Documents for wind resistant design of buildings in Japan

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ABSTRACT: This paper first introduces the background to structural design of buildings in Japan. Then it introduces the 2004 version of the wind load provisions of the AIJ Recommendations for Loads on Buildings. The major revisions are as follows: replacement of gust loading factor based on tip displacement by one based on base bending moment; introduction of wind directionality factor (8 wind directions); explicit introduction of wind load combinations; correction and addition of topographic effects; substantial fulfillment of aerodynamic coefficients; and so on. Finally, the 2004 version of the AIJ Guidelines for the Evaluation of Habitability to Building Vibration is also introduced.

KEYWORDS: Building Standard Law of Japan, AIJ Recommendations, AIJ Guidelines, Wind resistant design, Habitability to vibration, Wind directionality

1 INTRODUCTION

The design of buildings constructed in Japan should be based on the Building Standard Law of Japan (BSLJ), which specifies the minimum building design requirements. The BSLJ was completely revised in 2000 and the basic structural analysis method was shifted to Performance Based Design (PBD). However, this requires greater accuracy for structural design, as well as more accurate calculation methods reflecting the real situation in evaluating wind loads.

Since the 1993 version of AIJ-RLB was issued, many relevant studies have been carried out and revision work has continued to incorporate study results. In the revised version published in September 2004, the criteria classifying buildings and assessing necessary procedures for wind load estimation for a building of interest are given as the same as those of the AIJ-RLB-1993. Simplified Procedures for small rigid buildings, Detailed Procedures for along-wind load and roof wind load for structural frames for general buildings, crosswind load and torsional load for wind sensitive buildings, and vortex-resonance and aerodynamic instabilities particularly for wind-sensitive buildings are recommended, based on the aspect ratio H/B, natural frequency f_1, design wind speed U_H and so on.

The AIJ Guidelines for the Evaluation of Habitability to Building Vibration (AIJ-GBV, hereafter) was first published in 1991, and completely revised in 2004. While the AIJ-GBV-1991 proposed four deterministic guidelines, the AIJ-GBV-2004 is based on probabilistic human perception thresholds.

2 WIND LOAD PROVISIONS IN BUILDING STANDARD LAW OF JAPAN (BSLJ-2000)

2.1 *Design approval procedure and required performance in BSLJ-2000*

When a building is to be constructed in Japan, the building owner generally has to submit the plan to the local government for approval. However, the design of buildings higher than 60m has to be approved by the Minister of Land, Infrastructure and Transport (MLIT). This approval procedure is carried out on behalf of MLIT by a designated organization such as the Building Center of Japan (BCJ). Each designated organization has formed special committees consisting of experts in various building engineering fields from universities and the Japan Structural Consultants Association.

Essentially, structural designers are required to comply with the BSLJ. The required performance and design wind load levels specified in BSLJ-2000 are listed in Table 1. Design of buildings higher than 60m is required to take into account crosswind loads and torsional loads. However, BSLJ-2000 does not specify these wind loads, but AIJ-RLB does. As AIJ-RLB is not a law, it has only been used or consulted by structural designers requiring more sophisticated building designs or compensating parts not covered by BSLJ. Thus, structural designers of tall buildings higher than 60m commonly use AIJ-RLB.

The BSLJ-2000 includes the following items based on the Gust Loading Factor method (Davenport, 1961):

- Basic wind speed depending on geographic location;
- Four terrain roughness categories;
- Simplified Gust Loading Factor based on building tip displacement;
- Different wind load estimation methods for structural frames and cladding;
- Different aerodynamic factors for structural frames and cladding; and
- Design velocity pressure at building roof height

Table 1 *Required performance and design wind load levels (BSLJ)*

Wind Load Levels		Medium-level Winds	Strongest-level Winds
Recurrence Period		50 years	500 years
Required Performance		- No damage to main frames - Cladding does not fall down	Buildings never collapse
$H \leq 60$m	Design Methods	- Allowable Stress Method - Limit Strength Method	Limit Strength Method
	Load Factor	1	1.6
	Wind Loads	Along-wind shall be checked.	
$H > 60$m	Design Method	Dynamic response analyses in time domain	
	Wind Speed Factor	1	1.25
	Wind Loads	Along-wind, crosswind, torsional, and vertical loads shall be checked.	

2.2 *Level of wind load*

The wind load provisions of BSLJ-2000 are quite similar to those of AIJ-RLB-1993. The main revisions are on levels of load, clear separation of loads for design of structural frames and design of cladding and components, introduction of exposure factor and gust loading factor.

The basic wind speed in BSLJ-2000 is based on 50-year-recurrence 10-minute-mean wind speed at 10m above ground over open flat terrain. The minimum value is set at 30m/s for the basic wind speed. The result ranges from 30 to 46m/s. A map for the basic wind speed in BSLJ-2000 is given in Fig. 1.

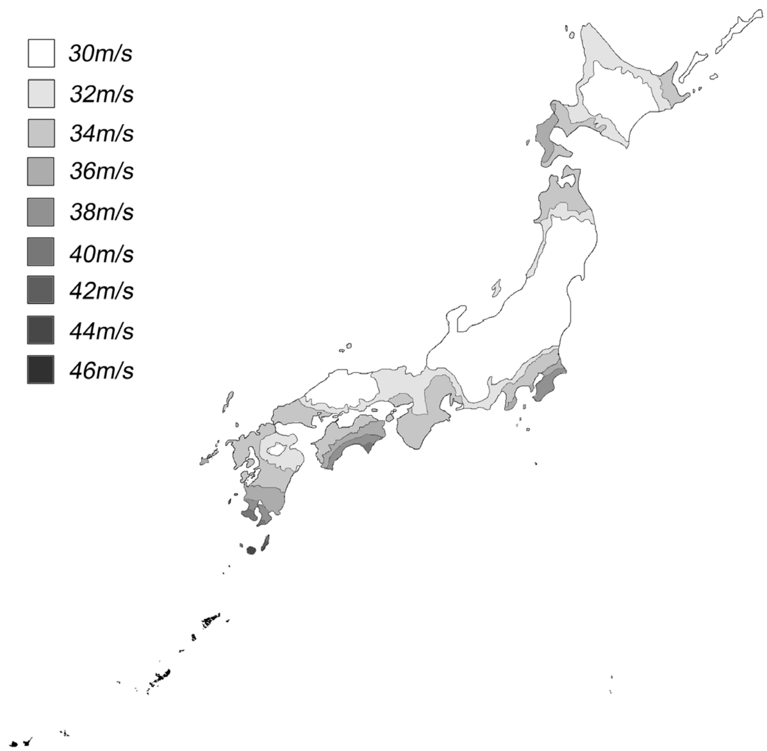

Figure 1 Basic wind speeds specified in BSLJ-2000 (50-year-recurrence)

Design wind speeds should be based on wind speeds prescribed as 1.0 and $\sqrt{1.6}$ times the basic wind speed for "No damage" and "No collapse" design criteria, respectively, as shown in Table 1. These correspond to return periods of 50 years and 500 years, respectively. "No damage" means that the stresses in the main frame members must be less than the yield stress of the material. "No collapse" means that the stresses must be less than 1.1 times the yield stress.

2.3 *Load estimation formula*

Wind loads for no damage design are given by the following formulas. The formula for structural frames is

$$W_f = 0.6 E_r^2 V_o^2 G_f C_f \tag{1}$$

where, W_f : wind load (N/m^2) for main frames, E_r : mean wind speed profile factor, G_f : gust loading factor, V_0 : basic wind speed (m/s), and C_f : wind force coefficient.

The formula for cladding and components is

$$W_c = 0.6 E_r^2 V_0^2 \hat{C}_f \qquad (2)$$

where, W_c : wind load (N/m^2) for cladding and components, and $\hat{C}_f$: peak wind force coefficient.

