No. 43 December 2014

STEEL CONSTRUCTION TODAY & TOMORROW

http://www.jisf.or.jp/en/activity/sctt/index.html

風

【 *k a z e* 】

"風 (*kaze*)" in Japanese, or wind in English

Serious building damage, particularly to exterior members, that is caused by the strong winds of typhoons and tornadoes occur worldwide.
How to deal wind load and how to mitigate strong wind-induced damage are discussed in this issue, No. 43.

In this issue.....

Special Feature: Wind Resistance of Buildings

Photo: SUZUKI hisao

Published Jointly by

The Japan Iron and Steel Federation

JSSC Japanese Society of Steel Construction

Advances in Wind-resistant Design and Wind Resistance Evaluation in Japan

by Yasushi Uematsu, Professor, Tohoku University

Establishment of the Enforcement Order of the Building Standard Law

In Japan, the Enforcement Order of the Building Standard Law of Japan was established in 1950. It was the first legal regulation that specified the wind-load evaluation method to be adopted in building design, which regulated the calculation of wind load P by use of the following equation:

$$P = C \cdot q \cdot A \qquad (1)$$

where

C : wind force coefficient

A : building area or tributary area of component under consideration (m^2)

q : velocity pressure (calculated by using Equation (2))

Meanwhile, the wind force coefficient C refers to the difference between external and internal pressure coefficients. Note that these two coefficients are not specified separately.

$$q = 60\sqrt{h}\ (\mathrm{kg/m^2}) \qquad (2)$$

where

h : height above ground level (m)

The equation is based on a maximum instantaneous wind velocity of 63 m/s that was observed at the top of the steel observation tower (15 m above ground level) of the Muroto Meteorological Observatory during the Muroto Typhoon in 1934. The equation was derived by assuming that the vertical distribution (profile) of the maximum instantaneous wind velocity is proportional to 1/4th the power of the height above ground level, and by substituting the above-mentioned observation value.

Incidentally, it is well accepted that the power exponent for the profile of the maximum instantaneous wind velocity during typhoons and other synoptic winds is about 1/2 the value for the mean wind velocity. Thus, the above-mentioned power exponent of 1/4 implies that the power exponent for the mean wind velocity profile corresponds to about 1/2. In those days, such a large value of power exponent was not used in any country in the world, which in fact did not reflect actual conditions. However in Japan, no so-called high-rise buildings had been constructed, and there were few buildings for which wind loads dictated the structural design, thereby causing no substantial problems. The wind force coefficients C were obtained from wind tunnel tests using uniform smooth flow, in which the effects of turbulence were not considered. Further, the social background that led to the establishment of Equation (2) included the following issues:

• In those days, because the Muroto Typhoon was of unprecedented scale in Japan, it was expected that, if wind loads of that level were adopted, buildings would have some safety margin for future typhoons.

• Unlike earthquakes, typhoons can be forecasted to a certain extent, and thus it is possible to take countermeasures against them. Therefore, it was considered that, even if lower wind loads were adopted for reasons of economic advantage, no serious problems would result.

Increasing Building Height Prompts Reexamination of Wind Load Calculation Method

Following the popularization of TV sets among general households during Japan's period of high economic growth, the nation's first large-scale television tower with a height of 180 m was constructed in Nagoya in June 1954. During the tower's design stage, the inadequacy of Equation (2) was pointed out. As a result, the profile of the maximum instantaneous wind velocity was reexamined in reference to the building codes and standards in foreign countries. This produced the following equation, which assumed a power exponent of 1/8 and was used in the design of the tower:

$$q = 120\sqrt[4]{h}\ (\mathrm{kg/m^2}) \qquad (3)$$

After completion of the Nagoya television tower, full-scale measurements taken during typhoons demonstrated that the measured results agreed relatively well with Equation (3). As a result, Equation (3) has played a great role in the blossoming of subsequent high-rise building construction in Japan.

