasure, such as altering the natural period of the member by changing the dimension of members, etc. Examinations for vibration caused by variable loads shall be carried out using appropriate methods to ensure that functions and serviceability of the structures are not impaired [15].

Verification on performance of serviceability incorporating probability of failure and probability of risk carried out using reliability analysis for certain aspects; deflection of beam for example. During service life, from the statistical data it known that certain level of deflection caused crack or damage on the nonstructural member attached on it. On the other hand, the certain level of deflection also reduces the conformity and makes distortion of appearance. Based on that, codes specify the limitation on maximum deflection. Reliability analysis performed to obtain the probability of failure – the deflection exceeds the limit – during service life. The procedures of structural reliability serviceability explained as follow.

The probability of serviceability failure is then the probability of exceeding the allowable deflection limit δ_L

$$p_f(t) = P_r\{G[\delta_L, \Delta(t)] \leq 0\} \tag{2.5}$$

where the serviceability limit state is given by

$$G[\delta_L, \Delta(t)] = \delta_L - \Delta(t) \tag{2.6}$$

The probability of serviceability failure during service live may be calculated as

$$p_f(t) = \Phi(-\beta_{serv}) \tag{2.7}$$

where Φ = standard normal distribution function; β_{servi} = serviceability reliability index; and t = service live period which may separated into several tenancy period.

Similar method can be applied to the other aspect of serviceability such as the crack width. The serviceability limit state for maximum crack width with the resistance R and load effect Q is written as

$$G[R, Q] = w_{lim} - w_m = w_{lim} - \varepsilon w_{max} \tag{2.8}$$

where w_{lim} is the allowable crack width specified in the design codes, w_m the observed maximum crack width, ε, the factor representing model uncertainty of evaluated maximum crack width and w_{max} the evaluated maximum crack width given by code.

2.1.5 Durability Reliability of Concrete Structures

The broad concept of design for durability is described in a flow chart in Figure 2.3. Concrete structures are designed for structural integrity and good serviceability. Crack size and distribution are limited by good design and steel distribution. Once this is achieved, the durability performance of the crack-freed (limited crack) concrete would depend on the type environment in service. In all environments, the durability performance of concrete structure is dependent on the *Quality* of the concrete. Where deterioration is resulted from steel corrosion, the *Quantity* or thickness of the concrete cover (sometimes referred to as Covercrete) is also of extreme important. Greater concrete cover usually means longer time it takes for aggressive agent to reach the steel causing corrosion. Too much a cover, on the other hand, could result in larger and more cracks allowing direct access of aggressive agent to the steel reinforcement [14].

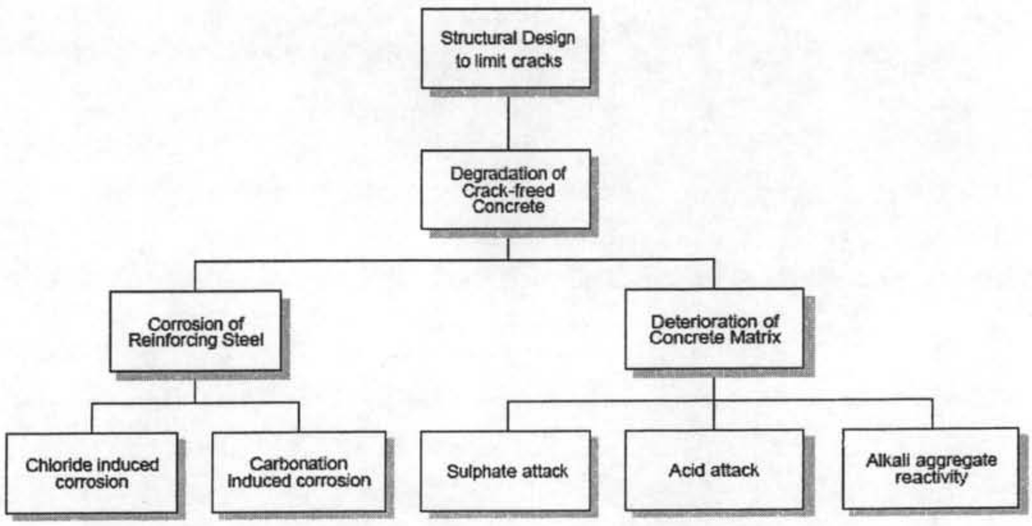


Figure 2.3 Design for durability concept [14].

The main criteria used in specifying concrete for durability remain to be both the quality and quantity of concrete cover. The quality is specified by a maximum limit on the water-to-cement (w/c) ratio or characteristic compressive strength (f_c') and the type of cement (in the case of concrete in sulphate environment). A corresponding minimum quantity of concrete cover is also given.

The exposure classifications are also defined slightly differently but there is a basic distinction between problems associated with corrosion of steel reinforcement and deterioration due to sulphate attack. In normal or industrial environment where the most common cause of steel corrosion is due to carbonation from the atmosphere, both

BS 8110 and AS 3600 recommend specific strength grade and minimum concrete cover for each specific situation. The approaches used by the two standards could be a very good guide to specifying concrete in the Asia/Pacific region. ACI 357 and BS 8110 adopted a pragmatic approach in specifying the maximum w/c ratio together with the more measurable criteria of both the strength grade and minimum cement content. Both codes recognize the practical value of identifying a strength grade consistent with the w/c ratio required for durability. On the other hand, AS 3600 used compressive strength as the sole criterion. The Commentary to AS 3600 provides recommendations on the minimum cement content. All codes specify a maximum limit for the chloride ion content in the 'as placed' concrete. AS 3600, for example, limits the maximum acid-soluble chloride-ion content to be 0.8 kg/m^3 for reinforced and post-tensioned concrete [14].

The theory of durability design is in principle based on the theory of safety (or structural reliability) traditionally used in structural design. In this context safety denotes the capacity of a structure to resist, with a sufficient degree of certainty, the occurrence of failure in consequences of several of potential hazards to which the structure is exposed. Now the use of this technique is increasingly advocated for dealing also with durability and service life problem. Although traditionally the methodology of safety has been almost exclusively applied to studies of structural mechanics, the method is by no means restricted to such design problem [31].

A new feature in the theory of safety is the incorporation of time into design problem. It allows the possibility of treating degradation of materials as an essential part of the problem. Safety against failure (falling below the performance requirements) is a function of time. Designing a structure with the required safety now includes a requirement of time during which the safety requirement must be fulfilled. In other words a requirement for the service life must be imposed.

The simplest mathematical model for describing the event 'failure' comprises a load variable Q and a resistance variable R. In principle the variable Q and R can be any quantities and expressed in any units. The only requirement is that they are commensurable. If R and Q are independent of time, the event of 'failure' can be expressed as follow:

$$\{\text{failure}\} = \{R<Q\} \tag{2.9}$$

In other words, the failure occurs if the resistance is smaller than the load.

In term of service live, or durability, the main concern is to ensure that the failure on durability aspects not occurs during service live. Thoft-Christensen (200_) [32] proposed the new definitions of service life for reinforced concrete structures. The definitions is

$$T_{service} = T_{crack} + \Delta T_{cr} = T_{corr} + \Delta t_{crack} + \Delta T_{cr} \tag{2.10}$$

where $T_{service}$ = service life; T_{corr} = initiation time for corrosion; Δt_{crack} is time from corrosion initiation to corrosion crack initiation; and ΔT_{cr} is the time from initial cracking to a critical crack is developed. Thoft-Christensen (200_) [32] also proposed the example on determining the service live related to corrosion of the reinforcement due to chloride penetration of the concrete and cracking of the concrete due to corrosion of the reinforcement. The deterioration steps can be seen on Figure 2.4 below.

1. Chloride penetration of the concrete
2. Initiation of the corrosion of the reinforcement
3. Evolution of corrosion of the reinforcement
4. Initial cracking of the concrete
5. Evolution of cracks in the concrete.
6. Spalling

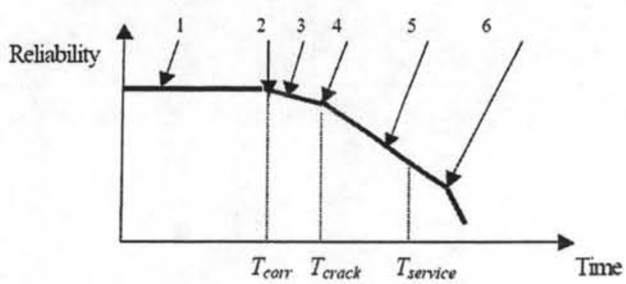


Figure 2.4 Deterioration steps [32].

On verifications of service live, the limit state could be formulated as

$$G[T_{design}, T_{serv}] = T_{design} - T_{serv} > 0 \tag{2.11}$$

$$G[T_{design}, T_{serv}] = T_{design} - (T_{crack} + \Delta T_{cr} = T_{corr} + \Delta t_{crack} + \Delta T_{cr}) > 0 \tag{2.12}$$

The probability of failure on service life can be calculated as

$$p_f(T) = P_r\{G[T_{design}, T_{serv}] > 0\} \tag{2.13}$$

There are some methods on predicting deterioration stepping time as shown in Figure 2.3 can be seen on paper by Thoft-Christensen (200_) [32].

Some other aspects of durability should get the proper attention on designing structural concrete durability as mention on ACI 201.2R Guide to Durable Concrete. These each aspects could be the critical aspect on determining the service life, i.e. freezing and thawing, chemical attack, abrasion resistance, corrosion of embedded metal, and alkali-aggregate reaction.

