CHAPTER 1 GENERAL CONCEPTS

General

This chapter is devoted to description of basic concept when a structural design of buildings is implemented, and basic terminology which are needed in the estimation of loads with reference to these recommendations.

1.1 Scope of Applications

These recommendations apply to the estimation of loads on ordinary buildings and similar structures or parts thereof. Estimated loads may be used in structural design of buildings and parts of them, and in assessment of structural performance.

1.2 Fundamental concept

1.2.1 Structural performance

(1) Safety Buildings must have an appropriate degree of safety against various loads. To meet this requirement, the appropriate load and its intensity have to be evaluated. The appropriate degree of safety must be determined based on social and economical considerations.

(2) Serviceability An appropriate degree of serviceability must be maintained to ensure that building functions are not lost under normal usage conditions. To meet this requirement, an appropriate load intensity has to be determined for relatively frequent loads. The appropriate degree of serviceability must be determined based on social and economical considerations.

(3) Reparability Buildings must be, if necessary, designed to maintain reparability when they are damaged. The appropriate degree of reparability must be determined based on social and economical considerations.

1.2.2 Structural analysis

Load effects in terms of the stress or deformation of structural members and/or joints are obtained from structural analysis based on estimated loads. Structural analysis methods and procedures are not specified in the recommendations, but loads are estimated, in principle, for static analysis. Dynamic loads caused by strong wind or earthquake ground motions are evaluated for design as equivalent static loads.

1.2.3 Proper design and construction

Design load intensities are evaluated by assuming that buildings are properly designed and constructed. Therefore, not only proper design and construction are necessary to minimize reductions in safety and serviceability due to a human error, but also load intensities must be specified under the appropriate management and compliance.

1.3 Definitions

Terminology used in the recommendations is defined as follows:

- Limit state: A state beyond which either structure or part thereof no longer satisfies the prescribed requirements relating to safety or serviceability.
- Limit state design: design for targeted safety as well as serviceability limit states.
- Load effect: Stress, deformation, displacement, etc. induced within the structure by a single load or combined loads.
- Basic value of load: representative intensity of load as a reference for estimation of load effects. Basically, it is based on 100-year return period value of load.
- Load factor: Partial safety coefficients by which the basic value of the load effects are multiplied to obtain the design load effects.
- Return period: The expected time interval between which events greater than a certain magnitude are predicted to occur.

CHAPTER 2 LOADS AND LOAD COMBINATIONS

2.1 Loads

The following loads are to be considered in structural design.

- Dead load (G)
- Live load (Q)
- Snow load (S)
- Wind load (W)
- Earthquake load (E)
- Temperature load (T)
- Earth or hydraulic pressure (H)
- Other loads

2.2 Basic Load Values

The following basic load values, which represent the characteristic intensity of loads, are to be determined.

- The basic dead load (G) is the nominal dead load, or is determined according to actual conditions.
- The basic live load (Q) is the 99 percentile non-exceedance live load under normal conditions, or is a corresponding value if it is difficult to statistically evaluate.
- The basic snow load (S) is the 100-year-return-period snow load, based on ground snow depth.
- The basic wind load (W) is the 100-year-return-period wind load, based on mean wind speed.
- The basic earthquake load (E) is the 100-year-return-period earthquake load, based on the horizontal peak ground acceleration on the engineering bed rock.
- The basic temperature load (T) is the 100-year-return-period wind load under normal conditions, or is a corresponding value if it is difficult to statistically evaluate.
- The basic earth pressure and hydraulic pressure (H) is the 99 percentile non-exceedance values in normal conditions or a corresponding.
- The basic values of other loads are the ones evaluated as in the above.

2.3 Load combinations and load factors

2.3.1 Basis of Load combinations

(1) Load combinations for design and assessment of buildings or parts are to be selected based on the required performance level of the buildings or the parts.

(2) The required performance level of buildings or parts must be determined by designers based on importance, sociality, economy, relevance to existing design codes, etc.

(3) Loading states to be considered are, according to actual conditions, as follows,

- Normal states
- State of live loading
- State of snow load
- State of strong wind
- State of earthquake
- State of temperature change

2.3.2 Load combination for Limit State Design (LSD) format

(1) When designing a building and/or a structural member, the following limit state shall be considered appropriately for each load combination.

- Safety limit state
- Serviceability limit state

(2) A target performance level shall be defined using a target reliability index considering the predetermined reference period.

(3) Load combinations shall be considered as the sum of the products of the load effect corresponding to the basic value of each load and the load factor as,

$$\gamma_{\mathrm{p}}\, S_{\mathrm{pn}} + \sum_k \gamma_k\, S_{k\mathrm{n}} \tag{2.1}$$

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

Load factors shall be determined appropriately considering the limit state, the target reliability index, the variability in the load effect of each load and resistance, the probability of load coincidence, etc.

2.3.3 Load combination for allowable stress design and ultimate strength design

Combinations of loads used for allowable stress design and ultimate strength design are to be taken based on service life, importance, failure consequence of buildings and properties of each load such as variability, occurrence frequency and duration time, and design loads factored by the return period conversion factor are used. Subsection 2.3.2 may also be applied for ultimate strength design.

(1) Loading states to be considered

Loading states to be considered in allowable stress design are to be categorized into either long-term or short term loading state according to loading period and are based on 2.3.1(3). Snow load during strong wind or earthquakes are to be taken according to actual conditions. Loading states to be considered in ultimate strength design are also based on 2.3.1(3).

(2) Return period conversion factor

The return period conversion factors below are to be used the design return period r different from 100 year period. The return period conversion factors for dead, live, and temperature loads are to be calculated according to actual conditions.

a. Case of snow load

For the case of non-controlled roof snow, the return period conversion factor k_{rS} equals the following.

$$k_{\mathrm{rS}} = \begin{cases} 0.36 + 0.14\ln(r) & : \text{heavy snowy area} \\ 0.08 + 0.20\ln(r) & : \text{other area} \end{cases} \tag{2.2}$$

b. Case of wind load

For calculation of wind load, the return period conversion factor k_{rW} is the following

$$k_{\mathrm{rW}} = 0.63\,(\lambda_{\mathrm{U}} - 1)\,\ln(r) - 2.9\,\lambda_{\mathrm{U}} + 3.9 \tag{2.3}$$

where $\lambda_{\mathrm{U}} = U_{500}/U_0$ is a ratio of the 10 minute mean wind speed corresponding to 500 year return period to the basic wind speed (refer to Appendix 6.1.7).

c. Case of earthquake load

For calculation of earthquake load, the return period conversion factor for peak ground acceleration k_{rE} equals to the following.

$$k_{\mathrm{rE}} = \left(\frac{r}{100}\right)^{1/k} \tag{2.4}$$

where k represents regional statistical characteristics of seismic hazard (refer to 7.2.2).

CHAPTER 3 DEAD LOADS

3.1 Dead Loads

Building dead load shall be estimated based on the actual state of the building concerned. The weight of each part of the building shall be calculated using the density of the materials, the unit weights or the combined weights.

CHAPTER 4 LIVE LOADS

4.1 General

4.1.1 Definition

Live loads are the vertical loads due to furniture and people, which act randomly and vary with time and space during the lifetime of the building. They are determined for each element of structures considering design limit states, particular use of the building, temporary concentration of people and furniture and dynamic effects of live loads.

4.2 Estimation of Live Loads

4.2.1 Basic value of live loads

The basic value Q (N/m^2) of live loads is estimated from the following equation :

$$Q = k_e k_a k_n Q_0 \tag{4.1}$$

where

k_e : conversion factor to equivalent uniformly distributed load (see 4.2.3)

k_a : area reduction factor (see 4.2.4)

k_n : multi-story reduction factor (see 4.2.5)

Q_0 : basic live load intensity (N/m^2) (see 4.2.2)

but k_n can be used for only calculating axial force of column and vertical loads of foundations.

4.2.2 Basic live load intensity

The basic live load intensity Q_0 is the 99 percentile value of the sustained live load intensity for the particular use, as shown in Table 4.1.

Table 4.1 Basic live load intensity Q_0

Categories*	①	②	③	④	⑤	⑥	⑦	⑧
Q_0 (N/m^2)	1,000	500	1,600	2,100	3,500	2,200	4,700	1,800

*Category means one of the following uses :

① residential flats, dwellings

② hotel rooms (not including unit bath)

③ offices, laboratories

④ supermarkets, department stores

⑤ computer rooms (not including cables)

⑥ lane loads and garage parking spaces

⑦ library stack rooms

⑧ buildings where loads are produced mainly by people, for example, theaters, movie theaters, halls, assembly rooms, conference rooms, classrooms, etc.

4.2.3 Conversion factor for equivalent uniformly distributed load

Conversion factors for equivalent uniformly distributed load k_e are listed in Table 4.2.

Table 4.2 Conversion factor to equivalent uniformly distributed load

Categories*	①	②	③	④	⑤	⑥	⑦	⑧
Slab	1.8	2.0	1.6	1.8	1.6	1.8	1.6	1.6
Girder, Column, Foundation	1.2							

4.2.4 Area reduction factor

Area reduction factor k_a can be calculated from the following equations.

(1) For categories ①～⑦ in Table 4.1

$$k_a = 0.6 + \frac{0.4}{\sqrt{A_f / A_{ref}}} \leqq 1.0 \tag{4.2}$$

(2) For category ⑧ in Table 4.1

$$k_a = 0.7 + \frac{0.3}{\sqrt{A_f / A_{ref}}} \leqq 1.0 \tag{4.3}$$

where, A_f is the influence area (m^2) and A_{ref} is the basic area (=18m^2).

4.2.5 Multi-story reduction factor

Multi-story reduction factor k_n can be calculated from the following equation.

$$k_n = 0.6 + \frac{0.4}{\sqrt{n}} \tag{4.4}$$

where, n is the number of floors supported.

No reduction shall be permitted for category ⑧ in Table 4.1 and $k_a\, k_n > 0.4$ for all categories.

4.3 Live Loads Considering Concentration, Deflections or Cracks

In the case that the effects of furniture concentration, deflections or cracks must be considered, it is necessary to determine the appropriate intensity of design live loads considering those effects.

4.4 Dynamic Effects of Live Loads

With regard to the dynamic effects of live loads, the effects of movements of people and objects must be considered when it is necessary to evaluate the serviceability performance of buildings in relation to vibrations, such as habitability for occupants, counter-vibration measures for precision equipments, etc. It is also desirable to consider the influence of ambient environments and the source (or sources) of vibrations located on other floor slabs inside the buildings.

CHAPTER 5 SNOW LOADS

5.1 Scope and Procedure

As a snow load, (1) snow load on the roof, (2) partial snow load on the roof, (3) other snow loads shall be defined. Snow load on the roof is defined as the product of the snow load on the ground for the region considered and a shape coefficient. When the snow load on the roof is certainly controlled through some devices or technique, it could be reduced.

5.2 Snow Load on the Ground

5.2.1 Equations for snow load on the ground

Snow load on the ground S_0 (kN/m^2) is determined from the following equation:

$$S_0 = k_{\mathrm{env}}\, d_0\, p_0 \tag{5.1}$$

where

k_{env}: environmental coefficient, as defined in 5.2.4,

d_0: basic snow depth (m) on the ground, as defined in 5.2.2,

p_0: equivalent unit weight for ground snow, as defined in 5.2.3.

The value d_0 p_0 could be defined with special consideration for the data of precipitation and temperature.

5.2.2 Basic snow depth on the ground

Basic snow depth on the ground d_0 is defined as the annual maximum value for the whole season with a return period of 100 years, and is estimated from meteorological data of the ground snow depth observed for a certain period.

5.2.3 Equivalent unit weight for ground snow

Equivalent unit weight for ground snow p_0 (kN/m^3) is given by the following equation:

$$p_0 = 0.72\sqrt{d_0/d_{\mathrm{ref}}} + 2.32 \tag{5.2}$$

where

d_0: basic snow depth (m) on the ground,

d_{ref}: reference snow depth (1 m).

5.2.4 Environmental coefficient

Environmental coefficient k_{env} is generally defined as unity. When the snow depth on the ground is estimated to locally increase because of geographical, man-made and natural features, k_{env} should be correspondingly larger than unity.

5.3 Ground Snow Load with Accumulation for *n* Days

5.3.1 Equation for ground snow load with accumulation for *n* days

When the roof snow is certainly controlled, snow load on the ground S_n is determined from the following equation:

$$S_n = k_{\mathrm{env}}\, d_n\, p_n \tag{5.3}$$

where

k_{env}: environmental coefficient, as defined in 5.2.4,

d_n: basic snow depth (m) on the ground when the snow load on the roof is controlled, as defined in 5.3.2,

p_n: equivalent unit weight for ground snow with roof snow control, as defined in 5.3.3.

5.3.2 Basic snow depth with accumulation for *n* days

Basic snow depth on the ground d_n is defined as the annual maximum value of snow accumulation for n days with a return period of 100 years, and is estimated from meteorological data of the ground snow depth observed for a certain period. The length of evaluation period n (day) for snow accumulation is decided with the performance and reliability of the roof snow control system.

5.3.3 Equivalent unit weight for ground snow with roof snow control

Equivalent unit weight for ground snow with roof snow control p_n is equal to p_0.

5.4 Snow Load on the Roof

5.4.1 Equation for snow load on the roof

Snow load on the roof is given by the following equation:

$$S = \mu_0\, S_0 \tag{5.4}$$

where

μ_0: shape coefficient defined in 5.4.2.

5.4.2 Shape coefficient

Shape coefficient μ_0 is defined by the following equation:

$$\mu_0 = \mu_{\mathrm{b}} + \mu_{\mathrm{d}} + \mu_{\mathrm{s}} \tag{5.5}$$

where

μ_{b}: basic shape coefficient defined in 5.4.2 (1)

μ_{d}: shape coefficient for irregular distribution caused by snow drift defined in 5.4.2 (2)

μ_{s}: shape coefficient for irregular distribution caused by sliding defined in 5.4.2 (3)

Shape coefficient for large or special shaped buildings should be defined after special field research or experiments.

(1) Basic shape coefficient

Basic shape coefficient μ_b is given in Figure 5.1. In the figure, wind speed V (m/s) means the average wind speed in January through February. For a fractional value of V, μ_b should be determined by the interpolation.

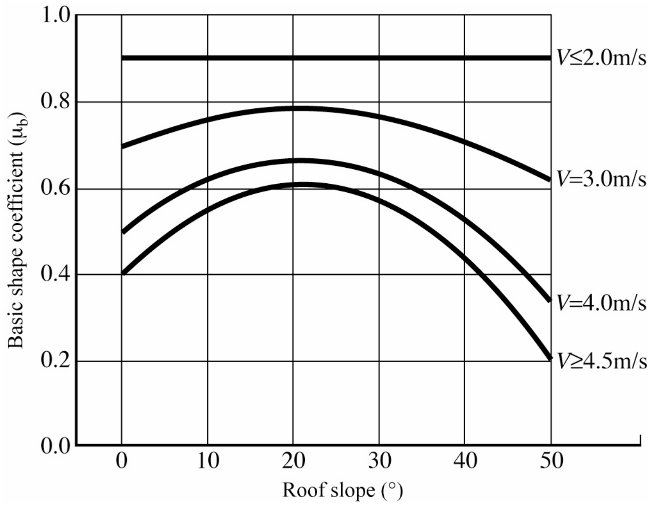


Figure 5.1 Relation between μ_b and roof slope.

(2) Shape coefficient for irregular distribution caused by snow drift

1) Shape coefficient for irregular distribution caused by snow drift μ_d in the troughs of M-shaped roofs, multiple pitched roofs and multispan roofs are given in Table 5.1. At the ridge, μ_d should be zero. At the halfway point, μ_d is calculated by linear interpolation. For a fractional value of V, μ_d should be determined by interpolation.

2) For multilevel roofs, distribution of shape coefficient for the lower level roof should be determined from Figure 5.2. In the figure, μ_d at point O is given in Table 5.2. For a fractional value of V, μ_d should be determined by interpolation.

Table 5.1 μ_d in the troughs of M-shaped roof, multiple pitched roof and multispan roof

Roof slope	M-shaped and multiple pitched roofs				multispan roof			
	average wind speed in Jan. through Feb.				average wind speed in Jan. through Feb.			
	<2 m/s	3 m/s	4 m/s	4.5 m/s<	<2 m/s	3 m/s	4 m/s	4.5 m/s<
< 10°	0	0	0	0	0	0	0	0
25°	0	0	0.15	0.20	0.10	0.20	0.35	0.55
40°	0	0.20	0.35	0.45	0.10	0.30	0.45	0.70
50° <	0	0.30	0.55	0.70	0.10	0.40	0.65	0.80

Table 5.2 μ_d for multilevel roofs

average wind speed in Jan. through Feb.	<2 m/s	3 m/s	4 m/s	4.5 m/s<
μ_d	0.10	0.30	0.50	0.60

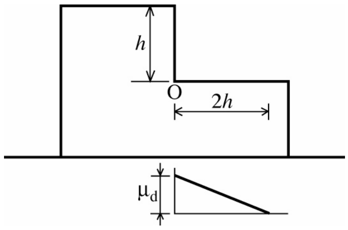


Figure 5.2 μ_d for multilevel roofs

(3) Shape coefficient for irregular distribution caused by sliding

Shape coefficient for irregular distribution caused by sliding μ_s on M-shaped roofs, multiple pitched roofs and multispan roofs is determined from either equation (5.6) or (5.7) according to the roof slope. μ_s is positive in the troughs of these roofs and negative at the ridge. At the halfway point, μ_s is determined by the linear interpolation. When the slope of the roof is between those defined in equation (5.6) and equation (5.7), μ_s is determined from the sliding performance of the roofing materials.

1) When roof slope is smaller than 10 degrees,

$$\mu_s = 0 \tag{5.6}$$

2) When roof slope is larger than 25 degrees,

$$\mu_s = \mu_b \tag{5.7}$$

5.5 Snow Load on the Roof with Snow Control

5.5.1 Equation for snow load on the roof

Snow load on the roof with snow control is given by the following equation:

$$S = \mu_n S_n - S_c \tag{5.8}$$

where

μ_n: shape coefficient with snow control corresponding to μ_0 defined in section 5.9

S_n: ground snow load with accumulation for n days (kN/m^2) defined in 5.3.1

S_c: controlled snow load (kN/m^2) defined in 5.5.2

5.5.2 Controlled snow load

Controlled snow load, S_c, is generally determined after field research and experiments investigating the capacity of sliding or melting devices, where S_c is the differential between initial snow load expected when heavy snow fall starts and removed snow load by the device whose performance is guaranteed even during heavy snow fall.

5.6 Partial Snow Load on the Roof

Partial snow load should be considered to exist on the roof as following cases:

(1) A case when snow load on the roof partially increases because of snowdrift caused by projecting structures.
(2) A case at the eaves or gable ends, snow load on the roof partially increases because of snow eaves.
(3) A case when snow is sliding from upper roofs to the eaves or lower levels of multilevel roofs. Distance and impact should also be considered.

5.7 Other Snow Loads

The following should be considered as additional snow loads.

(1) When side pressure from snow drifts on the outside wall of the building might not be negligible, it should be considered.
(2) When the building might be buried under snow, snow load caused by sedimentation should be considered.
(3) When snow adheres to the building or snow covers the building, snow load caused by adhering or covering snow should be considered.
(4) When snow blows into balconies or outside corridors and the load might not be negligible, it should be considered.
(5) For the high rise building or the structures with large roof, the effect of snowdrift on the surrounding buildings should be considered.
(6) Regarding points for disaster prevention should be considered.

CHAPTER 6 WIND LOADS

6.1 General

6.1.1 Scope of application

(1) This chapter describes wind loads for the design of buildings that respond elastically in strong winds.

(2) Two different wind loads are described. The first is for the design of structural frames, and the second is for the design of components/cladding of buildings.

6.1.2 Estimation principle

(1) Wind loads for the design of buildings are individually specified for horizontal wind load for structural frames, roof wind load for structural frames and wind load for components/cladding. The horizontal wind loads for the design of structural frames shall be individually determined in the along-wind, across-wind and torsional directions.

(2) For wind load for structural frames, combination of each horizontal wind load and combination of horizontal wind load and roof wind load shall be considered according to A6.8. For components of cladding and structural frame or particular joints of cladding and structural frames, combination of horizontal wind load on structural frames and local wind load on cladding shall be considered.

(3) The wind loads shall generally be determined from the design wind speed defined for each wind direction given in A6.1.2.

(4) The reference height is generally the mean roof height of the building. The wind loads are calculated from the velocity pressure at this reference height. However, wind loads on lattice type structures shall be calculated from the velocity pressure at each height, as shown in A6.6.

(5) The horizontal wind load on structural frames and the roof wind load on structural frames are given by the product of the velocity pressure given in A6.1, the wind force coefficient given in A6.2, the gust effect factor given in A6.3 and the projected area or subject area as shown in 6.2 and 6.3.

(6) The wind load on components/cladding is given by the product of the velocity pressure given in A6.1, the peak wind force coefficient given in A6.2 and the subject area.

