CHAPTER 2 LOADS AND LOAD COMBINATIONS

2.1 Loads

Temperature load cannot be ignored in addition to loads for ordinary buildings, i.e., dead, live, snow, wind and earthquake loads and soil, hydraulic pressure, since larger buildings have been built and buildings with special use have been generalized recently. Therefore, these recommendations incorporate the temperature load since the previous version in 1993. Alphabetical symbols for each load are adopted in the main text in order to harmonize with international standards. Other loads includes load from mechanical equipment, which is not covered in the live load, vibration or impact load from them. Vibration when a building is being used, temporary load during construction of buildings may be taken into consideration, depending upon construction sites, size, use and the way of construction of buildings.

2.2 Basic Load Values

A load denoted by L may be, in general, described using both coefficients $C_1, C_2, \cdots, C_n$ and a physical quantity X as in the following.

$$L = C_1 \cdot C_2 \cdots C_n \cdot X \tag{2.2.1}$$

X is an random variable with inherent uncertainty related to natural phenomena and $C_k(k = 1, 2, \cdots, n)$ are also random variables whose uncertainty may be possibly reduced by accumulation of information, research findings etc. Examples of X are ground snow depth for snow load, a squared wind speed for wind load, peak ground acceleration on the engineering bedrock for earthquake load. C_k is a parameter such as the roof shape factor for snow load, wind pressure coefficient for wind load. Furthermore, analytical errors, errors associated with modeling, error due to insufficient statistical data may be taken into account.

When considering various loads in a design, it is possible to evaluate each load on a common basis, by using the common rule for determining X. A basic value of load L_r is now described as

$$L_r = \overline{C_1} \cdot \overline{C_2} \cdots \overline{C_n} \cdot X_r \tag{2.2.2}$$

where X_r is an r-year return period value of load X and $\overline{C_k}$ is either a most likely value for C_k from engineering judgment or a mean of C_k in case its statistical data is available.

The design load in these recommendations is basically a product of the basic value of load thus specified and load factors. Consequently, it is recommended that appropriate load factors be used not only for limit state design but also for allowable stress design and ultimate strength design. For application to allowable stress design, a return period conversion factor is introduced by which design return periods are modified.

Since 1993, the basic value of a load has been the 100-year return period value. For general acceptance, a maximum observed load is prefered as the basic value. It is easy to adopt the maximum-level event in the past for a design, since snow, wind and earthquake loads come from natural phenomena. As maximum levels, for example, the 1963 or 1981 great snow falls, the Muroto typhoon and the great Kanto earthquake can be used. It is more rational, however, to objectively evaluate load intensity on the basis of past many events or an ample statistical database, rather than a single past event. Although Poisson, Markov, and various extreme-value distribution models are possibly selected as a probabilistic model, it is preferable to evaluate load intensity with disregard to selected models. Then, it is concluded that it would be natural to specify the basic value of load as the value corresponding to a specified return period r (equal to the annual exceedance probability $1/r$).

For future diversification of design methods, it is desirable to adopt the same return period for snow, wind and earthquake loads. It is not so rare that the maximum level load in the past exceeds the 50-year return period value. In addition, earthquakes as great as the Kanto earthquake are said to occur once within 60 to 200 years. It seems to be reluctant to regard a 50-year return period value as a maximum level of load. Under this circumstance, attention to statistical data of annual maxima has been paid in the last forty years and the historical data has been accumulated, from which the 100-year return period value can be estimated without abrupt extrapolation.

It could be generally acceptable to some extent to set a value occurring once within 100 years as a "maximum level" value, considering the life of human being is 70 to 80 years. From the past experience of the Hanshin-Awaji disaster and the Tottori-ken seibu earthuake, it would be expected to have safer buildings in a modern Japan. From the above, it can be considered that the social environment is ready to accept the safety level based on the 100-year return period.

Note that the above discussion will not be made since dead load, live load and soil, hydraulic pressures are stationary load, conceptual compatibility is kept by adopting the same annual exceedance probability. In other words, the load value with 99 percent non-exceedance probability is adopted for dead load, live load, and soil, hydraulic pressures. This probability coincides with the probability that the annual maximum load does not exceed the 100-year return period value.

2.3 Load Combinations and Load Factors

2.3.1 Basis of load combinations

Regarding the load combination in these recommendations, the load combination specified in conventional design methods, the states where a few loads are applied such as "$G + P$", "$G + P + K$" and "$G + P + 0.35S + W$" are, respectively, described as normal state, earthquake state and strong wind state for high snow-prone region , and these descriptions imply the states when various loads are imposed simultaneously. These descriptions imply "states" and it would be inadequate to include a figure such as "0.35".

Considering the above shortcomings, "load combination" denoted in these recommendations is understood as "combination of load effects". Borrowing the description from the Japanese building standard law, "$G + P$" exactly implies combined stress in a structural member of interest in which both the stress due to the dead load "G" and that due to the live load "P" are applied simultaneously. If so, "$G + P$" no longer describes some state and it is meaningful to adopt the description with a figure.

Since "$G + P$" represent a linear sum of stresses or deformations, superposition of each load effect can be justified rigorously only in the case that a building behaves linearly against external loads. The stresses or deformations within a structural member under combined loads are equal to the ones which are obtained by applying the loads all at once when the superposition rule holds. Note that when a building is in the plastic range of response or the stress state changes due to different order of loading, the superposition of load effects does not hold.

1) Required performance level

It is not an easy task to determine design loads and intensity of combined loads. As is described in each chapter of these recommendations, each load can take various values; that is, they can be expressed in terms of probability distributions. In these recommendations, the basic value of each load is defined in such a way that the basic value is the value corresponding to a 100-year return period for time-varying loads, and it is the value of 99 percent non-exceedance probability for time-invariant loads. By this treatment, the basic value of a load is specified to achieve equality among different variable loads in a probabilistic sense. As will be shown in the next section, this basic value of a load can provide good reference for determining a design load, but cannot be a design load straightforward.