2.2 Concept of Structural Reliability

Reliability methods first received significant academic attention in the period from 1967 to 1974. During this period researchers developed many of the theoretical tools necessary for handling the uncertainty present in design. Since the development and acceptance of these methods, there has been a gradual shift away from design approaches based on deterministic values toward new codes that are able to rationally take into consideration issues of reliability and uncertainty in structural design. Most traditional construction materials such as steel, concrete, and timber now use probabilistic based design codes [6].

Reliability can be defined as probabilistic measure of assurance of performance with respect to some prescribed condition(s). The condition can be referred to as an ultimate limit state (such as collapse) or serviceability limit state (such as excessive deflection and/or vibration) or durability limit state [7].

The need for probabilistic approach to assess service lives resulted from the often uncertain nature of loadings and the performance aspects of reinforced structural concrete. Since it cannot take the natural variation of the physical parameters into account, a deterministic approach, using fixed or arbitrary values for pertinent variables, should not be used to assess performance. These uncertainties can be dealt with effectively by using probabilistic methods in which the safety and service/performance requirements are measured by their reliabilities (defined as the probability of survival).

In the past, reliability theory has most often been identified with the military, aerospace and electronics fields. The importance of reliability theory in the area of civil engineering has been increasingly realized over the past number of years. Only over the past decade has the area of reliability in civil engineering applications received the attention necessary to ensure the optimal performance and safety of structures [8].

The importance of civil engineering reliability theory stems from the very nature of the various approaches to structural design. Most often in the past, a deterministic approach has been taken in civil engineering design where the design parameters usually consisted of selected factors of safety multiplied by expected service loads. However, these service loads are rarely known with certainty. As a result these loads should actually be treated as random variables. This different approach calls for the implementation of probabilistic and statistical techniques. These analytical tools have traditionally formed the basis of reliability theory and more recently provide the basic framework for the study of civil engineering related reliability problems.

In the structural reliability analysis, it is the maximum load (or load effect) to which the structure may be subjected over its useful life that is concern. Normally, the determination of the lifetime maximum load is done separately from that of the structural resistance. Presumably, therefore, the resistance R and the lifetime load Q are, respectively, function of basic resistance variables and load variables. That is,

$$R = g_R(R_1, R_2, \ldots, R_n) \tag{2.14}$$

and similarly

$$Q = g_Q(Q_1, Q_2, \ldots, Q_n) \tag{2.15}$$

This situation is represented by two density functions such that the overlapping region between the density functions constitutes the probability of the failure as shown in Figure 2.5.

From Figure 2.5, it is clear that the structural reliability analysis requires a determination of the safety factor and the underlying uncertainties of R and Q. In evaluating the safety of a particular structure, or the design of a proposed structure, the determination of the central safety factor clearly involves the calculation of the available mean strength and the mean lifetime load to which the structure may be subjected. However, in evaluating the level of safety implicit in the design code, the determination of the central safety factor would require information on the actual resistance relative to the design resistance, as well as the actual load versus the design load; in other words, the biases in the design resistance and the design load. This information is a function of the specified allowable resistance and load factors (load and resistance factor design, LRFD). Further explanation on this factor can be found in Sub-chapter 2.3 Limit State Design and Partial Safety Factors.

Figure 2.5 Density functions of resistance R and load Q, and the probability of failure [36].

The reliability of a structure or component is defined as its probability of survival, p_s, which are related to the probability of failure, p_f by:

$$p_s = 1 - p_f \tag{2.16}$$

Considering the uncertainties associated with load effects on structural resistance, the probability of failure (or reliability) provides a meaningful measure of the adequacy of a structure or member. Failure is defined in relation to different possible failure modes, commonly referred to as limit states. For example, ultimate limit states represent the inability of the structure to resist the imposed load effects and can be associated with large inelastic displacements and, for bridge decks, large cracks or punching shear failure. On the other hand serviceability limit states are defined as the inability of the structure to meet its normal use or durability requirements. Examples are excessive delaminations or deformations.

The concept of a "limit state" is used to help define failure in the context of structural reliability analyses. A limit state is a boundary between desired and undesired performance of a structure. This boundary is often represented mathematically by a limit state function of performance function. For example in, in a structure, failure could be defined as the inability to carry loads. This undesired performance can occur by many modes of failure: cracking, corrosion, excessive deformations, exceeding load-carrying capacity for shear or bending moment, or local or overall buckling.

The performance of a structure in relation to a certain limit state can be described as a function of a set of basic parameters (X_i, i = 1, 2, …, n). For example, the deformation of a structural component, d, can be expressed as a function of the load effects, material properties and geometric parameters:

$$d = d(X_1, X_2, \ldots., X_n) \qquad (2.17)$$

Assuming that the maximum allowable deformation for a certain serviceability condition is d_0 the boundary between failure and survival can, in this case, be described by $d = d_0$ or:

$$d - d_0 = 0 \qquad (2.18)$$

Failure occurs if $(d- d_0) > 0$ while the structure is safe for $(d - d_0) < 0$. Substituting Equation (2.17) into Equation (2.18) one can write:

$$f(X_1, X_2, \ldots. X_n) = 0 \qquad (2.19)$$

in which d_0 was incorporated in the function f, such that failure occurs if $f > 0$ and no failure occurs if $f < 0$. The function f is called the limit state function, and it separates the failure region ($f > 0$) and the safe region ($f \leq 0$). This concept is illustrated in Figure 2.6, where the problem is simplified by assuming that the limit state function depends only on two basic parameters, $f(X_1, X_2)$. This simplification is convenient because it allows the visual representation in Figure 2.6, but the concept is equally applicable to an n-dimensional space. The limit state function can then be plotted in the (X_1, X_2) plane, and the failure and safe regions are as depicted in Figure 2.6. If one envisages a perpendicular axis to the (X_1, X_2) plane on which a joint probability density function is defined, the probability of failure is represented by the volume under the density function and over the failure region. The calculation of the failure probability in the general n-dimensional case is a complex task. Much research has been devoted to this problem, and solutions with different levels of detail and accuracy are available.

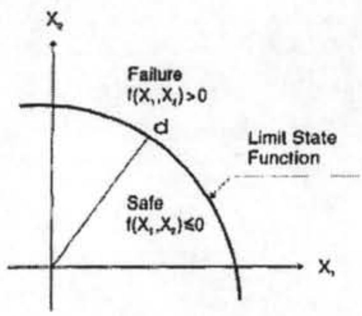


Figure 2.6 Illustration of reliability in two-dimensional space [8].

Since load and resistance parameters are random variables; therefore, it is convenient to measure the structural performance in term of reliability index, β. Various procedures for calculation of β are presented by Nowak and Collins [4].

The general format of the limit state function *g* is

$$g = R - Q \leq 0 \tag{2.20}$$

where g = safety margin; R = resistance; and Q = load effect. In this study, Q is a combination of load components.

The reliability index β can be considered as a function of the probability of failure P_F

$$\beta = -\Phi^{-1}(P_F) \tag{2.21}$$

where Φ^{-1} = inverse standard normal distribution function.

A version of the reliability index was defined as the inverse of the coefficient of variation. In other context of discussion, it can be defined as the shortest distance from the origin of reduced variables to the line $g(Z_R, Z_Q) = 0$. This definition, which was introduced by Hasofer and Lind (1974), is illustrated in Figure 2.7.

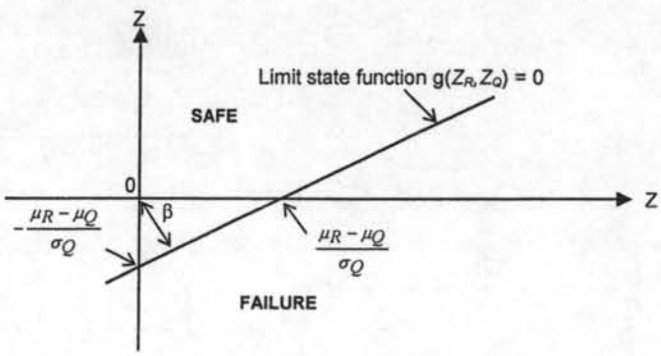


Figure 2.7 Reliability index defined as the shortest distance in the space of reduced variables [4].

Using geometry, the reliability index (shortest distance) can be calculated from the following formula:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{2.22}$$

where β is the inverse of the coefficient of variation of the function $g(R,Q) = R - Q$ where R and Q uncorrelated. For normally distributed random variables R and Q it can be shown that the reliability index is related to the probability of failure by

$$\beta = -\Phi^{-1}(P_f) \quad \text{or} \quad P_f = \Phi(-\beta) \tag{2.23}$$

This is the general definitions of the reliability index. The definition for a two-variable case can be generalized for n variables as follows. Consider a limit state function $g(X_1,X_2,\ldots\ldots,X_n)$ where the X_i variables are uncorrelated.

For examples, the Hasofer-Lind reliability index is defined as follows:

1. Define the set of reduced variables $(Z_1,Z_2,\ldots\ldots,Z_n)$ using

$$Z_i = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}} \tag{2.24}$$

2. Redefine the limit state function by expressing it in terms of the reduced variables $(Z_1,Z_2,\ldots\ldots,Z_n)$.
3. The reliability index is the shortest distance from the origin in the n-dimensional space of reduced variables to the curve described by $g(Z_1,Z_2,\ldots\ldots,Z_n) = 0$.

In structural reliability, the greatest concern regarding the variations in construction is the safety (reliability) of the structure, or inversely the probability that the structure will fail. Thus structural reliability methods are becoming prominent as a means to quantify uncertainty. Typically these methods are used to compute a reliability index, β, that is based on the materials and configuration of the structure, and the variability inherent therein.