(7) For relatively flexible buildings with large aspect ratios, the horizontal wind loads on structural frames in the across-wind and torsional directions given in A6.4 and A6.5 shall be considered. The criteria for this are described in 6.1.3(1).

(8) For flexible buildings with very large aspect ratios, the structural safety against vortex-induced vibration and aeroelastic instability shall be checked. The criteria for this are described in 6.1.3(2). The wind loads on structural frames and members of round sectional shape caused by vortex induced vibration shall be determined by A6.7.

(9) For small buildings and structures with large stiffness, a simplified procedure can be used, as given

in A6.11.

(10) The increase of wind-induced vibration caused by neighboring buildings shall be considered from A6.12.

(11) The response acceleration for checking the habitability of a building against wind-induced vibration shall be evaluated from A6.10. For this evaluation, the 1-year-reccurence wind speed can be obtained from A6.13.

(12) When the wind load shielding effects by surrounding topographies or buildings are considered, the future changes shall be confirmed, and the shielding effect shall be investigated by appropriate wind tunnel study or other suitable verification methods.

6.1.3 Buildings for which particular wind load or wind induced vibration is taken into account

(1) Buildings for which horizontal wind loads on structural frames in across-wind and torsional directions are taken into account

For the buildings that satisfy the following criteria, wind load in the across-wind direction as defined in A6.4 and wind load in the torsional direction as defined in A6.5 shall be checked.

$$\frac{H}{\sqrt{BD}} \geq 3 \tag{6.1}$$

where

H (m): reference height as defined in 6.1.2(4)

B (m): building breadth

D (m): building depth

(2) Vortex resonance and aeroelastic instability

For buildings that satisfy the following criteria, vortex-induced vibration and aeroelastic instability shall be checked by the appropriate wind tunnel tests and so on. For buildings with circular section, the wind load is prescribed in A6.7.

1) For buildings with rectangular section

$$\frac{H}{\sqrt{BD}} \geq 4 \quad \text{and} \quad \left(\frac{U_{\mathrm{H}}}{f_{\mathrm{L}}\sqrt{BD}} \geq 0.83 U^{*}_{\mathrm{Lcr}} \quad or \quad \frac{U_{\mathrm{H}}}{f_{\mathrm{T}}\sqrt{BD}} \geq 0.83 U^{*}_{\mathrm{Tcr}} \right) \tag{6.2}$$

where

U_{H} (m/s): design wind speed as defined in A6.1.2. (wind directionality factor $K_{\mathrm{D}} = 1$)

U^{*}_{Lcr}: non-dimensional critical wind speed for aeroelastic instability in across-wind direction calculated from Table 6.1

U^{*}_{Tcr}: non-dimensional critical wind speed for aeroelastic instability in torsional direction calculated from Table 6.2

f_{L}, f_{T} (Hz): natural frequency for first mode in across-wind and torsional directions

2) For buildings with circular cross-section

$$\frac{H}{D_{\mathrm{m}}} \geq 7 \quad \text{and} \quad \frac{U_{\mathrm{H}}}{f_{\mathrm{L}} D_{\mathrm{m}}} \geq 4.2 \tag{6.3}$$

where

D_m (m): building diameter at height $2H/3$

Table 6.1 Non-dimensional critical wind speed for aeroelastic instability in across-wind direction U^*_{Lcr}

Flat terrain categories	Side ratio D/B	Scruton number δ_L Note)	Critical speed U^*_{Lcr}
I, II	$D/B \le 0.8$	$\delta_L \le 0.7$	$16\delta_L$
		$\delta_L > 0.7$	11
	$0.8 < D/B \le 1.5$	all	$1.2\delta_L + 7.3$
	$1.5 < D/B \le 2.5$	$\delta_L \le 0.2$	2.3
		$0.2 < \delta_L \le 0.8$	12
		$\delta_L > 0.8$	$15\delta_L$
	$D/B > 2.5$	$\delta_L \le 0.4$	3.7
		$\delta_L > 0.4$	not necessary to evaluate
III, IV, V	$D/B \le 0.8$	all	$4.5\delta_L + 6.7$
	$0.8 < D/B \le 1.2$	all	$0.7\delta_L + 8.8$
	$D/B > 1.2$	all	11

Note) δ_L is the mass damping parameter defined as $\delta_L = \zeta_L M/(3\rho BDH)$, where ζ_L is the damping ratio for the first mode in the across-wind direction, M(kg) is the total building mass, ρ (1.22kg/m^3) is the air density.

Table 6.2 Non-dimensional critical wind speed for aeroelastic instability in torsional direction U^*_{Tcr}

Side ratio D/B	Scruton number δ_T Note)	Critical speed U^*_{Tcr}
$D/B \le 1.5$	$\delta_T \le 0.05$	2
	$0.05 < \delta_T \le 0.1$	11
	$\delta_T > 0.1$	not necessary to evaluate
$1.5 < D/B \le 2.5$	$\delta_T \le 0.05$	2
	$0.05 < \delta_T \le 0.15$	$4 + 8\delta_T$
	$\delta_T > 0.15$	$8.6 + 7.4\delta_T$
$2.5 < D/B \le 5$	$\delta_T \le 0.05$	2
	$\delta_T > 0.05$	$5 + 10.5\delta_T$

Note) δ_T is the mass damping parameter defined as $\delta_T = \zeta_T M(B^2 + D^2)/(36\rho B^2 D^2 H)$, where ζ_T is the damping ratio for the first mode in the torsional direction.

6.2 Horizontal Wind Loads on Structural Frames

6.2.1 Scope of application

This section defines the procedures for estimating horizontal wind loads on structural frames in the along-wind direction.

6.2.2 Procedure for estimating wind loads

Along-wind loads on structural frames are calculated from Eq.(6.4).

$$W_D = q_H C_D G_D A \tag{6.4}$$

where

W_D (N): along-wind load at height Z

q_H (N/m^2): velocity pressure as defined in A6.1.1

C_D : wind force coefficient as defined in A6.2

G_D : gust effect factor as defined in A6.3.1

A (m^2): projected area at height Z

6.3 Roof Wind Load on Structural Frames

6.3.1 Scope of application

This section defines the procedures for estimating roof wind loads on structural frames of buildings.

6.3.2 Procedure for estimating wind loads

Roof wind loads on structural frames are calculated from Eq.(6.5)

$$W_R = q_H C_R G_R A_R \tag{6.5}$$

where

W_R (N): wind load

q_H (N/m^2): design velocity pressure as defined in A6.1.1

C_R : wind force coefficient as defined in A6.2

G_R : gust effect factor for roof wind load as defined in A6.3.2

A_R (m^2): subject area

6.4 Wind Loads on Components/Cladding

6.4.1 Scope of application

This section defines the procedures for estimating wind loads on components/cladding of buildings.

6.4.2 Procedure for estimating wind loads

Wind loads on components/cladding of buildings are calculated from Eq.(6.6).

$$W_C = q_H \hat{C}_C A_C \tag{6.6}$$

where

W_C (N): wind load

q_H (N/m^2): design velocity pressure as defined in A6.1.1

$\hat{C}_{\mathrm{C}}$: peak wind force coefficient as defined in A6.2

A_{C} (m^2): subject area of components/cladding

A6.1 Wind Speed and Velocity Pressure

A6.1.1 Velocity pressure

The design velocity pressure, q_{H} (N/m^2), is calculated from:

$$q_{\mathrm{H}} = \frac{1}{2}\rho U_{\mathrm{H}}^2 \tag{A6.1}$$

where

ρ (kg/m^3): air density, assumed to be 1.22

U_{H} (m/s): design wind speed, which depends on wind direction and is defined in A6.1.2

A6.1.2 Design wind speed

Design wind speed, U_{H} (m/s), is calculated for each wind direction from:

$$U_{\mathrm{H}} = U_0 K_{\mathrm{D}} E_{\mathrm{H}} k_{\mathrm{rW}} \tag{A6.2}$$

where

U_0 : basic wind speed (m/s) depending on the geographic location of the construction site, defined in A6.1.3

K_{D} : wind directionality factor defined in A6.1.4.

E_{H} : wind speed profile factor at reference height H defined in A6.1.5.

k_{rW} : return period conversion factor defined in A6.1.7.

The 1-year-recurrence wind speed is defined in A6.13 for evaluation of habitability.

A6.1.3 Basic wind speed

Basic wind speed U_0 (m/s) corresponds to the 100-year-recurrence 10-minute mean wind speed over a flat, open terrain at an elevation of 10m. The wind speed is defined in Fig.A6.1 for various locations in Japan.

Izu Islands, not shown in the map	46
Ogasawara Islands, Satsunann Islands, Okinawa Islands, Daitou Islands, Sakishima Islands, not shown in the map	50

Figure A6.1 Basic wind speed U_0 (m/s)

A6.1.4 Wind directionality factor

Wind directionality factor K_D reflects the directional characteristics of the extreme wind, which are influenced by the geographical location and topographic feature of the construction site. It shall be determined as follows, with reference to the wind directionality factors for the 8 cardinal directions shown in Table A6.1.

(1) Where the aerodynamic shape factors for each wind direction are known from an appropriate wind tunnel experiment, the wind directionality factor K_D, which is used to evaluate the wind loads on structural frames and components/cladding for a particular wind direction, shall take the same value as that for the cardinal direction whose 45 degree sector includes that wind direction.

(2) Where the aerodynamic shape factors in A6.2 are used

1) When assessing wind loads on structural frames

a) Where the aerodynamic shape factors are dependent on wind direction, four wind directions should be considered that coincide with the principal coordinate axis of the structure. If the wind direction is within a 22.5 degree sector centered at one of the 8 cardinal directions, the wind directionality factor K_D for this direction shall be adopted. If the wind direction is outside of the 22.5 degree sector, the larger of the 2 nearest cardinal directions shall be adopted.

b) Where the aerodynamic shape factors are independent of wind direction, the wind directionality factor K_D shall take the same value as that for the cardinal direction whose 45 degree sector includes that wind direction.

2) When assessing wind loads on components/cladding

$K_D = 1$

Table A6.1 Wind directionality factor K_D

	Wakkanai	Kitamiesashi	Haboro	Omu	Rumoi	Asahikawa	Abashiri	Otaru	Sapporo	Iwamizawa
NE	0.95	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.9
E	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
SE	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	1	1
S	1	0.85	1	0.85	0.85	1	0.85	0.85	1	1
SW	1	1	1	1	0.95	1	0.85	1	0.85	0.85
W	0.85	1	1	1	0.95	1	0.95	1	0.95	1
NW	0.85	0.85	0.95	0.85	1	0.85	1	1	1	0.95
N	0.95	0.85	0.85	0.85	1	0.85	0.9	0.85	0.85	0.9

	Obihiro	Kushiro	Nemuro	Suttsu	Muroran	Tomakomai	Urakawa	Esashi	Hakodate	Kutchan
NE	0.85	0.85	0.9	0.85	0.85	0.85	0.85	0.85	0.95	0.85
E	0.85	0.9	0.9	0.85	0.85	0.85	0.85	0.85	0.95	0.95
SE	0.85	0.9	0.85	1	0.85	0.85	0.85	0.85	0.85	0.95
S	0.85	0.85	0.85	1	0.85	0.85	0.85	1	1	0.95
SW	0.85	0.85	0.85	0.85	0.85	0.85	0.85	1	1	0.95
W	1	1	0.95	1	1	0.85	1	1	0.95	1
NW	1	0.9	1	1	1	0.9	1	1	0.9	1
N	0.85	0.85	0.95	0.9	0.85	1	0.85	0.85	0.85	0.85

Table A6.1(continued) Wind directionality factor K_D

	Mombetsu	Hiroo	Ofunato	Shinjo	Wakamatsu	Fukaura	Aomori	Mutsu	Hachinohe	Akita
NE	0.9	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
E	0.9	0.85	0.85	0.85	0.9	0.85	0.85	0.85	0.85	0.85
SE	0.85	0.85	0.85	0.85	0.9	0.85	0.85	0.85	0.85	0.85
S	0.85	0.85	0.85	0.85	0.85	1	0.85	0.85	0.85	0.95
SW	1	1	0.85	0.85	0.85	1	1	0.95	1	0.95
W	1	1	0.85	1	1	0.95	1	1	1	1
NW	1	0.85	1	1	1	0.95	0.9	0.85	0.95	1
N	0.9	0.85	0.9	0.85	0.85	0.9	0.85	0.85	0.85	0.85

	Morioka	Miyako	Sakata	Yamagata	Sendai	Ishinomaki	Fukushima	Shirakawa	Onahama	Wajima
NE	0.85	0.85	0.85	0.85	0.85	1	0.85	0.85	1	0.9
E	0.85	0.85	0.85	0.85	0.85	0.95	0.85	0.85	0.85	0.85
SE	0.85	0.9	0.85	0.85	0.85	0.9	0.85	0.85	0.9	0.85
S	0.85	0.9	0.85	0.9	0.85	0.85	0.85	0.85	0.85	0.9
SW	0.95	0.95	0.9	1	0.85	0.85	0.85	0.85	0.85	1
W	1	1	1	1	1	1	1	1	0.95	1
NW	1	0.95	1	0.95	1	1	1	1	1	0.95
N	0.95	0.95	0.85	0.85	0.85	1	0.85	0.95	1	0.95

	Aikawa	Niigata	Kanazawa	Fushiki	Toyama	Nagano	Takada	Utsunomiya	Fukui	Takayama
NE	0.85	0.85	0.85	0.9	0.85	0.85	0.85	0.9	0.85	0.85
E	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
SE	0.85	0.85	0.85	0.85	0.9	0.85	0.85	0.85	1	0.9
S	0.85	0.85	0.9	0.85	1	0.85	0.85	0.85	1	1
SW	0.85	1	1	1	0.9	1	0.85	0.85	0.85	0.85
W	0.85	1	0.9	0.95	0.9	1	1	0.85	0.85	0.85
NW	1	1	0.85	0.85	0.85	0.9	0.95	0.85	1	0.85
N	1	0.9	0.85	0.9	0.85	0.85	0.85	1	1	0.85

	Matsumoto	Suwa	Kumagai	Mito	Tsuruga	Gifu	Nagoya	Iida	Kofu	Chichibu
NE	0.85	0.85	0.85	1	0.85	0.85	0.85	0.95	0.85	0.85
E	0.85	1	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
SE	1	1	0.85	0.85	0.85	1	1	0.85	0.85	0.85
S	1	0.95	0.85	0.85	0.85	0.95	1	0.85	0.85	0.85
SW	0.9	0.85	0.85	0.85	0.85	0.85	0.85	1	0.85	0.85
W	0.85	1	0.95	0.9	0.85	0.85	0.85	1	0.85	0.9
NW	0.85	1	1	1	1	0.9	0.85	1	1	1
N	0.85	0.85	0.95	1	0.85	0.85	0.85	0.95	1	0.85

	Choshi	Ueno	Tsu	Irako	Hamamatsu	Omaezaki	Shizuoka	Mishima	Tokyo	Owase
NE	0.9	0.85	0.85	0.9	0.95	1	0.85	1	0.85	0.85
E	0.85	1	1	0.95	0.95	1	0.85	0.85	0.85	0.95
SE	0.85	0.9	1	0.95	0.85	0.85	0.85	0.85	0.85	0.85
S	0.85	0.85	0.85	0.9	0.85	0.95	0.95	0.85	0.85	0.85
SW	0.85	0.9	0.85	0.85	0.85	0.95	1	0.85	0.85	0.95
W	0.85	0.85	0.95	1	1	1	1	0.85	0.85	1
NW	0.95	0.85	0.9	1	1	0.95	0.85	0.85	1	0.95
N	1	0.85	0.85	0.85	0.85	0.85	0.85	1	1	0.85

Table A6.1(continued) Wind directionality factor K_D

	Irozaki	Ajiro	Yokohama	Tateyama	Katsuura	Oshima	Miyakojima	Hachijojima	Chiba	Yokkaichi
NE	0.85	0.95	0.85	0.85	0.85	1	0.85	0.85	0.85	0.85
E	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.9	1
SE	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.9	1
S	0.85	0.85	0.85	0.85	1	0.85	0.85	1	0.95	0.85
SW	0.9	1	0.85	0.85	1	0.95	0.85	1	0.95	1
W	1	1	0.85	0.85	0.85	0.9	0.95	0.9	0.85	1
NW	0.85	0.85	1	1	1	0.85	1	0.9	1	0.85
N	0.85	0.95	1	1	0.9	1	1	0.9	1	0.85

	Saigo	Matsue	Sakai	Yonago	Tottori	Toyooka	Maizuru	Hagi	Hamada	Tsuyama
NE	0.9	0.9	0.95	0.85	0.85	0.85	0.85	0.85	0.85	0.85
E	0.85	0.85	0.95	0.85	0.85	0.85	0.85	0.85	0.85	0.95
SE	0.85	0.85	0.85	0.85	0.85	0.85	1	0.9	0.85	0.95
S	0.85	0.85	0.85	0.85	1	0.85	1	0.9	0.85	0.85
SW	0.85	0.9	0.95	0.95	0.85	0.85	0.85	0.85	0.9	0.85
W	0.85	1	1	1	0.85	0.85	0.85	0.9	1	0.95
NW	0.85	0.95	0.85	1	0.9	1	1	1	0.85	0.95
N	1	0.95	0.85	1	0.9	1	1	1	1	1

	Kyoto	Hikone	Shimonoseki	Hiroshima	Kure	Fukuyama	Okayama	Himeji	Kobe	Osaka
NE	1	0.85	0.85	0.85	1	0.95	0.85	0.85	1	0.9
E	0.95	0.85	0.85	0.85	0.85	0.85	0.85	0.85	1	0.85
SE	0.85	0.85	0.85	0.95	0.85	0.85	0.85	1	0.85	0.85
S	0.85	0.85	0.85	1	0.9	0.85	0.85	1	0.85	1
SW	0.85	0.85	0.85	1	0.9	0.85	1	0.85	0.85	1
W	0.85	0.95	0.85	0.85	0.9	0.85	0.85	0.9	0.85	1
NW	0.95	1	1	0.85	1	0.85	0.85	0.9	1	1
N	0.95	0.85	0.95	0.85	1	1	0.85	0.85	1	1

	Sumoto	Wakayama	Shionomisaki	Nara	Yamaguchi	Izuhara	Hirado	Fukuoka	Iizuka	Sasebo
NE	0.85	0.85	0.95	1	0.85	0.85	0.9	0.85	0.85	1
E	0.85	0.85	0.95	1	0.9	0.85	0.85	0.85	0.85	0.85
SE	0.9	1	0.85	0.85	1	0.85	0.85	1	0.85	0.85
S	1	1	0.9	0.85	0.85	0.95	0.85	1	0.9	0.85
SW	0.85	0.95	0.95	0.85	0.85	1	0.85	0.85	0.9	0.85
W	0.85	1	1	0.85	0.85	0.9	0.85	1	0.85	0.9
NW	0.85	1	1	0.85	0.85	0.95	0.95	1	1	0.9
N	0.85	0.85	0.85	0.85	0.85	1	1	1	1	1

	Saga	Hita	Oita	Nagasaki	Kumamoto	Nobeoka	Akune	Kagoshima	Miyakonojo	Miyazaki
NE	0.85	0.85	0.85	0.9	0.85	0.85	0.85	0.85	0.85	0.9
E	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.9	0.85
SE	0.85	0.85	1	0.85	0.85	0.95	0.85	0.95	1	1
S	1	1	0.85	0.9	1	1	1	0.95	1	0.85
SW	0.85	1	0.85	1	1	0.85	0.9	1	0.9	0.85
W	0.85	0.95	0.9	1	0.95	0.85	0.85	0.85	0.85	0.85
NW	0.95	0.85	0.9	0.95	1	0.9	0.85	0.85	0.85	0.85
N	0.95	0.85	0.85	0.9	1	0.9	0.85	0.85	0.85	0.9

Table A6.1(continued) Wind directionality factor K_D

	Makurazaki	Aburatsu	Yakushima	Tanegashima	Ushibuka	Fukue	Matsuyama	Tadotsu	Takamatsu	Uwajima
NE	0.85	0.95	0.85	0.85	0.85	0.85	0.9	0.85	0.9	0.95
E	1	0.9	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.95
SE	1	0.9	0.95	0.95	0.85	0.9	1	0.85	0.85	1
S	1	1	1	1	0.85	0.95	1	0.85	0.85	1
SW	1	1	1	0.9	0.9	0.95	0.95	1	0.95	0.85
W	0.95	0.85	0.85	0.9	1	0.95	0.95	1	1	0.85
NW	0.95	0.85	0.85	0.85	0.85	1	0.9	0.85	0.9	0.85
N	0.95	0.95	0.85	0.85	0.85	1	0.9	0.85	0.9	0.85

	Kouchi	Tokushima	Sukumo	Shimizu	Murotomisaki	Naze	Miyakojima	Kumejima	Naha	Nago
NE	0.85	0.85	0.85	0.85	1	0.85	0.95	0.85	0.85	0.85
E	1	0.85	0.95	0.95	1	0.85	0.85	0.85	0.85	0.85
SE	1	1	0.95	0.9	0.9	0.9	0.85	0.95	0.95	0.9
S	0.85	1	0.9	0.95	0.85	0.85	0.95	1	1	1
SW	0.85	0.85	1	0.95	0.95	0.85	0.85	1	1	1
W	0.85	0.85	1	1	1	0.85	0.95	0.85	1	0.85
NW	0.85	0.85	1	0.85	0.9	1	1	1	1	0.9
N	0.85	0.85	0.85	0.85	0.85	1	1	1	1	0.85

	Okinoerabu	Minamidaitojima
NE	0.85	0.9
E	0.85	0.85
SE	1	0.95
S	1	0.95
SW	0.85	0.85
W	0.85	0.85
NW	0.9	1
N	0.95	1

A6.1.5 Wind speed profile factor

(1) Wind speed profile factor

Wind speed profile factor E is calculated from:

$$E = E_r E_g \tag{A6.3}$$

where

E_r : exposure factor for flat terrain categories, defined in (2)

E_g : topography factor defined in (3)

(2) Exposure factor based on flat terrain categories

The exposure factor for flat terrain categories is defined in 2), according to the flat terrain categories defined in 1).