Following the revision of the Building Standard Law in 1963, Japan's first full-scale high-rise building, the Mitsui-Kasumigaseki Building (36 stories above ground, 156 m in height), was completed in Tokyo, marking the dawn of the high-rise building age in Japan. In addition, the National Indoor Stadium with a 126-m main span was constructed for the Tokyo Olympic Games held in 1964, marking the beginning of large-span buildings in Japan.

As the height or clear span of buildings increases, the natural frequency generally decreases, thereby causing the significant dynamic effect of wind. That is, the contribution of the resonance component (resonance effect) in the dynamic response of buildings becomes more significant. On the other hand, when the scale of a building increases, the net wind load acting on the building decreases due to scale effect. In the case of a small-scale building, the load effect (for example, the stress involved in the structural members) becomes the maximum when the maximum peak wind velocity occurs. In the case of a large-scale building, on the other hand, the wind pressures acting on the structural members do not reach the maximum peak values at the same time for all members, and thus the load effect does not become the maximum at the moment of the maximum peak wind velocity.

Emerging from this background was a gradually increasing understanding of the dynamic load effects on buildings, which led to many surveys and researches on the turbulent structure of wind, wind tunnel test methods, the actual conditions of wind pressure, wind-induced vibration and other factors.

Along with the appearance of high-rise buildings, it was urgently required to establish a reasonable wind-resistant design method for curtain walls, particularly to establish a method for testing the wind resistance of glass plate and the water proof of curtain walls. During that period, Japan was successively struck by super typhoons, such as the Ise-bay Typhoon (1959) and the Second Muroto Typhoon (1961), causing great damage to roofing, exterior walls and other exterior members. The damage to these exterior members frequently triggered severe damage to the main wind resisting systems (structural frames), thereby pointing out the importance of preventing damage to cladding/components and promoting safe design.

In such situations, Notification No. 109, the first regulation concerning the wind-resistant design of exterior members, was issued by the Ministry of Construction in 1971. This regulation focused mainly on the following two topics:

1) Design velocity pressure is sorted into two classes respectively for roofing materials and for external walls, which are calculated by using the following equations:
   - For roofing materials:
     $q = 120\sqrt[4]{h}\,(\mathrm{kg/m^2})$ (4)
   - For external walls of buildings higher than 31 m:
     $q = 60\sqrt{h}\,(\mathrm{kg/m^2})$ for $h \leq 16\,\mathrm{m}$ (5a)
     $q = 120\sqrt[4]{h}\,(\mathrm{kg/m^2})$ for $h > 16\,\mathrm{m}$ (5b)
2) The areas of local wind pressures in eaves, overhanging roofs, verges and wall surface corner sections are specified, where the design wind force coefficient is specified as $C = -1.5$.

Fig. 1 shows the profiles of velocity pressure provided by Equations 5(a) and 5(b). The curves of the two equations cross at a height of 16 m, and the lower value of these two curves at each height is used for evaluating wind loads.

Efforts to Establish a More Rational Method of Evaluating Wind Loads

Because the fluctuation of wind velocity is quite random in nature, the time-space correlation of wind velocity should be considered appropriately based on a statistical and probabilistic approach, when evaluating the wind loads on buildings. Prof. Alan G. Davenport of the University of Western Ontario, Canada proposed in 1967 a new approach, known as the gust loading factor method (Fig. 2). According to this method, the design wind load P is provided by the following equations:

$$P = q \times C \times G \times A \quad (6)$$

$$q = \frac{1}{2}\rho U^2 \quad (7)$$

where

U : mean wind velocity at height z above ground level

G : gust loading factor defined by the following equation:

$$G = \frac{\bar{X} + x_{\max}}{\bar{X}} = 1 + g_x \frac{\sigma_x}{\bar{X}} \quad (8)$$

where

$\bar{X}$: mean displacement of building due to the mean wind force

$x_{\max}$: maximum value of dynamic displacement ($= g_x \cdot \sigma_x$)

σ_x: standard deviation of dynamic displacement

g_x: peak factor.