2.4 *Category for exposure factor*

Four terrain categories, Categories I - IV, are specified in the BSLJ-2000, but five terrain categories, Categories I - V, are defined with descriptive expressions and/or photographs showing typical examples in AIJ-RLB-1993 and 2004. Category V, representing large city centers where tall buildings are heavily concentrated, is not categorized in the provisions of BSLJ-2000.

The mean wind speed profile factor for each terrain category is given as the same as those of the AIJ-RLB-2004.

2.5 *Gust loading factor*

Table 2 shows gust loading factors prescribed in the provisions. For simplicity, the gust loading factor was derived from the basic wind speed V_0 = 35m/s, and some typical building shapes, heights and dynamic properties of buildings in Japan.

Table 2 Gust loading factor in BSLJ-2000

Terrain category	Mean height of roof		
	H≤10m	10<H<40m	40m≤H
I	2.0	Linear interpolation	1.8
II	2.2		2.0
III	2.5		2.1
IV	3.1		2.3

2.6 *Wind force coefficients*

Wind force coefficients for main frames are given as follows:

$$C_f = C_{pe} - C_{pi} \qquad (3)$$

where, C_{pe} is external wind pressure coefficient and C_{pi} is internal wind pressure coefficient. Peak wind force coefficients for cladding loads are given as follows:

$$\hat{C}_f = \hat{C}_{pe} - \hat{C}_{pi} \qquad (4)$$

where, $\hat{C}_{pe}$: peak external pressure coefficient and $\hat{C}_{pi}$: peak internal pressure coefficient. Here, attention should be paid to the peak internal pressure coefficient. Note that this is not the one that defines the real peak internal pressure; the equivalent peak internal pressure causes the maximum instantaneous wind forces. The external pressure coefficients, peak internal pressure coefficients and wind force coefficients are tabulated in the BSLJ-2000. Wind tunnel tests are also available to determine these coefficients.

3 AIJ RECOMMENDATIONS

3.1 *Major Revisions*

The following revisions were made in AIJ-RLB-2004:

- Replacement of gust loading factor based on tip displacement by one based on base bending moment;
- Introduction of wind directionality factor (8 wind directions);
- Explicit introduction of wind load combinations;
- Correction and addition of topographic effects;
- Substantial fulfillment of aerodynamic coefficients;

In addition, many other items related to wind-resistant design of buildings have been corrected or newly introduced, such as one-year recurrence wind speed, to assess habitability under wind-induced vibrations.

3.2 *Design Wind Speed*

The design wind speed U_H defined at the average roof height (reference height) H is given as:

$$U_H = U_0 K_D E_H k_{rW} \tag{5}$$

where, U_0 : basic wind speed, K_D : wind directionality factor, E_H : wind speed profile factor, and k_{rW} : return period conversion factor.

3.2.1 *Basic Wind Speed*

The basic wind speed U_0 is defined as a 100-year-recurrence wind speed in the meteorological standard conditions, i.e. the 10min mean wind speed 10m above the ground for open flat terrain. It is given as a map depending upon the geographical location. Strong winds in Japan are mainly caused by typhoons, but the effects of synoptic winds and others cannot be neglected, especially in the north part of Japan. The basic wind speed was estimated considering the effects of both typhoon winds and synoptic winds. The typhoon winds were estimated by "Typhoon Simulation" using the Monte-Carlo technique. The synoptic winds were estimated from meteorological wind data. The relation of the probability of exceedence and the synoptic wind speed was evaluated by the Modified Jensen-Franck method (Cook, 1983) and the Gumbel Distribution. The combined probability of both wind climates was derived for the calculation of basic wind speed.

3.2.2 *Wind Directionality Factor*

One of the major revisions is the introduction of wind directionality factor. In AIJ-RLB-1993, a constant wind speed is given for the design wind speed regardless of wind direction, because it was difficult to reasonably estimate wind directionality in tropical-cyclone-prone regions such as Japan. The main cause of the extremely high wind speed in most parts of Japan is typhoons. Meteorological stations in Japan have approximately 75 years of records at most. However, the annual average number of landfalls of typhoons in Japan is only three, so the number of typhoons included in the records of a particular site is very limited. When the records were divided into 8 or 16 sectors of azimuth, each sector has very few typhoon data, and the sampling error becomes very large.

Figure 2 Basic wind speed specified in AIJ-RLB-2004 (100-year-recurrence)

Tamura et al. (1995) proposed to estimate virtual typhoon wind data at any meteorological station using the correlation between observed meteorological wind records and those by the Schlömer equation commonly used in Typhoon Simulation (Mitsuta & Fujii, 1979, Matsui et al., 1997) based on atmospheric pressure data and typhoon paths. A flow chart for generating virtual typhoon wind data at meteorological stations is shown in Fig.3.

The proposed method in Fig.3 uses the correlations between the meteorological records (wind speed U_{ME} and wind direction D_{ME}) at a meteorological station and friction-free winds (wind speed U_{FF} and wind direction D_{FF}) at the same station calculated by an objective analysis using meteorological data of typhoon paths and accompanying atmospheric pressure distributions surrounding meteorological stations. The friction-free wind speed U_{SFF} and wind direction D_{SFF} evaluated by a Typhoon Simulation are converted to wind speed U_{Vir} and wind direction D_{Vir} at the height of the anemometer at the target meteorological station based on the conditional probability $P(U_{ME}|U_{FF}, D_{FF})$ and $P(D_{ME}|U_{FF}, D_{FF})$ by the Monte-Carlo technique. The various effects such as terrain roughness, topography, orography, and characteristics of typhoon paths around the site are all reflected in the virtual typhoon data U_{Vir} and D_{Vir}. The virtual typhoon data were generated at 153 meteorological stations in Japan for 5,000 years. Then, sufficient virtual typhoon wind speed data were accumulated for 8 azimuth sectors, and an extreme value analysis was made for each sector.

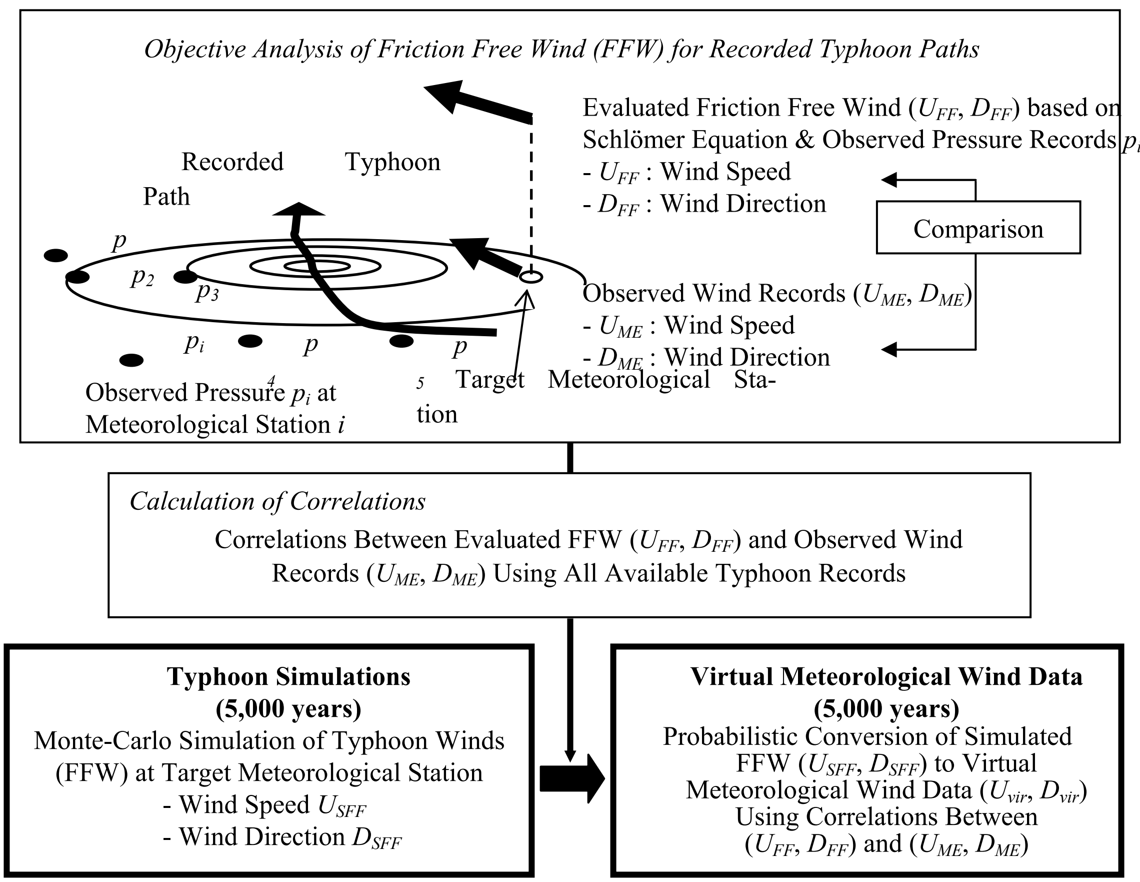

Figure 3 Generation of virtual meteorological data in tropical-cyclone-prone regions