1) The flat terrain categories of the construction site are defined in Table 6.2. However, if the terrain category changes from smooth to rough in the region of the smaller of 40H (H: reference height) and 3km upwind of the construction site, the terrain category of the construction site is assumed the same as that of the upwind smooth terrain.

Table A6.2 Flat terrain categories

	Category	Condition at construction site and upwind region
Smooth	I	Open, no significant obstruction, sea, lake
↑	II	Open, few obstructions, grassland, agricultural field
	III	Suburban, wooded terrain, few tall buildings (4 to 9-story)
↓	IV	City, tall buildings (4 to 9-story)
Rough	V	City, heavy concentration of tall buildings (higher than 10-story)

2）The exposure factor based on the flat terrain categories is defined in Eq.(A6.4), according to the terrain categories defined in 1).

$$E_r = \begin{cases} 1.7\left(\dfrac{Z}{Z_G}\right)^{\alpha} & Z_b < Z \le Z_G \\ 1.7\left(\dfrac{Z_b}{Z_G}\right)^{\alpha} & Z \le Z_b \end{cases} \tag{A6.4}$$

where

Z (m): height above ground

Z_b, Z_G, α : parameters determining the exposure factor E_r, defined in Table A6.3

Table A6.3 Parameters determining E_r

Category	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

(3) Topography factor

Topography factor, which reflects the change of the mean wind speed that occurs as wind passes at right angles over escarpments or ridge-shaped topography, as shown in Figs.A6.2 and A6.3, is defined in Eq.(A6.5). However, when the inclination θ_s calculated from Eq.(A6.6) is less than 7.5 degrees, or X_s / H_s is beyond the range shown in Tables A6.4 and A6.5, it is not necessary to consider the topography factor, i.e., $E_g = 1$.

$$E_g = (C_1 - 1)\left\{C_2\left(\frac{Z}{H_s} - C_3\right) + 1\right\}\exp\left\{-C_2\left(\frac{Z}{H_s} - C_3\right)\right\} + 1 \quad \text{and} \quad E_g \ge 1 \tag{A6.5}$$

$$\theta_s = \tan^{-1}\frac{H_s}{2L_s} \tag{A6.6}$$

where

C_1, C_2, C_3 : parameters determining the topography factor, are given in Tables A6.4 and A 6.5, and depend on the topography shape, inclination θ_s and distance X_s (m) from the top of the topographic feature to the construction site. When the inclination θ_s is greater than 60 degrees, the topography factor is assumed to be the same as that at 60 degrees.

Z (m): height above ground. It is assumed the same value as Z_b when it is smaller than Z_b.

H_s (m): height of the topography

L_s (m): horizontal distance from the top of topographic feature to the point where the height is half the topography height as shown in Figs. A6.2 and A6.3

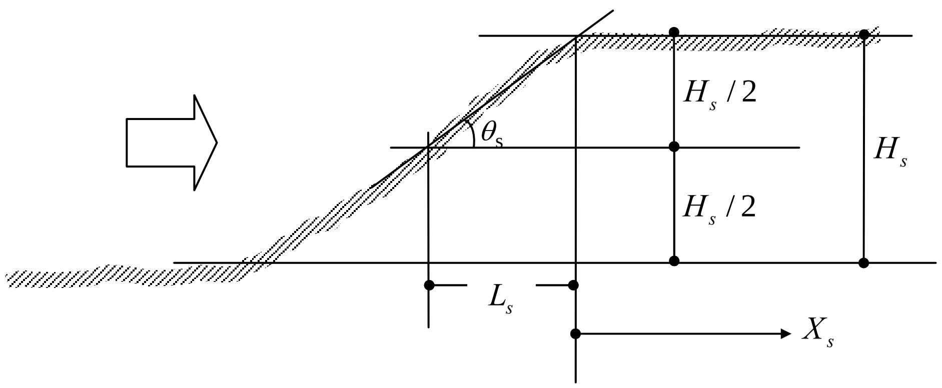


Figure A6.2 Escarpments

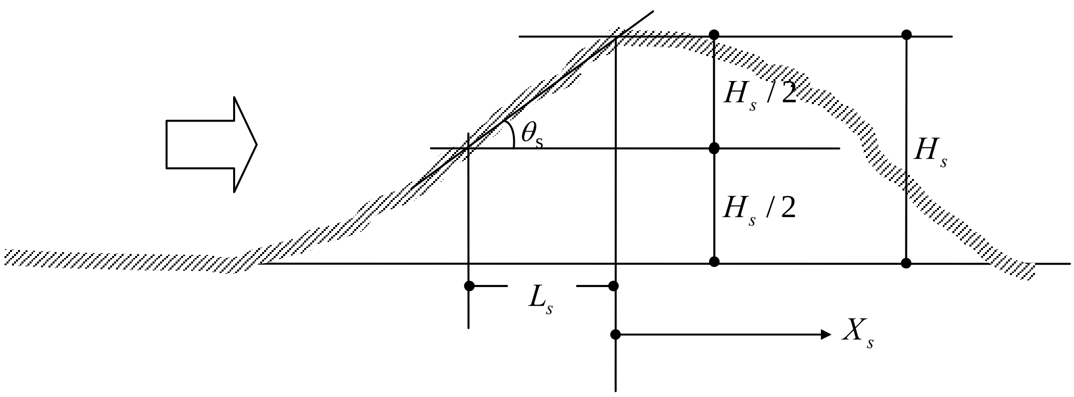


Figure A6.3 Ridge-shaped topography

Table A6.4 Parameters determining E_g (escarpments)

θ_s		X_s/H_s									
		–4	–2	–1	–0.5	0	0.5	1	2	4	8
7.5°	C_1	1.15	1.3	1.5	1.5	1.6	1.45	1.3	1.3	1.2	1.15
	C_2	0.8	0.8	0.8	0.8	0.8	0.7	0.6	0.6	0.5	0.4
	C_3	–2	–2	–2	–2	–2	–2	–2	–2	–2	–2
15°	C_1	0.4	1	1.2	1.55	2.1	1.65	1.5	1.3	1.2	1.15
	C_2	0.9	0	0.65	0.85	1	0.8	0.7	0.55	0.45	0.35
	C_3	–2	–2	–2	–2	–2	–2	–2	–2	–2	–2
30°	C_1	0.7	–0.5	1.05	1.1	1.3	1.3	1.25	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
45°	C_1	0.8	0	–3.5	1.1	1.2	1.35	1.3	1.2	1.15	1.1
	C_2	0.5	1	1.6	2	1.1	1.3	1.3	1.3	0.9	0.55
	C_3	–2	–2	–2	0.8	0.3	0.2	0.75	1.05	1.4	2
60°	C_1	0.6	0.1	–1.8	–2.4	1.2	1.4	1.35	1.25	1.15	1.1
	C_2	0.65	0.9	1.3	2.6	2	1.8	1.7	1.5	0.85	0.45
	C_3	–2	–2	–2	–1	0.5	0.5	0.8	1.2	1.9	3.1

Table A6.5 Parameters determining E_g (ridge-shaped topography)

θ_s		X_s/H_s									
		–4	–2	–1	–0.5	0	0.5	1	2	4	8
7.5°	C_1	1.1	1.2	1.35	1.35	1.4	1.3	1.3	1.2	1.1	1
	C_2	1	1	1	1	1.5	1.2	1.1	2	1.6	0
	C_3	0	0	0	0	0.2	0.2	0.2	0.5	0.9	0
15°	C_1	1	1.05	1.2	1.25	1.3	1.4	1.3	1.25	0.35	0.65
	C_2	0	0	1	1	1	1.5	1.5	2	3	2
	C_3	0	0	0	0	0	0.5	0.6	1.1	0.2	0.3
30°	C_1	0.75	0.55	0.85	1	1.2	1.3	1.25	1.2	1.1	1.02
	C_2	1.5	2	2	0	1	2	2	1.6	1.7	1.7
	C_3	0	0	0	0	0	1.1	1.3	2.1	2.2	2.8
45°	C_1	0.75	0.55	0.2	0.75	1.15	1.2	1.15	1.12	1.1	1.02
	C_2	1.5	2	2	3	1	2.5	2.5	2	1.6	1.3
	C_3	0	0	0	0	0	1.2	1.9	2.2	2.5	3.2
60°	C_1	0.75	0.55	0.2	0.2	1.15	1.12	1.15	1.12	1.1	1.02
	C_2	1.5	1.5	1.8	3	1	2.2	2.5	2	1.6	1.3
	C_3	0	0	0	0	0	1.8	2	2.3	2.6	3.4

Note) For a particular inclination θ_s and a horizontal location X_s / H_s, the topography factor is calculated by interpolating linearly from the values at the nearest inclinations and horizontal locations.

A6.1.6 Turbulence intensity and turbulence scale

Turbulence intensity and turbulence scale in A6.2, A6.3 are defined as follows.

(1) Turbulence intensity

1) Turbulence intensity I_Z is defined according to the conditions of the construction site as:

$$I_Z = I_{rZ} E_{gI} \tag{A6.7}$$

where

I_{rZ} : turbulence intensity at height Z on the flat terrain categories, defined in 2)

E_{gI} : topography factor defined in 3)

2) Turbulence intensity on flat terrain categories

Turbulence intensity I_{rZ} on flat terrain categories is defined in Eq.(A6.8) according to the terrain categories.

$$I_{rZ} = \begin{cases} 0.1\left(\dfrac{Z}{Z_G}\right)^{-\alpha-0.05} & Z_b < Z \le Z_G \\ 0.1\left(\dfrac{Z_b}{Z_G}\right)^{-\alpha-0.05} & Z \le Z_b \end{cases} \tag{A6.8}$$

where

Z (m): height above ground

Z_b, Z_G, α : parameters determining the exposure factor, defined in Table A6.3

3) Topography factor for turbulence intensity

Topography factor for turbulence intensity for the condition, in which the wind passes at right angles to the escarpments or ridge-shaped topography, as shown in Figs.A6.2 and A6.3, is defined as:

$$E_{gI} = \frac{E_I}{E_g} \tag{A6.9}$$

where

$$E_I = (C_1 - 1)\left\{C_2(\frac{Z}{H_s} - C_3) + 1\right\}\exp\left\{-C_2(\frac{Z}{H_s} - C_3)\right\} + 1 \quad \text{and} \quad E_I \ge 1 \tag{A6.10}$$

where

E_I : topography factor for the standard deviation of fluctuating wind speed. When the inclination θ_s calculated from Eq.(A6.6) is less than 7.5, or the distance from the top of the topographic feature X_s (m) is beyond the range of X_s / H_s in Tables A6.6 and A6.7, it is not necessary to consider the topography factor, i.e., $E_I = 1$.

E_g : topography factor for mean wind speed, defined in Eq.(A6.5)

C_1, C_2, C_3 : parameters determining the topography factor E_I, are given in Tables A6.6 and A6.7, and depend on the topography shape, inclination θ_s and the distance X_s (m) from the top of the topographic feature to the construction site. When the inclination θ_s is greater than 60 degrees, the topography factor is assumed to be the same as that at 60 degrees.

Z (m): height above ground. It is assumed to be the greater of Z_b and Z_c when it is smaller than Z_b in Table A6.3, or Z_c in Tables A6.6 and A6.7

H_s (m): height of topography

L_s (m): horizontal distance from the top of the topographic feature to the point where the height is half the topography height

Table A6.6 Parameters determining E_I (Escarpments)

θ_s		X_s/H_s									
		–4	–2	–1	–0.5	0	0.5	1	2	4	8
7.5°	Z_c/H_s	0	0	0	0	0	0	0	0	0	0
	C_1	1	1	1	1	1	1	1	1	1	1
	C_2	0	0	0	0	0	0	0	0	0	0
	C_3	0	0	0	0	0	0	0	0	0	0
15°	Z_c/H_s	0	0	0	0	0	0	0	0	0	0
	C_1	1	1.05	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
	C_2	0	0	0.2	0.2	0.3	0.4	0.5	0.5	0.3	0.3
	C_3	0	0	0	0	0	0	0	0	0	0
30°	Z_c/H_s	0	0	0	0	0	0	0	0	0	0
	C_1	1.05	1.05	1.1	1.15	1.2	1.3	2.5	1.8	1.4	1.25
	C_2	0	0	0	0.7	2	2.5	10	8	4	1.5
	C_3	0	0.5	1	1	0.5	0	0	0.1	0.2	0.6
45°	Z_c/H_s	0	0	0	0	0	0	0	0.3	0.3	0.2
	C_1	1.05	1.1	1.15	1.3	1.8	1.5	3	3	1.8	1.5
	C_2	0	0	0	1.4	1.7	1.6	6	5	3.5	2
	C_3	0	0	0.5	0	–1	–0.8	0	0	0.3	0.5
60°	Z_c/H_s	0	0	0	0	0	0	0	0.5	0.7	0.9
	C_1	1.1	1.15	1.2	1.3	6	8	4	3.5	2.2	1.7
	C_2	0	0	0	0.7	2.5	5	8	5	3	1.5
	C_3	0	0	0.3	0.5	–1.3	–0.4	0.2	0.2	0.4	0.5

Table A6.7 Parameters determining E_{I} (Ridge-shaped topography)

θ_s		X_s/H_s									
		–4	–2	–1	–0.5	0	0.5	1	2	4	8
7.5°	Z_c/H_s	0	0	0	0	0	0	0	0	0	0
	C_1	1	1	1	1	1	1	1	1	1	1
	C_2	0	0	0	0	0	0	0	0	0	0
	C_3	0	0	0	0	0	0	0	0	0	0
15°	Z_c/H_s	0	0	0	0	0	0	0	0.6	0	0
	C_1	1	1	1	1	1	3.4	4.2	4	2.8	2
	C_2	0	0	0	0	0	19	11	4.6	2	1.6
	C_3	0	0	0	0	0	0.1	0.2	0.3	0.6	0.7
30°	Z_c/H_s	0	0	0	0	0	0.6	0.8	1.5	1.6	2.2
	C_1	1	1	1	1	1	1.6	1.9	2.2	3.2	2.7
	C_2	0	0	0	0	0	5	4	2	1.7	1.3
	C_3	0	0	0	0	0	0.4	0.6	1	0.7	0.5
45°	Z_c/H_s	0	0	0	0	0	0.8	1.5	1.7	1.9	2.5
	C_1	1	1	1	1	1	1.4	1.7	2.1	2.8	2.8
	C_2	0	0	0	0	0	2.6	2.2	1.7	1.4	1.3
	C_3	0	0	0	0	0	0.8	1.1	1.2	0.9	0.5
60°	Z_c/H_s	0	0	0	0	0	1.35	1.6	1.8	2	2.6
	C_1	1	1	1	1	1	1.6	1.8	2.1	2.9	2.9
	C_2	0	0	0	0	0	2	1.6	1.3	1.3	1.2
	C_3	0	0	0	0	0	1	1.2	1.2	0.8	0.6

Note) For a particular inclination θ_s and a horizontal location X_s / H_s, the topography factor for fluctuating wind speed is calculated by interpolating linearly from the values at the nearest inclinations and horizontal locations.

(2) Turbulence scale

Turbulence scale is defined independently of the terrain categories of the construction site as:

$$L_Z = \begin{cases} 100\left(\dfrac{Z}{30}\right)^{0.5} & 30\text{m} < Z \le Z_G \\ 100 & Z \le 30\text{m} \end{cases} \tag{A6.11}$$

where

Z (m): height above ground

Z_G : parameter determining the exposure factor, defined in Table A6.3

A6.1.7 Return period conversion factor

Return period conversion factor k_{rW} is calculated from Eq.(A6.12).

$$k_{rW} = 0.63(\lambda_U - 1)\ln r - 2.9\lambda_U + 3.9 \tag{A6.12}$$

where

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

where

U_{500} (m/s): 500-year-recurrence 10-minute mean wind speed at 10m above ground over a flat and open terrain, defined in Fig.A6.4

U_0 (m/s): basic wind speed, defined in A6.1.3

r (year): design return period

Izu Islands, not shown in the map	52
Ogasawara Islands, Satsunann Islands, Okinawa Islands, Daitou Islands, Sakishima Islands, not shown in the map	58

Figure A6.4 500-year-recurrence 10-minute mean wind speed at 10m above ground over a flat and open terrain U_{500} (m/s)

A6.2 Wind force coefficients and wind pressure coefficients

Wind force coefficients and wind pressure coefficients fall into two categories corresponding to the design of the structural frames and components/claddings. The coefficients shall be estimated from wind tunnel experiments or from the following procedure using the wind pressure coefficients (external and internal pressure coefficients) and wind force coefficients provided in this clause.

A6.2.1 Procedure for estimating wind force coefficients

(1) Wind force coefficients for design of structural frames

1) Wind force coefficients C_D for estimating horizontal wind loads on structural frames

Wind force coefficients are given in A6.2.4(1) and A6.2.4(4) or calculated from Eq.(A6.13) using the external pressure coefficients provided in A6.2.2.

$$C_D = C_{pe1} - C_{pe2} \tag{A6.13}$$

where

C_{pe1}: external pressure coefficient on windward face

C_{pe2}: external pressure coefficient on leeward face

2) Wind force coefficients C_R for estimating roof wind loads on structural frames

Wind force coefficients are given in A6.2.5(2) or calculated from Eq.(A6.14) using the external pressure coefficients provided in A6.2.2 and the internal pressure coefficients provided in A6.2.3.

$$C_R = C_{pe} - C_{pi} \tag{A6.14}$$

where

C_{pe}: external pressure coefficient on roof

C_{pi}: internal pressure coefficient

3) Wind force coefficients C_D for estimating horizontal wind loads on lattice structures

Wind force coefficients are given in A6.2.4(3) or calculated from the wind force coefficients for individual members provided in A6.2.4(5).

(2) Peak wind force coefficients $\hat{C}_C$ for design of components/cladding

Peak wind force coefficients $\hat{C}_C$ are given in A6.2.7 or calculated from Eq.(A6.15) using the peak external pressure coefficients provided in A6.2.5 and the factor for the effect of fluctuating internal pressures provided in A6.2.6.

$$\hat{C}_C = \hat{C}_{pe} - C^*_{pi} \tag{A6.15}$$

where

$\hat{C}_{pe}$: peak external pressure coefficient

C^*_{pi}: factor for effect of fluctuating internal pressures

A6.2.2 External pressure coefficients for structural frames

(1) External pressure coefficients C_{pe} for buildings with rectangular sections and heights greater than 45m

For buildings with rectangular sections and heights greater than 45m, the external pressure coefficients on the windward and leeward walls and on the roof are given in Table A6.8. The values in Table A6.8 are applicable to buildings whose aspect ratios H / B are less than or equal to 8.

Table A6.8 External pressure coefficients C_{pe} for buildings with rectangular sections and heights greater than 45m

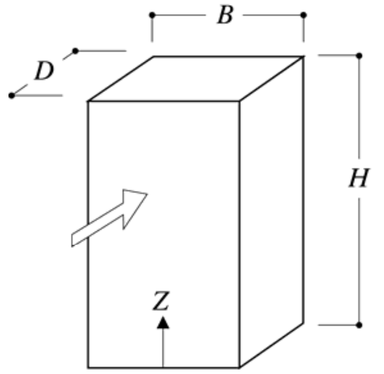
i) Wall

External pressure coefficient C_{pe}

	$D / B \leq 1$	$D / B > 1$
windward wall C_{pe1}	$0.8\,k_Z$	
leeward wall C_{pe2}	–0.5	–0.35

Factor for vertical profile k_Z

$Z \leq Z_b$	$Z_b < Z < 0.8H$	$Z \geq 0.8H$
$(Z_b / H)^{2\alpha}$	$(Z / H)^{2\alpha}$	$0.8^{2\alpha}$



B(m): building width

D(m): building depth

H(m): reference height

Z(m): height from ground

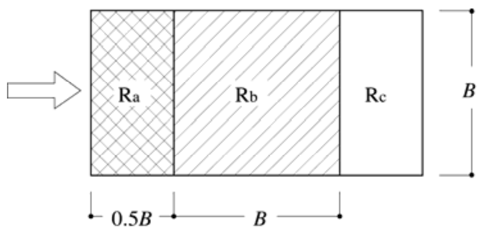
Z_b (m): height defined in Table A6.3

α : parameter defined in Table A6.3

ii) Roof

External pressure coefficient C_{pe}

zone R_a	–1.2
zone R_b	–0.6
zone R_c	–0.2



(2) External pressure coefficient C_{pe} for buildings with rectangular sections and heights less than or equal to 45m

1) Buildings with flat, gable and mono-sloped roofs

External pressure coefficients C_{pe} for buildings with rectangular sections and flat, gable and mono-sloped roofs whose heights are less than or equal to 45m are given in Table A6.9(1).