Comparison of Equations (6) to (8) with the corresponding provision in the Enforcement Order of the Building Standard Law shows the following features:

- When considering the wind-induced vibration of structure, the maximum instantaneous wind velocity does not always give the maximum load effects (i.e., stress, displacement and other loading effects).
- The maximum peak wind force at each point on the building does not occur simultaneously. Therefore, the net wind force acting on the entire building decreases with an increase in the size of building.

In light of such conditions, the gust loading factor method adopts a probabilistic and statistical approach to evaluate the design wind loads on buildings by considering the temporal and spatial fluctuation characteristics of wind velocity. Further, the method provides the "equivalent static wind load" that gives the maximum load effect. Therefore, the commonly used conventional static analysis can be applied in the structural design, nevertheless the wind loading is dynamic.

When comparing Equations (1) and (6) each other, it is found that the design wind velocity is specified based on the maximum instantaneous wind velocity in Equation (1), while it is based on the average wind velocity in Equation (6). Further, the dynamic effect of fluctuating wind velocity on the load effects is taken into consideration by using the maximum instantaneous wind velocity in Equation (1), while it is taken into consideration by using the gust loading factor G that is based on the maximum peak value of the building response.

Fig. 1 Comparison of Design Velocity Pressure

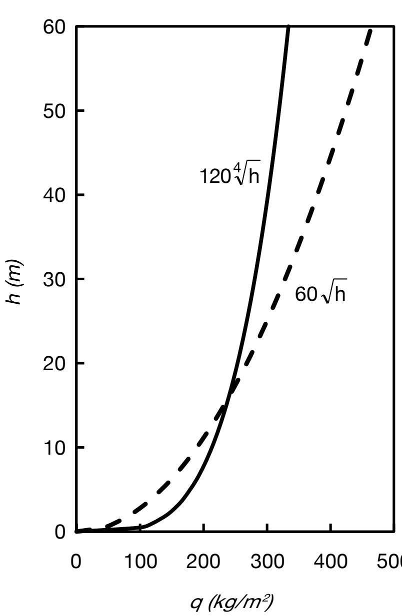
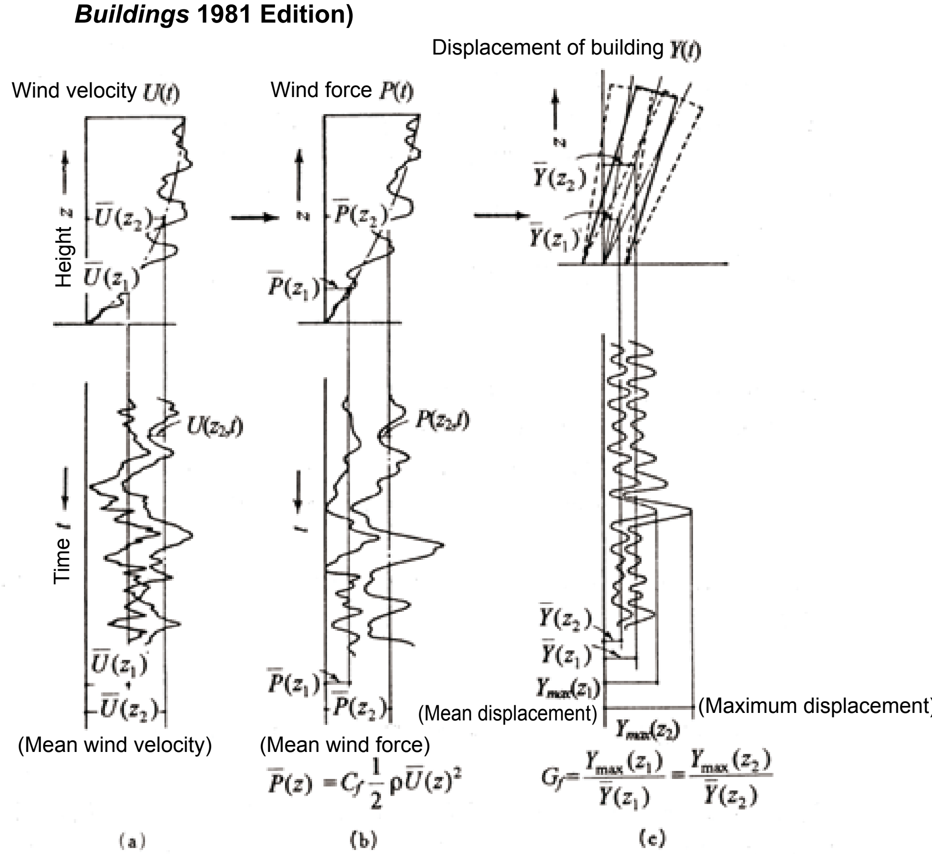