Davenport (1969) first pointed out wind directional effects on wind load estimation, and Holmes (1981, 1990), Cook (1983), Melbourne (1984) and Simiu & Heckert (1998) discussed some important problems related to wind directionality. Melbourne's results are reflected in AS1170.2(1989) and AS/NZS1170.2(2002), in which different design wind speeds are explicitly given for different wind directions, although the application scope is limited to non-tropical cyclone regions.

If a building is oriented so that the wind pressure/force coefficients can be small for the wind directions where the wind speed is high, an economic design becomes possible. However, the design would be more risky than the conventional designs using a unique wind speed regardless of the wind directions having a specified annual exceedence probability, if the same annual exceedence probability is adopted for directional winds. To evaluate Wind Directionality Factor, the equivalent annual exceedence probability of directional wind speed corresponding to an annual exceedence probability of load effects (base shear, base moment, etc.) corresponding to 100-year recurrence is assessed under different conditions: load effects, building shape, orientation, and geographic location. Conditions also change depending on the structural frames, components and cladding. Investigations have been carried out to determine the extent of the recurrence wind speed that is appropriate to determine the wind directionality factor, and the wind directionality factor was decided as follows:

1) Calculation of 100-year recurrence wind load effect (e.g. internal force, peak pressure) based on the actual wind climate
2) Calculation of equivalent return period causing the same 100-year recurrence wind load effect in the most unfavorable case
3) Calculation of average directional wind speeds U_D based on the equivalent return period for various cases at each meteorological site

Table 3 Example of wind directionality factor in AIJ-RLB-2004

Direction	Sapporo	Tokyo	Kyoto	Osaka	Fukuoka
NE	0.85	0.85	1	0.9	0.85
E	0.85	0.85	0.95	0.85	0.85
SE	1	0.85	0.85	0.85	1
S	1	0.85	0.85	1	1
SW	0.85	0.85	0.85	1	0.85
W	0.95	0.85	0.85	1	1
NW	1	1	0.95	1	1
N	0.85	1	0.95	1	1

The equivalent return period was estimated at around 150 - 200 years for various conditions. This is because the stress in the main structural frame is generally dominant in only one or two wind directions for many buildings. Consequently, the wind directionality factor K_D was estimated as the ratio of the average directional wind speed U_D to the 100-year-recurrence basic wind speed U_0, and the lower limit was set at 0.85. Thus, the wind directionality factors K_D specified for 8 sectors of N, NE, E, SE, S, SW, W, NW are tabulated for 142 cities and areas in Japan in AIJ-RLB-2004.

3.2.3 *Wind Speed Profile Factor*

The wind speed profile factor E_H, which accounts for the change in wind speed with height, surface roughness and topographical features, is given by the following formula by setting the height $Z = H$.

$$E = E_r E_g \tag{6}$$

Here, E_r is the exposure factor for flat terrains, and is given as follows.

$$E_r = \begin{cases} 1.7\left(\dfrac{Z}{Z_G}\right)^{\alpha} & Z_b < Z \le Z_G \\ 1.7\left(\dfrac{Z_b}{Z_G}\right)^{\alpha} & Z \le Z_b \end{cases} \tag{7}$$

where, Z_G : gradient height, Z_b : interfacial layer height, and α : power-law index. These parameters are listed in Table 4 for five flat terrain subcategories.

Table 4 Parameters for exposure factor

Flat Terrain Subcategories	I	II	III	IV	V
Z_b (m)	5	5	10	20	30
Z_G (m)	250	350	450	550	650
α	0.1	0.15	0.2	0.27	0.35

E_g in Eq.(6) is the topography factor for mean wind speed, and is given as follows.

$$E_g = (C_1 - 1)\left\{C_2\left(\frac{Z}{H_s} - C_3\right) + 1\right\}\exp\left\{-C_2\left(\frac{Z}{H_s} - C_3\right)\right\} + 1,\ (E_g \ge 1) \tag{8}$$

where, H_S : the height of an escarpment or a ridge, and C_1, C_2, and C_3 are parameters depending upon the angle of inclination θ_S of the upwind slope and the distance X_S from the upper edge of the escarpment or the ridge. These parameters are tabulated in AIJ-RLB-2004 for escarpments and ridges. Table 5 is an example of escarpments with a slope angle $\theta = 30°$.

Table 5 Example of parameters for escarpments in AIJ-RLB-2004

Escarpment ($\theta_S = 30°$)										
X_s/H_s	-4	-2	-1	-0.5	0	0.5	1	2	4	8
C_1	0.7	-0.5	1.05	1.1	1.3	1.3	1.2	1.2	1.15	1.1
C_2	0.65	1.2	1.65	1.5	1.45	1.3	0.9	0.9	0.85	0.6
C_3	-2	-2	1	0.8	0.3	0.3	0.5	0.7	1.2	1.4