Table A6.9(1) External pressure coefficients C_{pe} for buildings with rectangular sections and flat, gable and mono-sloped roofs whose heights are less than or equal to 45m

i) Wall

zone W_U (windward wall)

$B/H \le 1$	$B/H > 1$
$0.8k_Z$	0.6

zone S (side wall)

S_a	S_b	S_c
–0.7	–0.4	–0.2

k_Z is factor for vertical profile provided in Table A6.8.

When $0.8H < Z_b$, $k_Z = 0.8^{2\alpha}$.

zone L (leeward wall)

<table>
<tr><th rowspan="2">wind dir.</th><th rowspan="2">roof angle
θ (°)</th><th colspan="2">L_a</th><th colspan="2">L_b</th></tr>
<tr><th>$D/H \le 1$</th><th>$D/H > 1$</th><th>$B/H < 6$</th><th>$B/H \ge 6$</th></tr>
<tr><td>W_1</td><td>$\theta \le 45$</td><td rowspan="3">–0.6</td><td rowspan="2">–0.4</td><td rowspan="4">same value
as zone L_a</td><td rowspan="3">–0.8</td></tr>
<tr><td rowspan="3">W_2
W_3</td><td>$\theta < 20$</td></tr>
<tr><td>$20 \le \theta < 30$</td><td>–0.5</td></tr>
<tr><td>$30 \le \theta \le 45$</td><td colspan="2">–0.6</td><td>–1.0</td></tr>
</table>

ii) Roof

zone R_U (windward roof)

<table>
<tr><th rowspan="2"></th><th rowspan="2">roof angle
θ (°)</th><th colspan="2">$D/H \le 1$</th><th colspan="2">$D/H > 1$</th></tr>
<tr><th>$B/H \le 2$</th><th>$B/H \ge 6$</th><th>$B/H \le 2$</th><th>$B/H \ge 6$</th></tr>
<tr><td rowspan="3">Positive</td><td>$\theta < 10$</td><td colspan="4">not necessary to evaluate</td></tr>
<tr><td>$10 \le \theta < 15$</td><td colspan="4">0</td></tr>
<tr><td>$15 \le \theta \le 45$</td><td colspan="4">$0.014(\theta - 15)$</td></tr>
<tr><td rowspan="5">Negative</td><td>$\theta < 10$</td><td colspan="4">same value as zone R (roof)</td></tr>
<tr><td>$10 \le \theta < 30$</td><td rowspan="2">$-0.84\tan(70-2\theta)$</td><td rowspan="4">$-0.81\tan(72-1.6\theta)$</td><td>$0.04(\theta-30)$</td><td rowspan="3">$-0.5\tan(80-2\theta)$</td></tr>
<tr><td>$30 \le \theta < 35$</td><td rowspan="3">0</td></tr>
<tr><td>$35 \le \theta < 40$</td><td rowspan="2">0</td></tr>
<tr><td>$40 \le \theta \le 45$</td><td>0</td></tr>
</table>

* Linear interpolation is permitted for $2 < B/H < 6$.

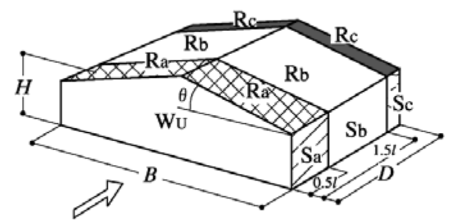
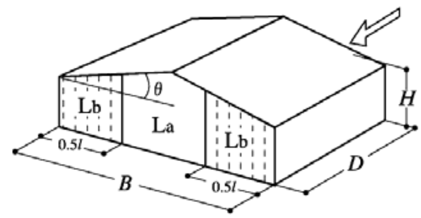
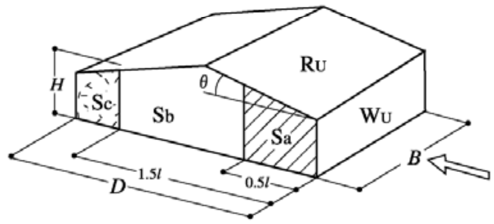
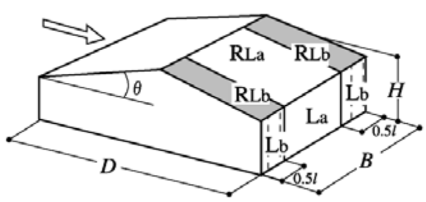
zone R_L (leeward roof)

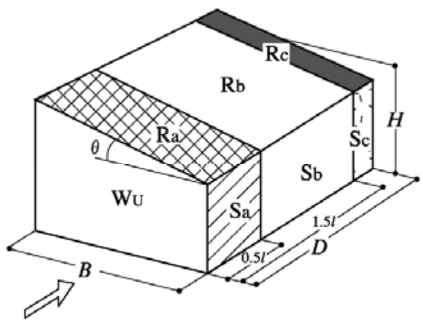
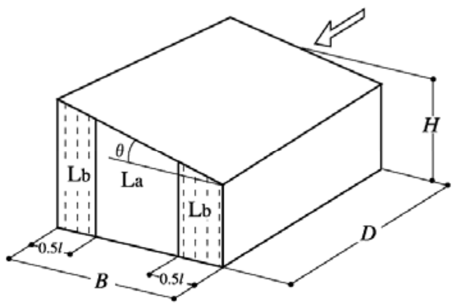
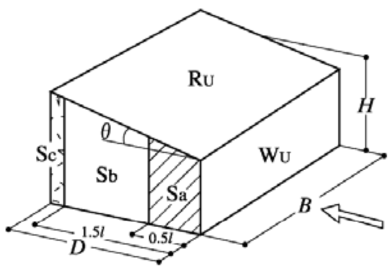
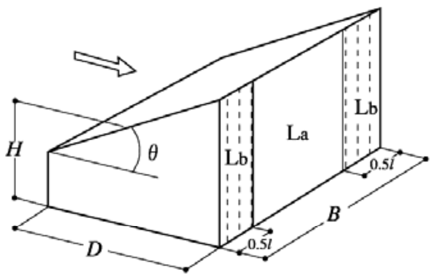
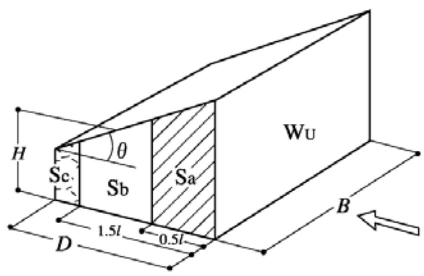
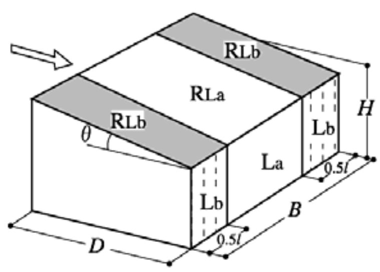
roof angle θ (°)	R_{La}		R_{Lb}	
	$D/H \leq 1$	$D/H > 1$	$B/H < 6$	$B/H \geq 6$
$\theta < 10$	same values as zone R (roof)			
$10 \leq \theta < 15$	–0.6	–0.5	same value as zone R_{La}	–1.1
$15 \leq \theta \leq 45$		–0.6		–1.4

zone R (roof)

	R_a		R_b	R_c
	$D/H \leq 1$	$D/H > 1$		
$B/H \leq 2$	–1.0	–0.8	–0.4	–0.2
$B/H \geq 6$	–1.2	–1.0	–0.6	–0.4

* Linear interpolation is permitted for $2 < B/H < 6$.

wind dir.	windward wall (zone Wu), windward roof (zone Ru), roof (zone R), side wall (zone S)	leeward wall (zone L), leeward roof (zone R_L)
W_1		
W_2		

W_1		
W_2		
W_3		

B(m) : building width

D(m) : building depth

H(m) : reference height

l (m) : the smaller of 4*H* and *B*

2) Buildings with vaulted roofs

External pressure coefficients for buildings with rectangular sections and vaulted roofs whose heights are less than or equal to 45m are given in Table A6.9(2).

Table A6.9(2) External pressure coefficients C_{pe} for buildings with rectangular sections and vaulted roofs whose heights are less than or equal to 45m

i) Wall

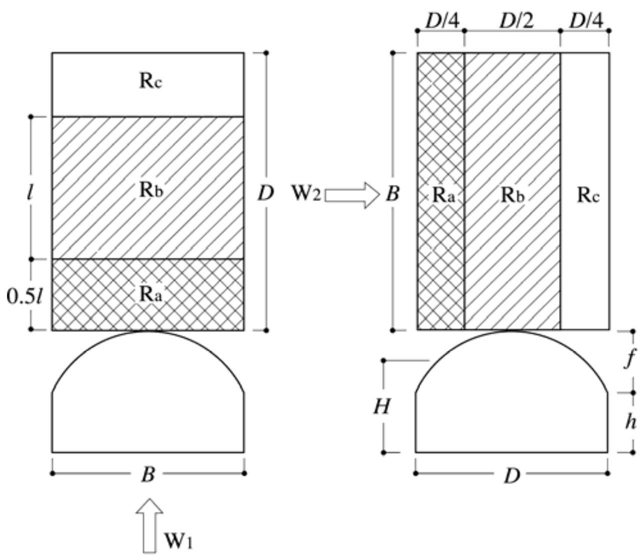
External pressure coefficients are defined in Table A6.9(1).

ii) Roof

wind dir.	*f/B*	zone R_a			zone R_b			zone R_c		
		h/B=0	*h/B*=0.3	*h/B*=0.7	*h/B*=0	*h/B*=0.3	*h/B*=0.7	*h/B*=0	*h/B*=0.3	*h/B*=0.7
W₁	0	–0.4	–0.9	–0.8	–0.4	–0.5	–0.4	–0.4	–0.3	–0.2
	0.1	–1.2	–1.1	–1.1	–0.7	–0.5	–0.5	–0.4	–0.4	–0.4
	0.3	–1.1	–1.1	–1.1	–0.6	–0.5	–0.5	–0.4	–0.4	–0.4
	0.4	–1.1	–1.1	–1.1	–0.5	–0.5	–0.5	–0.4	–0.4	–0.4

wind dir.	*f/D*	zone R_a			zone R_b			zone R_c		
		h/D=0	*h/D*=0.3	*h/D*=0.7	*h/D*=0	*h/D*=0.3	*h/D*=0.7	*h/D*=0	*h/D*=0.3	*h/D*=0.7
W₂	0	–0.4	–1.0	–0.9	–0.4	–1.0	–0.9	–0.4	–0.6	–0.9
	0.1	–0.5	–1.2	–1.5	–0.9	–1.0	–1.0	–0.5	–0.5	–0.5
	0.3	–0.1	–0.4	–0.9	–1.2	–1.4	–1.5	–0.5	–0.5	–0.5
	0.4	0.2	0	–0.5	–1.2	–1.3	–1.4	–0.5	–0.5	–0.5

* Linear interpolation is permitted for values *f/B*, *h/B*, *f/D* and *h/D* other than shown.



B(m): building width

D(m): building depth

H(m): reference height

f(m): rise

h(m): eaves height

l (m): the smaller of 4*H* and *B*

(3) External pressure coefficient C_{pe} for spherical domes

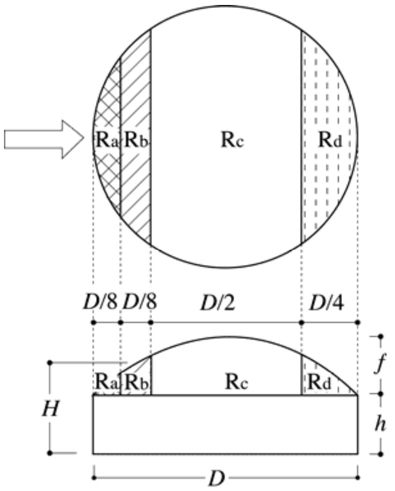
External pressure coefficients for spherical domes are given in Table A6.10.

Table A6.10 External pressure coefficients C_{pe} for spherical domes

f/D	zone R_a (positive)			zone R_a (negative)		
	h/D = 0	*h/D*=0.25	*h/D* = 1	*h/D* = 0	*h/D*=0.25	*h/D* = 1
0	Not necessary to evaluate			–0.6	–1.4	–1.2
0.05	0.3	0	0	0	–1.0	–1.6
0.1	0.4	0	0	0	–0.6	–1.2
0.2	0.5	0	0	0	0	–0.4
0.5	0.7	0.6	0.6	not necessary to evaluate		

f/D	zone R_b			zone R_c			zone R_d		
	h/D = 0	*h/D*=0.25	*h/D* = 1	*h/D* = 0	*h/D*=0.25	*h/D* = 1	*h/D* = 0	*h/D*=0.25	*h/D* = 1
0	0	–0.8	–1.2	0	–0.1	–0.4	0	–0.1	–0.3
0.05	0	–0.4	–0.8	–0.2	–0.4	–0.4	–0.1	–0.3	–0.3
0.1	0	–0.4	–0.6	–0.4	–0.6	–0.6	–0.2	–0.4	–0.4
0.2	0	–0.4	–0.6	–0.6	–0.8	–1.0	–0.2	–0.4	–0.4
0.5	0	–0.3	–0.4	–1.1	–1.2	–1.3	–0.2	–0.4	–0.4

* Linear interpolation is permitted for values *f/D* and *h/D* other than shown.



D(m): building diameter

H(m): reference height

h(m): eaves height

f (m): rise

A6.2.3 Internal pressure coefficients for structural frames

Internal pressure coefficients for structural frames shall be estimated appropriately considering the location and size of wall openings. Internal pressure coefficients for buildings without dominant openings are given in Table A6.11.

Table A6.11 Internal pressure coefficients C_{pi} for buildings without dominant openings

C_{pi}
0 or −0.4

A6.2.4 Wind force coefficients for design of structural frames

(1) Wind force coefficients C_D for buildings with circular sections

Wind force coefficients for buildings with circular sections are given in Table A6.12. The values in Table A6.12 are applicable to cases where $DU_H \geq 6\,(\mathrm{m}^2/\mathrm{s})$ and $H/D \leq 8$.

Table A6.12 Wind force coefficients for buildings with circular sections

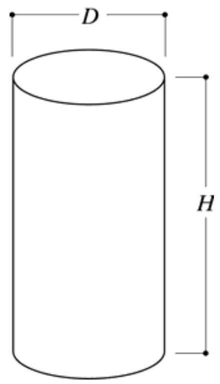
$$C_D = 1.2k_1k_2k_Z$$

where

k_1 : factor for aspect ratio

k_2 : factor for surface roughness

k_Z : factor for vertical profile defined in Table A6.8 and $k_Z = 0.8^{2\alpha}$ when $0.8H < Z_b$



k_1

$H/D < 1$	$1 \leq H/D \leq 8$
0.6	$0.6(H/D)^{0.14}$

k_2

smooth surface (metal, concrete, flat curtain walls, etc.)	0.75
rough surface (1% relative roughness, rough curtain walls, etc.)	0.9
very rough surface (5% relative roughness)	1

D(m): building diameter

H(m): reference height

Z_b (m): height defined in Table A6.3

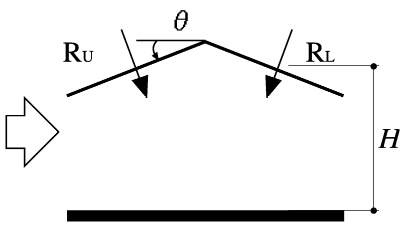
α : parameter defined in Table A6.3

(2) Wind force coefficient C_R for free roofs with rectangular base

Wind force coefficients for free roofs with rectangular base are given in Table A6.13. The values in Table A6.13 are applicable to small buildings specified in A6.11.

Table A6.13 Wind force coefficient C_R for free roofs with rectangular base

roof angle θ (°)	windward roof R_U positive	windward roof R_U negative	leeward roof R_L positive	leeward roof R_L negative
$-30 \leq \theta \leq -10$	$0.7+0.01\theta$	$-0.6+0.03\theta$	$0.05-0.025\theta$	$-1.2-0.03\theta$
$-10 < \theta < 10$	0.6	–0.9	0.3	–0.9
$10 \leq \theta \leq 30$	$0.3+0.03\theta$	$-1.15+0.025\theta$	0.3	$-0.6-0.03\theta$



H(m): reference height

θ (°): roof angle

Positive indicates downwards.

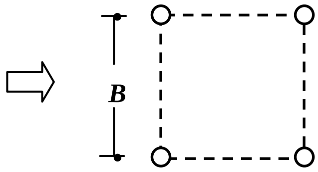
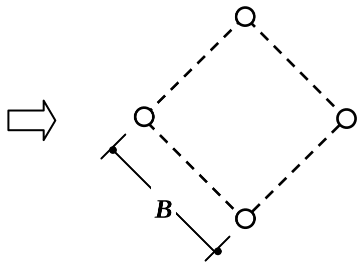
(3) Wind force coefficient C_D for lattice structures

6.5.6 Wind force coefficients for lattice structures are given in Table A6.14.

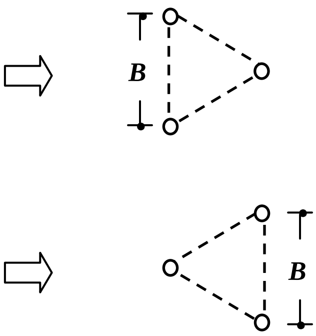
6.5.7

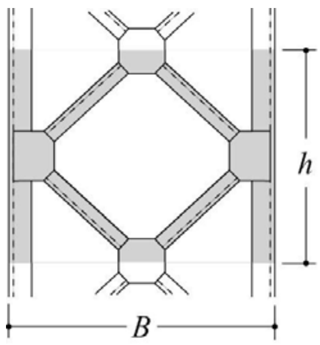
6.5.8 **Table A6.14** Wind force coefficients C_D for lattice structures

6.5.9 i) Square plan

				
solidity φ	angle	circular pipe	angle	circular pipe
0	3.8	2.3	4.4	2.5
0.5	1.9	1.4	2.3	1.7
0.6	1.9	1.4	2.3	1.7

ii) Triangular plan

		
solidity φ	angle	circular pipe
0	3.8	2.3
0.5	1.9	1.4
0.6	1.9	1.4



The solidity φ is defined by

$\varphi = A_F / A_0$

where

A_F(m^2): projected area per panel

()

A_0(m^2): whole plane area (=Bh)

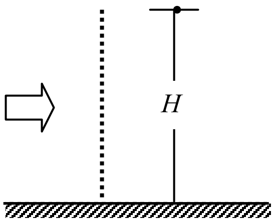
*Linear interpolation is permitted for values of φ other than shown.

(4) Wind force coefficients C_D for fences on ground

Wind force coefficients for fences on ground are given in Table A6.15.

Table A6.15 Wind force coefficients C_D for fences on ground

solidity φ	C_D
0	1.2
0.2	1.5
0.6	1.7
≥ 0.9 (solid fences included)	1.2



Note:

The area for calculating the wind loads is the overall area multiplied by the solidity φ.

The definition of φ is the same as that in Table A6.14.

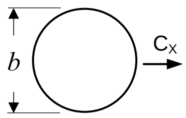
Linear interpolation is permitted for values of φ other than shown.

The height of fence H is used for calculating the wind load.