Fig. 2 Definition of Gust Loading Factor (*Recommendations for Loads on Buildings* 1981 Edition)



The building response depends not only on the wind velocity but also on many factors relating to the building; that is, the shape, scale and the dynamic characteristics, such as natural frequency and damping factor. All of these factors are reflected in the equation for G. In the case of small-scale buildings, it is thought that the maximum load effect occurs at the moment of the maximum peak wind velocity. Here, the ratio of the maximum peak wind velocity to the mean value is defined as the gust factor G_v. Because the wind force is proportional to the square of the wind velocity, Equations (1) and (6) imply that $G = G_v^2$.

It does without saying that the wind load evaluation by using Equation (6) is far more rational than the use of Equation (1). As a result, the use of Equation (6) to evaluate wind loads has been incorporated into the provisions of codes and standards in many countries. In Japan, the Architectural Institute of Japan (AIJ) published *Recommendations for Loads on Buildings* in 1981, in which the wind load evaluation was based on the Davenport method.

Then, because several inadequacies were found in the AIJ *Recommendations*, it was revised in 1993 to correct these inadequacies. The features of the revised *Recommendations* are as follows:

- The wind load calculation equation takes two forms: one for the main wind force resisting systems (structural frames) and the other is for cladding/components. This is because the scale and vibration characteristics of the structural frames are quite different from those of the cladding/components, and due considerations are made on the different ways that wind load works on the structural frames as opposed to cladding/components.
- The design wind velocity is settled by taking into account the frequency of the occurrence of strong winds and the safety level of the building during the service life of the building in respective areas. That is, the return period is settled according to the safety level required for the building, and the building is designed based on the wind velocity that corresponds to the return period thus settled. Meanwhile, the design wind velocity is evaluated by using the annual maximum wind velocity.
- The design velocity pressure q_H is to be set as the velocity pressure at a reference height H corresponding to the building (commonly the average height of the roof). Accordingly, the vertical distribution of the wind load is treated as the distribution of the wind force coefficient (or wind pressure coefficient).
- The effect of the time-space correlation of wind pressures on the building is evaluated using the probabilistic and statistical approach, which is expressed as the gust effect factor. While the gust effect factor is similar to the gust loading factor defined by Prof. Davenport, the gust effect factor is applied in a wider context as a factor that expresses the dynamic load effect of wind pressures and wind forces.
- The conditions of wind blowing at the site are classified according to "surface roughness," and the features of these conditions thus classified were reflected in the profiles of mean wind velocity and turbulence intensity.

Then in 2000, the Enforcement Order of the Building Standard Law was fully revised. The Order incorporated a wind load evaluation method that was based on the probabilistic and statistical approach, as in the case of many other countries. Although the method defined in the Order was simplified by imposing several restrictions, such as limiting the applicable building height to 60 m or less, the basic method of wind load evaluation used is almost the same as that provided in the AIJ *Recommendations* (1993).

The AIJ plans to revise the *Recommendations* approximately every 10 years by actively incorporating the latest information. In line with this policy, revisions were made in 2004 and preparations are being promoted to publish the 2015 revised version. In this revision, in order to allow more reasonable evaluation of design wind loads, due consideration is being given to many factors, such as the effect of local topography on wind velocity, the wind direction coefficient and seasonal coefficient, aerodynamic instability, and load combinations. Further, the revision will also employ the use of computational fluid dynamics (CFD) together with wind tunnel experiment.