Many different approximate techniques can be used to calculate β which is conventionally related to the probability of failure through the standard normal distribution. This approximate relationship in Equation (2.22) is exact if all the concerned variables are normal and the failure function is linear. While β can be related to the probability of failure, it is more often used as a basis of comparison between structures (or states of a single structure) with a higher value of β indicating a higher degree of structural reliability. In probabilistic based design procedures, a predefined value of β is often used as a target level for design [6]. There are several methods to determining reliability index, but the most advance is known as Rackwitz-Fiessler procedures, will be explained as follows and taken from Nowak and Collins 2002 [4].

2.2.1 Rackwitz-Fiessler Procedure; Modified Matrix Procedure

The Rackwitz-Fiessler procedure requires the knowledge of the probability distributions for all the variables involved. The basic idea behind the procedure begins with the calculation of equivalent normal values of the mean and standard deviation for each non-normal random variable. Suppose that a particular random variable X with mean μ_x and standard deviation σ_x is described by a CDF (cumulative distribution function $F_x(x)$) and a PDF (probability density function $f_x(x)$). To obtain the equivalent normal mean μ_x^e and standard deviation σ_x^e, the CDF and PDF at the value of the variable x^* on the failure boundary by g = 0. Mathematically, these requirements are expressed as

$$F_x(x^*) = \Phi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) \tag{2.25}$$

$$f_x(x^*) = \frac{1}{\sigma_x^e}\varphi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) \tag{2.26}$$

where Φ sis the CDF for standard normal distribution and ϕ is the PDF for the standard normal distribution. Equation 2.25 simply requires the cumulative probabilities to be equal at x^*. Equation 2.26 is obtained by differentiating both sides of Equation 2.25 with respect to x^*. By manipulating these equations, μ_x^e and σ_x^e can be expressed as follows:

$$\mu_x^e = x^* - \sigma_x^e\left[\Phi^{-1}\left(F_x(x^*)\right)\right] \tag{2.27}$$

$$\sigma_x^e = \frac{1}{f_x(x^*)}\phi\left(\frac{x^* - \mu_x^e}{\sigma_x^e}\right) = \frac{1}{f_x(x^*)}\phi\left[\Phi^{-1}\left(F_x(x^*)\right)\right] \tag{2.28}$$

The basic steps in the iteration procedure are applies to both linear and non-linear limit state functions. These steps in the matrix procedure implementing the Rackwitz-Fiessler modification are as follows;

1. Formulate the limit state function. Determine the probability distributions and appropriate parameters for all random variables X_i (i=1, 2, . . . , n) involved.
2. Obtain an initial design point $\{x_i^*\}$ by assuming values for n-1 of the random variables X_i. (Mean values are often a reasonable choice.) Solve the limit state equation g = 0 for remaining random variable. This ensures that the design point is on the failure boundary.
3. For each of the design point values x_i^* corresponding to a non-normal distribution, determine the equivalent normal mean $\mu_{x_i}^e$ and standard deviation $\sigma_{x_i}^e$ using Equations 2.27 and 2.28. If one or more x_i^* values correspond to a normal distribution, then the equivalent normal parameters are simply the actual parameters.
4. Determine the reduced variates $\{z_i^*\}$ corresponding to the design point $\{x_i^*\}$ using

$$z_i^* = \frac{x_i^* - \mu_{x_i}^e}{\sigma_{x_i}^e} \tag{2.29}$$

5. Determine the partial derivatives of the limit state function with respect to the reduced variates. For convenience, define a column vector {G} as the vector whose elements are these partial derivatives multiplied by -1:

$$\{G\} = \begin{Bmatrix} G_1 \\ G_2 \\ \cdot \\ \cdot \\ \cdot \\ G_n \end{Bmatrix} \text{ where } G_i = -\left.\frac{\partial g}{\partial Z_1}\right|_{\text{evaluated at design point}} \tag{2.30}$$

6. Calculate an estimate of β using the following formula:

$$\beta = \frac{\{G\}^T\{z^*\}}{\sqrt{\{G\}^T\{G\}}} \text{ where } \{z^*\} = \begin{Bmatrix} z_1^* \\ z_2^* \\ \cdot \\ \cdot \\ \cdot \\ z_n^* \end{Bmatrix} \tag{2.31a}$$

The superscript T denotes transpose. For linear performance function, Equation 2.31a simplifies to

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \mu_{x_i}^{e}}{\sqrt{\sum_{i=1}^{n} \left(a_i \sigma_{x_i}^{e}\right)^2}} \tag{2.31b}$$

7. Calculate a column vector containing the sensitivity factors using

$$\{\alpha\} = \frac{\{G\}}{\sqrt{\{G\}^T \{G\}}} \tag{2.32}$$

8. Determine a new design point in reduced variates for n-1 of the variables using

$$z_i^* = \alpha_i \beta \tag{2.33}$$

9. Determine the corresponding design point values in original coordinates for the n-1 values in Step 7 using

$$x_i^* = \mu x_i + z_i^* \sigma x_i \tag{2.34}$$

10. Determine the value of the remaining random variable (i.e., the one not found in Steps 8 and 9) by solving the limit state function g = 0.
11. Repeat Steps 3 – 10 until β and design points { x_i^* } converge.

2.2.2 Rackwitz-Fiessler Procedure; Graphical Procedure

A graphical version of the Rackwitz-Fiessler procedure (i.e., using equivalent normal parameters) can be applied when the CDFs of the basic variables are available as plots on normal probability paper. Each non-normal variable is approximated by a normal distribution, which is represented by a straight line. The value of the CDF of the approximating normal variable is the same at the design point as that of the original distribution. On normal probability paper this means that the straight line intersects with the original CDF at the design point. Furthermore, at the design point, the PDFs of the original variable and the approximating normal variable are the same. Since the PDF is a tangent (first derivative) of the CDF, the straight line (approximating normal) is tangent to the original CDF at the design point. The parameters of the approximating normal distribution (mean and standard deviation) can be read directly from the graph. This graphical approach is illustrated in Figure 2.8.

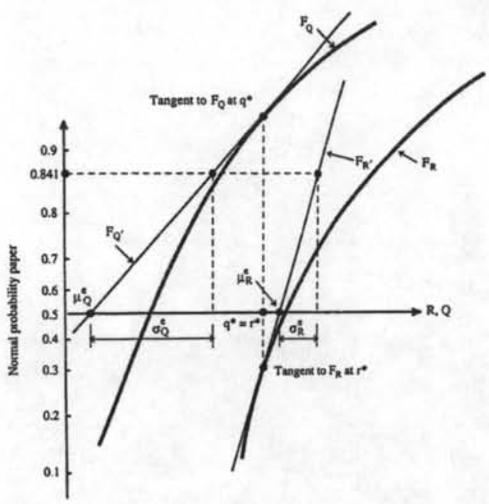


Figure 2.8 Graphical illustration of Rackwitz-Fiessler procedure.

2.3 Limit State Design and Partial Safety Factors

The basic mission of structural design is to accurately deal with the contradiction between the structural safety and economy, to choose a reasonable balance between them, and then to meet the planned structural requirement during the prescriptive period with the minimum price. In order to obtain the goals, for couples of decades the structural design has adopted several methods, such as allowable stress design, working stress design, and limit state design. Accepted as a probabilistic approach to structural safety, the limit state design (LSD) has been used by structural engineers in many countries since the mid 1970's. It provides more consistent safety for various load combinations and various combinations of materials than the past popular method, such as working stress design.

There are two basic functional requirements for all building structures: serviceability during the useful life of the building and safety from collapse during the construction and useful life of the building. LSD defines the various types of collapse and unserviceability that are to be avoided. Those concerning safety are called the ultimate limit states, others concerning unserviceability are called the serviceability limit states. Compared with the past structural design methods, LSD uses partial safety factors instead of the traditional single safety factors. The partial safety factors include the load factors, α, the load combination factor, ψ, the importance factor, γ, and the resistance factor, ϕ, which will be reviewed primarily latter. Moreover, safety and serviceability are also controlled by defining specified loads and material properties statistically, in terms of the probability level (e.g., 5% maximum probability of under-run for material properties) or the return period (10 to 100 years for snow, wind, and earthquake loads).

The LSD criteria can be expressed as follows:

$$\phi R \geq \gamma\left[\alpha_D D + \psi\left(\alpha_L L + \alpha_Q Q + \alpha_T T\right)\right] \quad (2.35)$$

where D, L, Q, and T refer to dead, live, wind (or earthquake) loads and imposed deformation (temperature, etc.) respectively, and ϕR is the factored resistance.

The establishment of resistance factors shows the development of reliability analysis through probability study. Resistance factors are derived for a number of factors causing variability in strength [9]:

- Variability in member strength due to variability of material properties in the structure.
- Variability in member strength due to variability of dimensions.
- Variability in member strength due to simplifying assumptions in the resistance equations, such as the use of a rectangular stress block in concrete design.
- Increased risk to building occupants if failure occurs without warning and the post-failure strength is less than the original strength.

Referring to the papers written by J. G. MacGregor, the resistance factors, ϕ_c and ϕ_s are established through following procedure. The derivation of the resistance factors is a huge job considered in many aspects: material variability, strength format, loading combination, etc. Then taking flexure analysis of reinforced concrete as the example introduces how to establish the resistance factors of reinforcement and concrete.

Consider a large family of similar floor beams each subjected to the same type of occupancy, each for a 50 years lifetime. Each of the beams has been designed to support an unfactored or service load moment, Q. Since the dead load and the live load are both random, each of the family of beams will experience a different lifetime maximum moment. Each of the beams concerned has a different moment capacity, R, due to random variations in the concrete and steel strengths, effective depth, etc. As a result, the families of Q and R have the distribution plotted in Figure 2.5. The 45-degree line in Figure 2.2 represents the case where the load effect, Q, equals the strength, R. Combinations of Q and R that fall above this line, results in failure (Q>R).