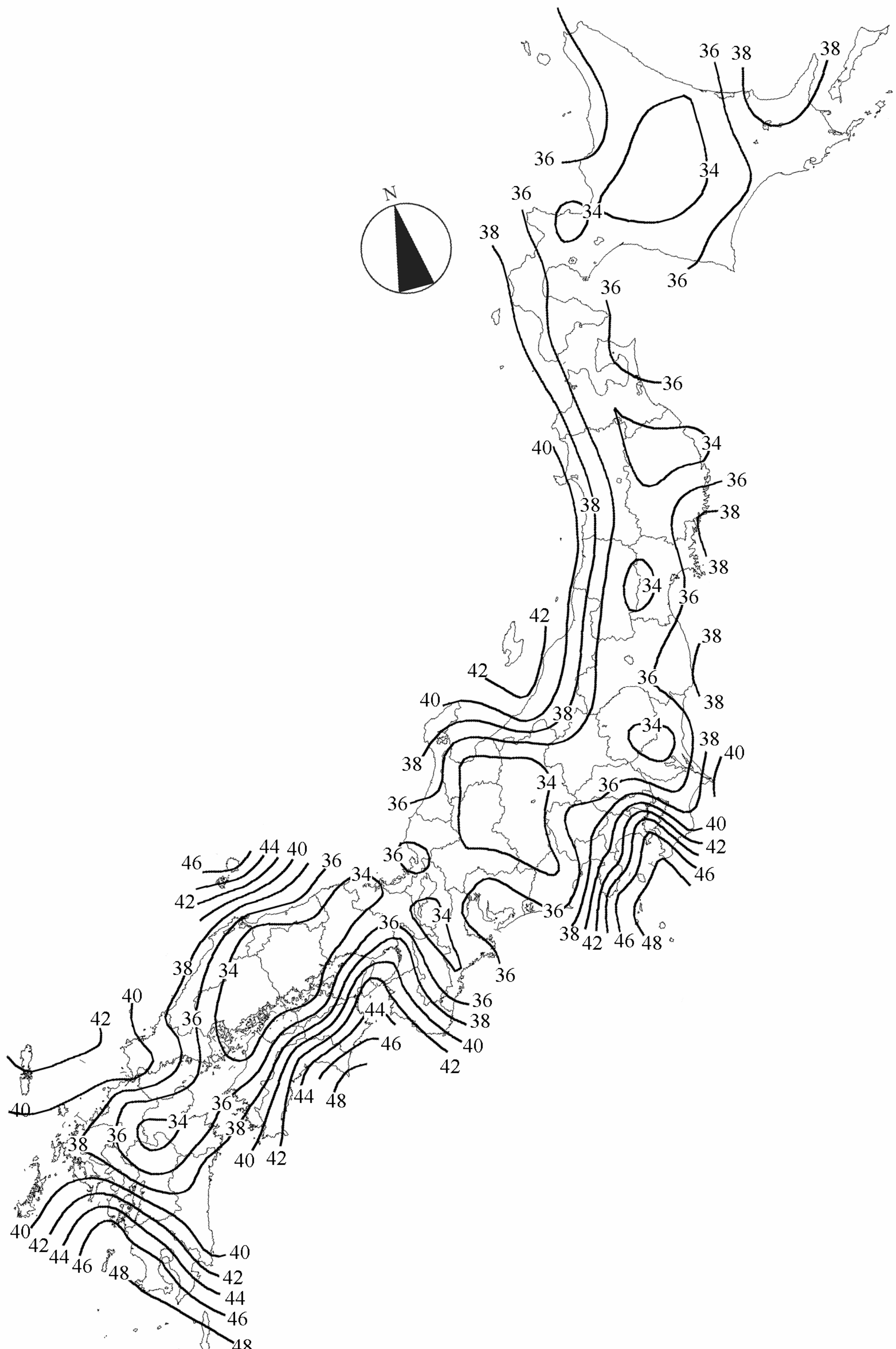

Figure 4 U_{500} specified in AIJ-RLB-2004 (500-year-recurrence)

The significance of topographic effects in Japan is well recognized, but AIJ-RLB-1993 specifies the speed-up ratio only for 2D escarpments. In this respect, a series of wind tunnel tests and numerical simulations has been carried out [Meng & Hibi (1998), Kondo et al. (2001, 2002), Kawai & Kondo (2003)], in an organized way, and speed-up ratios for escarpments and ridges are specified in AIJ-RLB-2004. The topographic effects on turbulence intensity are also specified in AIJ-RLB-2004.

3.2.4 *Return Period Conversion Factor*

The return period conversion factor k_{rW} is given as follows:

$$k_{rW} = 0.63(\lambda_U - 1)\ln r - 2.9\lambda_U + 3.9 \tag{9}$$

$$\lambda_U = \frac{U_{500}}{U_0} \tag{10}$$

where, U_{500} : 500-year-recurrence wind speed for the meteorological standard conditions, and U_0 : basic wind speed (100-year-recurrence). The contour map is also given for U_{500} in AIJ-RLB-2004 as shown in Fig.4.

3.2.5 *Turbulence Intensity and Turbulence Scale*

The turbulence intensity and the turbulence scale at height Z is given by the following formula.

$$I_Z = I_{rZ} E_{gI} \tag{11}$$

Here, I_{rZ} is the turbulence intensity for flat terrains, and is given as follows:

$$I_{rZ} = \begin{cases} 0.1\left(\dfrac{Z}{Z_G}\right)^{-\alpha-0.05} & Z_b < Z \le Z_G \\ 0.1\left(\dfrac{Z_b}{Z_G}\right)^{-\alpha-0.05} & Z \le Z_b \end{cases} \tag{12}$$

where, Z_G, Z_b, and α are the parameters given in Table 4. E_{gI} in Eq.(11) is the topography factor for the turbulence intensity.

$$E_{gI} = \frac{E_I}{E_g} \tag{13}$$

Here, E_g is the topography factor for the mean wind speed given by Eq.(8), and E_I is that for the fluctuating component of wind

$$E_I = (C_1 - 1)\left\{C_2\left(\frac{Z}{H_S} - C_3\right) + 1\right\}\exp\left\{-C_2\left(\frac{Z}{H_S} - C_3\right)\right\} + 1,\ (E_I \ge 1) \tag{14}$$

where, H_S is the height of an escarpment or a ridge, and C_1, C_2, and C_3 are tabulated in AIJ-RLB-2004 for escarpments and ridges.

The turbulence scale L_Z (m) at height Z is given by the following formula for every terrain category:

$$L_Z = \begin{cases} 100\left(\dfrac{Z}{30}\right)^{0.5} & 30\text{m} < Z \le Z_G \\ 100 & Z \le 30\text{m} \end{cases} \tag{15}$$

3.3 *Along-wind Loads*

3.3.1 *Along-wind Loads for Ordinary Buildings*

The Gust Loading Factor (GLF) used in AIJ-RLB-1993 is based on the tip displacement following the original GLF proposed by Davenport (1961). Holmes (1994, 1996) proposed a more sophisticated GLF for lattice towers, which enables estimation of any load effects such as bending moments and shear forces at any elevation. This is also applicable to general prismatic buildings (Holmes, 2002). Zhou & Kareem (2001) proposed to use the GLF based on the base bending moment (BBM) to provide more realistic equivalent static wind load (ESWL). Recently, an "e-database" of aerodynamic wind loads was introduced by Zhou et al. (2003).

Reviewing the recent development of the GLF method, AIJ-RLB-2004 will adopt the BBM-based GLF rather than the traditional GLF. However, it is not equal to that proposed by Zhou & Kareem (2001). The ESWL proportional to the mean wind load distribution will still be adopted in AIJ-RLB-2004 for continuity of the current version. It was also thought to be too early to completely change the concept and the method of GLF, considering the timing just after the BSLJ-2000 newly adopted the traditional GLF in the law. The ESWL at height *Z* is given as:

$$W_D = q_H C_D G_D A \tag{16}$$

where, $q_H = \rho U_H^2/2$: design velocity pressure at the mean roof height H (reference height), C_D: aerodynamic factor at height Z, G_D : BBM-based gust loading factor, and A: projected area at height Z. The BBM-based GLF is given as:

$$G_D = 1 + g_D \frac{C_g'}{C_g}\sqrt{1+{\phi_D}^2 R_D} \tag{17}$$

where, g_D : peak factor, C_g' and C_g: fluctuating and mean coefficients for along-wind overturning moment, ϕ_D: correction factor depend on the mode shape of along-wind vibration, and R_D: resonance factor. The formulae for calculating these parameters are given as functions of dimensions and dynamic characteristics of the building and various wind parameters. The correction factor ϕ_D depending on the mode shape is given as:

$$\phi_D = \frac{1-0.4\ln\beta}{2+\beta}\frac{M}{M_D} \tag{18}$$

Here, the vibration mode shape μ is approximated by the following equation:

$$\mu = \left(\frac{Z}{H}\right)^{\beta} \tag{19}$$

and M : total building mass excluding underground part, and M_D: generalized mass of fundamental mode of along-wind vibration.

3.3.2 *Along-wind Loads for Tower-like Lattice Structures*

Wind loads on tower-like lattice structures that are not given in AIJ-RLB-1993 are incorporated in the new revision. Unlike the cases for general prismatic buildings, wind force can be evaluated from combination of member wind forces by velocity pressure based on the local wind speed at the positions of individual members. Therefore, only wind loads on tower-like structures can be determined based not on the velocity pressure at the reference height q_H but on the velocity pressure at individual heights q_z.