These recommendations clearly state that the magnitude of the design load should be determined according to the performance level required in the building to be designed. The design loads here are not "design loads" prescribed as minimum level in conventional standard or recommendations, but rather should reflect the use and importance and design service life of the building. It then follows that it would be nonsense that design loads already exist before the performance levels required for a building or parts of building are discussed. The higher the required performance level is, the larger the design load should be. The above way of thinking leads to building performance-clarified design ,i.e., performance-based design in short. Note that although it is widely practically used to adjust allowable criteria on a building capacity side under the constant design load, it is needed to relate with the required performance level.

How the required performance should be treated and which level should be taken does begin with clarification of the performance required for a building. The performance considered in a structural design is in general divided into safety and serviceability, but more sub-divided performance exists. Classification of performance will be studied and identified in the future. The most important issue herein is that building designers should be responsible to design a building with direct attention to the performance and to let building owners properly understand this performance.

The performance required for a building is diverse. How should the performance be quantified? What kinds of measure to represent degree of the performance should be desirable? The ordinary way of assuring the performance is that the degree of performance most often is expressed in terms of the building behaviour or physical state of a building under the specified design load. For example, the way is based on checking that a building does not reach the specified state under the predetermined design load. In this case, how the design load is determined is often out of concern. This checking procedure is quite simple; however, the background on how to determine the design load and what kind of performance can be assured are very vague. The design criteria that the stress in a structural member is less than the allowable stress does not clearly aim at the required performance of the member.

The above indicates that in a conventional design method, consistent design margin is not assured not only when the design load is specified, but also when the allowable capacity criteria is set. The limit state design is the method in which these design margin are allocated in an integrated manner, based on rational reasoning. The margins on the design load and on the capacity are quantified by means of the probability theory, and the required performance level is expressed in terms of allowable probability of failure or a target reliability index. Both load and capacity sides are equally treated. This treatment can facilitate to quantify the degree of limit state exceedance and eventually can help structural designers to explain the performance level of buildings.

The design loads are determined by selecting the return periods, as can be seen in Vision2000[1] and a draft of performance-based design in AIJ[2]. Table 2.3.1 shows the required performance matrix made by SEAOC, in which return periods from 75 to 475 years or equivalently, exceedance probabilities in 30 or 50 years are the key parameters to determine the earthquake loads. Some committees of AIJ have proposed earthquake design loads based on the return periods. The use of return periods for determining the design load may be somewhat related to design life of a building; the capacity criteria of parts or structural members are not directly linked with design limit states. Therefore, the performance level under the above procedure is still unclear.

Table 2.3.1 a required performance matrix for seismic design in Vision2000

Ground motion intensity		Damage level			
Frequency (Return period)	Exceedence probability	Negligible (Fully operational)	Light (Operational)	Moderate (Life safe)	Severe (Near collapse)
Frequent (43 yrs)	50% in 30yrs	●			
Occasional (72 yrs)	50% in 50yrs	■	●		
Rare (475 yrs)	10% in 50yrs	★	■	●	
Very Rare (970 yrs)	10% in 100yrs		★	■	●

2) Determination of performance level

A method of determining the magnitude of the design load or the performance level of a building is stated below. The performance level of a building should be determined by a building designer with great responsibility. Before determination, consensus between the building owner, users and the building designer has to be reached through extensive discussion. To do this, the possible state of the building under the specified loading state should be anticipated, and the use of a building, required state have to be clarified. Furthermore, the service life during which the building is to be used from the time of design is also an important parameter for determining the performance level. This is a trade-offs between the performance level and economical issues, and assessment of economy during the service life (so-called life-cycle cost management) will become more important, where the balance between the economy and the performance level should be studied. As one of the determination methods of the performance level, the reference value for the levels are often used by assessing the levels of buildings which follow the existing design standards or guidelines. This method is called “a code-calibration method” for existing design methods. In this chapter, reference levels are set by this method.

It is noted in the beginning of these recommendations that in an actual building design, it is illegal to adopt a performance level lower than the minimum level required in current building design standards. However, the recommendations would be extremely useful when a performance level greater than the one of the current design standard levels.

3) Loading state

The loads which are taken into consideration in a structural design are carefully selected by adequately judging the use of a building, building environment and site conditions. In these recommendations, using the term “loading state,” which is newly introduced, the state of exposure of a building or structural members, the structural design is carried out for each state.

The loading state can be defined either for structural members or a whole building. This is closely related to how the required performance is specified and how the performance is verified. When verifying the performance, the loading state has to be selected for the structural member according to the actual condition. To adequately satisfy the required performance, the corresponding cost is needed. Since there is no way of determination of the performance level with disregarding the economical consideration, the required performance level, as a trade-off relationship with economy, has to be clarified, and eventually rational performance levels from economic as well as performance perspective has to be sought.

2.3.2 Load combination for Limit State Design (LSD) format

This section discusses load combinations to be considered in Limit State Design. Although “limit state design” has broader meanings, it is considered as a reliability-based Limit State Design in this recommendation.

Except for dead load, which can be predicted fairly accurately based on a building’s weight, a great deal of uncertainty/variability exits in the intensity of loads such as live load, snow load, wind load, and earthquake load. These loads depend, for instance, on the size and the location of a building, when it is to be designed or constructed, etc.

Limit State Design is a fairly flexible structural design method. In Limit State Design, a limit state, that is the border between a favorable state and an unfavorable state of a building or a structural member, is explicitly determined. The performance of a building is measured quantitatively by accounting for the variability in structural model, resistance, strength of materials, etc., and thus, it can be controlled intentionally. The performance is often measured by a limit state probability, (P_f), the probability that a limit state is exceeded during a reference period, or a reliability index.

Limit states can be classified into two categories, a safety limit state and a serviceability limit state. A safety limit state is the border after which human lives are in danger. It includes an ultimate limit state such as building collapse, or partial collapse, due to a lack of load carrying capacity and/or ductility. A serviceability limit state is the border after which there exists some difficulty in daily use of the building. It includes damages that decrease inhabitability, durability, and/or appearance such as cracking, excessive deformation or vibrations that create discomfort of the occupants and affect non-structural members and equipment. It also includes the initiation of non-reversable behavior such as yielding of members of a building.