The appearance of high-rise buildings has posed new and unexpected problems. In 1979, Typhoon No. 20 struck the Tokyo metropolitan area with winds of a strength seen only once every ten years. Also, it brought attention to the issue of wind-induced vibration of high-rise buildings, especially in the Shinjuku new urban center. Building vibration did not cause any serious problems to the structures. However, because of the typhoon's long duration and incessant blowing, a considerable number of people had discomfort and queasiness as they experienced in seasickness.

Triggered by such situations, the issue of habitability of high-rise buildings has received a great deal of attention, which led to the publication of *Guidelines for the Evaluation of Habitability to Building Vibration* in 1991 by AIJ. The *Guidelines* was then revised in 2004 by incorporating the latest available knowledge. In the *Guidelines*, the criterion for evaluating the habitability is given by use of the relationship between the maximum response acceleration for the wind velocity with a 1-year return period and the natural frequency of building.

Evaluation of Wind Resistance

In June 1998, the Building Standard Law was revised. In this revision, the design concept was widely shifted to the "performance-based design" while remaining the conventional concept of specification design. In building design, three limit states are generally assumed—serviceability, damage and safety limits, and the design criteria are given to each of these limit states. For example, the serviceability limit for high-rise buildings is determined by taking wind-induced vibration (habitability) into account.

In this regard, based on the *Guidelines for the Evaluation of Habitability to Building Vibration* mentioned above, the criterion for evaluating habitability is given employing the relation between the maximum response acceleration for wind velocity with a 1-year return period and the natural frequency of buildings. Particularly in the response and limit strength calculation in the Enforcement Order of the Building Standard Law, it is stipulated that, when determining damage limit, structural members should remain in the elastic range when subjected to rare strong winds with an approximately 50-year return period, and that, when determining safety limit, buildings should not collapse even when subjected to extremely rare strong winds with a 500-year return period. However, neither the *Guidelines* nor the Enforcement Order offers a clear prescription for cladding/components suffering great damage due to strong winds. Because of this, the AIJ has provided specific design approaches in its publication *Manual for Cladding Wind Resistance Evaluation for Designers and Engineers* published in 2013. ■

Reference

1) A.G. Davenport, Gust loading factors, Proc. of ASCE, Struc. Div., 1967.

Strong Wind-induced Damage to Buildings and Concepts for Mitigating Such Damage

by Hitomitsu Kikitsu, Building Research Institute

Serious Damage to Buildings Caused by Strong Wind

It is recently reported that serious tornado damage has frequently occurred both in Japan and abroad with great social impact. The scenes are still fresh in our memory of the damage inflicted by tornadoes in several Japanese cities, centering on Tsukuba in Ibaragi Prefecture in 2012 (Photo 1) and Koshigaya in Saitama Prefecture in 2013. Meanwhile, it is also true that damage is caused by typhoons, but this damage is liable to be obscured by the scale of the damage caused by tornadoes (Photos 2~3).

Among a building's various structures, it is the roofing members, exterior walls, openings and other external claddings and components that are vulnerable to the effect of strong wind. The primary measure for preventing wind-induced building damage is to mitigate the damage to them.

Concepts Conducive to Mitigating Damage to External Claddings and Components

• Specific Damage Conditions

When a building is subjected to strong wind of typhoon, the wind force attributable to the turbulence of approach flow generates on the windward roof and wall, and this results in localized peak negative pressure occurring along the edges of roof and sidewall. The Building Standard Law of Japan prescribes the method for calculating localized peak pressure. According to this method, the action of wind gusts generated by a translating tornado can be considered to follow those of a typhoon. However, unlike a typhoon, particularly as the whirling center of a tornado approaches closer to a building, the force produced by the updraft works on the building to increase the damage (refer to Fig. 1), to which due attention should be paid.