3.4 *Wind Loads on Roof Structures*

Roof wind loads were given in the AIJ-RLB-1993 as follows:

$$W_R = q_H\left(C_{pe}G_{pe} - C_{pi}G_{pi}\right)A_R$$

In an expedient manner, it seems to be described such that the difference between peak external pressure and peak internal pressure is considered. The expression has been modified to meet the essential concept of the GLF describing the maximum load effect by multiplying the GLF by the mean wind load effect. However, because there are cases where the difference between internal and external pressure coefficients becomes zero, the GLF method is not always valid. In such cases, the wind load calculation takes into account only the fluctuating component of pressure. There are cases where the downward wind load becomes critical, considering the combination with fixed load, snow load, and so on, even if its absolute value is small. The revised version also provides roof wind load for such cases. Then, the ESWL for the roof structures of buildings without any predominant openings is given as:

$$W_R = q_H C_R G_R A_R \tag{20}$$

$$C_R = C_{pe} - C_{pi} \tag{21}$$

where q_H: design velocity pressure, C_{pe}: external pressure coefficient, C_{pi}: internal pressure coefficient, G_R : gust loading factor, and A_R: subjected area for a roof beam.

The GLF is given as:

$$G_R = 1 \pm \frac{\sqrt{12.3 r_{\mathrm{Re}}^2\left(1+R_{Re}\right)+0.3 r_c^{\ 2}}}{\left|1-r_c\right|}, \qquad \text{for the case } C_{pi} = -0.4 \text{ and } C_R \neq 0 \tag{22}$$

$$C_R G_R = \pm 0.25\sqrt{12.3 r_{\mathrm{Re}}^2\left(1+R_{Re}\right)+0.3}, \quad \text{for the case } C_{pi} = -0.4 \text{ and } C_R = 0 \tag{23}$$

$$G_R = 1 \pm \sqrt{12.3 r_{\mathrm{Re}}^2\left(1+R_{Re}\right)+0.3 r_c^{\ 2}}, \qquad \text{for the case } C_{pi} = 0 \tag{24}$$

Here, the three parameters in Eqs.(22), (23) and (24) are given as follows. For the roof beam parallel to the wind direction:

$$r_c = 0.08\frac{L}{H} + 0.25$$

$$r_{Re} = \begin{cases} \left(0.23 + 3.5 I_H{}^2\right)\exp\left(-0.15\dfrac{L}{H}\right) & L/H \le 4 \\ 0.13 + 1.9 I_H^2 & L/H > 4 \end{cases}$$

$$R_{\mathrm{Re}} = 0.006\left(\frac{f_R H}{U_H}\right)^{-3}\frac{\pi}{4\zeta_R}.$$

For the roof beam normal to the wind direction:

$$r_c = -\frac{0.4}{C_{pe}}$$

$$r_{Re} = \begin{cases} \left(0.15 + 5 I_H{}^2\right)\exp\left(-0.1\dfrac{L}{H}\right) & L/H \le 6 \\ 0.082 + 2.7 I_H^2 & L/H > 6 \end{cases}$$

$$R_{\mathrm{Re}} = 0.015\left(\frac{f_R H}{U_H}\right)^{-3}\frac{\pi}{4\zeta_R},$$

where I_H: turbulence intensity at the reference height H, L: roof span, f_R and ζ_R: natural frequency and damping ratio of the fundamental vibration mode of the roof beam.

It is noted that the product of the wind force coefficient C_R and the GLF G_R in Eq.(20) is given by Eq.(23), and the GLF is not separately defined for the case $C_R = 0$.

3.5 *Crosswind Loads and Torsional Loads*

The crosswind load and torsional wind loads should be examined in design of slender and flexible buildings to satisfy the following conditions:

$$\frac{H}{\sqrt{BD}} \ge 3 \tag{25}$$

where, H: building height, B and D: building width and depth, and f_1: the smaller of the fundamental natural frequencies of crosswind vibration and torsional vibration.

3.5.1 *Crosswind Loads*

The crosswind load at height Z is given by the following formulae:

$$W_L = 3 q_H C_L' A \frac{Z}{H} g_L \sqrt{1 + \phi_L^2 R_L} \tag{26}$$

where $C_L' = 0.0082(D/B)^3 - 0.071(D/B)^2 + 0.22(D/B)$: fluctuating crosswind overturning moment coefficient, A : projected area of the building, g_L: peak factor, ϕ_L: correction factor depending on the mode shape of the crosswind vibration, and R_L: resonance factor. The formulae for calculating these parameters are given as functions of dimensions and dynamic characteristics of the building and various wind parameters. The correction factor ϕ_L is given as:

$$\phi_L = \frac{M}{3M_L}\left(\frac{Z}{H}\right)^{\beta-1}(1-0.4\ln\beta) \tag{27}$$

where, M_L : generalized mass of fundamental mode of crosswind vibration.

3.5.2 *Torsional Wind Loads*

The torsional wind load at height Z is given by:

$$W_T = 1.8 q_H C'_T AB\frac{Z}{H} g_T\sqrt{1+\phi_T{}^2 R_T} \tag{28}$$

where, $C'_T = \{0.0066+0.015(D/B)^2\}^{0.78}$: fluctuating torsional moment coefficient, A : projected area of the building, B : breadth of the building, g_T : peak factor, ϕ_T : correction factor depending on the mode shape of the crosswind vibration, and R_T : resonance factor. The formulae for calculating these parameters are given as functions of dimensions and dynamic characteristics of the building and various wind parameters. The correction factor ϕ_T is given as:

$$\phi_T = \frac{M(B^2+D^2)}{36I_T}\left(\frac{Z}{H}\right)^{\beta-1}(1-0.4\ln\beta) \tag{29}$$

where, M : total building mass excluding underground part, and I_T : generalized inertial moment of the fundamental mode of torsional vibration.

3.6 *Vortex Resonance and Aerodynamic Instabilities*

Vortex resonance and aerodynamic instabilities should be examined for particularly wind-sensitive buildings and structures satisfying the following conditions. The condition for structures with a rectangular plan is given by:

$$\frac{H}{\sqrt{BD}} \ge 4 \quad \text{and} \quad \left(\frac{U_H}{f_L\sqrt{BD}} \ge 0.83U^*_{Lcr} \quad \text{or} \quad \frac{U_H}{f_T\sqrt{BD}} \ge 0.83U^*_{Tcr}\right) \tag{30}$$

That for structures with a circular plan is given by:

$$\frac{H}{D_m} \ge 7 \quad \text{and} \quad \frac{U_H}{f_L D_m} \ge 4.2 \tag{31}$$

Here, f_L and f_T: fundamental natural frequencies of crosswind and torsional vibrations, respectively; and U^*_{Lcr} and U^*_{Tcr} : non-dimensional onset wind speeds for crosswind and torsional instabilities. U^*_{Lcr} and U^*_{Tcr} are tabulated for structures with a rectangular plan, and vary with the side ratio D/B, the mass-damping-parameter δ, and the terrain categories.

In design of these types of buildings or structures, some appropriate investigations including wind tunnel tests are recommended. Incidentally, AIJ-RLB-2004 specifies wind load formulae only for buildings and structures with a circular plan or with cylindrical members.

3.7 *Combinations of Wind Load Components*

Wind pressure acting on a building surface spatio-temporally fluctuates in a complicated manner and never acts symmetrically even for a second. Therefore, for example, even if the along-wind force is the largest value, the other components are not zero. Even for rigid low- and middle-rise buildings, shear force and normal stress in the columns are affected by these combined wind loads. High-rise buildings, etc. should take into account the combination of along-wind, across-wind and torsional components of dynamic response.

In AIJ-RLB-1993, the wind load combinations are not strongly recommended, because the same design wind speed estimated from the annual maximum wind speeds regardless of the wind direction has to be applied for the most severe wind direction. However, if wind directionality is adopted in AIJ-RLB-2004, it will become important to take the wind load combinations into account in design.

3.7.1 *Combinations of Horizontal Wind Loads*

The wind load combinations for low- medium- and high-rise building models have been investigated comprehensively (Tamura et al., 2002a, 2002b, Kikuchi et al., 2002), and the combination method is clearly shown in the revised version. AIJ-RLB-2004 gives two methods.