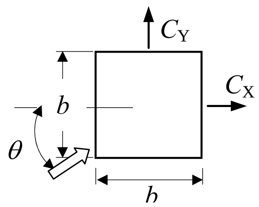
(5) Wind force coefficients C for components

Wind force coefficients for components are given in Table A6.16

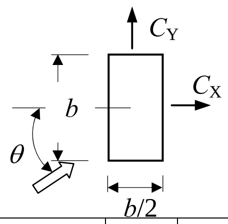
Table A6.16 Wind force coefficients C for members



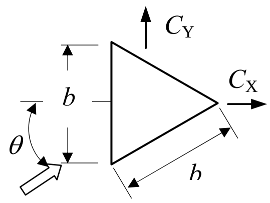
C_X
1.2



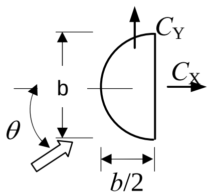
θ (°)	C_X	C_Y
0	2.1	0
45	1.6	1.6



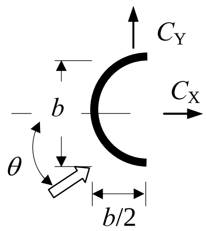
θ (°)	C_X	C_Y
0	2.4	0
45	1.6	0.7
90	0	0.8



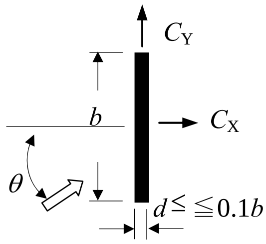
θ (°)	C_X	C_Y
0	2.1	0
30	2.1	–0.2
60	0.7	1.1



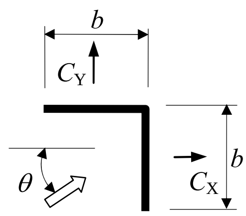
θ (°)	C_X	C_Y
0	1.2	0
45	0.8	0.8
90	0.6	0.5
135	–1.7	0.6
180	–2.3	0



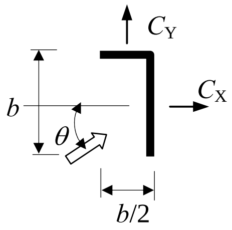
θ (°)	C_X	C_Y
0	1.1	0
45	0.8	0.7
90	0.9	0.5
135	–2.3	0.6
180	–2.5	0

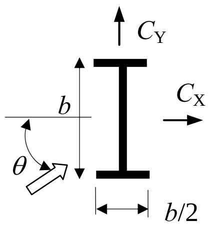


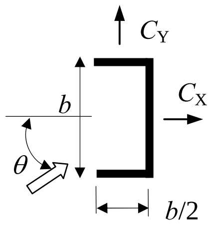
θ (°)	C_X	C_Y
0	2.0	0
45	1.8	0.1
90	0	0.1



θ (°)	C_X	C_Y
0	1.9	2.2
45	2.3	2.3
90	2.2	1.9
135	–1.9	–0.6
180	–2.0	0.3
225	–1.4	–1.4



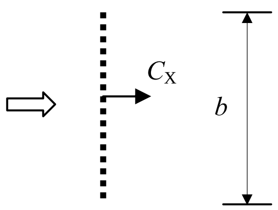




θ (°)	C_X	C_Y	θ (°)	C_X	C_Y	θ (°)	C_X	C_Y
0	2.0	1.1	0	2.1	0	0	2.6	0
45	2.3	1.1	45	2.1	0.6	45	2.0	0.8
90	1.8	0.8	90	±0.6	0.7	90	±0.6	0.8
135	–1.7	0				135	–1.6	0.6
180	–2.0	0.1				180	–2.0	0
225	–1.5	–0.6						
270	0.6	–0.8						
315	1.2	–0.2						

The area for calculating wind loads is bl (b = member width, l = member length) irrespective of wind direction.

net



solidity φ	C_X
0	2
0.2	2
0.6	2.7
≥ 0.9 (solid plate included)	2

The area for calculating wind loads is $bl\varphi$ (l = net length).

The definition of φ is the same as that in Table A6.14.

Linear interpolation is permitted for values of φ other than shown.

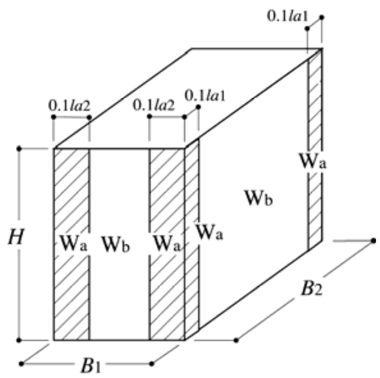
A6.2.5 Peak external pressure coefficients for components/claddings

(1) Peak external pressure coefficients $\hat{C}_{\mathrm{pe}}$ for buildings with rectangular sections and heights greater than 45 m

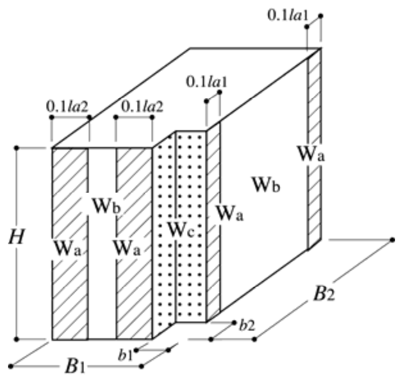
For buildings with rectangular sections and heights greater than 45m, peak external pressure coefficients are given in Table A6.17. The values in Table A6.17 are applicable to buildings whose aspect ratios H / B_1 are less than 8.

Table A6.17 Peak external pressure coefficients $\hat{C}_{\mathrm{pe}}$ for buildings with rectangular sections and heights greater than 45m

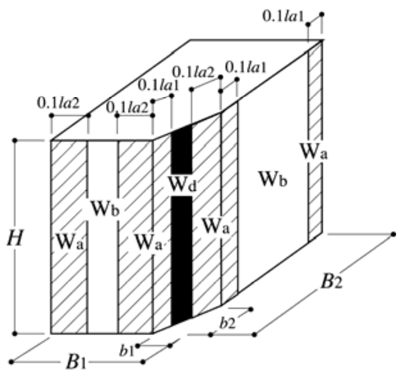
Building with rectangular section



Building with recessed corners



Building with chamfered corners



B_1(m): smaller side length of plan

B_2(m): larger side length of plan

H(m): reference height

l_{a1}(m): the smaller of H and B_1

l_{a2}(m): the smaller of H and B_2

i) Wall

a) Positive peak external pressure coefficients

$$\hat{C}_{\mathrm{pe}} = k_Z (1 + 7 I_Z)$$

where k_Z: factor for vertical profile defined in Table A6.8

I_Z: turbulence intensity at the height Z defined by Eq. (A6.7)

When the effect of local topography is considered, the values of k_Z and I_Z at reference height H ($z = H$) can be used in the above equation.

b) Negative peak external pressure coefficients

zone	case	$\hat{C}_{pe}$
W_a	all	–3.0
W_b	all	–2.4
W_c	$b / B \leq 0.2$	–3.0
	$b / B > 0.2$	–2.4
W_d	$b' / B \leq 0.2$	–3.0
	$b' / B > 0.2$	–2.4

b/B: the smaller of b_1/B_1 and b_2/B_2

b'/B: the larger of b_1/B_1 and b_2/B_2

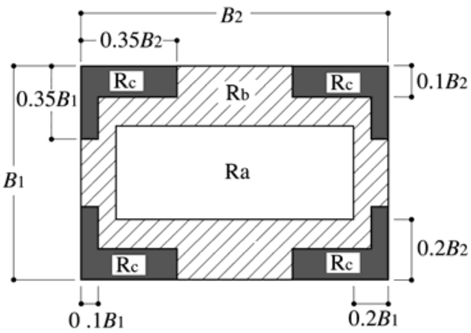
ii) Roof

a) Positive peak external pressure coefficients

Not necessary to evaluate

b) Negative peak external pressure coefficients

zone	$\hat{C}_{pe}$
R_a	–2.5
R_b	–3.2
R_c	$-5.0k_C$



Reduction factor for area subject to local suction k_C

subject area A_C (m^2)	k_C
$A_C < 1$	1
$1 \leq A_C \leq 5$	$1/ A_C^{0.18}$
$5 < A_C$	0.75

(2) Peak external pressure coefficients $\hat{C}_{\mathrm{pe}}$ for buildings with rectangular sections and heights less than or equal to 45m

1) Buildings with flat, gable and mono-sloped roofs

Peak external pressure coefficients for buildings with rectangular sections and flat, gable and mono-sloped roofs whose heights are less than or equal to 45m are given in Table A6.18(1).

Table A6.18(1) Peak external pressure coefficient $\hat{C}_{\mathrm{pe}}$ for buildings with rectangular sections and flat, gable and mono-sloped roofs whose heights are less than or equal to 45m

i) Wall

a) Positive peak external pressure coefficients

$$\hat{C}_{\mathrm{pe}} = 0.9(1+7I_{\mathrm{H}})$$

where I_{H} is turbulence intensity at height of H, obtained by substituting H for Z in Eq. (6.7)

b) Negative peak external pressure coefficients

zone $\mathrm{W_a}$	–3.0
zone $\mathrm{W_b}$	–2.4

ii)Roof

a) Positive peak external pressure coefficients

$$\hat{C}_{\mathrm{pe}} = C_{\mathrm{pe}}(1+7I_{\mathrm{H}})$$

where C_{pe} is positive external pressure coefficient for zone $\mathrm{R_U}$ provided in Table A6.9(1)

b) Negative peak external pressure coefficients

buildings with flat and gable roofs

zone	roof angle θ (^{o})		
	$\theta \le 10$	20	$30 \le \theta$
R_a	–3.2	–2.5	–2.5
R_b	$-5.4k_C$	–3.2	–3.2
R_c	–3.2	–3.2	–3.2
R_d	–3.2	$-5.4k_C$	–3.2
R_e	–2.5	–3.2	–3.2
R_f	–2.5	–2.5	–2.5
R_g	–2.5	$-5.4k_C$	–3.2

buildings with mono-sloped roofs

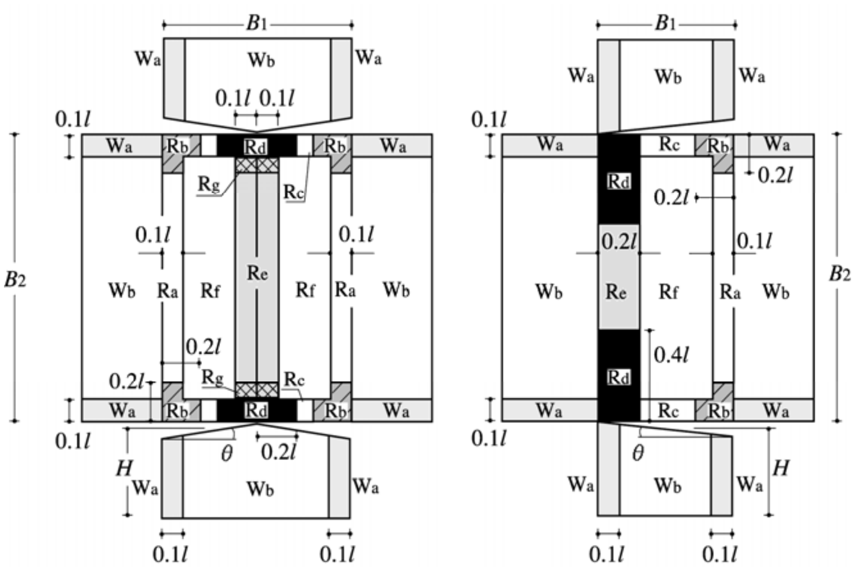
zone	roof angle θ (^{o})		
	10	20	$30 \le \theta$
R_a	–3.2	–2.5	–2.5
R_b	$-5.4k_C$	–3.2	–3.2
R_c	–3.2	–3.2	–3.2
R_d	$-5.4k_C$	$-6.5k_C$	$-5.4k_C$
R_e	–3.7	–3.7	–3.7
R_f	–2.5	–2.5	–2.5

k_C represents reduction factor for area subjected to local suction provided in Table A6.17.

Linear interpolation is permitted for values of θ other than shown.

For mono-sloped roofs with $\theta < 10^o$, the value for flat and gable roofs with $\theta \le 10^o$ is used.

Buildings with flat and gable roofs Buildings with mono-sloped roofs



l (m): the smallest of $4H$, B_1 and B_2

2) Buildings with vaulted roofs

Peak external pressure coefficients for buildings with rectangular sections and vaulted roofs whose heights are less than or equal to 45m are given in Table A6.18(2).

Table A6.18(2) Peak external pressure coefficient $\hat{C}_{pe}$ for buildings with rectangular sections and vaulted roofs whose heights are less than or equal to 45m

i) Wall

a) Positive peak external pressure coefficients

Positive peak external pressure coefficients are defined in Table A6.18(1).

b) Negative peak external pressure coefficients

Negative peak external pressure coefficients are defined in Table A6.18(1).

ii) Roof

a) Positive peak external pressure coefficients

f/B_1	zone R_a			zone R_b			zone R_c		
	h/B_1=0	$h\ B_1$ = 0.3	h/B_1 = 0.7	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7
0.1	0.8	0.8	0.5	0.5	0.4	0.3	0.2	0.1	0
0.3	2.0	2.3	1.8	1.6	1.4	1.2	0.6	0.4	0.4
0.4	2.2	2.4	2.4	1.9	1.8	1.8	0.8	0.6	0.5

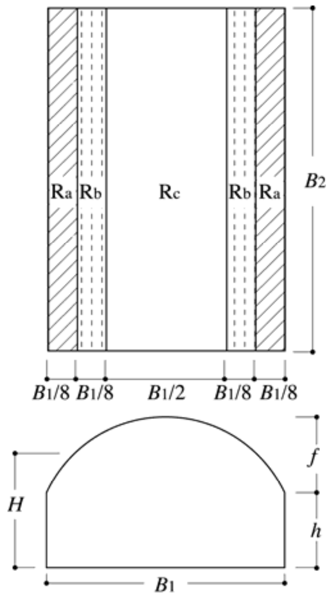
Linear interpolation is permitted for values h/B_1 and f/B_1 other than shown.

b) Negative peak external pressure coefficients

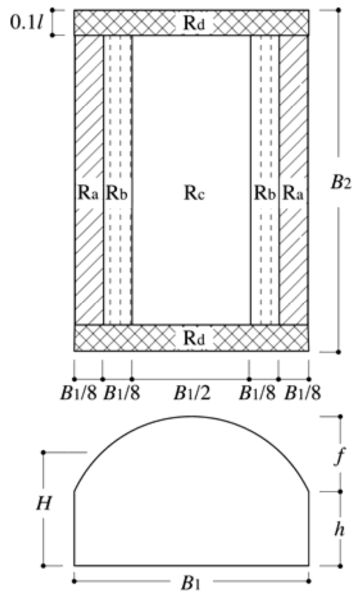
f/B_1	R_a			R_b			R_c			R_d		
	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7	h/B_1 = 0	h/B_1 = 0.3	h/B_1 = 0.7
0	–2.5	–3.2	–3.2	–2.5	–2.5	–2.5	–2.5	–2.5	–2.5	–2.5	$-5.4k_C$	$-5.4k_C$
0.1	–1.4	–4.2	–4.8	–1.8	–2.2	–3.2	–2.5	–2.5	–2.5	–3.4	–4.8	–4.4
0.3	–1.4	–2.4	–2.6	–2.0	–3.2	–3.2	–3.8	–4.4	–4.5	–4.0	–4.4	–4.5
0.4	–1.8	–2.4	–2.6	–2.4	–3.2	–3.2	–4.3	–4.4	–4.6	–4.0	–4.4	–4.8

k_C represents reduction factor for area subjected to local suction provided in Table A6.17.

Liner interpolation is permitted for values h/B_1 and f/B_1 other than shown.



zones for positive peak pressure coefficients



zones for negative peak pressure coefficients

h(m): eaves height
f(m) : rise
B_1(m): building length in span direction
B_2(m): building length in ridge direction
H(m): reference height
l (m): the smallest of $4H$, B_1 and B_2

(3) Peak external pressure coefficients $\hat{C}_{pe}$ for buildings with circular sections

Peak external pressure coefficients for buildings with circular sections are given in Table A6.19.

Table A6.19 Peak external pressure coefficients $\hat{C}_{pe}$ for buildings with circular sections

i) Wall

a) Positive peak external pressure coefficients

$$\hat{C}_{pe} = k_Z(1+7I_Z)$$

where k_Z: factor for vertical profile defined in Table A6.8

When $0.8H < Z_b$, $k_Z = 0.8^{2\alpha}$.

I_Z: turbulence intensity at reference height Z defined by Eq. (A6.7)

When the effect of local topography is considered, the values of k_Z and I_Z at the reference height H ($Z = H$) can be used in the above equation.

b) Negative peak external pressure coefficients

$$\hat{C}_{pe} = -\{(k_1-1)k_2 + k_3 + 1.4\}(1+7I_H)$$

where k_1: factor for aspect ratio defined in TableA6.12

k_2: factor for surface roughness defined in Table A6.12

k_3: factor for end effects defined in the following table

I_H: turbulence intensity at height of H, obtained by substituting H for Z in Eq.(6.7)

k_3

lower part	upper part		
	$H/D \le 2$	$2 < H/D \le 7$	$7 < H/D \le 8$
0.2	0.2	$0.1(H/D)$	0.7

ii) Roof

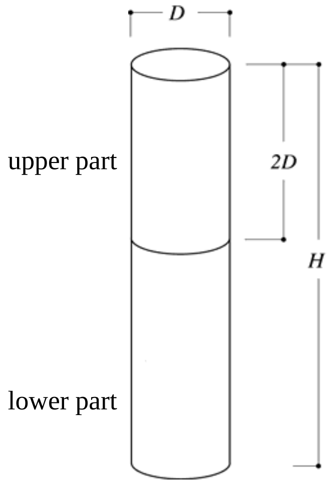
a) Positive peak external pressure coefficients

Not necessary to evaluate

b) Negative peak external pressure coefficients

The values defined in Table A6.20 for $f/D = 0$ can be used.

When $H/D > 1$, the value for $h/D = 1$ should be used.



D(m) : building diameter
H(m) : reference height

(4) Peak external pressure coefficients $\hat{C}_{pe}$ for buildings with circular sections and spherical domes

Peak external pressure coefficients $\hat{C}_{pe}$ for buildings with circular sections and spherical domes are given in Table A6.20. The values in Table A6.20 are applicable to buildings whose aspect ratios h/D are less than 1.

Table A6.20 Peak external pressure coefficients $\hat{C}_{pe}$ for buildings with circular sections and dome roofs

i) Wall

a) Positive peak external pressure coefficients

Positive peak external pressure coefficients are defined in Table A6.19.

b) Negative peak external pressure coefficients

Negative peak external pressure coefficients are defined in Table A6.19.

ii) Roof

a) Positive peak external pressure coefficients

f/D	zone R_a			zone R_b			zone R_c		
	h/D =0	*h/D* =0.25	*h/D* =1	*h/D* =0	*h/D*=0.25	*h/D* =1	*h/D* =0	*h/D*=0.25	*h/D* =1
0	0.6	0.4	0.4	1.1	0.5	0.5	0.6	0.6	0.6
0.05	1.3	0.5	0.5	1.0	0.4	0.4	0.5	0.1	0.1
0.1	1.7	0.7	0.7	0.9	0.3	0.3	0.4	0	0
0.2	$0.9(1+7I_H)$	$0.6(1+7I_H)$	$0.4(1+7I_H)$	1.2	0.6	0.6	0.2	0	0
0.5	$1+7I_H$	$1+7I_H$	$1+7I_H$	1.9	1.3	0.7	0.3	0	0

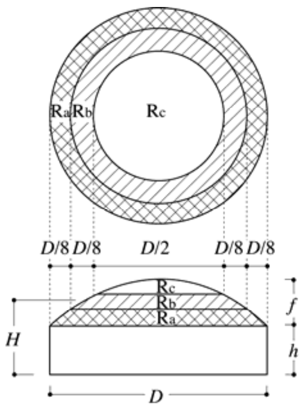
I_H: turbulence intensity at reference height *H*, obtained by substituting *H* for *Z* in Eq. (A6.7).

Linear interpolation is permitted for values *f/D* and *h/D* other than shown.

b) Negative peak external pressure coefficients

f/D	zone R_a			zone R_b			zone R_c		
	h/D =0	*h/D*=0.25	*h/D* =1	*h/D* =0	*h/D*=0.25	*h/D* =1	*h/D* =0	*h/D*=0.25	*h/D* =1
0	–4.4	–5.1	–3.3	–1.5	–3.7	–3.0	–0.4	–2.3	–2.3
0.05	–3.0	–4.8	–3.3	–1.5	–2.7	–2.7	–1.3	–1.3	–1.3
0.1	–2.0	–4.2	–3.0	–1.5	–2.2	–2.2	–1.4	–1.4	–1.4
0.2	–2.0	–2.0	–2.0	–1.9	–1.9	–1.9	–2.1	–2.1	–2.1
0.5	–2.6	–2.6	–2.6	–2.8	–2.8	–2.8	–3.0	–3.0	–3.0

Linear interpolation is permitted for values *f/D* and *h/D* other than shown.



D(m): building diameter

f(m): rise

H(m): reference height

h(m): eaves height

A6.2.6 Factor for effect of fluctuating internal pressures

The factor for the effect of fluctuating internal pressures for designing components/cladding shall be estimated appropriately considering the location and size of wall openings. The values of C^*_{pi} for buildings without dominant openings are given in Table A6.21.

Table A6.21 Factor C^*_{pi} for effect of fluctuating internal pressures for buildings without dominant openings

C^*_{pi}
0 or –0.5

A6.2.7 Peak wind force coefficients for components/cladding

Peak wind force coefficients for free roofs with rectangular base are specified as shown in Table A6.22. The values in Table A6.22 are applicable to small buildings specified in A6.11.

