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Design load and structural safety - Developments in Japan

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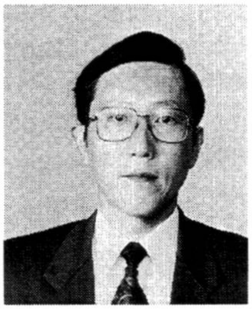

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Summary

Developments of Japanese building codes are reviewed with emphasis on load specifications. Current load specifications are briefly summarised for live loads, snow loads, wind loads and earthquake loads mostly defined for the allowable stress design. Specified Load intensity values are based on those determined in an empirical manner in or before1950. Recent developments are introduced paying attention to Recommendations for Loads on Buildings published by the Architectural Institute of Japan. Further discussions are developed for recent activities towards a new concept of performance-based structural design.

1. Introduction

The major purpose of structural design is to make structures safe against anticipated actions and loads in their lifetime. As far as Japanese building code is concerned, the intensities of all design loads are specified numerically and most engineers can easily take those numbers for their structural analysis calculations. Since environmental actions could often exceed the specified intensity, the engineer should consider the safety margin in various ways. However the degree of safety is not explicitly stated in current regulations, then individual engineers have to face the difficulty to make their judgments on the structural safety. They have to accept the safety according to codified numbers without any quantitative measure of safety, although such codified values tend to determine automatically the safety degree irrespective of indivitual environmental conditions and users' demands.

Among many parameters related to the structural safety, the maximum load intensity generally has most significant uncertainty. This means that the design load controls the structural safety to a fairly large extent. Then engineers should pay much more attention to the design load determination. Developments in Japanese building code are reviewed and current specifications of design loads are critically discussed. Then activities for new concept of structural design are introduced for future developments towards a performance-based structural design and/or a limit state design.

2. Developments of Japanese building codes [1), 2)]

The first Japanese building code, Urban Building Law and Urban Planning Law were promulgated in 1919 to regulate building constructions and city planning in six major cities. Seven chapters in strength requirements are 1). General, 2). Wood construction, 3). Masonry and brick work, 4). Steel construction, 5). Reinforced concrete construction, 6). Independent chimney and 7). Strength calculation. The allowable stress design method was used specifying allowable stresses for structural materials. Only vertical loads were specified and no descriptions were given for the snow, wind and earthquake loads. Design live loads were similar to those in then New York City Building Code.

The 1923 Kanto Earthquake caused serious damages to the capital city, then earthquake resistant regulations were introduced according to proposals by Professor Riki Sano. The seismic coefficient of 0.1 was specified. The anticipated maximum seismic coefficient was estimated as 0.3 and was reduced to one third by considering the safety factor of 3 used in determining allowable stress level relative to the material strength.

The Urban Building Law was effective until 1950, although proposals for the revision were often discussed in the Architectural Institute of Japan (A.I.J.). The 1937 proposal by A.I.J. included 1). detailed classification of building use for live loads, 2). detailed classification of structural woods, 3). increase of allowable stresses for steel and 4). introduction of specifications for snow (unit weight of 29.4 Pa/cm) and wind (1 kPa velocity pressure for the height less than 15 m).

In 1944, Temporal Japanese Standard 532 "Loads on Buildings" and 533 "Fundamentals of structural calculations of buildings" were enacted to replace the Urban Building Law during the war time. Major revisions may be summarised as, 1). increase of the design load for important structures, 2). reduction of live loads, 3). introduction of snow load (unit weight of 19.8 Pa/cm), 4). introduction of wind load (392 $\sqrt{h}$ (Pa) as velocity pressure, where h is the height (m)), 5). horizontal seismic coefficient 0.15 for ordinary soil and 0.20 for soft soil, 6). allowable stress values are twice those specified in the Urban Building Low and 7). consideration for calculation error, construction error and variability of materials. The intentional reduction of structural safety was clearly observed in these war-time standards.