Recently, longer building service lives such as 100 years have been sought due to concerns about environmental issues, while temporal commercial buildings are also expected for economic reasons. The longer the service life of a building, the higher the possibility that the building is subjected to a larger loading and accordingly the larger the design load shall be. However, the concept of a service life is not explicitly considered in the past design method, the design load intensity for ordinary buildings are also employed for such buildings without considering the difference in design service life. Although there are some cases that design load is determined based on the return period, it is not clearly stated how the return period is determined considering the service life.

On the contrary, the limit state probability for a reference period is used as a quantitative measure of structural performance, and accordingly, a building can be designed considering the difference in the length of service life by alternating the reference period with the same target limit state probability.

Note here that the reference period is used to estimate the limit state probability: that is the probability of exceeding a limit state during a reference period. There are other “periods/lives” in structural design such as “service life,” “design service life,” and “(expected) return period.” These are the periods during which a building is physically in use, the service life assumed in design stage, and the mean interval that loading larger than a certain intensity occurs, respectively. These “periods” should be distinguished from one another. Also, it should be noted that either the same reference periods or the same limit state probabilities should be considered in the comparison of structural performance, as the limit state probability varies depending on the reference period.

A reliability index, β, is a measure of reliability for a considered limit state, and can be defined using limit state probability, P_f, as

$$\beta = \Phi^{-1}(1 - P_f) \tag{2.3.1}$$

in which $\Phi(\bullet)$ = the standard normal probability distribution function. Fig.2.3.1 illustrates the relation between a reliability index, β, and a limit state probability, P_f.

Knowledge of probability and statistics is required to estimate a limit state probability. However,

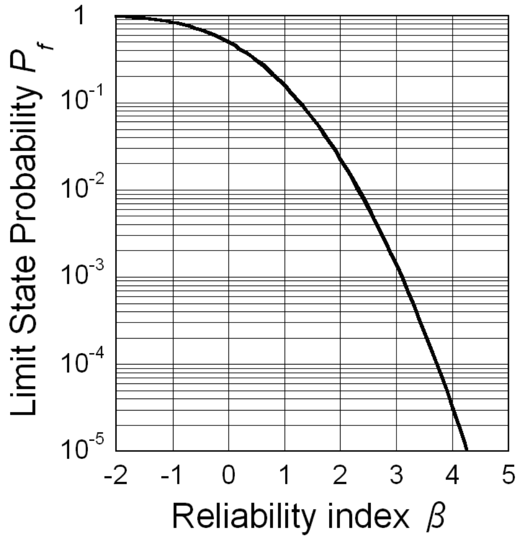

Fig.2.3.1 Relation between a reliability index, β, and a limit state probability, P_f

reliability-based Limit State Design can also be carried out in a similar manner to past deterministic design procedures by using load and resistance factors determined using probability and statistics by considering the variability in strength of materials and load intensity and the target performance level. Such design procedures have already been used in practice in North America and Europe [3][4][5]etc., and the following recommendations have already been published from AIJ in Japan: Standard for Limit State Design of Steel Structures (draft) (1990)[6], Recommendations for Limit State Design of Steel Structures (1998)[7], and Recommendations for Limit State Design of Buildings (2002)[8].

It has been recognized among researchers and engineers that load intensity should be treated as random. In Recommendation for Load on Buildings, wind load and snow load have been treated as random since the revision in 1981, and live load and earthquake load has been treated as random since the revision in 1993. The current revision also has a database of statistical characteristics for each load.

1) Load combination

The design requirement in Limit State Design is that the limit state probability of a building and/or a structural member during a reference period shall be equal to or smaller than the target (or acceptable) probability. The design format can be expressed generally as

$$P_f = \mathrm{Prob}\{g(\mathbf{X}) \leq 0\,;\; 0 < t \leq T_{\mathrm{ref}}\} \leq P_{f\mathrm{a}} = 1 - \Phi(\beta_{\mathrm{T}}) \qquad (2.3.2)$$

in which $\mathbf{X}$ = the vector of basic random variables, T_{ref} = the reference period, $P_{f\mathrm{a}}$ = maximum acceptable limit state probability, $\Phi(\bullet)$ = the standard normal probability distribution function, β_{T} = target reliability index, and $g(\mathbf{x})$ = the limit state function. A limit state function indicates the state of a building and/or a structural member; $g(\mathbf{x}) > 0$, $g(\mathbf{x}) < 0$, and $g(\mathbf{x}) = 0$ indicate that the building is in a favorable state, in an unfavorable state, and at the boarder (limit state) between the favorable and unfavorable states.

A building is subjected to more than one load processes which vary in time. Let's consider the combination of all loading (load combination) as follows:

$$G(t) + Q(t) + S(t) + W(t) + E(t) \tag{2.3.3}$$

in which $G(t)$,$Q(t)$, $S(t)$, $W(t)$, and $E(t)$ are load effect due to the dead load, G, live load, Q, snow load, S, wind load, W, and earthquake load, E, at time t. Other load effect due to such as temperature load shall be combined considering the site conditions.

A limit state function can be expressed as,

$$g(R, \mathbf{S}(t)) = R - \{G(t) + Q(t) + S(t) + W(t) + E(t)\} \tag{2.3.4}$$

in which R = resistance (either in strength or deformation) and $\mathbf{S}(t)$ is a vector consist of the load effect of each loading at time t under load combination. When the possibility of simultaneous occurrence of two loadings is small, one of them can be excluded from the load combination.

At a glance it seems cumbersome to evaluate Eq.(2.3.2) with Eq.(2.3.4). However, when the change of resistance in time can be neglected, the limit state probability can be simply expressed by Eq.(2.3.5).

$$P_f = \mathrm{Prob}\{R - \max_{0<t\leq T_{\mathrm{ref}}} \{G(t) + Q(t) + S(t) + W(t) + E(t)\} \leq 0\} \leq 1 - \Phi(\beta_{\mathrm{T}}) \tag{2.3.5}$$

The probability distribution of the maximum load combination during a reference period can be estimated directly [10, 11, 12]etc. or approximated by Turkstra's rule[9]. Turkstra's rule states that this maximum value can be approximately estimated as the sum of the maximum load effect of the principal load during the reference period and the arbitrary-point-in-time intensity of the other (secondary) load effects. Applying Turkstra's rule, the combination of time-variant loads can be treated as the combination of time-invariant loads as,

$$\max_{0<t\leq T_{\mathrm{ref}}} \left\{ \sum_{i=1}^{n} S_i(t) \right\} \approx S_{\mathrm{p}} + \sum_k S_k \tag{2.3.6}$$

in which $S_i(t)$ = the load effect of the i-th load (including both the principal load and the secondary loads) at time t, S_{p} = the maximum load effect during the reference period due to the principal load, and S_k = the annual maximum load effect due to the k-th secondary load.