The first is applicable even without information on crosswind or torsional responses. For buildings not satisfying the conditions given by Eq.(25), the crosswind force W_{LC} given by the following formula should be applied simultaneously with the along-wind load W_D given by Eq.(16) as shown in Fig. 5.

$$W_{LC} = 0.35\frac{D}{B}W_D\,, \quad (W_{LC} \geq 0.2W_D) \tag{32}$$

This formula is based on studies by Tamura et al. (2002a) and Hibi et al. (2003). Tamura et al. (2002a) reported that the peak normal stresses in the columns could be almost 30% underestimated on average for both low-rise and middle-rise building models if only the along-wind force component is taken into account in structural design. Hibi et al. (2003) discusses the wind load combination effects based on comprehensive pressure data for various building models in terms of the peak normal stress in columns, including the dynamic resonant effects. It has proposed a wind load combination factor γ defined as $W_{LC} = \gamma W_D$, which is the crosswind load applied with the long-wind load. The formula of $\gamma = 0.34(D/B) + 0.05$ is recommended for low- and medium-rise buildings, where D and B are the along-wind and crosswind dimensions of the building plan. This formula was slightly modified and used in AIJ-RLB-2004 as Eq.(32).

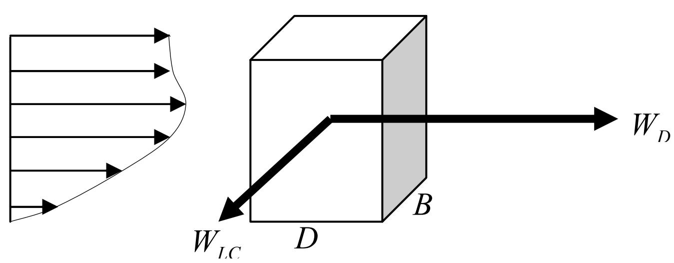

Figure 5 Wind load combination for low- and medium-rise buildings in AIJ-RLB-2004

For buildings satisfying the conditions of Eq.(15), AIJ-RLB-2004 applies a more detailed method, based on analytical results using the correlation method by Asami (2000, and 2002) for the combination of load effects by along-wind, crosswind, and torsional actions. Asami (2000) proposed wind load combination methods considering the correlations of along-wind, crosswind and torsional responses based on the spectral modal technique, where 8 design points are used to approximate the response trajectory. The load combination factor is defined as a function of the correlation coefficient between the crosswind response and the torsional response [Asami (2002)]. As is well known, Solari & Pagnini (1999) proposed a wind load combination method using 12 design points enveloping the possible elliptic trajectory of the along-wind and cross-wind responses. Table 6 gives the wind load combinations to be considered.

W_D, W_L, and W_T in Table 6 are given by Eqs.(16), (26), and (28). ρ_{LT} in Table 6 is the correlation coefficient between the crosswind response and the torsional response, and is tabulated as a parameter depending upon the side ratio D/B, the frequency ratio f_θ/f_L, and the reduced frequency $f_1 B/U_H$. Here, f_θ and f_L are the fundamental natural frequency of the torsional vibration and the crosswind vibration, respectively. f_1 is the smaller of f_θ and f_L.

Table 6 Wind load combinations for high-rise buildings in AIJ-RLB-2004

Combination	Along-wind Load	Crosswind Load	Torsional Load
1	W_D	$0.4W_L$	$0.4W_T$
2	$W_D\left(0.4+\dfrac{0.6}{G_D}\right)$	W_L	$\left(\sqrt{2+2\rho_{LT}}-1\right)W_T$
3	$W_D\left(0.4+\dfrac{0.6}{G_D}\right)$	$\left(\sqrt{2+2\rho_{LT}}-1\right)W_L$	W_T

3.7.2 *Combinations of Horizontal Wind Loads and Roof Wind Load*

Simultaneously acting horizontal wind loads and roof wind loads and their combinations need to be considered. Horizontal wind loads might be dominant in high-rise buildings and roof wind loads might be dominant in long-span structures. However, demarcation is difficult because there are many intermediate buildings. As an expedient method, it is recommended to simply superimpose the two loads. The following recent results support this method. Tamura et al. (2003) reported that the vertical component of the wind force acting on medium-rise buildings tended to become largest when one of the horizontal wind force components, i.e. along-wind, crosswind or torsional component, reached its maximum value.

3.8 *Wind Loads for Components and Cladding*

The wind loads for the components and cladding is expressed by the following equation in AIJ-RLB-1993.

$$W_C = q_H\left(C_{pe}G_{pe} - C_{pi}G_{pi}\right)A_C$$

However, this expression may also be misunderstood by structural designers if the external wind pressure and the internal pressure always reach their peak values simultaneously. This expression was also modified to:

$$W_C = q_H \hat{C}_C A_C \tag{33}$$

in AIJ-RLB-2004, where, $\hat{C}_C$ is a peak wind force coefficient and is given by:

$$\hat{C}_C = \hat{C}_{pe} - C^*_{pi} \tag{34}$$

Here, $\hat{C}_{pe}$: peak external pressure coefficient, and C^*_{pi}: coefficient accounting for the effect of the internal pressure fluctuation. $\hat{C}_{pe}$ is given for various building surface locations, and is a function of a tributary area A_C. Regarding C^*_{pi}, 0 and -0.5 should both be considered for buildings without any dominant openings.

3.9 *Aerodynamic Shape Factors*

Attempts have been made to thoroughly review the wind force coefficient and wind pressure coefficient reflecting data accumulated for about 10 years since the publication of AIJ-RLB-1993. These expressions have been revised in line with the revised formula for the roof wind load and for the components and cladding. Furthermore, the wind force coefficients and wind pressure coefficients for domes, canopies, eaves, lattice structures, various cross-sectional shaped members, etc. have been added to assist designers based on recent wind tunnel results, e.g. Uematsu & Yamada (1994), Ueda et al. (1997), Ohtake (2000, 2001), Noguchi & Uematsu (2003), Kikuchi et al. (2003), Uematsu (2003).

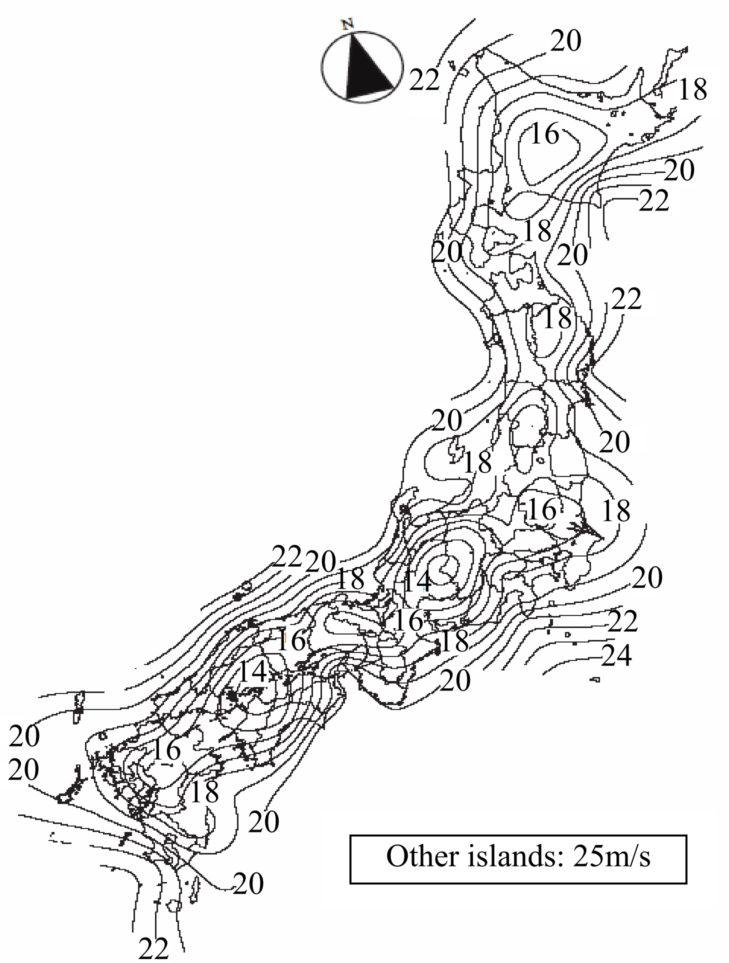

Figure 6 One-year-recurrence wind speed for evaluation of habitability to building vibrations

3.10 *One-Year-Recurrence Wind Speed*

In order to assess the habitability of buildings subject to wind-induced vibrations, the AIJ Guidelines for the Evaluation of Habitability to Building Vibration (AIJ-GBV) was published in 1991, in which the one-year-recurrence peak acceleration has been applied for the evaluation. The evaluation method for wind-induced acceleration and the one-year-recurrence directional wind speeds in the major cities of Japan are given in the Appendices. Incidentally, the AIJ-GBV was revised in May of 2004.