Under the new Japanese constitution, the Building Standard Law was proclaimed in 1950. The principle of requirements to structures is stated in Article 20 as, [3)]

> 1). Buildings shall be of structure safe from dead load, live load, snow load, wind pressure, ground pressure and water pressure as well as earthquake or other vibration or shock.
>
> 2). In preparing drawings/specifications for buildings as mentioned in Article 6 paragraph 1 item (2) or (3), the safety of the structure thereof shall be confirmed through structural calculation, where Article 6 paragraph 1 item (2): Wooden buildings which have three or more stories, or have a total floor area exceeding 500 square meters and item (3): Buildings other than wooden buildings, which have two or more stories or have a total floor area exceeding 200 square meters.

Design load values and related equations are specified in Articles 83 to 88 in Enforcement Order, based on the allowable stress design procedure. Allowable stresses are also specified in Articles 89 to 106 for structural materials. Special attentions have been paid for seismic resistant design after major earthquakes, e.g., the Tokachi-oki earthquake, 1968, the Miyagiken-oki earthquake, 1978. But otherwise specified values for loads and allowable stresses have been mostly unchanged since 1950.

3. Current load specifications [3)]

3.1. Live loads

A table is provided in Article 85 of Enforcement Order as alternative values to actual ones as summarised in Table 1. Although in Article 85 it is mentioned that live-load values can be estimated according to actual conditions, values of Table 1 are used in most cases of practices.

Table 1 Current Live load values (kPa) for various uses —summary

member	floor	girder /column
houses	1.76	1.27
offices	2.94	1.76
shopping stores	2.94	2.35
meeting rooms/no seats	3.53	3.23
garages	5.39	3.92

Live loads are combined with dead loads to calculate stresses due to permanent loads to be compared with the long-term allowable stress, f_L. f_L for the steel tensile stress is equal to 2/3 of the nominal yielding stress and f_L for the concrete compression is equal to 1/3 of the nominal ultimate compressive stress. Recent live load survey data are summarised as in Figure 1. [4)] When 99 percentile values of load intensities are compared with live load values in Table 1, e.g. for houses, offices and shopping stores, the latter is 1.2 to 1.8 times greater than the former by considering typical unit areas for the floor and the girder as 20 m^2 and 50m^2 respectively.

3.2 Snow loads

Deepest snow fall values are specified by special administrative agencies. The ratio of those values to statistically obtained values associated with 50 year return period varies from 0.61 to 1.5. [5)] These ratios indicate that the snow load in current design practices varies in terms of the return period in a very wide range such as 5 years to 2000 years. The unit weight of snow is specified as 19.8 Pa/cm or more, and in heavy snow regions, special administrative agencies increase its value to 29.4 Pa/cm.

Stresses due to snow loads are combined with stresses due to permanent loads and compared with the short-term allowable stress, f_S. In heavy snow regions long-term stress checking is also in practice for reduced snow loads. f_S for the steel tensile stress is equal to the nominal yielding stress and f_S for the concrete is twice f_L.

3.3 Wind loads

The velocity pressure is given by 588 $\sqrt{h}$ (Pa) for $h \leqq 16$ (m) and 1176 $h^{1/4}$ (Pa) for $h > 16$ (m). The latter was introduced in 1981 by considering significant conservatism of the former when applied to a part of the height greater than 16 m. 1176 $h^{1/4}$ was originally used for wind load for the first tall building in Japan, Kasumigaseki Building constructed in 1968 whose height is 147 m, by Dr. Kiyoshi Muto, and has been introduced in the cladding design in a form of notice of Ministry of Construction Since 1978.

Zoning factor was prepared in a form of notice of Ministry of Construction in 1959, but has not been used in practices in most administrative agencies. The ratio of the velocity pressure value at h = 10 (m) to corresponding statistically obtained values associated with 50 year returm period for flat open terrain varies from 1.1 to 2.2 covering most of Japanese islands. [5] These ratios indicate that the design wind load in terms of the return period in a wide, mostly conservative, range such as 80 years to 6000 years.

Stresses due to wind loads are combined with stresses due to permanent loads and compared with the short-term allowable stress. Such conservatism mentioned above may not be seriously criticised by practice engineers as earthquake loads often dominate the wind loads except for very light and/or very tall structures. For tall buildings with height over 60 m, return period based wind loads have recently been used according to A.I.J. Recommendation.