If we know the means and standard deviations of R and Q (R_0, σ_R, Q_0 and σ_Q) we can define a new function:

$$Y = R - Q > 0 \text{ or } Y = R/Q > 1 \text{ safety margin} \tag{2.36}$$

with mean

$$Y_0 = R_0 / Q_0 \tag{2.37}$$

and standard deviation

$$\sigma_Y = \left(\sigma_R^2 + \sigma_Q^2\right) \tag{2.38}$$

This is plotted in Figure 2.8. The shaded portion of this figure represents cases where R - Q < 0, in other words, where failure occurs. The probability of failure is the probability that R - Q < 0, indicated by the shaded portion of the curve. If the type of distribution is known, the probability of failure can be computed from the number of standard deviation by which Y_0 exceeds zero. This is shown in Figure 2.9 as $\beta\sigma_Y$, where β is referred to as the safety index:

$$\beta = Y_0 / \sigma_Y \tag{2.39}$$

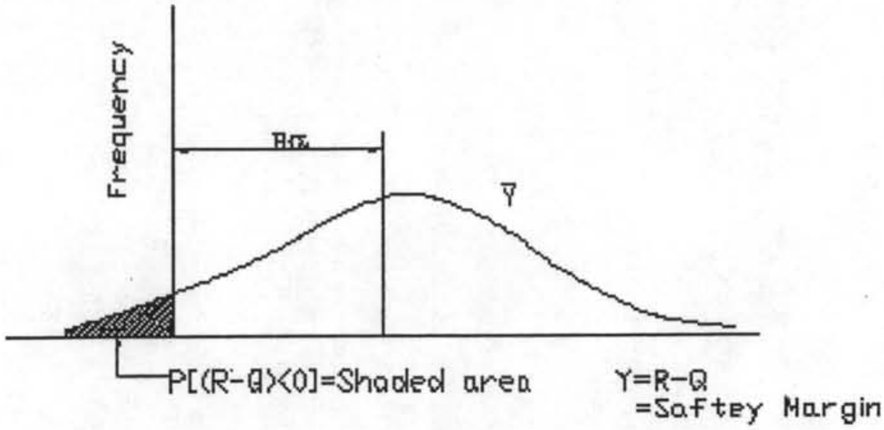


Figure 2.9 Definition of failure [21].

If β is increased by increasing the value of Y_0, the shaded area in Figure 2.9 is reduced and the probability of failure is reduced. Thus β is a measure of the reliability of a structural member, ϕ_c and ϕ_s are determined by the reliability target, β_T. Literature suggests that a desirable range of β_T values for safety is from 3.0 to 3.5. A target reliability index, β_T, of 3.00 to 3.25 for ductile failure and 3.25 to 3.5- for brittle failure was selected. These β values apply to loads that are defined at a probability of exceedance of 0.033 and for normal structures with a design life of 50 years [9].

Using the equation from MacGregor (1976) β is shown in a practical format:

$$\gamma_R = R_o / R \tag{2.40}$$

$$\gamma_U = U_o / U \tag{2.41}$$

Where R and U are design strength and design loading respectively

$$R\gamma_R\left[e^{-\beta\alpha V_r}\right] \geq U\gamma_U\left[e^{\beta\alpha V_u}\right] \tag{2.42}$$

Rearranging this gives,

$$\beta = \ln\left(R\gamma_R / U\gamma_U\right) / \left(\alpha V_R + \alpha V_Q\right) \tag{2.43}$$

where α is a "separation function" having value 0.75 (Lind 1971); V_R and V_Q are the coefficients of variation of R and Q [9].

The format of partial safety factors separates the influences of uncertainties and variabilities originating from different causes by means of design values assigned to basic variables. The design condition is expressed in terms of design values, e.g. as

$$g(F_d, f_d, a_d, \theta_d, C, \gamma_n) \geq 0 \tag{2.44}$$

where F_d are design values of actions; f_d are design values of material properties; a_d are design values of geometrical quantities; θ_d are design values of the variables θ which account for the model uncertainties; C are serviceability constraints; γ_n is a coefficient by which the importance of the structure and the consequences of failure, including the significance of the type of failure, are taken into account. The value γ_n could be made dependent on the specified degree of reliability of the actual structure or structural element.

Equation 2.44 should be regarded only as a symbolic description of the principles. Each symbol in equation 2.44 may represent a single variable or a vector containing several variables [10].

The basic variables are separated into:

- primary basic variables, and
- other basic variables.

The primary basic variables are those whose values are of primary importance for the design results. They should be specified in those codes which treat actions and structures of specific materials.

The design values of the primary basic variables F, f, a and θ are obtained in the following way:

$$F_d = \gamma_f F_r \tag{2.45}$$

$$f_d = \frac{f_k}{\gamma_m} \tag{2.46}$$

$$a_d = a_k \pm \Delta_a \tag{2.47}$$

$$\theta_d = \gamma_D \text{ or } 1/\gamma_D \tag{2.48}$$

where F_r are the representative values of actions; f_k are characteristic values of material properties; a_k are characteristic values of geometrical quantities; γ_f are partial factors for actions; γ_m are partial factors for materials; Δ_a are additive geometrical quantities; γ_D are partial factors for model uncertainties.

γ_f takes account of:

- the possibility of unfavorable deviations of the action values from the representative values, and
- the uncertainty in the action model.

γ_m takes account of:

- the possibility of unfavorable deviations of the material properties from the characteristic values, and
- uncertainties in the conversion factors.

Δ_a takes account of:

- the possibility of unfavorable deviations of the geometrical parameters from the characteristic (specified)
- values including the importance of variations in a, the tolerance specifications for a and the control of the deviations from a; and
- the cumulative effect of a simultaneous occurrence of several geometrical deviations.

γ_D takes account of uncertainties of models as far as can be found from measurements or comparative calculations.

For basic variables other than the primary ones, the partial factors are, a priori, set to unity and the additive quantities to zero; i.e. the design values are equal to the characteristic values. In some cases mean values may be used.

The partial factors for actions may include the effect of uncertainties of an action effect model. In a similar way the partial factors for resistance may include the effect of uncertainties in the geometrical parameters and in the resistance models. In such cases the notations γ_f and γ_m should be substituted by γ_F and γ_M respectively.

The values of the partial factors depend on the design situation and the limit state considered. If the deformation capacity is governing the design, Equation 2.44 has to be given a different form and part of the variables have to be substituted by other kinds of variables. This may, for example, be the case in design for seismic situations.

In many cases the basic variables and the factors θ which describe the uncertainties of the calculation models can be separated into groups so that some groups give action effects:

$$S(F, f, a, \theta_S) \qquad (2.49)$$

and other groups give resistances:

$$R(F, f, a, \theta_R) \qquad (2.50)$$

In the expression for S, the material properties, f is a primary basic variable only in special cases, for example, calculations according to a second-order theory. In the expressions for R, the actions F are of importance only in very special cases.

Thus design values, S_d and R_d, can be defined as:

$$S_d = S(F_d, f_d, a_d, \theta S_d) \qquad (2.51)$$

$$R_d = R(F_d, f_d, a_d, \theta R_d) \qquad (2.52)$$

Then equation 2.30 can be written

$$g(S_d, R_d) \geq 0 \qquad (2.53)$$

As for equation 2.44, equation 2.53 should be regarded only as a symbolic description. Each symbol S and R may represent several action effects and resistances respectively. In the simplest case, equation 2.53 can be written

$$R_d \geq S_d \tag{2.54}$$

Equations 2.539 and 2.54 can be applied in the ultimate limit state and the serviceability limit state. For the serviceability limit state, concerning, for example, deflections, the design condition is often of the type

$$S_d \leq C \tag{2.55}$$

where C is a serviceability constraint, for example, acceptable deflection.

Let assume that the limit state considered can be specified by a calculation model in terms of one (or several) function(s) g(...) of a set of variables $X_1, X_2, \ldots, X_n$, comprising actions, material properties, etc., so that a condition for the structure not to fail of the form $g(X_1, X_2, \ldots, X_n) \geq 0$ can be associated with the limit state. The design requirement may then be written as

$$g(x_{1d}, x_{2d}, \ldots, x_{nd}) \geq 0 \tag{2.56}$$

where $x_{1d}, x_{2d}, \ldots, x_{nd}$ are design values.

The design value of x_{id} of variable X_i depends on

- the parameters of variable X_i
- the assumed type of distribution
- the target safety index β for the limit state and design situation of concern
- a factor α_i describing the sensitivity in X_i with regard to attaining the limit state, according to the definition given in First Order Reliability Method (FORM) calculation.

2.3.1 Partial Safety Factor Method Based on Design Values

In design codes, design values x_d are not introduced directly. Random variables are first introduced by means of representative values x_k. In addition, there is a set a partial safety factors and load combination factors. In most cases the basic requirement can be formulated as [10]:

$$g(x_d) = R_d - S_d \geq 0 \tag{2.57}$$

with:

$$S_d = S(F_d, a_d, \theta_d \ldots) \tag{2.58}$$

$$R_d = R(f_d, a_d, \theta_d \ldots) \tag{2.59}$$

Here S is the load effect, and R is the corresponding resistance, with:

$F_d = \gamma_f F_k$ or $F_d = \gamma_f \psi_0 F_k$ = design value of a load parameter

$f_d = f_k/\gamma_m$. = design value of a material property

$\underline{a}_d = a_{nom} \pm \Delta_a$ = design value of geometrical property

$\underline{\theta}_d$ is the design value of a model factor

The index k denotes characteristic value.