3.11 *Miscellaneous*

There are several provisions for assisting structural designers and wind engineering practitioners, such as the Simplified Method for wind loads on small rigid buildings, evaluation formulae for along-wind, crosswind and torsional acceleration responses, interference effects of neighboring buildings, which are given in AIJ-RLB-2004.

Furthermore, in order to achieve reliability based design, the uncertainty and dispersion of parameters included in AIJ-RLB-2004 are evaluated and discussed.

4 AIJ GUIDELINES FOR EVALUATION OF HABITABILITY TO BUILDING VIBRATION (AIJ-GBV)

AIJ-GBV-2004 consists of three parts: human- and machine-induced vertical vibrations; traffic-induced vertical/horizontal vibrations; and wind-induced horizontal vibrations. Here, the provisions of the evaluation of habitability to wind-induced horizontal vibrations are introduced. After publication of the AIJ-GBV-1991, a series of experiments on the human perception threshold and researches on habitability to building vibrations were conducted, and many results were published, e.g. Shioya et al. (1992), Nakata et al. (1993), Denoon (2000) and Inoue et al. (2003). AIJ-GBV-2004 is based on those data.

Major revisions have been made in the expressions of the guidelines and the expansion of the frequency range up to 5Hz. The latter were made because of problems occurring in urban areas in Japan in wind-induced vibrations of relatively low-rise wooden or steel residential buildings. Instead of giving some deterministic recommended line, the AIJ-GBV-2004 only gives five curves: H-10, H-30, H-50, H-70 and H-90 shown in Fig.7. The number of each curve indicates the perception probability as a percentage, i.e. 10% of people can perceive the vibration specified by the H-10 curve.

The essential concept of AIJ-GBV-2004 is that the criteria for building habitability to vibration should be decided by a building owner. Although AIJ-GBV-1991 specified the H-2 curve and H-3 curve as the standard level for residential buildings and office buildings, respectively, no specified criteria is recommended in AIJ-GBV-2004. However, it may be difficult to judge the most appropriate vibration level and to select one of the curves as the design target for a given building. For designers' reference, calculated along-wind and crosswind responses are compared with the curves in the commentary of AIJ-GBVs-2004. Wind-induced responses of 286 buildings were calculated from formulae for the maximum along-wind and crosswind acceleration responses. In the response calculations, the measured full-scale values of the natural frequency and the damping ratio in the database collected by the AIJ Sub-Committee on Damping in Buildings (Tamura et al., 2000) were used. Several figures are plotted to show different building usages, e.g. offices, hotels and residences, and different building types, e.g. steel buildings, steel encased reinforced concrete buildings, and reinforced concrete buildings. Figure 8 shows an example. Owners and structural designers can select the design target or criteria that the building should satisfy based on such information.

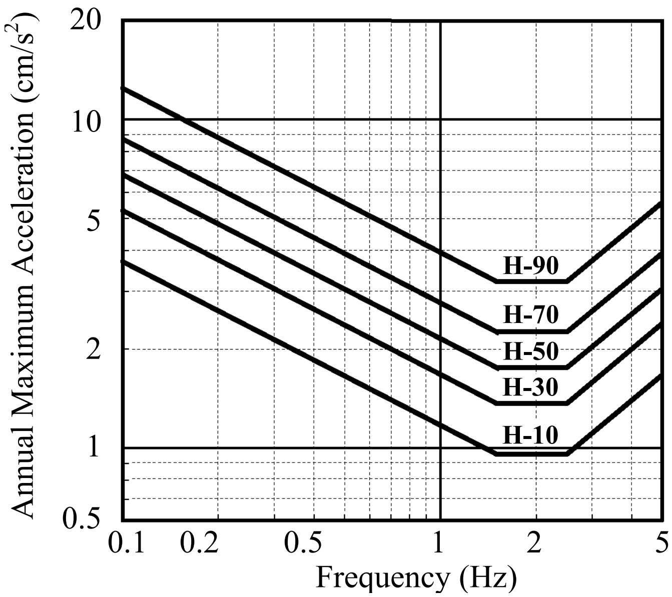

Figure 7 Probabilistic perception thresholds given in AIJ-GBV-2004

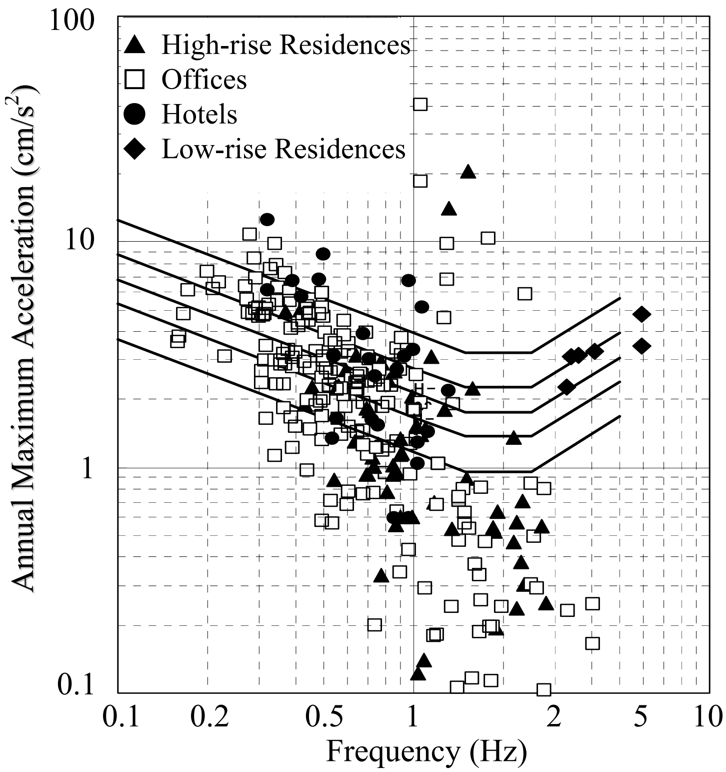

Figure 8 Comparisons with existing buildings given in AIJ-GBV-2004

5 CONCLUDING REMARKS

This paper has briefly introduced three important documents for wind resistant design of buildings in Japan: BSLJ-2000; AIJ-RLB-2004; and AIJ-GBV-2004. The most important revision in AIJ-RLB-2004 was to the design wind speed, i.e. the adoption of wind directionality in tropical-cyclone-prone regions. The probabilistic perception thresholds are given in the AIJ-GBV-2004 instead of the deterministic guidelines in the previous version.