3.4 Earthquake loads

Basic base shear coefficients are specified as 0.2 for the short-term allowable stress design and 1.0 for the capacity design. The latter was introduced in 1981 by considering the necessity of introduction of capacity design. Zoning factor is applied to reduce seismic shear force to 0.9 or 0.8 in lower seismicity regions, except for Okinawa where Zoning factor of 0.7 is used.

Vibration characteristic factor is defined as a function of natural period of the structure and the estimated dominant period of the soil, and is applied to multiply the basic base shear coefficient. Structural characteristic factor is specified to take into account the ductility performance of post-yielding structural behavior to the earthquake load in the capacity design. Values vary between 0.3 and 0.7 for reinforced concrete structures and between 0.25 and 0.5 for steel structures.

Many seismic hazard maps have been available and as far as statistical estimations concerned in a range of relatively short period such as less than 100 years, a fairly good agreement among maps can be pointed out.[5] The ratio of design earthquake load for the allowable stress design to the 50 year return period value varies from 0.42 at Tokyo to 1.3 at Fukuoka for six major cities.[5] These values seem to correspond to the return period of 10 to 80 years.

Dynamic response analyses are commonly used to examine the elastic and inelastic response behaviour of tall buildings with height over 60 m. The basic intensity of input earthquake ground motions is 25 cm/s for the elastic response and 50 cm/s for inelastic response to the criteria of story ductility factor of 2. Both El Centro, NS, 1940 and Taft, EW, 1952 motions have been still used as respresentative input motions since the time of Kasumigaseki building, although the irrationality has been pointed out for their particular spectral characteristics.

4. A.I.J. load recommendations [4)]

The Architectural Institute of Japan has been producing various types of standards and recommendations. Design specifications for steel structures and Standard for structural calculation of reinforced concrete structures have been used widely in practice in accordance with Building Standard Law and Enforcement Order. Recommendations for Loads on Buildings was first published in 1975 then revised in 1981, in 1986 and in 1993.

The principles of 1993 version may be summarised as, 1). common basic load intensity for various loads based on statistical data, 2). design loads for both allowable stress design and limit state design, 3). equivalent static loads for dynamic actions such as winds and earthquakes, and 4). providing variability information for physical parameters involved in load estimation.

Values associated with 100 year return period are used commonly as a basic load intensity for snow, wind and earthquake and 99 percentile values are used for a basic live load intensity. Return period conversion factor, R, was introduced and formulated as,

$$R = 0.40 + 0.13 \ln r \quad \text{for snow depth in heavy snow regions} \tag{1}$$

$$R = 0.22 + 0.17 \ln r \quad \text{for snow depth in other regions} \tag{2}$$

$$R = 0.54 + 0.1 \ \ln r \quad \text{for wind speed} \tag{3}$$

and

$$R = \left(\frac{r}{100}\right)^{0.54} \quad \text{for peak ground acceleration and velocity} \tag{4}$$

where r is the retrurn period.

Design loads for the allowable stress design are determined by taking an appropriate return period by applying return period conversion factor of Equations (1) to (4). Design loads for the limit state design are defined as products of the load factor and basic load values. The load factor is formulated by a commonly used form derived for log-normal ramdom variables as,

$$\gamma = \frac{1}{\sqrt{1+V_s^2}} \exp(\alpha_s \beta_T \sigma_{\ln s}) \frac{\bar{S}}{S_n} \tag{5}$$

where V_S is the coefficient of variation of load effect S, α_s is the separation facotr, β_T is the target reliability index, $\sigma_{\ln s}$ is the standard deviation of logarithm of S, $\bar{S}$ is the mean of S for a reference period and S_n is the basic value of S.

4.1. Live loads

A formula for the basic live load, L, is given by

$$L = L_o \times C_E \times C_{R1} \times C_{R2} \tag{6}$$

where L_o is the basic live load intensity corresponding to the 99 percentile value of arbitrary-point-in-time statistics for a reference influence area of 18 m², C_E is a conversion factor to Equivalent Uniformly Distributed Load (E.U.D.L.), C_{R1} is a reduction factor for changing of influence area and C_{R2} is a reduction factor for nultiple-story column loads.