Table A6.22 Peak wind force coefficient $\hat{C}_C$ for free roofs with rectangular base

a) Positive peak wind force coefficients

zone	roof angle θ (°)	$\hat{C}_C$
R_a	$-30 \le \theta \le -10$	$(0.65-0.015\theta)(1+7I_H)$
	$-10 < \theta < 10$	$0.8(1+7I_H)$
	$10 \le \theta \le 30$	$(0.55+0.025\theta)(1+7I_H)$
R_b	$-30 \le \theta \le -10$	$(0.9-0.02\theta)(1+7I_H)$
	$-10 < \theta < 10$	$1.1(1+7I_H)$
	$10 \le \theta \le 30$	$(0.85+0.025\theta)(1+7I_H)$

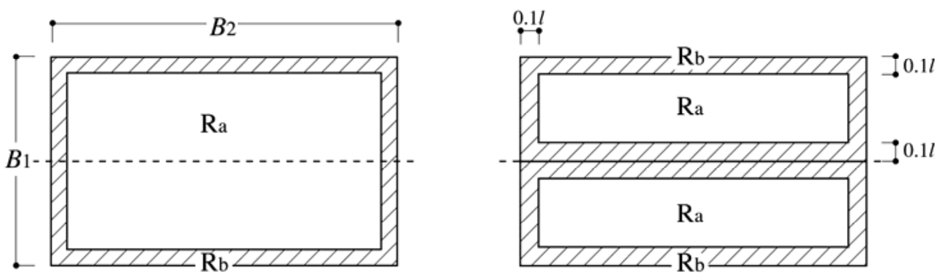
I_H: turbulence intensity at reference height H, obtained by substituting H for Z in Eq. (A6.7)

θ (°): roof angle specified in Table A6.13

l (m): smallest of $4H$, B_1 and B_2

b) Negative peak wind force coefficients

zone	roof angle θ (°)	$\hat{C}_C$
R_a	$\lvert\theta\rvert < 10$	-3.5
	$10 \le \lvert\theta\rvert \le 30$	$-2.9-0.06\lvert\theta\rvert$
R_b	$\lvert\theta\rvert < 10$	-4.5
	$10 \le \lvert\theta\rvert \le 30$	$-3.8-0.075\lvert\theta\rvert$



$\lvert\theta\rvert < 10^\circ$ $10^\circ \le \lvert\theta\rvert \le 30^\circ$

A6.3 Gust Effect Factors

A6.3.1 Gust effect factor for along-wind loads on structural frames

Gust effect factor G_D for along-wind loads on structural frames is estimated from Eq.(A6.16).

$$G_D = 1 + g_D \frac{C'_g}{C_g} \sqrt{1+\phi_D^2 R_D} \tag{A6.16}$$

where

$$g_D = \sqrt{2\ln(600\nu_D) + 1.2}$$

$$C_g = \frac{1}{3+3\alpha} + \frac{1}{6}$$

$$C'_g = 2I_H \frac{0.49-0.14\alpha}{\left\{1+\dfrac{0.63\left(\sqrt{BH}/L_H\right)^{0.56}}{(H/B)^k}\right\}} \qquad \begin{cases} k = 0.07 & (H/B \geq 1) \\ k = 0.15 & (H/B < 1) \end{cases}$$

$$R_D = \frac{\pi F_D}{4\zeta_D}$$

$$\nu_D = f_D \sqrt{\frac{R_D}{1+R_D}}$$

$$F_D = \frac{I_H^2 F S_D \left(0.57 - 0.35\alpha + 2R\sqrt{0.053-0.042\alpha}\right)}{C'^2_g}$$

$$R = \frac{1}{1+20\dfrac{f_D B}{U_H}}$$

$$F = \frac{4\dfrac{f_D L_H}{U_H}}{\left\{1+71\left(\dfrac{f_D L_H}{U_H}\right)^2\right\}^{5/6}}$$

$$S_D = \frac{0.9}{\left\{1+6\left(\dfrac{f_D H}{U_H}\right)^2\right\}^{0.5}\left(1+3\dfrac{f_D B}{U_H}\right)}$$

where

ϕ_D : mode correction factor given in Eq.(A6.32)

f_D (Hz): natural frequency for first mode in along-wind direction

ζ_D : critical damping ratio for first mode in along-wind direction

H (m): reference height as defined in 6.1.2(4)

B (m): projected breadth of building

U_H (m/s): design wind speed as defined in A6.1.2

I_H : turbulence intensity at reference height given in Eq.(A6.7) in which H is substituted for Z

L_H (m): turbulence scale at reference height given in Eq.(A6.11) in which H is substituted for Z

α : exponent of power law in wind speed profile defined in A6.1.5

A6.3.2 Gust effect factor for roof wind loads on structural frames

Gust effect factor G_R for roof wind loads on structural frames of buildings without dominant openings for internal pressure coefficient and wind force coefficient is specified as follows.

(1) Internal pressure coefficient C_{pi} is equal to –0.4.

1) Wind force coefficient C_R on tributary area of roof beam is not equal to 0.

Gust effect factor G_R for roof wind loads on structural frames is estimated from Eq.(A6.17).

$$G_R = 1 \pm \frac{\sqrt{12.3 r_{Re}^2 (1 + R_{Re}) + 0.3 r_c^2}}{|1 - r_c|} \tag{A6.17}$$

2) Wind force coefficient C_R on tributary area of roof beam is equal to 0.

The product of wind force coefficient C_R and gust effect factor G_R is estimated from Eq.(A6.18).

$$C_R G_R = \pm 0.25 \sqrt{12.3 r_{Re}^2 (1 + R_{Re}) + 0.3} \tag{A6.18}$$

(2) Internal pressure coefficient C_{pi} is equal to 0.

Gust effect factor G_R for roof wind loads on structural frames is estimated from Eq.(A6.19).

$$G_R = 1 \pm \sqrt{12.3 r_{Re}^2 (1 + R_{Re}) + 0.3 r_c^2} \tag{A6.19}$$

where parameters r_{Re}, R_{Re}, r_c in Eqs.(A6.17), (A6.18) and (A6.19) are defined as follows for direction of roof beam.

a) For roof beam parallel to wind direction

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

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

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

b) For roof beam normal to wind direction

$$r_c = -0.4 / C_{pe}$$

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

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

where

C_{pe} : external pressure coefficient as defined in A6.2.1
H (m): reference height as defined in 6.1.2(4)
I_{H} : turbulence intensity at reference height given in Eq.(A6.7) in which H is substituted for Z
L (m): span of roof beam
U_{H} (m/s): design wind speed as defined in A6.1.2
f_{R} (Hz): natural frequency for first mode of roof beam
ζ_{R} : critical damping ratio for first mode of roof beam

A6.4 Across-wind Vibration and Resulting Wind Load

A6.4.1 Scope of application

This section defines the procedures for estimating horizontal across-wind loads on structural frames. The procedure can be applied to buildings that satisfy the following conditions when wind is normal to the front face.

i) Buildings have a uniform rectangular section from bottom to top.

ii) $\dfrac{H}{\sqrt{BD}} \le 6$

iii) $0.2 \le \dfrac{D}{B} \le 5$

iv) $\dfrac{U_{\text{H}}}{f_{\text{L}}\sqrt{BD}} \le 10$

where

H (m): reference height as defined in 6.1.2(4)
B (m): projected breadth
D (m): depth
U_{H} (m/s): design wind speed as defined in A6.1.2
f_{L} (Hz): natural frequency for first mode in across-wind direction

A6.4.2 Procedure

Wind loads on structural frames caused by across-wind vibration are calculated from Eq.(A6.20).

$$W_{\text{L}} = 3q_{\text{H}}C'_{\text{L}}A\frac{Z}{H}g_{\text{L}}\sqrt{1+\phi_{\text{L}}^2 R_{\text{L}}} \qquad \text{(A6.20)}$$

where

$$C'_{\text{L}} = 0.0082(D/B)^3 - 0.071(D/B)^2 + 0.22(D/B)$$
$$g_{\text{L}} = \sqrt{2\ln(600 f_{\text{L}}) + 1.2}$$
$$R_{\text{L}} = \frac{\pi F_{\text{L}}}{4\zeta_{\text{L}}}$$

$$F_L = \sum_{j=1}^{m} \frac{4\kappa_j (1+0.6\beta_j)\beta_j}{\pi} \frac{(f_L / f_{sj})^2}{\left\{1-(f_L / f_{sj})^2\right\}^2 + 4\beta_j^2 (f_L / f_{sj})^2}$$

$$\kappa_1 = 0.85$$

$$\kappa_2 = 0.02$$

$$m = \begin{cases} 1, & D/B < 3 \\ 2, & D/B \geq 3 \end{cases}$$

$$f_{s1} = \frac{0.12}{\{1+0.38(D/B)^2\}^{0.89}} \frac{U_H}{B}$$

$$f_{s2} = \frac{0.56}{(D/B)^{0.85}} \frac{U_H}{B}$$

$$\beta_1 = \frac{(D/B)^4 + 2.3(D/B)^2}{\{2.4(D/B)^4 - 9.2(D/B)^3 + 18(D/B)^2 + 9.5(D/B) - 0.15\}} + \frac{0.12}{(D/B)}$$

$$\beta_2 = \frac{0.28}{(D/B)^{0.34}}$$

where

W_L (N): across-wind load at height Z

q_H (N/m^2): velocity pressure as defined in A6.1.1

A (m^2): projected area at height Z

B (m): projected breadth

D (m): depth

Z (m): height

H (m): reference height as defined in 6.1.2(4)

ϕ_L : correction coefficient for vibration mode as defined in Eq.(A6.33)

f_L (Hz): natural frequency for first mode in across-wind direction

ζ_L : critical damping ratio for first mode in across-wind direction

U_H (m/s): design wind speed as defined in A6.1.2

A6.5 Torsional Vibration and Resulting Wind Load

A6.5.1 Scope of application

This section defines the procedures for estimating torsional wind load structural frames. They can be applied to buildings that satisfy the following conditions when wind is normal to the front face.

i) Buildings have a uniform rectangular section from bottom to top.

ii) $\dfrac{H}{\sqrt{BD}} \leq 6$

iii) $0.2 \leq \dfrac{D}{B} \leq 5$

iv) $\dfrac{U_H}{f_T \sqrt{BD}} \leq 10$

where

H (m): reference height as defined in 6.1.2(4)

B (m): projected breadth

D (m): depth

U_{H} (m/s): design wind speed as defined in A6.1.2

f_{T} (Hz): natural frequency for first mode in torsional direction

A6.5.2 Procedure

Torsional wind loads on structural frames are calculated using Eq.(A6.21).

$$W_{\mathrm{T}} = 1.8 q_{\mathrm{H}} C'_{\mathrm{T}} AB \frac{Z}{H} g_{\mathrm{T}} \sqrt{1+\phi_{\mathrm{T}}^2 R_{\mathrm{T}}} \tag{A6.21}$$

where

$$C'_{\mathrm{T}} = \{0.0066 + 0.015(D/B)^2\}^{0.78}$$

$$g_{\mathrm{T}} = \sqrt{2\ln(600 f_{\mathrm{T}}) + 1.2}$$

$$R_{\mathrm{T}} = \frac{\pi F_{\mathrm{T}}}{4\zeta_{\mathrm{T}}}$$

$$U_{\mathrm{T}}^* = \frac{U_{\mathrm{H}}}{f_{\mathrm{T}}\sqrt{BD}}$$

$$F_{\mathrm{T}} = \begin{cases} \dfrac{0.14 K_{\mathrm{T}}^2 (U_{\mathrm{T}}^*)^{2\beta_{\mathrm{T}}}}{\pi} \dfrac{D(B^2+D^2)^2}{L^2 B^3} & [U_{\mathrm{T}}^* \le 4.5,\ 6 \le U_{\mathrm{T}}^* \le 10] \\ F_{4.5} \exp\left[3.5 \ln\left(\dfrac{F_6}{F_{4.5}}\right) \ln\left(\dfrac{U_{\mathrm{T}}^*}{4.5}\right)\right] & [4.5 < U_{\mathrm{T}}^* < 6] \end{cases}$$

$F_{4.5}$, F_6: values of F_{T} when U_{T}^* =4.5 and 6, respectively

$$K_{\mathrm{T}} = \begin{cases} \dfrac{-1.1(D/B)+0.97}{(D/B)^2+0.85(D/B)+3.3} + 0.17 & [U_{\mathrm{T}}^* \le 4.5] \\ \dfrac{0.077(D/B)-0.16}{(D/B)^2-0.96(D/B)+0.42} + \dfrac{0.35}{D/B} + 0.095 & [6 \le U_{\mathrm{T}}^* \le 10] \end{cases}$$

$$\beta_{\mathrm{T}} = \begin{cases} \dfrac{(D/B)+3.6}{(D/B)^2-5.1(D/B)+9.1} + \dfrac{0.14}{D/B} + 0.14 & [U_{\mathrm{T}}^* \le 4.5] \\ \dfrac{0.44(D/B)^2-0.0064}{(D/B)^4-0.26(D/B)^2+0.1} + 0.2 & [6 \le U_{\mathrm{T}}^* \le 10] \end{cases}$$

where

W_{T} (Nm): torsional wind load at height Z

q_{H} (N/m^2): velocity pressure as defined in A6.1.1

A (m^2): projected area at height Z

B (m): projected breadth

D (m): depth

L (m): the larger of B and D

Z (m): height

H (m): reference height as defined in 6.1.2(4)

ϕ_{T} : correction coefficient for vibration mode as defined in Eq.(A6.34)

f_{T} (Hz): natural frequency for first mode in torsional direction

ζ_{T} : critical damping ratio for first mode in torsional direction

U_{H} (m/s): design wind speed as defined in A6.1.2

A6.6 Horizontal Wind Loads on Lattice Structural Frames

A6.6.1 Scope of application

This section defines the procedures for estimating horizontal wind loads on lattice structures built directly on the ground, due to gust action.

A6.6.2 Procedure for estimating wind loads

Horizontal wind loads on lattice structures are calculated from Eq.(A6.22).

$$W_{\mathrm{D}} = q_{\mathrm{Z}} C_{\mathrm{D}} G_{\mathrm{D}} A_{\mathrm{F}} \tag{A6.22}$$

where

W_{D} (N): wind load

q_{Z} (N/m^2): velocity pressure at height Z, as acquired by changing H to Z in Eq.(A6.1)

C_{D} : wind force coefficient, as defined in A6.2.4(3)

G_{D} : gust effect factor calculated by the method described in A6.6.3

A_{F} (m^2): projected area of one face of lattice structure at height Z

A6.6.3 Gust effect factor

Gust effect factor is estimated from Eq.(A6.23).

$$G_{\mathrm{D}} = 1 + g_{\mathrm{D}} \frac{C'_{\mathrm{g}}}{C_{\mathrm{g}}} \phi_{\mathrm{D}} \sqrt{1 + R_{\mathrm{D}}} \tag{A6.23}$$

where

$$g_{\mathrm{D}} = \sqrt{2\ln(600 v_{\mathrm{D}}) + 1.2}$$

$$C'_{\mathrm{g}} = \frac{2 I_{\mathrm{H}}}{\alpha + 3} \sqrt{B_{\mathrm{D}}}$$

$$C_{\mathrm{g}} = \frac{1}{2\alpha + 3} - \frac{\lambda_{\mathrm{B}}}{2\alpha + 4}$$

$$R_D = \frac{\pi}{4\zeta_{\mathrm{D}}} \frac{S_{\mathrm{D}} F}{B_{\mathrm{D}}}$$

$$v_{\mathrm{D}} = f_{\mathrm{D}} \sqrt{\frac{R_{\mathrm{D}}}{1 + R_{\mathrm{D}}}}$$

$$B_{\mathrm{D}} = \left(1 - \frac{3}{4}\lambda_{\mathrm{B}}\right)^2 \frac{1}{1 + \frac{2\sqrt{HB}}{L_{\mathrm{H}}}}$$

$$\lambda_B = 1 - \frac{B_H}{B_0}$$

$$S_D = \left(1 - \frac{3}{4}\lambda_B\right)^2 \frac{1}{\left(1 + 3.5\frac{f_D B}{U_H}\right)\left(1 + 2\frac{f_D H}{U_H}\right)}$$

$$F = \frac{4\frac{f_D L_H}{U_H}}{\left\{1 + 71\left(\frac{f_D L_H}{U_H}\right)^2\right\}^{5/6}}$$

$$B = \frac{B_H + B_0}{2}$$

where

ϕ_D : mode shape correction factor, as calculated from (A6.32)

I_H : turbulence intensity at reference height, as acquired by changing Z to H in Eq.(A6.7)

L_H (m): turbulence scale at reference height, as acquired by changing Z to H in Eq.(A6.11)

α : exponent of power law in wind speed profile defined in A6.1.5

f_D (Hz): natural frequency for first mode in along-wind direction

ζ_D : critical damping ratio for first mode in along-wind direction

H (m): reference height, i.e. height of lattice structure

B_0 (m): width at ground level

B_H (m): width at height H

U_H (m/s): design wind speed as defined in A6.1.2

A6.7 Vortex Induced Vibration

A6.7.1 Scope of application

This section describes wind load on buildings with circular sections and their components caused by vortex induced vibration.

A6.7.2 Vortex induced vibration and resulting wind load on buildings with circular sections

Wind loads on buildings with circular sections caused by vortex-induced vibration are calculated from Eq.(A6.24).

$$W_r = 0.8\rho U_r^2 C_r \frac{Z}{H} A \tag{A6.24}$$

where U_r (m/s) is the resonance wind speed calculated from Eq.(A6.25).

$$U_r = 5 f_L D_m \tag{A6.25}$$

where

W_r (N): wind load at height Z

ρ (kg/m^3): air density (=1.22)

C_r : wind force coefficient at resonance, as defined in Table A6.23

Z (m): height from ground level

H (m): reference height as defined in 6.1.2(4)

A (m^2): projected area at height Z

f_L (Hz): natural frequency of first mode in across wind direction

D_m (m): diameter of building at height $2H/3$

Table A6.23 Wind force coefficient at resonance C_r

$U_r D_m$	$\rho_s\sqrt{\zeta_L} < 5$	$\rho_s\sqrt{\zeta_L} \geq 5$
$U_r D_m < 3$	$\dfrac{1.3}{\sqrt{\zeta_L}} + \dfrac{1.5}{\zeta_L}\dfrac{\rho}{\rho_s}$	$\dfrac{1.7}{\sqrt{\zeta_L}}$
$3 \leq U_r D_m < 6$	Linear interpolation	Linear interpolation
$6 \leq U_r D_m$	$\dfrac{0.53}{\sqrt{\zeta_L}} + \dfrac{0.16}{\zeta_L}\dfrac{\rho}{\rho_s}$	$\dfrac{0.57}{\sqrt{\zeta_L}}$

where

ζ_L : critical damping ratio for first mode in across wind direction

ρ_s (kg/m^3): building density as given by $M/(HD_m D_B)$

M (kg): total building mass

D_B (m): building diameter at base

A6.7.3 Vortex-induced vibration and resulting wind load on building components with circular sections

Wind loads on building components with circular sections caused by vortex-induced vibration are calculated from Eq.(A6.27) when the conditions of Eq.(A6.26) are satisfied.

$$\frac{L}{D} \geq 15 \quad \text{and} \quad \frac{U_H}{f_L D} \geq 4.2 \tag{A6.26}$$

where

L (m): length of component

D (m): diameter of component

U_H (m/s): design wind speed at height H which is the mean height of the component as defined in 6.1.2(4)

f_L (Hz): natural frequency for first bending mode

$$W_r = (2\pi f_L)^2 \sin\left(\frac{\pi x}{L}\right)\frac{M}{L}\frac{0.26U_r^*}{0.75\delta^{1.1} + 0.36U_r^*}A \tag{A6.27}$$

where U_r^* (m/s) is the non-dimensional resonance wind speed, and δ is a mass-damping parameter, calculated from Eqs.(A6.28) and (A6.29), respectively.

$$U_r^* = 5 + \frac{3}{\delta} \tag{A6.28}$$

$$\delta = \frac{4\pi\zeta_L M}{\rho D^2 L} \tag{A6.29}$$

where

W_r (N): wind load at x distant from the end of the component

x (m): distance from end of component

M (kg): total mass of component

L (m): span of component

A (m^2): projected area at x

ζ_L : critical damping ratio for first bending mode of component

ρ (kg/m^3): air density (=1.22)

A6.8 Combination of Wind Loads

A6.8.1 Scope of application

This section defines the procedures for estimating the combination of horizontal wind loads and roof wind loads.

For buildings not satisfying the conditions of Eq.(6.1), combination of along-wind load and across-wind load should be considered by reference to A6.8.2. For buildings satisfying the condition of Eq.(6.1), combination of horizontal wind loads should be considered by reference to A6.8.3.

For horizontal wind load and roof wind load, the combination load must be considered by reference to A6.8.4

A6.8.2 Combination of horizontal wind loads on buildings not satisfying the conditions of Eq.(6.1)

Along-wind load calculated by 6.2 and across-wind load calculated from Eq.(A6.30) must be considered together.

$$W_{LC} = \gamma W_D \tag{A6.30}$$

where

$$\gamma = 0.35\frac{D}{B} \quad \text{and} \quad \gamma \geq 0.2$$

where

W_{LC} (N): combined across-wind load

W_D (N): along-wind load defined in 6.2

B (m): projected breadth of building

D (m): depth of building

A6.8.3 Combination of horizontal wind loads for buildings satisfying the conditions of Eq.(6.1)

The three load combinations described in Table A6.24 must be considered.