The design value θ normally enters the equations by means of partial factors γ_{Sd} and γ_{Rd} for the total model, such that:

$$S_d = \gamma_{Sd} S(\gamma_f F_k, \gamma_f \psi_0 F_k, a_{nom} \pm \Delta_a \ldots) \tag{2.60}$$

$$R_d = \frac{1}{\gamma_{Rd}} R\left(\frac{f_k}{\gamma_m}, a_{norm} \pm \Delta_a \ldots\right) \tag{2.61}$$

$$\gamma_f = F_d/F_k,\ \gamma_m = f_k/f_d \tag{2.62}$$

The procedure described above is cumbersome from a practical point of view. Therefore, the following simplifications are often made:

$$\text{on the loading side: } \rightarrow S_d = S(\gamma_F F_k, a_{nom}) \tag{2.63}$$

on the resistance side: $\rightarrow R_d = R\left(\frac{f_k}{\gamma_M}, a_{nom}\right)$ or $R_d = \frac{1}{\gamma_R} R(f_k, a_{nom})$ (2.64)

In this case γ_F and γ_M (or γ_R) should be calibrated in such a way that they result in the same values as the original equations.

2.3.2 Partial Safety Factor Based on Calibration

In the procedure outlined in 2.3.1, the Partial Factor method is introduced as an elaboration of the Design Value method. An alternative method is to start with some arbitrary partial factor format and to require that the partial factors are chosen in such a way that the reliability of the resulting structures is as close as possible to some selected target value [10].

Assume the partial factor format can be written as:

$$g\left(\frac{f_{k1}}{\gamma_{m1}}, \frac{f_{k2}}{\gamma_{m2}}, \ldots, \gamma_{f1} F_{k1}, \gamma_{f2} F_{k2}, \ldots\right) \geq 0 \qquad (2.65)$$

where f_{ki} is the characteristic strength of material i; γ_{mi} is the partial factor for material i; F_{kj} is the representative value for load j; γ_{fj} is the partial factor for load j.

Now, define a representative set of n test elements, which should be chosen to cover adequately the scope of application of the code in terms of:

- types of actions
- types of structural dimensions
- types of materials
- types of limit states

For a given set of partial factors (γ_{m1}, γ_{m2} ... γ_{f1}, γ_{f2} ...) the set of representative structural elements can be designed. Each element will then possess a level of reliability which will deviate more or less from the target value. Using the reliability index β, the aggregate deviation D can be expressed as:

$$D = \Sigma\left[\beta_k(\gamma_{mi}, \gamma_{fj}) - \beta_t\right]^2 \tag{2.66}$$

β_t is the target value of β

$\beta_k = \beta$ for element k as a result of a design using (γ_{m1}, γ_{m2}, γ_{f1}, γ_{f2})

Clearly, the set of partial factors which minimizes this aggregated deviation D can be considered as the best set of factors. If not all elements are considered of equal importance, weight factors may be introduced. Instead of β, one may also use the probability of failure itself. It may be realistic to penalize values smaller than the target probability to a lesser degree than values exceeding the target. One may also try to optimize the economic criteria for a wide set of representative structural elements.

2.3.3 Procedure on Estimating Global Partial Safety Factor γ_M

Procedure on calculating γ- values based on FIB Bulletin No. 202 "Reliability of Concrete Structures" [33] explained as follows.

The design criterion is assumed to have relatively simple general form

$$\varsigma_S(m \cdot G + n \cdot Q) \leq R \tag{2.67}$$

where G is permanent action

Q is variable action

m and n are coefficients which transform action values to action effect values

ζ_S is a coefficient which describe the variability of the calculation model for the load effect

R is resistance

G, Q, ζ_S and R are stochastic variables, m and n are deterministic values. In general, ζ_S may be different for G and Q but here it is assumed to be equal for G and Q.

It is assumed that the resistance can be described by the relation

$$R = \varsigma_R \cdot a \cdot \eta \cdot f \tag{2.68}$$

where ζ_R is a coefficient which describe the variability of the calculation model for the resistance

a is a geometric quantity, for example, thickness of a plate, cross section area, moment of inertia etc.

η is a conversion factor which transform the strength of control test specimens (cylinder, cube etc) to the strength in the structure.

f is the strength of control test specimens or other strength values derived from the strength of such specimens

ζ_R, a, η and f are all assumed to be stochastic variables.

To simplify the equation the notation

$$\zeta = \frac{\zeta_R}{\zeta_S}$$

is introduced. ζ_S and ζ_R are assumed to be independent and thus

$$\mu_\zeta = \frac{\mu_{\zeta R}}{\mu_{\zeta S}}$$

$$V_\zeta^2 = V_{\zeta R}^2 + V_{\zeta S}^2$$

In the method of partial coefficient the formulation of the design criteria (2.67) and (2.68) will be

$$m \cdot \gamma_G \cdot G_k + n \cdot \gamma_Q \cdot Q_k \le \zeta_k \cdot a_k \cdot \eta_k \cdot \frac{f_k}{\gamma_M} \tag{2.69}$$

where G_k, Q_k, a_k, and f_k are characteristic values. $\zeta_k = \zeta_{Rk}/\zeta_{Sk}$ where ζ_{Rk} and ζ_{Sk} are characteristic values. Thus ζ_k should not be regarded as a characteristic value. η is not introduced with a characteristic value but with a fixed value. γ_G and γ_Q are assumed to be known partial coefficient.

In the probabilistic method the formulation of the design criteria (1) and (2) will be

$$m \cdot G^* + n \cdot Q^* \le \zeta^* a^* \eta^* f^* \tag{2.70}$$

where the general expressions for the design values G^* etc are

for actions: $X^* = \mu_x(1 - \alpha_x \beta V_x)$ (2.71)

X is G and Q

for resistance: $Y^* = \mu_y \exp(\alpha_y \beta V_y)$ (2.72)

Y is R, ζ, a, η, and f

where μ are mean values

V are coefficient of variations

α are sensitivity factors

β is the reliability index

The reliability index β is related to the acceptable formal probability of failure through the equation

$$\in = \phi(-\beta) \tag{2.73}$$

where $\phi(.)$ is the Gaussian probability distribution function.

The parameters of the problem are:

- The coefficient of variation

V_G, V_Q, V_ζ, V_a, V_η and V_f

- A Lumber of values describing the relation between the characteristic values (and the fixed value η_k of η) and the mean values ($G_k = \lambda_G \cdot \mu_G$ etc):

λ_G, λ_Q, λ_ζ, λ_a, λ_η, and λ_f

- the ratio ν between the effects of the mean values of the variable action and the permanent action

$$\nu = \frac{n\mu_Q}{m\mu_G}$$

A small ν means dominating permanent action.

- the reliability index β.

- the partial coefficient γ_G, γ_Q and γ_M.

If the parameters, mentioned above, except γ_M are known the value $\xi = \mu_R/\mu_G$ can be calculated from eq. (2.70). With this value of ξ, γ_M can be calculated from eq. (2.69). If β is the unknown parameter the way of calculation is the opposite.

2.4 Variables on Reliability Analysis of Reinforced Concrete Structures

2.4.1 Characteristics of Material and Member Dimension

The basic variables that influence the resistance of the component are concrete strength in compression and tension, yield strength of the reinforcement, and dimensions of the cross section. In quantifying the variability of the resistance, several important assumptions must be noted:

- The variability of the material properties and dimensions correspond to the "average" quality of construction expected in practice.
- Material strengths are assumed to be representative of relatively slow loading rates for dead, live and snow or rain loads.
- Long-term strength changes of the concrete and steel (due to increasing maturity of the concrete, reduction strength through fatigue, and possible future corrosion of the reinforcement) are ignored.

For concrete elements used in buildings, the statistical parameters for material properties and dimension are summarized in Table 2.4. Table 2.5 summarizes the statistical parameter for resistance for reinforced concrete beams and columns and prestressed elements.

Table 2.4 Statistical parameter of material properties and dimension [4]

Property	Mean value*	Coefficient of Variation	Standard Deviation*†
	Concrete compressive strength		
f_c'= 3000 psi (20.70 Mpa)	2760 psi (19.04 Mpa)	0.18	-
f_c'= 4000 psi (27.60 Mpa)	3390 psi (23.39 Mpa)	0.18	-
f_c'= 5000 psi (34.50 Mpa)	4028 psi (27.79 Mpa)	0.15	-
	Concrete tensile strength		
f_c'= 3000 psi (20.70 Mpa)	306 psi (2.11 Mpa)	0.18	-
f_c'= 4000 psi (27.60 Mpa)	339 psi (2.34 Mpa)	0.18	-
f_c'= 5000 psi (34.50 Mpa)	366 psi (2.53 Mpa)	0.18	-
	Reinforcement		
Grade 40 yield	45.3 ksi (312.57 Mpa)	0.116	5.3 ksi
Drade 60 yield	67.5 ksi (465.75 Mpa)	0.098	6.6 ksi
Grade 270 prestressing strand tensile strength	281 ksi (1938.90 Mpa)	0.025	7.0 ksi
	Errors in Dimension†		
Overall nominal depth of slab	+0.03 to +0.21 in (+0.76 to +5.33 mm)	-	0.26 to 0.47 in (6.60 to11.94 mm)
Overall nominal depth of beam	-0.12 to +0.81 in (-3.048 to 20.57 mm)	-	0.25 to 0.55 in (6.35 to 13.97 mm)
Effective depth, one way slab, top bars	-0.4 in (10.16 mm)	-	0.50in (12.70 mm)
Effective depth, one way slab, bottom bars	-0.13 in (3.30 mm)	-	0.35 in (8.89 mm)
Effective depth of beam, top bars	-0.22 in (5.59 mm)	-	0.53 in (13.46 mm)
Nominal width of beam stem	+0.10 in (+2.54 mm)	-	0.15 in (3.81 mm)
Nominal column dimensions	+0.06 in (1.52 mm)	-	0.25 in (6.35 mm)
Cover, bottom steel in beams	-0.35 to 0.06 in (-8.89 to 1.52 mm)	-	0.28 to 0.45 in (7.11 to 11.43 mm)