4.2 Snow loads

Two types of snow loads are defined; one is roof snow loads without control based on the maximum snow depth as in conventional practices and the other is roof snow loads with control based on 7 day snow accumulation.

A formula for the basic snow load, S, is given by

$$S = d_o \times \rho_s \times \mu \times g \times C_e \tag{7}$$

where d_o is the basic snow load intensity, i.e. the 100 year return period value of maximum snow depth on the ground, ρ_s is the equivalent snow density, μ is the roof shape coefficient consisting of the basic coefficient as a function of the average wind speed in winter and the slope of the roof, a coefficient for the irregularity due to snow drift and a coefficient for the irregularity due to sliding, g is the gravity acceleration and C_e is the environmental coefficient.

The equivalent snow density is expressed as a function of design snow depth to meet recent data available as shown in Figure 2. The snow temperature seems not to be a significant parameter for the equivalent snow depth and a unique formula in Figure 2 was employed for ρ_s in Japan.

4.3 Wind loads

Basic wind load is estimated by Equation (8).

$$W = \frac{1}{2}\rho(U_o E_H)^2 C_f G_f A \tag{8}$$

where ρ is the air density, U_o is the basic wind speed, i.e. the maximum wind speed (10 minute mean) associated with 100 year return period over a flat opten terrain at an elevation of 10 m above the ground, E_H is the wind speed profile factor at the height H and defined as a product of exposure factor E_r and topography factor E_g, where five terrain categories are introduced to specify E_r with different power law index varying between 0.10 and 0.35 for a power law wind profile model, C_f is the wind force coefficient, G_f is the gust effect factor, and A is the projected area.

A contour map of basic wind speed is provided by exmanining meteorological data with a new terrain correction scheme.[6] The annual change of terrain roughnesses seem to be a considerable factor to the variation of averaged maximum wind speed over meteorological observation sites as shown in Figure 3.

Prediction procedures for wind-induced responses in both windward and lateral directions have been improved significantly based on recent experimental and analytical works and are extensively utilized to improve the accuracy of G_f.[7] For example non-dimensional critical wind speeds for buildings with a rectangular section for aeroelastic instability are tabulated for various side ratios in open and rough terrains.

A simplified procedure for the estimation of wind loads is provided for buildings satistying following conditions; 1). Shapes and structural systems of buildings are not special, 2). Mean roof height is less then 15 m, 3). Projected breadth is at least half the mean roof height but less than 30 m. Wind loads based on a simplified procedure yields slightly more conserrative estimation than that by a detailed one.

4.4 Earthquake loads

Detailed descriptions of earthquake loads appeared in A.I.J. recommendation at the first time in 1993 version, mostly bacause of the difficulty in reaching a general consensus, although many state of the art reports have been published. [8]

Basic horizontal story shear force of the i-th story is estimated by a response spectrum method as,

$$Q_i = D_s \sqrt{\sum_{m=1}^{k} \left[\left(\sum_{j=i}^{n} W_j \beta_m U_{jm} \right) S_A (T_m, h_m) / g \right]^2} \tag{9}$$

where D_s is the structural characteristic factor and equals unity in the elastic response, k is the number of necessary modes, n is the number of story, W_j is the gravity load of the j-th story, β_m is the participation factor for the m-th vibration mode, U_{jm} is the m-th vibration mode of the j-th story, g is the gravity acceleration, T_m , h_m are the natural period and the damping ratio of the m-th mode respectively and

$$S_A(T,h) = \begin{cases} \left(1 + \dfrac{f_A - 1}{d} \dfrac{T}{T_C} \right) F_h G_A A_o & 0 < T \le dT_c \\ F_h f_A G_A A_o & \text{for} \quad dT_c < T \le T_c \\ \dfrac{2\pi}{T} F_h f_v G_v V_o & T_c < T \end{cases}$$

where f_A is the acceleratioin response amplification ratio for $dT_c < T \leq T_c$, f_v is the velocity response amplification ratio for $T_c < T$, dT_c and T_c are the lower and upper bound periods of the range, where $S_A(T,h)$ is constant, respectively, F_h is the damping modification factor and $F_h = 1$ for $h = 5$ % and A_o and V_o are basic peak acceleration and velocity of earthquake ground motion at the reference firm soil accociated with 100 year return period respectively, and G_A and G_v are soil type modification factors for the peak acceleration and velocity respectively.