Neglecting the cases that the possibility of coincidence is very small, the following load combinations are considered in design.

(1) When live load, Q, is the principal load: $G + Q$

(2) When snow load, S, is the principal load: $G + Q + S$

(3) When wind load, W, is the principal load: $G + Q + W$ (ordinary region), $G + Q + S + W$ (heavy snow region)

(4) When earthquake load, E, is the principal load: $G+Q+E$ (ordinary region), $G+Q+S+E$ (heavy snow region)

Note that it is not appropriate to apply Turkstra's rule to the cases when more than two load processes are intermittent or impulse. Also, the rule could provide erroneous estimates when the principal load is not dominant (approximately, when the standard deviation of the principal load $<$ the standard deviation of the sum of the secondary loads).

2) Load factors

In the implementation of Limit State Design, limit state probabilities can be evaluated directly; however, this is cumbersome in practice. Alternatively, the load and resistance factor format has been proposed. The load and resistance factors can be determined based on the target performance levels determined by the clients, users, and structural engineers. The design format employing load and resistance factors is expressed as,

$$\phi\, R_{\mathrm{n}} \geq \gamma_{\mathrm{p}}\, S_{\mathrm{pn}} + \sum_{k} \gamma_k\, S_{k\mathrm{n}} \qquad (2.3.7)$$

in which S_{pn} and $S_{k\mathrm{n}}$ are load effect due to the basic value of the principal load and the k-th secondary load, respectively, and γ_{p} and γ_k are the corresponding load factors.

It is difficult to evaluate directly the load factors for time-variant loads[12]. However, applying Turkstra's rule, the combination of time-variant loads can be approximated as the combination of time-invariant loads. The flowchart of the load factors are shown in Fig.2.3.2.

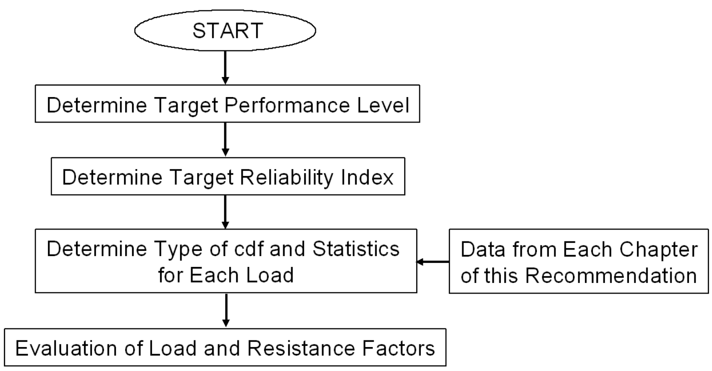

Fig.2.3.2 Flowchart for Load Factors

Fig.2.3.3 schematically illustrates the concept of load and resistance factors. In structural design, the nominal value of resistance R_{n} is determined so that the target reliability index is satisfied. If the ratio of the nominal value and the mean value is constant, the location of the joint probability density function of resistance R and load effect S, $f_{RS}(r,s)$, is determined adjusting $f_{RS}(r,s)$ in the horizontal direction so that the limit state probability, that is the volume of $f_{RS}(r,s)$ in the domain of $(r-s<0)$, shall be smaller than the maximum acceptable limit state probability. Once $f_{RS}(r, s)$ is fixed, the design point is determined as the most likelihood point of $f_{RS}(r, s)$ along the limit state surface $r - s = 0$, and the load factor and the resistance factor are defined as the ratio of s^* to the basic value of load S_n and the ratio of r^* to the nominal value of the resistance R_n.

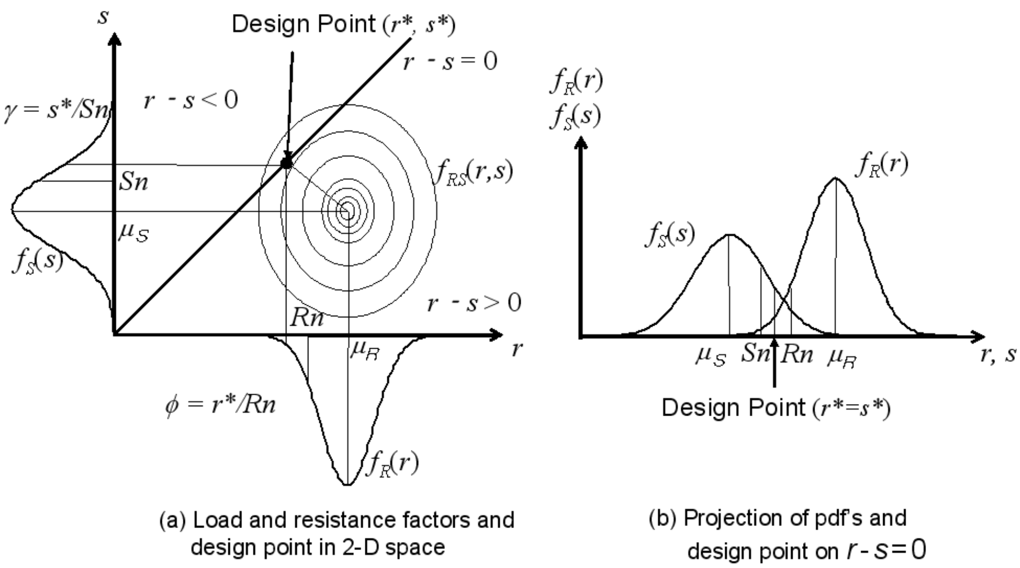

Fig.2.3.3 Load and Resistance Factor and Design Point

Load and resistance factors can be evaluated using AFOSM[13]; however, it requires probability analysis and is cumbersome in practice. In "Recommendations of Limit State Design for Buildings[8]," tables of load and resistance factors are presented considering representative statistical characteristics of loads. When the conditions are different from the assumptions used to evaluate these factors, the factors should be evaluated using an approximation method such as Third Moment Method[14] or the simplified method[15] presented in "Recommendations of Limit State Design for Buildings." The simplified method is briefly described in the following.