1 ksi = 6900 Pa; 1 in = 25.4 mm

† A range of values is presented in some instances because data from multiple sources were used. See Ellingwood et.al., (1980) for details

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

Table 2.5 Statistical parameter of concrete beams and columns [4]

Action	Type of Member	Details*	λ_R	V_R
Felxure (R/C)	One-way slab	5 in (12.7 mm) Grade 40	1.22	0.16
		5 in (12.7 mm) Grade 60	0.21	0.15
	Two-way slab	5 in (12.7 mm) Grade 40	1.16	0.15
		5 in (12.7 mm) Grade 60	1.12	0.14
	Beams†	Grade 40, f_c' = 5 ksi (34.5 Mpa)	1.14-1.18	0.14
		Grade 60, f_c' = 5 ksi (34.5 Mpa)	1.01-1.09	0.08-0.12
	Overall		1.05	0.11
Flexure (Prestressed)	Plant precast pretensioned†	-	1.04-1.06	0.057-0.097
	Overall, precast		1.06	0.08
	Cast-in-place, posttensioned†	-	1.02-1.05	0.061-0.144
	Overall, posttensioned		1.04	0.095
Axial + flexure	Short column, composite failure	f_c' = 3 ksi (20.7 Mpa)	1.05	0.16
		f_c' = 5 ksi (34.5 Mpa)	0.95	0.14
	Short column, tension failure	f_c' = 3 & 5 ksi (20.7 &34.5 Mpa)	1.05	0.12
	Slender column, composite failure	f_c' = 3 ksi (20.7 Mpa)	1.10	0.17
	Slender column, tension failure	f_c' = 5 ksi (34.5 Mpa)	0.95	0.12
Shear	Beams†		0.93-1.09	0.17-0.21

* ksi = 6900 Pa; 1 in = 25.4 mm.

† Various reinforcing details were considered. See Ellingwood et., all. (1980) for details

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

Table 2.4 shows that the mean and coefficient of variation of compressive and tensile strength of concrete generally depend on the specified compressive strength f_c' of the mix. The nominal compression strength is calculated as $0.85 f_c'$. The tensile strength of concrete is significantly less than its compressive strength. Moreover it turns out that the standard deviation for dimension is roughly independent of the beam size. Therefore, the coefficient of variation tends to be smaller for larger beam sizes.

Nowak and Szersen in 2003 [16] published the new characteristics of material and dimension-related characteristics of reinforced structural members. The materials surveyed include ordinary ready-mix concrete, ordinary plant-cast concrete, high-strength concrete and light-weigh concrete. Characteristic on reinforcement related to yield strength and bar diameter. Some of the results related to this research presented as follows;

Table 2.6 Statistical parameters for ordinary ready-mix concrete [16]

f_c' kpa (psi)	Number of samples	Mean f_c' kpa	V	λ
20,670 (3000)	88	27,970	0.102	1.35
24,115 (3500)	25	29,214	0.079	1.21
27,560 (4000)	116	34,310	0.145	1.235
31,005 (4500)	28	35,310	0.042	1.14
34,450 (5000)	30	39,480	0.058	1,15
41,340 (6000)	30	46,163	0.042	1.12

Table 2.7 Statistical parameters for reinforcing steel, Grade 420 Mpa (60 ksi) [16]

Bar size	Number of samples	Mean f_y Mpa	V	λ
9.5 mm (N0.3)	72	496.1	0.04	1.2
12.5 mm (N0.4)	79	473.3	0.065	1.145
15.5 mm (No.5)	116	465.1	0.04	1.125
19 mm (No.6)	38	476.1	0.05	1.15
22 mm (No.7)	29	481.6	0.05	1.165
25 mm (No.8)	36	473.7	0.05	1.145
28 mm (No.9)	28	475.7	0.05	1.15
31 mm (No.10)	5	470.2	0.04	1.14
34.5 (No.11)	13	473.7	0.035	1.145

From the data surveyed presented on Table 2.6 and Table 2.7 of all concrete materials [16], the average coefficient of variation of f_c' is V = 0.101, therefore, in the calibration, Szersen and Nowak taken V = 0.10 used for all considered types and grades of concretes. Furthermore, based on the findings, the equation for bias factor recommended for ready-mixed, plant cast, high-strength, and lightweight concretes can be calculated from following formula

$$\lambda = -0.0081 \times f_c'^3 + 0.1509 \times f_c'^2 - 0.9338 \times f_c' + 3.0649 \quad (2.74)$$

As for reinforcement, Szersen and Nowak [16] recommended bias factor for f_y is $\lambda = 1.145$ and the recommendation coefficient of variation of f_y for calibration on that study is $V = 0.05$. For comparison, the bias factor for f_y used in previous studies as shown in Table 2.7 was $\lambda = 1.125$, and coefficient of variation, $V = 0.10$ [16]. The study also reports the survey on fabrication factor, F. Fabrication factor represents the variation in dimension and geometry of the considered structural elements. The recommended statistical parameters are based on studies by Ellingwood et al. (1980). For the dimension of concrete components, the recommendation parameters are listed in Table 2.8.

Table 2.8 Statistical parameters of fabrication factor for dimension of concrete in USA [17]

Item	V	λ
Width of beam, cast-in-place	0.04	1.01
Effective depth of reinforced concrete beam	0.04	0.99
Effective depth of prestressed concrete beam	0.025	1.00
Effective depth of slab, cast-in-place	0.12	0.92
Effective depth of slab, plant-cast	0.06	1.00
Effective depth of slab, post-tensioned	0.08	0.96
Column width and breadth	0.04	1.005

2.4.2 Statistical Parameter of Resistance

Nowak and Szersen [16] also report the statistical parameters of reinforced concrete members in USA. The study used deterministic expressions for resistance following the code. Statistical parameters of resistance were calculated by Monte Carlo simulations, using the statistical parameters determined for material and fabrication factor. The report stated that the data on concrete compressive strength used in the study was obtained from different sources (from different construction sites and/or from different concrete mixture plants), so it includes the so-called batch-to-batch variation, which is higher than the within-test variation. Furthermore, it also stated that the investigated data also include the variation caused by different testing methods (data come from different labs) and event different concrete mixture and design ingredients. The resulting statistical parameters are given in Table 2.9 which is only show the result for ordinary concrete.

Table 2.9 Statistical parameters of resistance for ordinary concrete [16]

Structural type and limit state	Old material data		New material data	
	λ	V	λ	V
R/C beam cast-in-place, flexure	1.114	0.119	1.190	0.089
R/C beam plant-cast, flexure	1.128	0.133	1.205	0.081
R/C beam cast-in-place, shear	1.159	0.120	1.230	0.109
R/C beam plant-cast, shear	1.170	0.116	1.242	0.105
P/S beam plant-cast, flexure	1.034	0.081	1.084	0.073
P/S beam plant-cast, shear	1.130	0.105	1.194	0.103
R/C slab cast-in-place	1.052	0.169	1.077	0.146
RC Slab, plant-cast	1.146	0.116	1.174	0.082
P/S slab plant-cast	1.053	0.070	1.075	0.070
Post-tensioned slab cast-in-place	0.961	0.146	0.982	0.145
R/C column cast-in-place, tied	1.107	0.136	1.260	0.107
R/C column plant-cast, tied	1.102	0.134	1.252	0.103
R/C column cast-in-place, spiral	1.163	0.124	1.316	0.097
R/C column plant-cast, spiral	1.156	0.122	1.323	0.091
P/S column plant-cast, tied	1.017	0.094	1.080	0.090
P/S column plant-cast, spiral	1.068	0.076	1.133	0.071
Plain concrete, flexure, shear	1.004	0.082	1.105	0.082

The report by Nowak and Szersen [16] conclude that the comparison with previous tests (1970) confirmed that there is an improvement in quality of materials; in particular, it is observed that variation of strength is reduced. The most significant difference between the older data and recent results is in the strength of concrete and yield strength of steel reinforcing bars. It was observed that the safety margin in strength of concrete, in terms of bias factor (ration of mean to nominal value), decreases for higher values of strength. Previously, result study by Ellingwood et al (1980) [18] on the statistical resistance of reinforced concrete members shown in Table 2.10.

Study by Amatayakul [37] for conditions in Thailand, the actual yield strength of reinforcing steel is 15-30% higher than the specified value. The average concrete compressive strength of class equal and above 200 kg/cm^2 is about 20 kg/cm^2 less than specified value. The member cross section tend to about 0.2 – 0.5 cm larger than those shown on the construction drawings. The effective depths are 0.1 – 1.8 cm less than the specified values.

Furthermore, Amatayakul [37] reported that the capacity reduction factors obtained from reliability analysis were found to be 0.77 – 0.85 for sehar. By considering the materials and construction qualities in Bangkok area, the capacity reduction factors of 0.80, 0.65, and 0.80 may be appropriate for flexure, compression and shear, respectively, without considering the load factors.