As discussed in 3.4, there are significant discrepancies between the difference of earthquake loads in current practices in low seismicity and high seismicity regions and that appeared in return period consistent peak ground accerelation(PGA). From the optium reliability view-point, a higher safety is justified for a low seismicity area and a lower safety has to be accepted for a high seismicity area[9], therefore unique retrun period value throughout the country may not be appropriate. Variation of the annual maximum PGA is much greater than that of the annual maximum snow depth or the annual maximum wind speed. Nevertheless the return period conversion factor, R, in Equation (4) shows a representatine tendency of the probabilistic characteristics as shown in Figure 4, where the Frechet distribution is consistant to the formula by Equation (4).

5. New concept of structural design

A draft standard for limit state design for steel structures was published by A.I.J. in 1990.[10] However it has not been approved by the Ministry of Construction yet and never been used in practices. The target reliability index for ultimate limit states was determined by calibrating to the current allowable stress design. The reference period for the ultimate limit state is 50 years. $\beta_T = 2.5$ for live loads, $\beta_T = 2.0$ for snow and wind loads and $\beta_T = 1.5$ for earthquake loads are used to calculate load and resitance factors for the ultimate limit state.

The ministry of construction formed new committees in 1995 to carry out a three year project to develop a new structural design frame-work, where performance-based design is discussed to replace specification-based design such as the allowable stress design. What is the performance-based design will not be answered soon, however the required performance for a structure is generally the safety and the serviceability, therefore the limit state design is regarded as one form of performance-based design.

The great Hanshin earthquake, January 17, 1995 shifted discussions of committees towards the seismic safety. The principle of 1986 revison of Building Standard Law Enforcement order explains that buildings may be slightly damaged by earthquakes occurring a few times in structural lifetime and may be seriously damaged by an possible maximum earthquake during lifetime but without human losses. Although the frequency of Jannuary 17, 1995 event is very low, when it occurred, people tend to think that their buildings should be damage free to this kind of earthquake. The retrum period of P.G.A. can be estimated as 500 years or over when estimated from statistics in A.I.J. load recommendation (1993) [11], and it seems reasonable that current seismic design criteria can not prevent property losses. Current design practices seem to have worked satisfactorily considering technical viewpoints in 1986, as most collapses of houses and buildings were caused by their deteriorations or poor maintainance or old standard design or poor workmanship.

Nevertheless demands for higher safety standard are discussed after observing many damages due to the earthquake. The minimum requirement is not necessarily to be the standard and engineers should have opportunities to provide higher safety according to owners' or users' demands. Probabilistic approahes are convenient to provide a rational measure for the safety or the frequency of earthquake occurrence, although the reliability concept has not been commonly accepted even in the engineering society.

Since building constructions are parts of economic activities, the target safety cannnot free from economic considerations. Optimum reliability based on the minimum total cost principle certainly provides a good guidance to determine the design load level.[9] Now in Japan people can see various states of damages due to strong motions and know how expensive to restore them. The performance of buildings under various levels of P.G.A. has to be described not only by engineering measures such as the maximum acceleration, the deflection, the ductility ratio etc, but also by an economical measure such as repair or replacement costs.

The reliability concept has been getting familiar throughout the world for engineered products. At the same time people have difficulties to measure the safety in a probabilistic manner. In particular when the structural safety is closely related to human losses, the appropriateness of target reliability in structural design is not easily understood by people who actually suffered from the recent earthquake. The reliability of seismic hazard in a long return period renge, i.e. a very low probability range, is also relatively poor in comparison with other variables of load intensities such as wind speed or snow depth. A great amont of works still seem necessary to include most recent findings in earthquake engineering such as active fault data, soil amplification and soil-structure interaction mechanism and so on in order to estimate lifetime maximum design earthquake load in a sophisticated manner.