Table A6.24 Horizontal wind load combinations

Combination	along-wind force	across-wind force	torsional moment
1	W_{D}	$0.4W_{\mathrm{L}}$	$0.4W_{\mathrm{T}}$
2	$W_{\mathrm{D}}\left(0.4+\dfrac{0.6}{G_{\mathrm{D}}}\right)$	W_{L}	$\left(\sqrt{2+2\rho_{\mathrm{LT}}}-1\right)W_{\mathrm{T}}$
3	$W_{\mathrm{D}}\left(0.4+\dfrac{0.6}{G_{\mathrm{D}}}\right)$	$\left(\sqrt{2+2\rho_{\mathrm{LT}}}-1\right)W_{\mathrm{L}}$	W_{T}

Note) $W_{\mathrm{D}}, W_{\mathrm{L}}, W_{\mathrm{T}}$ are the load effects due to along-wind load, across-wind load and torsional load, defined in 6.2, A6.4 and A6.5, respectively. G_{D} is the gust effect factor for along-wind loads defined in A6.3.1. ρ_{LT} is the correlation coefficient between across-wind vibration and torsional vibration defined in Table A.6.25.

Table A6.25 Correlation coefficient, ρ_{LT}

D/B	$f_1 B/U_{\mathrm{H}}$	ρ_{LT}		
		$\xi=1.0$	$\xi=1.1$	$\xi\geq 1.4$
≤ 0.5	≤ 0.1	0.9	0.3	0.2
	0.2	0.2	0.4	0.3
	0.3	0.2	0.5	0.4
	0.6	0.6	0.6	0.6
	≥ 1	0.7	0.7	0.7
1	≤ 0.1	0.7	0.2	0.2
	0.2	0.3	0.2	0.2
	0.3	0.2	0.2	0.2
	0.6	0.4	0.4	0.4
	≥ 1	0.5	0.5	0.5
≥ 2	≤ 0.05	0.6	0.2	0.2
	≥ 0.1	0.2	0.2	0.2

Note) For intermediate values of ξ, $f_1 B/U_{\mathrm{H}}$ and D/B, linear interpolation may be applied.

$$\xi=\begin{cases} f_{\mathrm{L}}/f_{\mathrm{T}} & f_{\mathrm{L}}\geq f_{\mathrm{T}} \\ f_{\mathrm{T}}/f_{\mathrm{L}} & f_{\mathrm{L}}< f_{\mathrm{T}} \end{cases}$$

f_{L} (Hz): natural frequency for first mode in across-wind direction

f_{T} (Hz): natural frequency for first mode in torsional direction

f_1 (Hz): the smaller of f_{L} and f_{T}

A6.8.4 Combination of horizontal wind loads and roof wind load

Combination of horizontal wind loads defined in A6.8.2 or A6.8.3 and roof load calculated from 6.3 shall be considered together.

A6.9 Mode Shape Correction Factor

A6.9.1 Scope of application

This section defines the procedures for mode shape correction to adjust the horizontal wind loads on structural frames calculated by a linear mode shape to the true mode shape.

A6.9.2 Procedure

The mode shape correction factors ϕ_{D} for along-wind wind load, ϕ_{L} for across-wind wind load and ϕ_{T} for torsional wind load are calculated from Eqs.(A6.32), (A6.33) and (A6.34), respectively, approximating each first mode shape by Eq.(A6.31).

$$\mu = \left(\frac{Z}{H}\right)^{\beta} \tag{A6.31}$$

$$\phi_{\mathrm{D}} = \begin{cases} \dfrac{1}{2+\beta}\dfrac{M}{M_{\mathrm{D}}}\lambda & \text{conventional building} \\ \dfrac{M}{5M_{\mathrm{D}}}\left\{\left(0.5\dfrac{B_{\mathrm{H}}}{B_0}-0.3\right)(\beta-2)+1.4\right\}\lambda & \text{lattice structure} \end{cases} \tag{A6.32}$$

$$\phi_{\mathrm{L}} = \frac{M}{3M_{\mathrm{L}}}\left(\frac{Z}{H}\right)^{\beta-1}\lambda \tag{A6.33}$$

$$\phi_{\mathrm{T}} = \frac{M\left(B^2+D^2\right)}{36I_{\mathrm{T}}}\left(\frac{Z}{H}\right)^{\beta-1}\lambda \tag{A6.34}$$

where

$\lambda = 1-0.4\ln\beta$

μ : first mode shape in each direction

B (m): projected breadth of building

B_0 (m): projected breadth at base of lattice structure

B_{H} (m): projected breadth at top of lattice structure

D (m): depth

I_{T} (kgm^2): generalized inertial moment for torsional vibration

M (kg): total mass of building above grand

M_{D} (kg): generalized mass of building for along-wind vibration

M_{L} (kg): generalized mass of building for across-wind vibration

A6.10 Response Acceleration

A6.10.1 Scope of application

This section defines the maximum along-wind response acceleration for ordinary buildings, the maximum across-wind response acceleration for buildings with rectangular plan satisfying the conditions of A6.4.1, and the maximum torsional response acceleration for buildings with rectangular plan satisfying the conditions of A6.5.1.

A6.10.2 The maximum response acceleration in along-wind direction

Maximum response acceleration in along-wind direction at the top of a building is calculated from Eq.(A6.36).

$$a_{\mathrm{Dmax}} = \frac{q_{\mathrm{H}} g_{\mathrm{aD}} B H C_{\mathrm{H}} C'_{\mathrm{g}} \lambda \sqrt{R_{\mathrm{D}}}}{M_{\mathrm{D}}} \qquad \text{(A6.36)}$$

where

$$g_{\mathrm{aD}} = \sqrt{2\ln(600 f_{\mathrm{D}}) + 1.2}$$

where

a_{Dmax} (m/s^2): maximum response acceleration in along-wind direction at top of building

q_{H} (N/m^2): velocity pressure as defined in A6.1.1

B (m): projected breadth of building

H (m): reference height as defined in 6.1.2(4)

C_{H} : value of wind force coefficient C_{D} at reference height as defined in A6.2

C'_{g} : rms overturning moment coefficient as defined in A6.3

λ : mode correction factor of general wind force calculated from Eq.(A6.35)

R_{D} : resonance factor as defined in A6.3

M_{D} (kg): generalized mass of building for along-wind vibration

f_{D} (Hz): natural frequency for first mode in along-wind direction

A6.10.3 Maximum response acceleration in across-wind direction

The maximum response acceleration in the across-wind direction at the top of a building is calculated from Eq.(A6.37).

$$a_{\mathrm{Lmax}} = \frac{q_{\mathrm{H}} g_{\mathrm{aL}} B H C'_{\mathrm{L}} \lambda \sqrt{R_{\mathrm{L}}}}{M_{\mathrm{L}}} \qquad \text{(A6.37)}$$

where

$$g_{\mathrm{aL}} = \sqrt{2\ln(600 f_{\mathrm{L}}) + 1.2}$$

where

a_{Lmax} (m/s^2): maximum response acceleration in across-wind direction at top of building

q_{H} (N/m^2): velocity pressure as defined in A6.1.1

B (m): projected breadth of building

H (m): reference height as defined in 6.1.2(4)

C'_{L} : rms overturning moment coefficient as defined in A6.4

λ : mode correction factor of general wind force calculated from Eq.(A6.35)

R_L : resonance factor as defined in A6.4

M_L (kg): generalized mass of building for across-wind vibration

f_L (Hz): natural frequency for first mode in across-wind direction

A6.10.4 Maximum torsional response acceleration

The maximum torsional response acceleration at the top of a building is calculated from Eq.(A6.38).

$$a_{\mathrm{Tmax}} = \frac{0.6 q_{\mathrm{H}} g_{\mathrm{aT}} B^2 H C'_{\mathrm{T}} \lambda \sqrt{R_{\mathrm{T}}}}{I_{\mathrm{T}}} \tag{A6.38}$$

where

$$g_{\mathrm{aT}} = \sqrt{2\ln(600 f_{\mathrm{T}}) + 1.2}$$

where

a_{Tmax} (rad/s^2): maximum torsional response acceleration at top of building

q_H (N/m^2): velocity pressure as defined in A6.1.1

B (m): projected breadth of building

H (m): reference height as defined in 6.1.2(4)

C'_T : rms torsional moment coefficient as defined in A6.5

λ : mode correction factor of general wind force calculated from Eq.(A6.35)

R_T : resonance factor as defined in A6.5

I_T (kgm^2): generalized inertia moment of building for torsional vibration

f_T (Hz): natural frequency of first mode in torsional direction

A6.11 Simplified Procedure

A6.11.1 Scope of application

This section defines the estimation of wind loads by a simplified procedure for small buildings. This procedure can be applied to buildings that satisfy the following conditions.

i) $H \le 15$ m

ii) $H/2 \le B \le 30$ m

where

H (m): reference height defined in 6.1.2(4)

B (m): projected breadth

A6.11.2 Procedure

(1) Wind loads on structural frames

Horizontal wind loads and roof wind loads on structural frames are calculated from Eq.(A6.39).

$$W_{\mathrm{Sf}} = 0.4 U_0^2 H^{0.4} C_{\mathrm{e}} C_{\mathrm{f}} A \tag{A6.39}$$

where

W_{Sf} (N): wind loads on structural frames

U_0 (m/s): basic wind speed defined in A6.1.2

H (m): reference height defined in 6.1.2(4), not less than 10m.

C_e : exposure factor, which is generally 1.0 and shall be 1.4 for open terrain with few obstructions (Category II). When wind speed is expected to increase due to local topography, this factor shall be increased accordingly.

C_f : wind force coefficient. For horizontal wind loads, the wind force coefficient C_D defined in A6.2 with $k_Z = 0.9$ shall be used. For roof wind loads, the wind force coefficient C_R defined in A6.2 shall be used.

A (m^2): subject area

(2) Wind loads on components/cladding

Wind loads on components/cladding of buildings are calculated from Eq.(A6.40).

$$W_{SC} = 0.15 U_0^2 H^{0.4} C_e \hat{C}_C A_C \tag{A6.40}$$

where

W_{SC} (N): wind loads on components/cladding

C_e : exposure factor, which is generally 1.0 and shall be 1.4 for open terrain with few obstruction (Category II). When wind speed is expected to increase due to local topography, this factor shall be increased accordingly.

$\hat{C}_C$: peak wind force coefficient defined in A6.2. When calculating $\hat{C}_C$, the value of I_Z or I_H shall be 0.26.

A_C (m^2): subject area of components/cladding

A6.12 Effects of Neighboring Tall Buildings

Effects of mutual interference by neighboring buildings and structures shall be considered for estimation of design wind loads on buildings and claddings, when the effects may increase the wind loads.

A6.13 1-Year-Recurrence Wind Speed

1-year-recurrence wind speed U_{1H} (m/s) is calculated from Eq.(A6.41).

$$U_{1H} = U_1 E_H \tag{A6.41}$$

where

U_1 (m/s): 1-year-recurrence 10-minute mean wind speed at 10m above ground over a flat and open terrain, defined in Fig.A6.5

E_H : wind speed profile factor at reference height H, defined in A6.1.5 according to the flat terrain category of the construction site

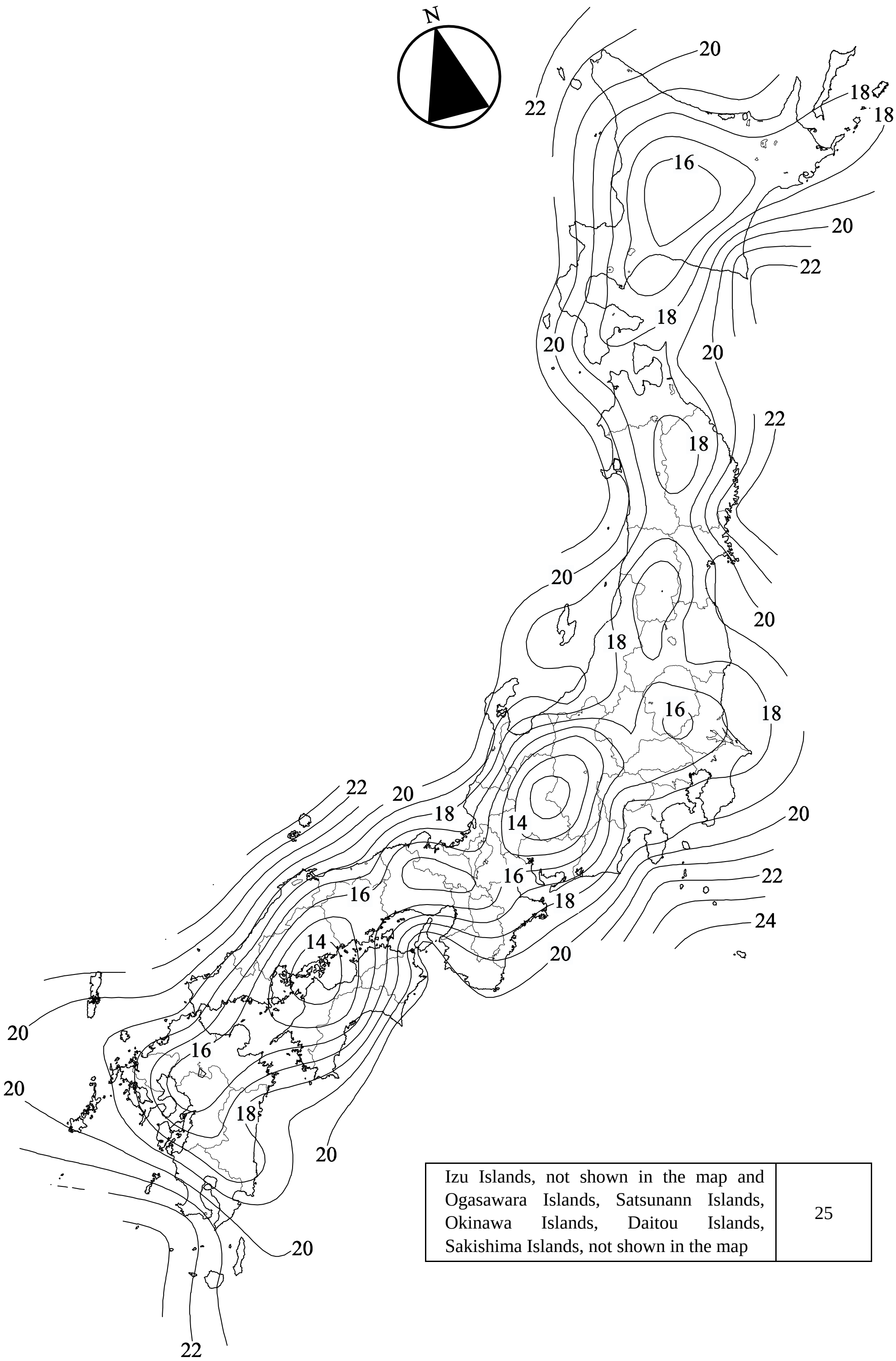


Figure A6.5 1-year-recurrence 10-minute mean wind speed at 10m above ground over a flat and open terrain U_1 (m/s)

7 SEISMIC LOADS

7.1 Estimation of Seismic Loads

7.1.1 Seismic load and design earthquake motion

(1) For ordinary buildings, seismic load is evaluated using the acceleration response spectrum (see Sec.7.2) and the equivalent static force procedure is applied.

(2) For buildings that are irregular in plan or elevation, design earthquake motion is represented by acceleration time history (see Sec.7.3) and time history dynamic analysis is applied. In this case it is recommended that the equivalent static force procedure is also applied.

7.1.2 Idealization of building and location of input ground motion

Prior to the estimation of seismic loads or design earthquake motions, the building and its foundation (including basement) should be appropriately idealized. The seismic load and design earthquake motion are then estimated for the idealized model.

(1) In the case of a regular shaped building with rigid floors, the building is idealized as a multi-degree-of-freedom system with the mass lumped at the floor levels, considering the effects of sway and rocking of foundation. In this case the input ground motion is assumed to be transmitted at the bottom of the foundation.

(2) When the building is resting on hard soil, the building can be idealized as being fixed at the foundation. In this case the seismic input motion is assumed to be transmitted at the ground level. It is also recommended to consider the effects of sway and rocking of foundation.

(3) For irregular shaped buildings or those where torsional vibration is expected to be dominant, the building is idealized as a three dimensional multi-degree-of-freedom system. The seismic input motion is assumed to be transmitted at the base of the foundation or ground level according to the idealized model.

(4) In the case where the soil layers above the engineering bedrock are idealized, the assumed location of seismic input motion is at the surface of the engineering bedrock.

7.2 Calculation of Seismic Loads

7.2.1 Methods for calculating seismic load

(1) Procedure with eigenvalue analysis

The seismic shear V_{Ei} of the i-th story of a building is estimated as follows:

$$V_{Ei} = k_{Di}\, k_{Fi} \sqrt{\sum_{j=1}^{j_c} V_{ij}{}^2} \tag{7.1}$$

where k_{Di} : reduction factor related to ductility of the building (see Sec.7.2.3),

k_{Fi} : amplification factor due to the irregularities of the building (see Sec.7.2.4),

j_c : maximum number of natural modes considered,

V_{ij} : seismic shear of the i-th story for the j-th natural mode.

V_{ij} is calculated as follows:

$$V_{ij} = \frac{S_{\mathrm{a}}(T_j, \zeta_j)}{g} \sum_{k=i}^{n} (w_k\, \beta_j\, \phi_{kj}) \tag{7.2}$$

where
- $S_{\mathrm{a}}(T_j, \zeta_j)$: acceleration response for the natural period T_j and damping ratio ζ_j,
- g : acceleration due to gravity,
- n : total number of stories of the building,
- w_k : weight of the k-th story,
- β_j : participation factor of the j-th natural mode,
- ϕ_{kj} : j-th natural mode shape of the k-th story.

where the j-th natural mode of the k-th story ϕ_{kj} and the j-th natural period T_j and participation factor β_j are obtained through eigenvalue analysis of the idealized model. w_k is calculated as the sum of the dead load and applicable portion of live load (snow load should be considered in heavy snow districts). The live load Q_{E} for calculating seismic load is evaluated from the actual conditions of the building, including fixed objects, etc., or from the values given in Table 7.1.

Table 7.1 Live load for seismic load Q_{E}

Room type†	①	②	③	④	⑤	⑥	⑦	⑧
Q_{E} (N/m^2)	300	200	700	900	1,100	900	2,200	300

† Room types are given in Table 4.1.

In the case where the natural periods for different modes are expected to be close to each other, instead of Eq.(7.1), $V_{\mathrm{E}i}$ is calculated as follows:

$$V_{\mathrm{E}i} = k_{\mathrm{D}i}\, k_{\mathrm{F}i} \sqrt{\sum_{j=1}^{j_c} \sum_{k=1}^{j_c} V_{ij}\, \rho_{jk}\, V_{ik}} \tag{7.3}$$

where ρ_{jk} is the correlation factor for the j-th and k-th natural modes, given by:

$$\rho_{jk} = \frac{8\sqrt{\zeta_j\, \zeta_k}(\zeta_j + r_{jk}\, \zeta_k) r_{jk}^{3/2}}{(1 - r_{jk})^2 + 4\, \zeta_j\, \zeta_k\, r_{jk}(1 + r_{jk}^2) + 4(\zeta_j^2 + \zeta_k^2) r_{jk}^2} \tag{7.4}$$

where r_{jk} is the natural frequency ratio of the j-th and k-th modes ($r_{jk} = \omega_j/\omega_k = T_k/T_j$), ζ_j and ζ_k are the j-th and k-th mode damping ratios.

(2) Procedure without eigenvalue analysis

For ordinary buildings that are regular in plan and elevation, the seismic shear can be estimated without eigenvalue analysis as follows:

$$V_{\mathrm{E}i} = k_{\mathrm{D}i}\, k_{\mathrm{F}i}\, k_{\mathrm{V}i} \frac{\mu_{\mathrm{m}}\, S_{\mathrm{a}}(T_1, \zeta_1)}{g} \sum_{k=i}^{n} w_k \tag{7.5}$$

where $k_{\mathrm{V}i}$: seismic shear distribution factor for the ***i***-th story,
μ_{m} : adjustment factor for single-degree-of-freedom assumption of multi-degree-of-freedom system,
T_1 : fundamental natural period of the building,
ζ_1 : damping ratio for the fundamental mode of the building.

$k_{\mathrm{V}i}$ can be calculated as follows:

$$k_{\mathrm{V}i} = 1 + k_1(1 - \alpha_i) + k_2\Big(\frac{1}{\sqrt{\alpha_i}} - 1\Big) \tag{7.6}$$

where the normalized weight α_i is given by:

$$\alpha_i = \frac{\sum_{k=i}^{n} w_k}{\sum_{k=1}^{n} w_k} \tag{7.7}$$

The factors k_1 and k_2 are determined depending on the number of stories (or height or fundamental natural period T_1),

- for low-rise buildings, $k_1 \approx 1$ and $k_2 \approx 0$,
- for high-rise buildings, $k_1 \approx 0$ and $k_2 \approx 1$,
- for other buildings, k_1 and k_2 have intermediate values.