Table 2.10 Statistical parameter for resistance of reinforced concrete members [18]

Limit state	λ	V	Distribution
Flexure ($f_c' = 5$ ksi, grade 60 steel)			
$\rho = 0.31\rho_b$	1.09	0.11	Lognormal
$\rho = 0.73\rho_b$	1.01	0.12	Lognormal
Axial and flexure (short columns)			
compression failure ($f_c' = 3$ ksi)	1.05	0.16	Lognormal
tension failure ($f_c' = 3$ or 5 ksi)	1.05	0.12	Lognormal
Shear (beams with a/d > 2.5)			
minimum stirrups	1.00	0.19	Lognormal
$\rho_v f_v = 150$ psi	1.09	0.17	Lognormal

2.4.3 Characteristics of Loadings

Types of Load

Loads of many types act on structures. These loads can be classified into three categories based on the types of statistical data that are available and the characteristics of the load phenomenon [4]:

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

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

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

Load Models

Szersen and Nowak [17] considered load components include deal load, Live load, snow, wind and earthquake. The statistical parameters for load components are taken from the available literature (Ellingwood et al. 1980; Ellingwood and Rosowsky 1996; and Nowak 1999). For each load combination, the statistical parameters are determined using so-called Turkestra's rule (Turkestra 1970; Turkestra and Madsen 1980; and Nowak and Collins 2000). Turkestra observed that the extreme value of load combinations corresponds to the occurrence of an extreme value of only one load component, while all the other load components take the corresponding average (arbitrary-point-in-time) values. The load statistical parameters are summarized in Table 2.11. The parameters shown are bias factor and coefficient of variation.

Table 2.11 Statistical parameters for load combinations [8] [17]

Load component	Arbitrary-point-in-time load		Maximum 50-year load	
	Bias λ	Coef. of Variation V	Bias λ	Coef. of Variation V
Dead load (cast-in-place)	1.05	0.10	1.05	0.10
Dead load (plant-cast)	1.03	0.08	1.03	0.008
Live load	0.24	0.65	1.00	0.18
Snow	0.20	0.87	0.82	0.26
Wind	0.00	0.00	0.78	0.37
Earthquake	0.00	0.00	0.66	0.56

On the loading aspects, Indonesia has it loading code The Indonesian Loading specification for houses and buildings SNI 03-1727-1989. The distributed loads were similar from those stated in ASCE 7-02. The live load for buildings in Indonesia listed in Table 2.12 below.

Table 2.12 Minimum floor uniformly distributed load for buildings in Indonesia [35]

Occupancy or use	load (kg/m^2)
a. Floor, stairs for normal housing	200
b. Floor and stairs for temporary housing, warehouse (not for store, factory or workshop)	125
c. Classroom for school and college, office, store, shopping mall, restaurant, apartments and hospital	250
d. Gymnasium	400
e. Dancing room	500
f. Floor and balcony besides a. to e., for mosque, theatre, meeting room, cinema, and tribune for fixed seat	400
g. tribune for non fixed seat or standing spectators	500
h. Stairs, borders and corridor for c.	300
i. stairs, borders and corridor for d, e, f, and g	500
j. Complementary floor for c, d, e, f and g	250
k. Floor for factory, workshop, warehouse, library, document room, book store, steel store, equipment room, machine room and special function, minimum	400
l. Park building - lower/first floor - upper floors	 800 400
m. cantilever balcony has to be designed as the adjacent room minimum	300

2.4.4 Previous Research Related to Structural Reliability and Comparison of Partial Safety Factor from Several Codes

Structures should with appropriates degrees of reliability, during their construction and whole intended lifetime, perform adequately and more particularly withstand all actions and environmental influences, liable to occur, and withstand accidental circumstances without damage disproportionate to the original events (this is called the insensitivity requirement). In the performance-based design, each performance needs to be verified. Reliability analysis which based on probabilistic approach is one method on the verification process.

In principle a degree of reliability should correspond to a statistically determined rate of failures with regards to the whole set of requirements (for set of similar structures under similar conditions). For a given structure this rate should be represented by an assessed probability referable to a given period of time. In the present state of knowledge, degrees of reliability can only correspond to individual verifications and can be associated with nominal probabilities which do not account for gross errors; therefore, they do not represent any actual rat of failures. In this sub-chapter, the previous research related to structural reliability is presented.

Many researchers studied the performance of concrete structures. Both Ultimate limit state (ULS) and serviceability limit state (SLS) are important for design of structures. Reliabilities for ULS have been extensively researched. Target reliability indexes for different ULS have been calibrated and applied to different structural design codes in many countries.

Tabsh [19] studied the safety of reinforced concrete members designed following ACI 318 building code. The study was using both Chapter 9 and Appendix C load combinations and strength reduction factor of ACI 318 1995 edition. Safety is evaluated in term of reliability index, since load and resistance are random variables. The strength limit states considered in the study include flexure, shear, and combined axial compression plus flexure. The effect of low and high reinforcement ratios for beams in flexure is taken into account. Beam with minimum stirrups and with normal shear reinforcement are also considered. For the case of columns that subjected to axial load plus flexure, both compression and tension failures are included. The load variables considered in the limit state are dead load, live load, and wind effect. The reliability of typical members is determined for a range of love-to-dead load. The results of the study indicate that design based on Appendix C can result in more uniform safety than design based on Chapter 9 of the ACI 318-95. Research conclusion stated that for the usual range of loading and reinforcement, the reliability index is about 3.0 for flexure and shear, and range between 3.5 and 3.75 for combined compression and flexure. Furthermore, designs based on Chapter 9 of ACI 318-95 have significantly low values of reliability index (less than 2) for beams in flexure and subjected to a combination if D and W, with low L. This is not the case for designs based on Appendix C because of the high demand required in the range of $L/D < 1$.

Ellingwood and Galambos [20] presented the probability-based criteria for structural design. The criteria are based on a comprehensive analysis of statistical data on structural loads and resistances and an examination of levels of reliability implied by the use of design standards and specifications. The criteria are intended to be used in specifications that are oriented towards limit state design. The result of study regarding to prescribed value of reliability index β_0 corresponding to the resistance factors ϕ using *first order second moment* (FOSM) are shown in Table 2.13. The paper described how safety-related performance criteria for structural design can be selected using reliability analysis to integrate available statistical data on resistances and loads.

The development of practical of practical probability-based loading and resistance criteria that would be acceptable to the professional design community at large has five essential components: (1) Develop statistical data to describe the basic load and resistance variables; (2) Establish procedures for calculating reliabilities of structural members and systems. Ideally, the performance criteria should be based on a system reliability requirement. However, current practice usually is to check performance on the basis of individual member behavior; (3) Establish target reliabilities associated with structural members designed according to existing criteria. This enables the probability-based criteria to be related to existing acceptable practice and provides the continuity that is necessary from one design specification to the next; (4) select a deterministic format that balances theoretical appeal with the need for simple safety and serviceability checking procedures in professional practice. Determine general load criteria suitable for all construction materials; (5) Develop resistance criteria that are consistent with the load criteria selected in step 4 such that reliabilities are close to the target values selected in step 3 [20].

Table 2.13 Selection of resistance factors [20]

Member, Limit state	β_0	ϕ
Structural steel		
Tension Member, yield	3.0	0.83
Beams in flexure	2.5	0.89
Beams in flexure	3.0	0.78
Column, intermediate slenderness	3.5	0.75
Reinforced Concrete		
Beam in flexure	3.0	0.85
Beam in shear	3.0	0.70
Tied column, compressive failure	3.5	0.62
Masonry, unreinforced		
Wall in compression, uninspected	5.0	0.41
Wall in compression, uninspected	7.5	0.22

Qi [21] reported the study on resistance factors for reinforced concrete. The report reviews the concept of resistance factors in the limit state design. A brief comparison of resistance factor among Canada, United States and China is presented. Then the development of limit state design is introduced from an international academic view. Finally, a procedure is illustrated to deduce resistance factors in the 1995 National Building Code (NBC) of Canada.

The report [21] stated that the resistance factors in NBC of Canada have been developed progressively from being applied to the structural action to being applied to material strength directly. In the 1994 edition of the CSA A23.3 Standard, the resistance factor is applied to the material strength, ϕ_c, f_c' or ϕf_y, rather than to the nominal resistance, ϕM_n, ϕV_n etc, which are indicated in the latest American Concrete Institute (ACI) Building Code. However, the application of resistance factors in Canada is similar to the application in China. Three different factors are summarized in Table 2.14 to Table 2.16.

Table 2.14 Resistance factors in CSA Standard A23.3 [21]

Kind of material	Resistance factor ϕ
Concrete, regular, ϕ_c [1]	0.60
Concrete, precast elements manufactured and erected, ϕ_c	0.65
Reinforcing bars, ϕ_s [2]	0.85
Prestressing tendon, ϕ_p	0.90
Structural steel, ϕ_s	0.90

1. The factored concrete strengths used in checking ultimate limit state shall be taken as $\phi_c * f_c^{1/2}$, where f_c' is the compressive strength of concrete.
2. The factored force in reinforcing bars, tendons, and structural shapes shall be taken as the product of the resistance factor, $\phi_s f_y$ where f_y is the yield strength of reinforcement.

Table 2.15 Strength reduction factors in the ACI Code [21]

Kind of strength	Strength reduction factor ϕ
Flexure, without axial load	0.90
Axial load, and axial tension with flexure	
Axial tension, and axial tension with flexure	0.90
Axial compression, and axial compression with flexure	
Members with spiral reinforcement	0.75
Other members	0.70
Shear and torsion	0.85
Bearing on concrete	0.70

Note: the factored resistance used in checking ultimate limit states shall be taken as ϕM_n, ϕV_n etc, where M_n is the nominal moment and V_n is the nominal shear.