3) Simplified method for load factors

When load effects (both the maximum of the principal load effect during the reference period and the arbitrary point-in-time effect of the secondary load) and resistance are lognormally distributed and statistically independent of one another, the load and resistance factors can be estimated by[13]

$$\gamma_i = \frac{1}{\sqrt{1 + V_{S_i}^2}}\ \exp(\alpha_{S_i}\ \beta_\mathrm{T}\ \sigma_{\ln S_i})\ \frac{\mu_{S_i}}{S_{\mathrm{n}_i}} \tag{2.3.8}$$

$$\phi = \frac{1}{\sqrt{1 + V_R^2}}\ \exp(-\alpha_R\ \beta_\mathrm{T}\ \sigma_{\ln R})\ \frac{\mu_R}{R_\mathrm{n}} \tag{2.3.9}$$

in which β_{T} = target reliability index, μ_X, σ_X, and V_X are the mean, standard deviation, and coefficient of variation of a random variable X, respectively, $\sigma_{\ln X}$ is the standard deviation of $\ln X$ (called a logarithmic standard deviation), and α_X is the sensitivity factor, that shows the relative importance of each random variable and expressed as,

$$\alpha_{S_i} = \frac{\sigma_{\ln S_i}\ \gamma_i\ \mu_{S_i}}{\sqrt{(\sigma_{\ln R}\ \phi\ \mu_R)^2 + \sum_i (\sigma_{\ln S_i}\ \gamma_i\ \mu_{S_i})^2}} \tag{2.3.10}$$

$$\alpha_R = \frac{\sigma_{\ln R}\ \phi\ \mu_R}{\sqrt{(\sigma_{\ln R}\ \phi\ \mu_R)^2 + \sum_i (\sigma_{\ln S_i}\ \gamma_i\ \mu_{S_i})^2}} \tag{2.3.11}$$

ϕ, γ_i, and μ_R in Eqs.(2.3.10) and (2.3.11) are unknown variables; they can be evaluated analytically when a building or a structural member is subjected to a single load.

$$\alpha_S = \frac{\sigma_{\ln S}}{\sigma_{\ln M}},\ \alpha_R = \frac{\sigma_{\ln R}}{\sigma_{\ln M}} \tag{2.3.12}$$

in which $\sigma_{\ln M} = \sqrt{(\sigma_{\ln R})^2 + (\sigma_{\ln S})^2}$.

When the load effects are not lognormally distributed, load and resistance factors can be estimated by the simplified method, which does not require probability analysis[8, 15].

For a considered limit state, the load factors for the load effect due to the basic load intensity (basic load effect) and the resistance factor for the nominal resistance are evaluated by the following three equations.

The load factor for the principal load, γ_{p}:

$$\gamma_{\mathrm{p}} = \frac{1}{\sqrt{1 + \tilde{V}_{S_{\mathrm{p}}}^2}}\ \exp(\alpha_{S_{\mathrm{p}}}\ \beta_{\mathrm{T}}\ \tilde{\sigma}_{\ln S_{\mathrm{p}}})\ \frac{\mu_{S_{\mathrm{p}}}}{S_{\mathrm{pn}}} \tag{2.3.13}$$

The load factor for the k-th secondary load, γ_k:

$$\gamma_k = \frac{1}{\sqrt{1 + \tilde{V}_{S_k}^2}}\ \exp(\alpha_{S_k}\ \beta_{\mathrm{T}}\ \tilde{\sigma}_{\ln S_k})\ \frac{\mu_{S_k}}{S_{k\mathrm{n}}} \tag{2.3.14}$$

The resistance factor, ϕ:

$$\phi = \frac{1}{\sqrt{1 + {V_R}^2}}\ \exp(-\alpha_R\ \beta_{\mathrm{T}}\ \sigma_{\ln R})\ \frac{\mu_R}{R_{\mathrm{n}}} \tag{2.3.15}$$

in which

β_{T}: Target reliability index during the reference period, (that is 50 years for an ultimate limit state and one year for a serviceability limit state),

$\mu_{S_{\mathrm{p}}}$: The mean value of the maximum load effect of the principal load during the reference period, S_{p} (S_{p} is the 50 year maximum value for an ultimate limit state and the annual maximum value for a serviceability limit state.),

μ_{S_k}: The mean value of the annual maximum load effect of the k-th secondary load,

μ_R: The mean value of the resistance of a structural component, R,

S_{pn}: The load effect due to the basic load intensity of the principal load,

$S_{k\text{n}}$: The load effect due to the basic load intensity of the k-th secondary load,

R_{n}: The nominal value of the resistance of a structural component, R,

$\tilde{V}_{S_\text{p}}$: The coefficient of variation of a lognormal random variable, $\tilde{S}_\text{p}$, of which statistical parameters are determined based on the probability distribution of S_p. Such a random variable as $\tilde{S}_p$ is called as the lognormally approximated random variable of S_p.

$\tilde{V}_{S_k}$: The coefficient of variation of, $\tilde{S}_k$, that is the lognormally approximated random variable of S_k,

V_R: The coefficient of variation of the resistance of a structural component,

$\tilde{\sigma}_{\ln S_\text{p}}$: The standard deviation of the natural logarithm of $\tilde{S}_\text{p}$,

$\tilde{\sigma}_{\ln S_k}$: The standard deviation of the natural logarithm of $\tilde{S}_k$,

$\sigma_{\ln R}$: The standard deviation of the natural logarithm of R,

α_{S_p}: The sensitivity factor of the principal load, S_p,

α_{S_k}: The sensitivity factor of the k-th secondary load, S_k,

α_R: The sensitivity factor of the resistance of a structural component, R.