T_1 is estimated from the height, dimension, type of structural system, etc., and ζ_1 is estimated appropriately. The factor μ_{m} is given as:

- 0.82 for T_1 shorter than the predominant period of the ground,
- 0.90 for longer periods,
- 1.0 for single-degree-of freedom system.

(3) Consideration of soil-structure interaction

In soft soil when it is not appropriate to assume that the ground is rigid, eigenvalue analysis is performed for the idealized model considering sway and rocking motions.

Even if eigenvalue analysis is not performed, the fundamental natural period T_1 and the damping ratio ζ_1 of the building are calculated, considering soil-structure interaction, as follows:

$$T_1 = \sqrt{T_{\mathrm{f}}^2 + T_{\mathrm{s}}^2 + T_{\mathrm{r}}^2} \tag{7.8}$$

where T_{f} : fundamental natural period of the building fixed at the base,
T_{s} : natural period of sway assuming the building is rigid,
T_{r} : natural period of rocking assuming the building is rigid.

$$\zeta_1 = \zeta_{\mathrm{f}}\Big(\frac{T_{\mathrm{f}}}{T_1}\Big)^3 + \zeta_{\mathrm{s}}\Big(\frac{T_{\mathrm{s}}}{T_1}\Big)^3 + \zeta_{\mathrm{r}}\Big(\frac{T_{\mathrm{r}}}{T_1}\Big)^3 \tag{7.9}$$

where ζ_{f} : damping ratio for the fundamental mode of the building fixed at the base,
ζ_{s} : damping ratio of sway assuming the building is rigid,
ζ_{r} : damping ratio of rocking assuming the building is rigid.

7.2.2 Acceleration response spectrum

The acceleration response spectrum $S_a(T_j, \zeta_j)$ at the ground level or at the base of foundation to evaluate seismic load is given by:

$$S_a(T_j, \zeta_j) = k_p(T_j, \zeta_j)\, \sigma_a(T_j, \zeta_j) \tag{7.10}$$

where $k_p(T_j, \zeta_j)$: peak factor of the acceleration response of a single-degree-of-freedom system,
$\sigma_a(T_j, \zeta_j)$: root mean square of the acceleration response of a single-degree-of-freedom system.

$k_p(T_j, \zeta_j)$ is usually assumed to be 3.0, but may also be calculated by appropriate methods.

$\sigma_a(T_j, \zeta_j)$ is calculated as follows:

$$\sigma_a^2(T_j, \zeta_j) = \int_0^\infty \left|H_j(\omega)\right|^2 G_a(\omega)\, d\omega \tag{7.11}$$

$H_j(\omega)$ is the transfer function of acceleration, and is calculated as follows:

$$\left|H_j(\omega)\right|^2 = \frac{(\omega_j^2)^2 + (2\,\zeta_j\,\omega_j\,\omega)^2}{(\omega_j^2 - \omega^2)^2 + (2\,\zeta_j\,\omega_j\,\omega)^2} \tag{7.12}$$

where $\omega_j = 2\pi/T_j$.

$G_a(\omega)$ is the acceleration power spectrum at the ground level or at the base of foundation, and is calculated as follows:

$$G_a(\omega) = \left|H_{GS}(\omega)\right|^2 \left|H_{SSI}(\omega)\right|^2 G_{a0}(\omega) \tag{7.13}$$

where $H_{GS}(\omega)$: soil amplification function to represent the amplification characteristcs of surface soil above the engineering bedrock,
$H_{SSI}(\omega)$: adjustment function to represent soil-structure interaction,
$G_{a0}(\omega)$: acceleration power spectrum at the engineering bedrock.

Instead of Eq.(7.10), the $S_a(T_j, \zeta_j)$ given in the previous recommendations (1993) may also be used.

(1) Acceleration power spectrum at the engineering bedrock

The acceleration power spectrum $G_{a0}(\omega)$ at the engineering bedrock is evaluated, converting the acceleration response spectrum $S_{a0}(T, 0.05)$ at the engineering bedrock given by:

$$S_{a0}(T, 0.05) = k_{rE}\, a_0\, S_0(T, 0.05) = \begin{cases} k_{rE}\, a_0 \left(1 + (k_{R0} - 1)T/T_c'\right) & : \quad (T < T_c') \\ k_{rE}\, a_0\, k_{R0} & : \quad (T_c' \leqq T < T_c) \\ k_{rE}\, a_0\, k_{R0} T_c/T & : \quad (T_c \leqq T) \end{cases} \tag{7.14}$$

where k_{rE} : return period conversion factor given in Sec.7.2.2(5),
a_0 : basic peak acceleration at the engineering bedrock,
$S_0(T, 0.05)$: normalized acceleration response spectrum at the engineering bedrock,
T_c', T_c : corner periods related to the predominant period of the engineering bedrock, $T_c = 0.3$~0.5 s, $T_c'/T_c = 0.2$~0.5,
k_{R0} : acceleration response ratio, approximately 2~3, where the acceleration response becomes approximately constant.

The duration for spectral conversion is assumed to be 20s.

(2) Soil amplification function

The amplification of seismic motion from the engineering bedrock to the ground level or to the base of foundation is evaluated using the transfer function obtained from shear wave propagation analysis or by the simplified formula of soil amplification function $H_{GS}(\omega)$ given by:

$$|H_{GS}(\omega)|^2 = \left|\frac{1}{\cos A + i\,\alpha_G \sin A}\right|^2 \tag{7.15}$$

$$A = \frac{\omega\, T_G}{4\sqrt{1 + 2\,i\,\zeta_G}} \tag{7.16}$$

where α_{G} : impedance ratio from the engineering bedrock to surface soil,

T_{G} : predominant period (s) of soil above the engineering bedrock,

ζ_{G} : damping ratio of soil above the engineering bedrock,

i : imaginary unit ($i = \sqrt{-1}$).

(3) Adjustment function of soil-structure interaction

The adjustment function to represent kinematic soil-structure interaction is evaluated considering the embedment effects of basement, or calculated as follows:

$$\left|H_{\mathrm{SSI}}(\omega)\right|^2 = \begin{cases} \dfrac{1}{1 + 2\,\eta\,\delta_{\mathrm{d}}^2} & : \quad \delta_{\mathrm{d}} \leqq 1 \\ \dfrac{1}{1 + 2\,\eta} & : \quad \delta_{\mathrm{d}} > 1 \end{cases} \tag{7.17}$$

where η : ratio between embedment depth of foundation d and equivalent width l

($l = \sqrt{A_{\mathrm{f}}}$, A_{f} : area of foundation),

δ_{d} : normalized depth of foundation embedment $\left(\dfrac{2\omega d}{\pi V_{\mathrm{S}}}\right)$,

V_{S} : shear wave velocity of the soil adjacent to the basement.

(4) Basic peak acceleration

The basic peak acceleration a_0 at the surface of the engineering bedrock is given by the standard hazard map of Fig.7.1. It may also be evaluated by appropriate seismic hazard analysis.

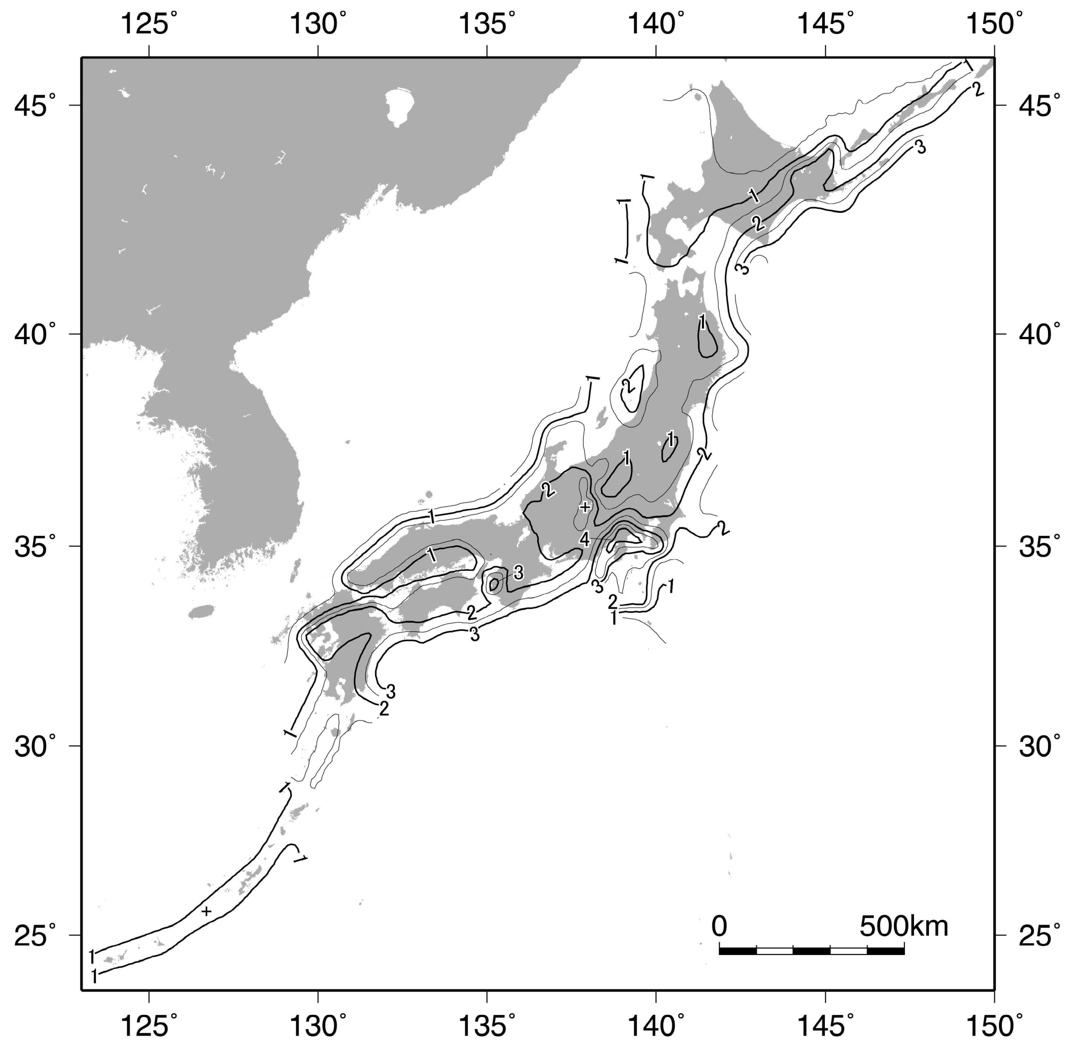


Fig.7.1 Basic peak acceleration a_0 (m/s^2)
at the surface of the engineering bedrock for a return period of 100 years.
"+" indicates that the value in that area is greater than that of the adjacent area.

(5) Return period conversion factor

The return period conversion factor k_{rE} of peak acceleration of seismic motion is determined from hazard analysis at the construction site.

7.2.3 Reduction factor related to ductility and response deformation

The reduction factor k_{Di} related to the ductility of the i-th story is calculated according to the inelastic characteristics and limit deformation of the building. The deformation of each story of the building caused by equivalent static force is modified using the reduction factor related to ductility. It is verified that the global safety and member damage of the building are within the allowable limit.

7.2.4 Amplification factor due to structural irregularities of the building

The amplification factor k_{Fi} of the i-th story of a building with irregularities in plan or elevation is calculated considering the possibility of damage concentration to a certain story, torsional vibration, etc.

7.3 Design Earthquake Motions

7.3.1 Fundamental concept for generating design earthquake motions

For a building whose dynamic behavior should be precisely captured, such as a very important building, the design earthquake motions are generated according to this section and the dynamic (time history) analysis is carried out for the appropriate model taking into account the soil-structure interaction and dynamic properties of the building.

The procedures to generate the design earthquake motions are classified into two types as follows:

- generation of design earthquake motions that are compatible with the design response spectrum,
- generation of design earthquake motions based on scenario earthquakes considering local site conditions and design requirements of the building.

7.3.2 Design earthquake motions compatible with the design response spectrum

The design earthquake motions are generated as simulated earthquake motions (time history) that are compatible with the design response spectrum described in Sec.7.2 as a target response spectrum.

The target response spectrum is defined either

- at the free surface of the engineering bedrock, or
- at the location of input motion to the idealized model of the building.

In the latter case, the target response spectrum is evaluated considering the soil conditions between the engineering bedrock and the location of input motion.

7.3.3 Design earthquake motions based on the scenario earthquakes

The engineering bedrock and the service life are selected according to the local site conditions and requirements of the building. Then scenario earthquakes are established considering the earthquake patterns. For the senario earthquakes, earthquake motions at the site are evaluated and the design earthquake motions are generated considering the source, path, soil conditions, etc.

(1) Scenario earthquakes

Considering the site conditions (seismicity, geography, geology, etc.), the engineering bedrock is selected and the service life is determined based on the importance and properties of the building (building use, structural characteristics, function, etc.). From these, earthquake patterns (e.g. inter-plate, intra-plate earthquakes or crustal shallow earthquakes) considered for the design are evaluated and the scenario earthquakes are established. The parameters of the scenario earthquakes are determined considering the locality and characteristics of the earthquakes.

(2) Evaluation of earthquake motions

Earthquake motions at the construction site are evaluated considering the characteristics of the source, the path, the local soil conditions, etc. for the scenario earthquakes.

Appropriate methods are adopted to evaluate earthquake motions according to the quality and the quantity of information on the conditions of the construction site and of the building.

(3) Generation of design earthquake motions

Earthquake motions at the construction site are evaluated considering the structural properties and design requirements of the building, etc., and the design earthquake motions are finally determined at the engineering bedrock at the construction site.

8 THERMAL LOADS

8.1 Scope of Applications

(1) This chapter describes thermal loads for the design of buildings.

(2) When a building is subject to temperature change due to the site layout, the time, the size of the building, the use of the building, or the service conditions, thermal load needs to be considered.

(3) When the thermal stress or deflection in structural members is reduced by having expansion joints or some other effective measure, thermal load may not be considered.

8.2 Thermal Load

The basic thermal load is the 100-year-return period of the change in outdoor air temperature, solar radiation, underground temperature or equivalent value.

CHAPTER 9 EARTH PRESSURE AND HYDRAULIC PRESSURE

9.1 Overview

(1) Earth pressure and hydraulic pressure acting against structures such as exterior basement walls and retaining walls, which touch the soil directly, shall be considered. The buoyancy shall also be considered for the structures below groundwater level.

(2) The earth pressure that permanently acts on exterior basement walls, etc. shall be assumed to be the earth pressure at rest in general, and the influence the surcharge on the ground surface shall be appropriately considered when it exists.

(3) The earth pressure that permanently acts on retaining walls shall be assumed to be the active earth pressure in general, and its influence shall be appropriately considered when there is surcharge on the ground surface.

(4) When the earth pressure and hydraulic pressure increase remarkably due to an earthquake, such an increase shall be taken into account appropriately.

(5) When calculating the earth pressure that acts permanently, the value of 99% non-exceedance probability considering the accuracy of the calculation method and the variation of geotechnical parameters shall be assumed to be the characteristic value of the load. The characteristic value of hydraulic pressure shall be assumed to be the highest free water level with the 100-year return period, and the influence of time variance shall also be considered.

9.2 Earth Pressure and Hydraulic Pressure that Act on Exterior Basement Walls

The earth pressure and hydraulic pressure that permanently act on exterior basement walls shall be calculated according to Eq. (9.1) for above groundwater level, and shall be calculated according to Eq. (9.2) for below groundwater level:

$$p = K_0 \gamma_t z + \Delta p_0 \tag{9.1}$$

$$p = K_0 \gamma_t h + K_0 \gamma' (z - h) + \gamma_w (z - h) + \Delta p_0 \tag{9.2}$$

where p : earth pressure and hydraulic pressure for each unit area in depth z (kN/m^2),

γ_t : total unit weight of soil (kN/m^3)

γ' : submerged unit weight of soil (kN/m^3)

γ_w : unit weight of water (= 9.8 kN/m^3)

K_0 : earth pressure coefficient at rest

h : groundwater depth (m)

Δp_0 : earth pressure increment for surcharge on the ground surface (kN/m^2)

9.3 Soil Pressure that Permanently Acts on Retaining Walls

The earth pressure that acts on retaining walls shall be generally assumed to be an active earth pressure. When there is no surcharge on the ground surface, the active earth pressure is calculated as follows:

$$p_A = K_A \gamma_t z - 2c\sqrt{K_A} \tag{9.3}$$

where P_A : active earth pressure for each unit area in depth z (kN/m^2)

K_A : active earth pressure coefficient

γ_t : total unit weight of soil (kN/m^3) (submerged unit weight of soil below the water table)

c : cohesion of soil (kN/m^2)

However, when P_A becomes negative, it is assumed to be zero. The active earth pressure coefficient, K_A, can be calculated by the following equation:

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta\cos(\theta+\delta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\cos(\phi+\delta)\cos(\theta-\alpha)}}\right\}^2} \tag{9.4}$$

where ϕ : angle of internal friction of backfill soil (deg)

θ : angle between backfill soil and the vertical plane

δ : angle of wall friction

α : angle between ground surface and horizontal plane

Here, it is assumed that $\sin(\phi-\alpha)=0$ at $\phi-\alpha\le 0$. When it is necessary to consider hydraulic pressure, the influence should be taken into account. The influence the surcharge on the ground surface shall be appropriately considered when it exists.

9.4 Earth Pressure During Earthquakes that Acts on Retaining Walls

The active earth pressure acting on retaining walls shall be calculated on the basis of reliable ground investigations and proper analyses, or using either of the following methods.

(1) Active earth pressure at earthquake proposed by Monobe and Okabe

$$p_{EA} = K_{EA}\gamma_t z \tag{9.5}$$

where P_{EA} : active earth pressure at the earthquake for each unit area in depth z (kN/m^2),

K_{EA} : earth pressure coefficient at the earthquake of Monobe and Okabe

γ_t : total unit weight of soil (kN/m^3) (submerged unit weight of soil below the water table)

z : depth (m) where active earth pressure P_{EA} acts on the wall

$$K_{EA} = \frac{\cos^2(\phi-\theta-\theta_k)}{\cos\theta_k\cos^2\theta\cos(\theta_k+\theta+\delta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\alpha-\theta_k)}{\cos(\phi+\delta+\theta_k)\cos(\theta-\alpha)}}\right\}^2} \tag{9.6}$$

where ϕ : angle of internal friction of backfill soil (deg)

θ : angle between backfill soil and the vertical plane

δ : angle of wall friction

α : angle between ground surface and horizontal plane

θ_k : earthquake synthesis angle $\theta_k = \tan^{-1} k_h$ (deg)

k_h : design horizontal seismic intensity

Here, it is assumed that $\sin(\phi-\alpha-\theta_k)=0$ at $\phi-\alpha-\theta_k\le 0$. When it is necessary to consider hydraulic pressure, the influence should be taken into account. The influence the surcharge on the ground surface shall be appropriately considered when it exists

(2) Trial wedge method

a) When the cohesion in the Monobe and the Okabe expression is not considered

$$P_{EA} = \frac{\cos^2(\psi - \phi + \theta_k) \cdot W}{\cos(\psi - \phi - \delta - \theta) \cdot \cos\theta_k} \qquad (9.8)$$

b) When the cohesion in the Monobe and the Okabe expression is considered

$$P_{EA} = \frac{W \cdot \sec\theta_k \sin(\psi - \phi + \theta_k) - c \cdot l \cos\phi}{\cos(\psi - \phi - \delta - \theta)} \qquad (9.8)$$

where P_{EA} : resultant force of active earth pressure at the earthquake (kN/m)

W : weight of the soil wedge (kN/m)

ψ : active slip angle at the earthquake (deg)

δ : angle of wall friction

c : cohesion in the Monobe and the Okabe expression (kN/m^2)

l : length of active slip surface (m)

9.5 Groundwater Level for Design

(1) The groundwater level for the design shall be determined as the highest water level with the 100-year return period considering the free water level based on continuity of soil strata, etc.

(2) Buoyancy ${}_vP_w$ that acts on the bottom of foundation shall be calculated by the following equation:

$$ {}_vP_w = \gamma_w(z - h) \qquad (9.9)$$

where γ_w : unit weight of water (kN/m^3)

z : depth (m) to the position at which buoyancy is required from the ground surface level

h : depth (m) from the ground surface level to the water table

9.6 Uncertainties of Earth Pressure and the Geotechnical Parameters for Design

(1) The earth pressure (active earth pressure, earth pressure at rest, and earth pressure during earthquakes, etc.) that acts on exterior basement walls and retaining walls shall be determined taking into consideration the influence of the uncertainties of the parameters used in the calculations.

(2) The value of 99% non-exceedance probability shall be adopted for the characteristic value of the geotechnical parameter used for the earth pressure calculation.