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REFERENCES

AIJ-GBV (1991), *Guidelines for the evaluation of habitability to building vibration*, Architectural Institute of Japan, Maruzen, pp.82. (in Japanese)

AIJ-GBV (2004), *Guidelines for the evaluation of habitability to building vibration*, Architectural Institute of Japan, Maruzen, pp.132. (in Japanese)

AIJ-RLB (1993), Recommendations on Loads for Buildings, Architectural Institute of Japan, pp.512 (in Japanese, English version: 1996)

AIJ-RLB (2004), Recommendations on Loads for Buildings, Architectural Institute of Japan, pp.651 (in Japanese)

AS 1170.2 (1989), Australian Standard, SAA Loading Code, Part 2: Wind loads

Asami, Y. (2000) Combination method for wind loads on high-rise buildings, Proceedings of the 16th National Symposium on Wind Engineering, Tokyo, Japan, pp.531-534 (in Japanese)

Asami, Y. (2002) Correlation method for wind load combinations, Document presented at Subcommittee on Wind Loading, Architectural Institute of Japan, 2002 (in Japanese)

AS/NZS 1170.2 (2002), Australian / New Zealand Standard, Structural design actions, Part 2: Wind actions

BSLJ (2000), Building Standard Law of Japan, Enforcement Orders, Regulations and Notifications by Ministry of Land, Infrastructure and Transport

Cook, N.J. (1983), Note on directional and seasonal assessment of extreme winds for design, *Journal of Wind Engineering and Industrial Aerodynamics*, **12**, pp.365-372

Davenport, A. G. (1961), The application of statistical concepts to the wind loading of structures, *Proceedings of Institution of Civil Engineers*, London, **19**, pp.449-472.

Davenport, A.G., (1969), Structural safety and reliability under wind action, *Proceedings of International Conference on Structural Safety and Reliability*, pp.131-145

Denoon, R.O. (2000), "Designing for serviceability accelerations in buildings", *PhD Thesis*, The University of Queensland, Australia.

Holmes J.D. (1981), Reduction factors for wind direction for use in codes and standards, *Colloque, Construire Avec Le Vent*, Nantes, France, pp.VI.2.1-VI.2.15

Holmes, J.D. (1990), Directional effects on extreme wind loads, *Civil Engineering Transactions*, Institution of Engineers, Australia, **CE32.1**, pp.45-50

Holmes, J.D. (1994), Along-wind response of lattice towers: part I - derivation of expressions for gust response factors. *Engineering Structures*, **16**, pp.287-292

Holmes, J.D. (1996), Along-wind response of lattice towers: part II - aerodynamic damping and deflections. *Engineering Structures*, **18**, pp.483-488

Holmes, J. D. (2002), Gust loading factor to dynamic response factor (1967- 2002), *Symposium Preprints, Engineering Symposium to Honour Alan G. Davenport for his 40 Years of Contributions*, The University of Western Ontario, June 20 - 22, 2002 (AGD2002), pp. A1-1-A1-8

Inoue, K., Ishikawa, T. and Noda, C. (2003), "Evaluation curve and material to explain habitability of horizontal vibration -- Part 1 Evaluation curve based on perception by physical and visual feeling --", *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, pp. 299-300 (in Japanese).

Kawai, H. and Kondo, K. (2003) Topographic multiplier around micro-topography, -- In case of two dimensional escarpment and hill --, *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.103-104 (in Japanese)

Hibi, K. Tamura, Y. and Kikuchi, H. (2003) Peak normal stresses and wind load combinations of middle-rise buildings, *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.113-114 (in Japanese)

Kikuchi, H., Ueda, H. and Hibi, K. (2003) Characteristics of wind pressures acting on curved roofs, *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.147-148 (in Japanese)

Kondo, K., Kawai, H. and Kawaguchi, A. (2001) Topographic multipliers for mean and fluctuating wind velocities around up-slope cliffs, *Summaries of Technical Papers of Annual Meeting,* Architectural Institute of Japan, Structures I, pp.105-106 (in Japanese)

Kondo, K., Tsuchiya, M. and Sanada, S. (2002) Evaluation of effects of micro-topography on design wind velocity, *Journal of Wind Engineering and Industrial Aerodynamics*, **90**, pp.1707-1718

Matsui M., Meng, Y. and Hibi, K. (1997), Extreme typhoon wind speeds considering the random variation in a full-scale observation, *Proceedings of ICOSSAR'97*, pp.1343-1349

Melbourne, W.H. (1984), Designing for directionality, *1st Workshop on Wind Engineering and Industrial Aerodynamics*, Highett, Victoria, Australia, July 1984, pp.11

Meng, Y. and Hibi, K. (1998) An experimental study of turbulent boundary layer over steep hills, Proceedings of the 15th National Symposium on Wind Engineering, pp.61-66 (in Japanese)

Mitsuta, Y., Fujii T. and Kawahira K., Analysis of typhoon pressure patterns over Japanese Islands, *National Disaster Science*, Vol.1, N0.1, pp.3-19, 1979

Nakata, S., Tamura, Y. and Otsuki, T. (1993), "Study on habitability to horizontal vibration of low rise buildings", *International Colloquium on Structural Serviceability of Buildings*, Göteborg, Sweden, IABSE Reports, Vol.69, pp.39-44.

Noguchi, M. and Uematsu, Y. (2003) Design wind pressure coefficients for spherical domes, JAWE, *Journal of Wind Engineering*, **95**, pp.177-178 (in Japanese)

Ohtake, K. (2000) Peak wind pressure coefficients for cladding of a tall building, Part 1, Characteristics of peak wind pressure, *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.193-194 (in Japanese)

Ohtake, K. (2001) Peak wind pressure coefficients for cladding of a tall building, Part 2, Stretch of peak wind pressure, *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.143-144 (in Japanese)

Simiu, E. and Heckert, N.A. (1998), Wind direction and hurricane-induced ultimate wind loads, *Journal of Wind Engineering and Industrial Aerodynamics*, **74-76**, pp.1037-1046

Shioya, K., Kanda, J., Tamura, Y. and Fujii, K. (1992), "Human perception thresholds of two dimensional horizontal motion", *Structures Congress ’92, Compact Papers*, ASCE, San Antonio, USA, pp.480-483.

Solari, G. and Pagnini, L.C. (1999) Gust buffeting and aeroelastic behavior of poles and monotubular towers. Journal of Fluid and Structures, Vol.13, pp.877-905.

Tamura, Y., Goto S. and Watanabe, Y. (1995), Design wind speed estimation by simulation of typhoon winds - Evaluation of wind direction factor based on wind-direction correlation method-, *Proceedings of the 3rd Japan Conference on Structural Safety and Reliability*, pp.483-486

Tamura, Y., Kikuchi, H. and Hibi, K. (2002a) Quasi-static wind load combinations for low- and middle-rise buildings, *AGD2002 Symposium Preprints*, Engineering Symposium to Honor Alan G. Davenport for His 40 Years of Contributions, The University of Western Ontario, Canada, pp.C3-1 - C3-12

Tamura, Y., Kikuchi, H. and Hibi, K. (2002b) Quasi-static wind load combinations for buildings, *The Second International Symposium on Wind and Structures*, Busan, Korea, August 21-23, pp.61-68

Tamura, Y., Kikuchi, H. and Hibi, K. (2003) Wind load combinations for middle-rise buildings (Part1 Wind force combinations), *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, Structures I, pp.111-112

Ueda, H., Hagura. H. and Oda, H. (1997) Characteristics of stress generated by wind pressures and wind loads acting on stiff two-hinged arches supporting a barrel roof, Journal of Structural and Construction Engineering, (Transactions of AIJ), Architectural Institute of Japan, pp.29-35 (in Japanese)

Uematsu, Y. (2003) Design wind force coefficients for free-standing canopy roofs, JAWE, *Journal of Wind Engineering*, **95**, pp.181-182 (in Japanese)

Uematsu, Y. and Yamada, M. (1994) Aerodynamic forces on circular cylinders of finite height, *Journal of Wind Engineering and Industrial Aerodynamics*, **51**, pp.249-265

Zhou, Y. and Kareem, A. (2001), Gust loading factor: new model, *Journal of Structural Engineering*, ASCE, **127(2)**, pp.168-175

Zhou, Y. Kijewski, T. and Kareem, A. (2003) Aerodynamic loads on tall buildings: an interactive database, *Journal of Structural Engineering*, ASCE, **129 (3)**, pp.394-404.