Table 2.16 Material subentry factors in the GBJ10-89[1] [21]

Kind of material	**Material subentry factor $1/\gamma$**
Concrete, compressed	0.74
Concrete, tensioned	0.74
Hot-roll steel, II, III, IV; cold drawing steel, I	0.91
Hot-roll steel, I	0.87
Cold drawing steel II, III, IV	0.83
Others[2]	

Note:
1. In building code of China, the strength design value is equal to the strength standard value divided by γ. The strength standard value is a characteristic value based on probability reliability.
2. The subentry factors of other material are detailed in GBJ10-89

Report conclusion [21] stated that the resistance factor of ACI slightly increases simplicity in application; the others can account for the effect of varying the portion of the load assigned to the steel and concrete. $\phi_s = 0.85$ and $\phi_c = 0.65$, which are higher than the value given in CSA A23-3-94 for cast-in-place concrete, could provide the necessary reliability in all cases of reinforced concrete.

Szersen and Nowak [17] reported the result of Calibration of design code for buildings (ACI 318), in the second part of the report study which is reliability analysis and resistance factors. Reliability indexes are calculated using various load combinations, the basic combinations of dead load and live load, and other combinations with snow, wind, and earthquake for the two resistance models considered (older database and new material database). The resulting values of the reliability index calculated for the old data base and load models, and potential consequences of failure, served as a basis for the selection of the target reliability index. For each type of structural element and load combination case, several possible values of the resistance factor were considered (rounded to the nearest 0.05). The target reliability levels based on the old material shown in Table 2.17 leads to recommended values as shown in Table 2.18.

Table 2.17 Selected target reliability indexes [17]

Structural type and limit state	Range of β	β_T
R/C beam cast-in-place, flexure	3.4 to 3.6	3.5
R/C beam plant-cast, flexure	3.2 to 3.4	3.5
R/C beam cast-in-place, shear	3.8 to 4.0	3.5
R/C beam plant-cast, shear	4.1 to 4.2	3.5
P/S beam plant-cast, flexure	4.2 to 4.4	3.5
P/S beam plant-cast, shear	4.3 to 4.4	3.5
R/C slab cast-in-place	2.3 to 2.5	2.5
RC Slab, plant-cast	3.8 to 3.9	3.5
P/S slab plant-cast	4.7 to 5.0	3.5
Post-tensioned slab cast-in-place	2.3 to 2.5	2.5
R/C column cast-in-place, tied	3.8 to 4.1	4.0
R/C column plant-cast, tied	3.9 to 4.2	4.0
R/C column cast-in-place, spiral	4.0 to 4.4	4.0
R/C column plant-cast, spiral	4.2 to 4.5	4.0
P/S column plant-cast, tied	5.0 to 5.3	4.0
P/S column plant-cast, spiral	5.8 to 6.2	4.0
Plain concrete, flexure, shear	5.7 to 6.2	4.0

Table 2.18 Recommended resistance factors [17]

Structural type and limit state	Resistance factor φ
R/C beam cast-in-place, flexure	0.90
R/C beam plant-cast, flexure	0.90
R/C beam cast-in-place, shear	0.85
R/C beam plant-cast, shear	0.85
P/S beam plant-cast, flexure	0.90
P/S beam plant-cast, shear	0.85
R/C slab cast-in-place	0.90
RC Slab, plant-cast	0.90
P/S slab plant-cast	0.90
Post-tensioned slab cast-in-place	0.90
R/C column cast-in-place, tied	0.75
R/C column plant-cast, tied	0.75
R/C column cast-in-place, spiral	0.80
R/C column plant-cast, spiral	0.80
P/S column plant-cast, tied	0.75
P/S column plant-cast, spiral	0.80
Plain concrete, flexure, shear	0.65

As stated previously that little research effort has been carried out on reliabilities for SLS. Perhaps this is mainly due to more complicated models of SLS and due to the difficulties encountered in defining serviceability failures, especially for reinforced concrete. Result studies from among these researchers will be summarized as follow.

Ellingwood [22] presented paper on probability-based criteria for serviceability limit states. According to several researches, excessive structural movement is a main cause of unserviceability in buildings. The paper stated that practical serviceability checking procedures can be developed using principles of probability, statistics and structural reliability, which resemble the criteria used for checking ultimate limit state. The load factors and load combination are different, because the consequence of attaining the serviceability limit state is not as severe. Building frames and floor system can be checked for structural motion using procedures that reflect the general dynamic characteristics of the structural system.

Stewart [23] carried out study on serviceability analysis of reinforced concrete structures, developing a probabilistic model to estimate immediate, creep, and shrinkage deflection, and using Monte Carlo simulation. The research was aimed to estimate deflection and probabilities of serviceability failure (lifetime and for each 8 year tenancy) for reinforced concrete beams sized according to the span-to-depth ratio serviceability requirements of Australian, British, and American Concrete Structures Codes. Serviceability failure is divined to occur when a deflection exceeds an allowable deflection limit as a result of flexure. The results suggested that the probabilities of serviceability failure are not consistent across a range of beam spans and the span-to-depth ratio serviceability requirements specified in Australian, British, and American concrete structures codes produce significantly different serviceability reliabilities. This is probably a consequence of the fact that the serviceability specifications of these codes have not been subjected to code calibration.

Quan and Gengwei [24] reported study on calibration of reliability index of RC beams for serviceability limit state of maximum crack width. Based on a group of tests on RC beams for maximum crack widths, model uncertainty in complicated expression for the limit state stipulated by China code for design of concrete beams was estimated. Formulations of the first-order reliability method for evaluating the reliability indexes for limit state were developed. Sensitivities of the reliability index to the ratio of variable load to permanent load, ratio of reinforcement, section depth, radius of longitudinal reinforcing steel, nominal cover, concrete cube strength, and allowable width limit were discussed. Research resulted that evaluated reliability index β for SLS of maximum crack width in the China structural code has values within the range 0.0-1.8, that is close to the recommendation $\beta = 0$ by ISO 2394 for reversible SLSs. That means that the provisions on crack width in RC beams in that code are acceptable. Furthermore, it showed that the reliability index for office building is higher that for residence buildings on the basis of China structural code.

For complete comparison of partial safety factors used in codes, the following are presented the partial safety factor from European and Japanese practices as shown in Table 2.19 and 2.20.

Table 2.19 Standard values for safety factors in Japan [15]

Limit states \ Safety Factor	Material factor, γm for concrete, γ_c	Material factor, γm for steel, γ_s	member factor γ_b	Structural analysis factor, γ_a	Load factor γ_f	Structure factor γ_i
Ultimate Limit State	1.3	1.0 or 1.05	1.1 ~ 1.3	1.0	1.0 ~ 1.2	1.0 ~ 1.2
Serviceability limit state	1.0	1.0	1.0	1.0	1.0	1.0
Fatigue Limit State	1.3	1.05	1.0 ~ 1.1	1.0	1.0	1.0 ~ 1.1

Table 2.20 Standard values for safety factors in Europe [12]

Partial factor γ_M of fundamental basic variable	with design situation: Persistent/transient	with design situation: Accidental	Action, γ_F	Unfavorable effect, γ_{sup}	Favorable effect, γ_{inf}
concrete			Permanent γ_G (exlude P)	1.35	1.0
Compressive strength, γ_c	1.5	1.2			
Tensile strength, γ_{ct}	see relevant clause	see relevant clause	Prestress γ_P	1.1	1.0
Reinforcing steel					
Tensile strength γ_s	1.15	1.0	Variable, γ_Q	1.5	Usually neglected
Compressive strength, γs_c	1.15	1.0			
Note: for serviceability verification the value of γ_M, γ_G, and γ_Q are taken to be 1.0 for fatigue loading, γ_M taken similar to strength of ULS, wile γ_G, and γ_Q are taken to be 1.1					

On the reliability of structures, ISO has been released document ISO 2394 General Principles on Reliability for Structures [10]. The document constitutes a common basis for defining design rules relevant to the construction and use of the wide majority of buildings and civil engineering works, whatever the nature or combination of the material used. The document of international standard is intended to serve as a basis for those committees responsible for the task of preparing national standards or codes of practice in accordance with the technical and economic conditions in a particular country, and which take into account the nature, type and conditions of use of the structure and the properties of the materials during its design working life.

ISO 2394 [10] also stated that it is important to recognize that structural reliability is an overall concept comprising models for describing actions, design rules, reliability elements, structural response and resistance, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one factor in isolation could therefore disturb the balance of reliability inherent in the overall concept. It is therefore important that the modification of any one factor should be accompanied by a study of the implications relating to the overall reliability concept. It is therefore important that the modification of any one factor should be accompanied by a study of the implications relating to the overall reliability concept.

In determining target reliability index, ISO 2394 suggest some values regarding the limit states. For serviceability, it recommend to use $\beta = 0$ for reversible and $\beta = 1.5$ for irreversible limit states. For fatigue limit state, it recommend to use $\beta =$ 2.3 to 3.1, depending on the possibility of inspection, and for ultimate limit state design, it recommend to use safety classes $\beta = 3.1$, 3.8 and 4.3. Typical value of β and the corresponding value of probability of failure P_f are shown in Table 2.21.

Table 2.21 Relationship between reliability index and probability of failure [10]

Reliability index	Probability of failure	Reliability index	Probability of failure
0.0	0.500	4.0	3.17×10^{-5}
0.5	0.309	4.5	3.40×10^{-6}
1.0	0.159	5.0	2.87×10^{-7}
1.5	6.68×10^{-2}	5.5	1.90×10^{-8}
2.0	2.28×10^{-2}	6.0	9.87×10^{-10}
2.5	6.21×10^{-3}	6.5	4.02×10^{-11}
3.0	1.35×10^{-3}	7.0	1.28×10^{-12}
3.5	2.33×10^{-4}	7.5	3.19×10^{-14}