6. Conclusions

The allowable stress design procedure has been used for buildings in Japan since 1919. Many improvements have been reflected in regulations in particular for the seismic resitance after every major earthquakes. However basic design load intemsity specifications have not been changed since 1950. Some attempts in Architectural Institute of Japan have been made to introduce the limit state design and to provide rational load estimation procedures including load intensity statistics. Performance-based design is now under discussion in committees formed by the Ministry of Construction to create a new structural design framework. Description of performances of structure under various levels of load conditions are to be explicitly used for design criteria. Reliability concept is also expected to be reflected in the new design procedure.

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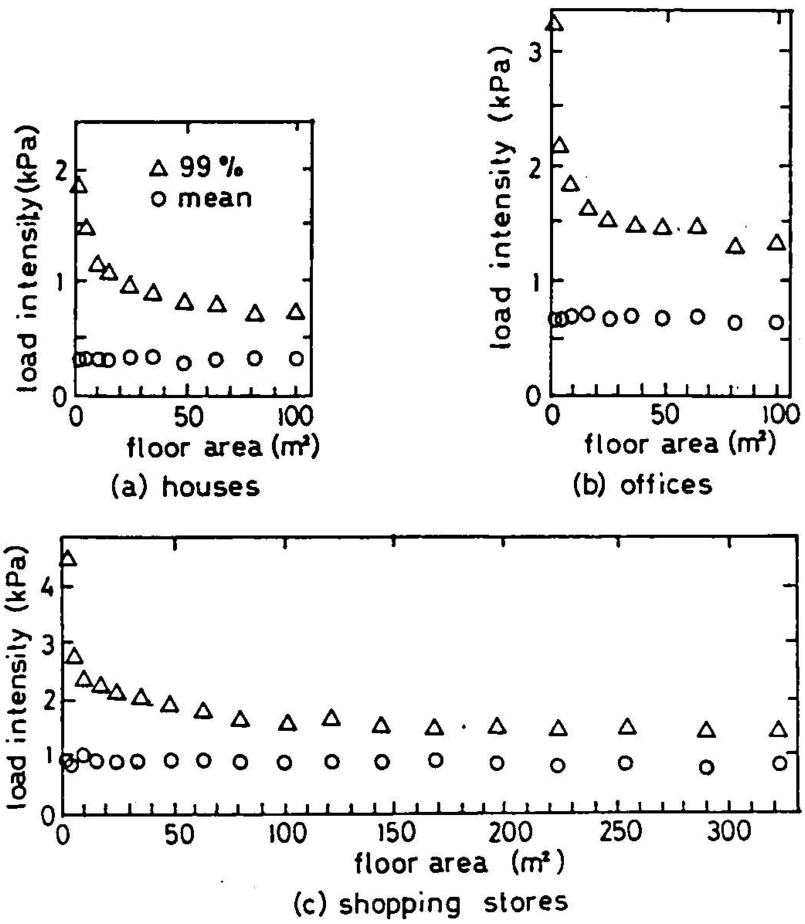

Fig. 1 Live load statistics with floor area [4)]