The sensitivity factors are evaluated by the following equations. Note that S_i represents either S_p or S_k, and $\tilde{S}_i$ represents the lognormally approximated random variable of S_i.

$$\alpha_R = \frac{V_R}{\sqrt{{V_R}^2 + \sum (c_j \, \tilde{V}_{S_j})^2}} \, u \tag{2.3.16}$$

$$\alpha_{S_i} = \frac{c_i \, \tilde{V}_{S_i}}{\sqrt{{V_R}^2 + \sum (c_j \, \tilde{V}_{S_j})^2}} \, u \tag{2.3.17}$$

in which u is the safety factor considering the approximation error of the simplified method and 1.05 is generally considered for u. c_i is the ration of the mean value of $\tilde{S}_i$ to the mean value of the total sum of $\tilde{S}_i$'s and evaluated by

$$c_i = \frac{\exp\left(\tilde{\lambda}^*_{S_i} + \frac{1}{2}\tilde{\sigma}^2_{\ln S_i}\right) \dfrac{S_{\text{n}_i}}{G_\text{n}} \dfrac{\mu_{S_i}}{S_{\text{n}_i}}}{\sum_j \exp\left(\tilde{\lambda}^*_{S_j} + \frac{1}{2}\tilde{\sigma}^2_{\ln S_j}\right) \dfrac{S_{\text{n}_j}}{G_\text{n}} \dfrac{\mu_{S_j}}{S_{\text{n}_j}}} \tag{2.3.18}$$

in which μ_{S_i} is the mean value of the annual maximum of S_i, $\tilde{\lambda}^*_{S_i}$ is the mean value of the natural logarithm of $\tilde{S}_i/\mu_{S_i}$ (called as the normalized logarithmic mean in the following).

Especially when the ratio of the mean value of the principal load to that of the dead load, μ_{S_p}/μ_D, is larger than unity, when the ratio of the mean value of the sum of the secondary loads excluding the dead load to the mean value of the dead load is about 0.2 - 0.7, and when the coefficient of variation of the dead load is about 0.1, the sensitivity factors can be simply evaluated by the following equations.

$$\alpha_R = \frac{(2 + 2\ \mu_{S_\mathrm{p}}/\mu_D + m)\ V_R}{A}\ u \tag{2.3.19}$$

$$\alpha_{S_\mathrm{p}} = \frac{3\ \tilde{V}_{S_\mathrm{p}}}{A}\ u \tag{2.3.20}$$

$$\alpha_D = \frac{0.2}{A}\ u \tag{2.3.21}$$

$$\alpha_{S_k} = \frac{\tilde{V}_{S_k}}{A}\ u \quad \text{(excluding dead load)} \tag{2.3.22}$$

in which α_D is the sensitivity factor of the dead load and m is the number of the secondary loads excluding the dead load. A in Eqs.(2.3.19)-(2.3.22) is the normalizing factor so that $\sum \alpha_\bullet^2 = u^2$, and expressed as,

$$A = \sqrt{\{(2 + 2\ \mu_{S_\mathrm{p}}/\mu_D + m)\ V_R\}^2 + (3\ \tilde{V}_{S_\mathrm{p}})^2 + \sum_{k=1}^{m} \tilde{V}_{S_k}^2 + 0.04} \tag{2.3.23}$$

The statistics of $\tilde{S}_i$ (normalized logarithmic mean, $\tilde{\lambda}^*_{S_i}$, logarithmic standard deviation, $\tilde{\sigma}_{\ln S_i}$, and coefficient of variation, $\tilde{V}_{S_i}$) can be evaluated by the following equations, as presented in "Recommendations of Limit State Design for Buildings."

$$\tilde{\lambda}^*_{S_i} = e_0 + e_1\ V_{S_i} + e_2\ {V_{S_i}}^2 + e_3\ {V_{S_i}}^3 \tag{2.3.24}$$

$$\tilde{\sigma}_{\ln S_i} = s_0 + s_1\ V_{S_i} + s_2\ {V_{S_i}}^2 + s_3\ {V_{S_i}}^3 \tag{2.3.25}$$

$$\tilde{V}_{S_i} = \sqrt{\exp\left\{(\tilde{\sigma}_{\ln S_i})^2\right\} - 1} \tag{2.3.26}$$

The coefficients e_j's and s_j's to be used in Eqs.(2.3.24) and (2.3.25) are presented in Table 2.3.1 for the secondary loads and the principal load for a serviceability limit state, and in Table 2.3.2 for the principal load for an ultimate limit state. The coefficients are given for different types of the probability distribution function that the annual maximum of the considered load is described. Note that μ_{S_i} and V_{S_i} in the tables are the mean value and coefficient of variation of the annual maximum of the load effect S_i (before it is lognormally approximated).

The load and resistance factors can be evaluated more accurately using the coefficients $s_0, \cdots, s_3$ in Eq.(2.3.25) estimated by Eq.(2.3.27) as a function of β_T using the coefficients in Table 2.3.3 or Table 2.3.4[15].

$$S_k = b_0 + b_1\ \frac{1}{\beta_\mathrm{T}} + b_2\ \frac{1}{{\beta_\mathrm{T}}^2} + b_3\ \frac{1}{{\beta_\mathrm{T}}^3} \tag{2.3.27}$$

For an ultimate limit state the ratio of the mean value of the maximum load effect of the principal load during the reference period of 50 years (50 year maximum value) to the basic load effect is estimated by

$$\frac{\mu_{S_\mathrm{p}}}{S_\mathrm{pn}} = \exp\left(\tilde{\lambda}^*_{S_\mathrm{p}} + \frac{1}{2}\tilde{\sigma}^2_{\ln S_\mathrm{p}}\right)\ \frac{\mu_{S_\mathrm{a}}}{S_\mathrm{pn}} \tag{2.3.28}$$

in which μ_{S_p} and μ_{S_a} are the mean value of the 50-year maximum and the annual maximum of the principal load, respectively. $\tilde{\lambda}^*_{S_\mathrm{p}}$ and $\tilde{\sigma}_{\ln S_\mathrm{p}}$ are the normalized logarithmic mean and logarithmic standard deviation of the 50-year maximum value of the principal load, respectively, and can be evaluated by Eq.(2.3.24) and Eq.(2.3.25), respectively, substituting the coefficient presented in Table 2.3.2.

For wind load, the statistics of BASIC wind velocity rather than the load effect are generally given. For such a case, the statistics of the annual maximum and those of the 50-year maximum of the wind load effect are estimated based on the logarithmic standard deviation of the corresponding (annual maximum or 50-year maximum) BASIC wind velocity, $\tilde{\sigma}_{\ln U}$, as,

$$\tilde{\lambda}^*_W = -2\ \tilde{\sigma}^2_{\ln U} \tag{2.3.29}$$

$$\tilde{\sigma}_{\ln W} = 2\ \tilde{\sigma}_{\ln U} \tag{2.3.30}$$

$$\tilde{V}_W = \sqrt{\exp\left(4\ \tilde{\sigma}^2_{\ln U}\right) - 1} \approx 2\ \tilde{V}_U \tag{2.3.31}$$

Eq.(2.3.31) provides a good approximation when $\tilde{V}_U \leq 0.2$.

The ratio of the mean value of the annual maximum wind load effect, μ_{W_a}, to the basic load effect, W_n, can be approximately estimated using the ratio of the mean value of the annual maximum wind velocity, μ_{U_a}, and the BASIC wind velocity, U_0, as,

$$\mu_{W_a}/W_\mathrm{n} \approx \left(\mu_{U_a}/U_0\right)^2 \tag{2.3.32}$$

When wind load is the principal load and an ultimate limit state is considered, the ratio of the mean value of the maximum value during the reference period (50-year maximum value) to the basic load effect can be evaluated by

$$\frac{\mu_{W_{50}}}{W_\mathrm{n}} = \exp\left\{2\left(\tilde{\lambda}^*_{U_{50}} - \tilde{\lambda}^*_{U_\mathrm{a}}\right) + 2\left(\tilde{\sigma}^2_{\ln U_{50}} - \tilde{\sigma}^2_{\ln U_\mathrm{a}}\right)\right\}\ \frac{\mu_{W_\mathrm{a}}}{W_\mathrm{n}} \tag{2.3.33}$$

in which $\mu_{W_{50}}$ and μ_{W_a} are the mean value of the 50-year maximum and the annual maximum of the wind load effect, respectively, $\tilde{\lambda}^*_{U_{50}}$ and $\tilde{\sigma}_{\ln U_{50}}$ are the normalized logarithmic mean and logarithmic standard deviation of the 50-year maximum of the wind velocity, respectively, and $\tilde{\lambda}^*_{U_\mathrm{a}}$ and , $\tilde{\sigma}_{\ln U_\mathrm{a}}$ are the normalized logarithmic mean and logarithmic standard deviation of the annual maximum of the wind velocity, respectively.

Generally, the uncertainty exists not only in the load intensity, P, which is the most basic physical parameter of load such as ground snow depth and basic wind velocity, but also in the estimation of the force based on load intensity and the estimation of the load effect through a structural model. Such uncertainties are often called as model uncertainties. If the bias in model uncertainty can be neglected, the ratio of the mean value to the basic value remains unchanged, while the variability in the load effect V_{S_i}, is increased as

$$V_{S_i} \approx \sqrt{V_{P_i}^2 + \sum_j V_{Mo_{ij}}} \tag{2.3.34}$$

in which V_{S_i} andV_{P_i} are the coefficient of variation of load effect, S_i, and load intensity, P_i, and $V_{Mo_{ij}}$ is the coefficient of variation of modeling error, Mo_{ij}, in estimating load effect, S_i, from load intensity, P_i. Note that the coefficient of variation of the maximum load intensity during the reference period is used for the coefficient of variation of the principal load, and the value estimated by Eq.(2.3.31) is used for the coefficient of variation of wind load.

Table 2.3.1 For lognormal approximation of the annual maximum value

Cdf of annual max.		Coefficients in Eq.(2.3.24)				Coefficients in Eq.(2.3.25)			
		e_0	e_1	e_2	e_3	s_0	s_1	s_2	s_3
Normal		0.00	0.00	0.00	0.00	0.01	0.85	-0.49	0.14
Gumbel		0.00	-0.16	-0.01	0.00	0.02	1.13	-0.67	0.20
Fréchet	$\beta_T \leq 2.5$	0.00	-0.28	-0.05	0.07	0.00	1.44	-0.98	0.26
	$\beta_T > 2.5$	0.00	-0.28	-0.05	0.07	0.00	1.68	-1.14	0.30

Table 2.3.2 For lognormal approximation of the 50-year maximum value

Cdf of annual max.		Coefficients in Eq.(2.3.24)				Coefficients in Eq.(2.3.25)			
		e_0	e_1	e_2	e_3	s_0	s_1	s_2	s_3
Normal		0.02	1.89	-1.05	0.30	0.02	0.34	-0.32	0.11
Lognormal		-0.01	2.34	-1.07	0.16	0.00	0.61	-0.13	0.00
Gumbel		0.04	2.32	-1.43	0.43	0.05	1.59	-0.60	0.22
Fréchet	$\beta_T \leq 2.5$	0.01	2.82	-2.15	0.62	0.00	1.44	-0.98	0.26
	$\beta_T > 2.5$	0.01	2.82	-2.15	0.62	0.00	1.68	-1.14	0.30

4) Target performance level

The performance level (target reliability index) required in a building or component should be determined considering the consequence and the nature of failure, importance of the building, usage, type of loading, type of building, minimum requirement prescribed in regulations, economic losses, social inconvenience, and the amount of expense and effort required to reduce the probability of failure.

In some cases a building as a whole has a required reliability level, while in the other cases required levels are different from part to part in a building.