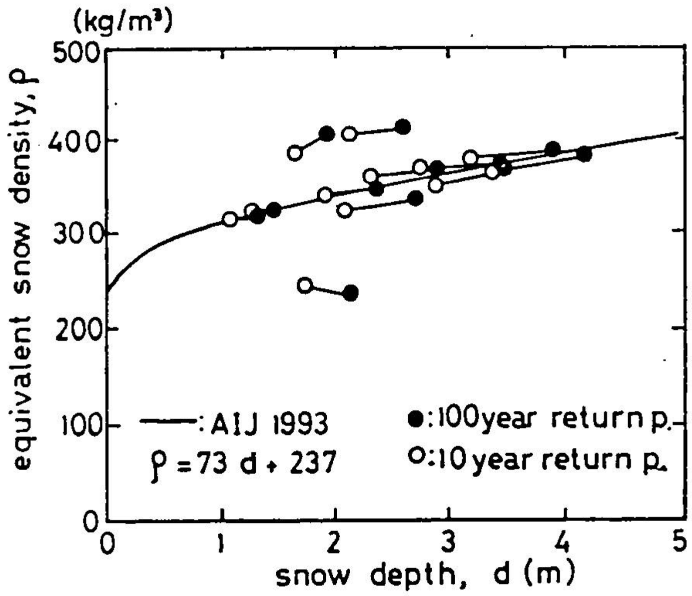

Fig. 2 Equivalent snow density with annual maximum snow depth for 12 sites in Japan [4)]

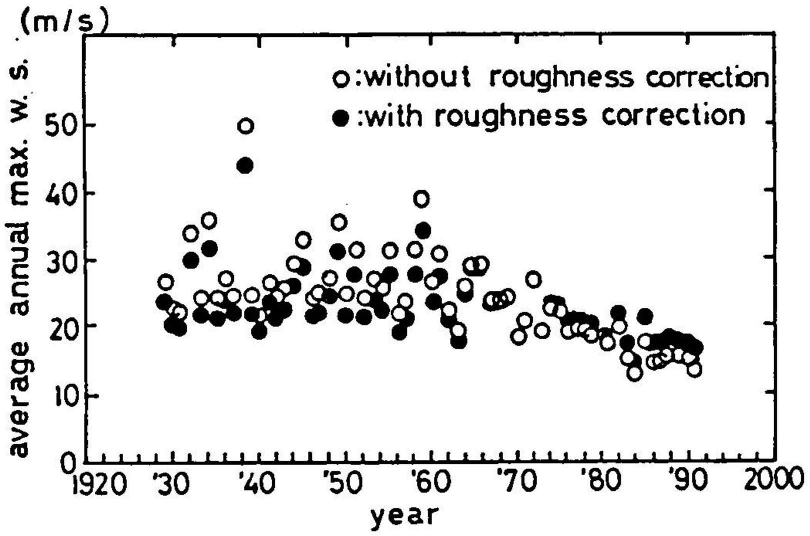

Fig. 3 *Variation of annual maximum wind speed averaged over meteorological stations in Japan* [4)]

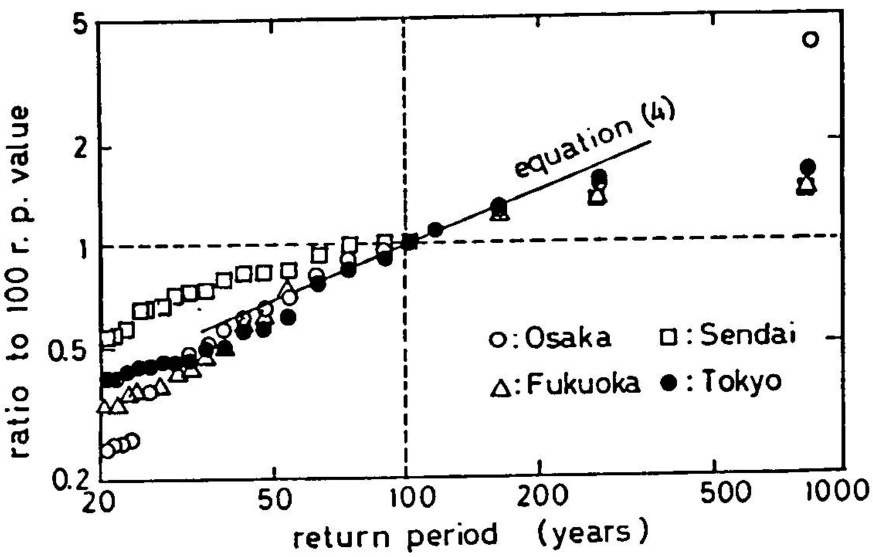

Fig. 4 *Ratio to 100 year return period value for PGA in Japan* [4)]