The reliability indices of a building designed based on the current Building Standard Law and enforcement orders in Japan vary depending on the site of construction, the type of a building, the type of a component, the load combination and limit state considered, etc. Examples of reliability indices for columns subjected to a load combination of $G + Q + E$ are as follows:

Table 2.3.3 For lognormal approximation of annual maximum value for Eq.(2.3.27)

Cdf of annual max.	Coefficients in Eq.(2.3.27)				
	S_k	b_0	b_1	b_2	b_3
Normal	s_0	0.03	-0.12	0.22	-0.19
	s_1	0.54	2.39	-4.89	3.94
	s_2	0.06	-7.09	21.67	-18.84
	s_3	-1.23	12.34	-33.21	28.25
Gumbel	s_0	0.01	0.01	-0.14	0.11
	s_1	2.03	-4.16	6.61	-3.25
	s_2	-3.81	15.82	-24.06	10.63
	s_3	2.22	-10.79	17.06	-6.07
Fréchet	s_0	0.01	-0.11	0.18	-0.07
	s_1	3.24	-7.7	9.88	-4.47
	s_2	0.76	-6.27	8.43	-3.72
	s_3	-0.56	2.52	-2.67	0.94

Table 2.3.4 For lognormal approximation of 50-year maximum value for Eq.(2.3.27)

Cdf of annual max.	Coefficients in Eq.(2.3.27)				
	S_k	b_0	b_1	b_2	b_3
Normal	s_0	0.01	-0.01	0.03	-0.01
	s_1	0.47	-0.35	0.45	-0.2
	s_2	-0.59	0.78	-1.21	0.58
	s_3	0.26	-0.43	0.75	-0.37
Lognormal	s_0	-0.03	0.13	-0.14	0.06
	s_1	1.08	-2.2	2.79	-1.24
	s_2	-0.32	1.51	-2.12	1.02
	s_3	0.1	-0.75	1.03	-0.48
Gumbel	s_0	0.094	-0.283	0.383	-0.166
	s_1	0.844	-0.361	0.324	-0.121
	s_2	-1.195	1.305	-1.78	0.803
	s_3	0.564	-0.774	1.141	-0.526
Fréchet	s_0	0.01	-0.12	0.24	-0.11
	s_1	3.21	-7.19	8.97	-3.97
	s_2	0.26	-5.81	9.38	-4.36
	s_3	-0.76	4.69	-7	3.16

- RC column: β_{U} =1.4for an ultimate limit state with reference period of 50 years, and β_{S} =0.9∼1.1 for a serviceability limit state with 1 year reference period.
- Steel column: β_{U} =1.9∼2.4 and β_{S} =1.3∼2.4

2.3.3 Load combination for allowable stress design and ultimate strength design

Design loads for allowable stress design and ultimate strength design are determined by selecting the design return period of loads. The design loads based on the return period is effective because either an exceedance probability or a non-exceedance probability of the load can be always taken into account. This treatment has the great advantage of possibility of relatively comparing with other load intensities. In a conventional design method, however, qualitative adjustment of design loads has been made based on the service life, importance and failure consequence of a building.

Let us consider the service life of a building. The degree of occurrence of large earthquakes increases as the service life of the building becomes longer. This implies that on the basis of probability, the longer the service life of a building is, the larger seismic design load is needed. When the probability that a certain load exceeds the magnitude Q within an arbitrary one year P_1, the probability that the load exceeds the magnitude within the service life T_{L}, P_{T} can be written as

$$P_{\mathrm{T}} = 1 - (1 - P_1)^{T_{\mathrm{L}}} \approx P_1 \, T_{\mathrm{L}} \tag{2.3.35}$$

Where each exceedance event is assumed to occur independently. If the annual probability P_1 is small, P_{T} is approximately equal to $P_1 T_{\mathrm{L}}$.

Determination of design loads based on return periods has not been done explicitly in the conventional allowable stress design and ultimate strength design. The service life, however, has been paid attention to in a life cycle cost management and life of buildings. Therefore, even though conventional design methods are used, it is necessary to account for design loads based on the service life of the building.

Importance and failure consequences of a building are mentioned in the following. The concept of importance and consequences is very vague. If loss of human lives and property and external influence due to building failure can be assumed to quantify, buildings with larger loss must have higher degree of importance and larger failure consequences. It then follows that in a conventional design, the procedure that larger design loads are used for buildings with greater importance. To increase design load, “an importance factor” or “a factor of building use” has been introduced. These factors are the ones by which a basic load intensity is multiplied. The adjustment of design loads by these factors is equivalent to that based on selecting the return period. In other words, the longer the return period is, the larger the load intensity is, as is increase of the above factors. Furthermore, it is recently adopted that besides the importance or building use factor, factored design loads are introduced to easily achieve the performance grading in the current provision on securing quality of housing in Japan. In the Vision 2000 shown in

2.3.1, design loads are specified based on the return period. The return period concept has been playing an extremely important role in performance-based design.

It is not easy to select and justify an appropriate return period in allowable stress or ultimate strength designs. If the uncertainty associated with building capacity could be small enough to neglect, a reciprocal of the selected return period is approximately equal to the performance level, i.e., a probability of failure. Since the uncertainty, however, cannot be neglected, a load and resistance factor design (LRFD) is preferably adopted, which is based on the probability of failure or reliability index closely related to the importance of a building.

(1) Loading states to be considered

Loading states in a conventional design are determined according to conditions where a building is located. In general, long- and short-term loading states are used. The long-term loading state implies the ordinary state under the normal state of a building, while the short-term loading state, which rarely occurs during the service life, are states of, large earthquakes, typhoons, extraordinary snow falls, etc. The loading states should follow prevailing rules in a conventional design method.

(2) Return period conversion factor

Let the cumulative distribution function of an annual maxima of load X be noted $F_X(x)$. The interval between the events that the annual maximum value X exceeds x is also a random variable. An expected value of the interval T_R is taken. The probability that X exceeds x just after one year is $p = 1 - F_X(x)$, and the probability that X exceeds x for the first time just after two years is $(1-p)^{i-1}p$, and the probability that X exceeds x for the first time at exactly i-th year is y just after two years is $(1-p)^{i-1}p$, Using these expressions, the expected value of the time interval t_R can be estimated as follows.

$$T_\mathrm{R} = E[T_\mathrm{R}] = \sum_{i=1}^{\infty} i\ (1-p)^{i-1}p = \frac{1}{p} = \frac{1}{1-F_X(x)} \tag{2.3.36}$$

This t_R is called “mean return period” or simply “return period”, the value corresponding to this period x is referred to as the value corresponding to the return period $x(t_\mathrm{R})$. $x(t_\mathrm{R})$ is the value by solving Eq. (36) with respect to x and can be obtained using an inverse function of $F_X(x)$. The longer t_R is, of course, the larger $x(t_\mathrm{R})$ is.

$$x(T_\mathrm{R}) = {F_X}^{-1}\left(1 - \frac{1}{T_\mathrm{R}}\right) \tag{2.3.37}$$

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