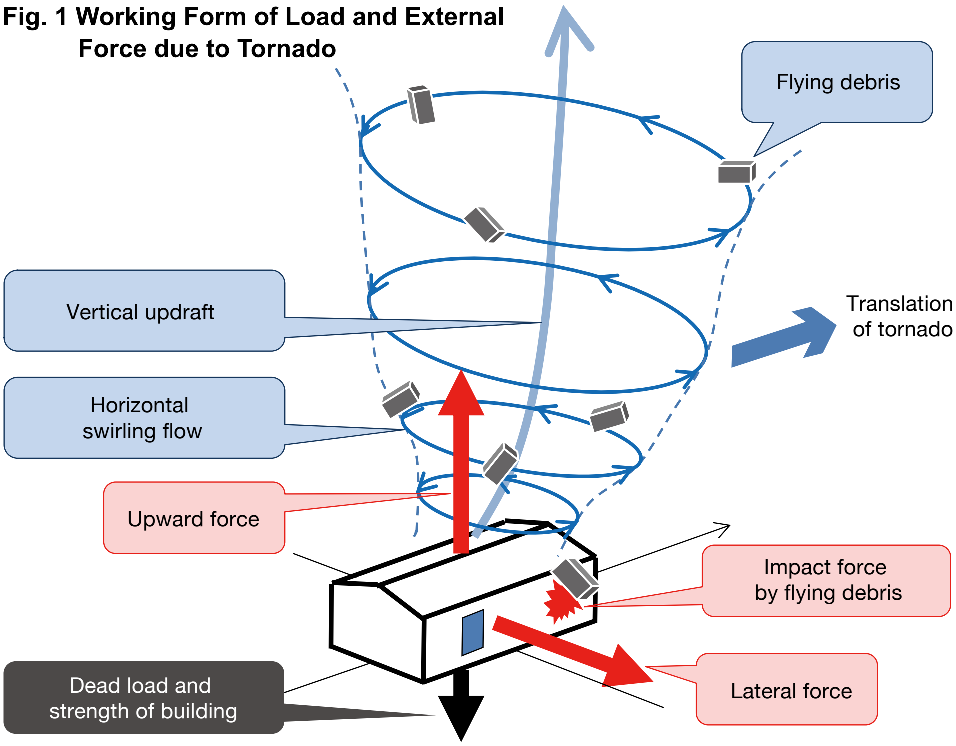
The damage caused by strong wind can be understood as the apparent damage of the most vulnerable section among wind-induced load paths in a building. Most of the damage can be found in the external cladding and components of the building. Accordingly, in order to mitigate such wind damage, it is important to settle the design load after gaining an appropriate understanding of how wind force will work on the building and, then, to give due consideration when selecting the specifications for the external cladding and components.

Photo 1 Example of damage caused by tornado (Tsukuba in 2012)

Photo 2 Example of damage caused by typhoon (Miyakojima, Okinawa in 2003)

Photo 3 Example of damage caused by typhoon (Miyakojima, Okinawa in 2003)

Fig. 1 Working Form of Load and External Force due to Tornado



• Concepts for Verification of the Strength of External Members

When verifying the strength of external cladding and components, two approaches are commonly applied: structural design based on the standard specification of the members and structural design based on their allowable strength.

In the former approach, the wind pressure resistance is secured by selecting standard specifications (distance between supports, plate thickness, etc.) from a product catalog according to the necessary level of the design load, and thus structural calculations are not required and wind pressure resistance can be easily verified.

In the latter approach, the wind pressure resistance is verified by calculating the allowable strength of each structural section based on the strength test results (Photo 4). In this verification process, for example in the case of steel roofing and walls, it is common that a value of 2.0 or higher be settled as the safety factor required to find the allowable strength of such members.

Based on the results of these verification processes, due care is taken in the design work, such as increasing the plate thickness or narrowing the fastening and installation spaces of the supporting members, in order to reduce the possibility of damage occurrence.

Further, there are many cases of production and commercial facilities that are large in scale and that require asset security and the preservation of building functionality. Facilities of this nature can be forecasted to incur several types of wind-induced damage. That is, when roofing and other exterior members are stripped off and scattered about, the resulting inflow of rain can extensively damage indoor equipment and make the entire facility functionally useless. In important facilities that house highly advanced functions, even if the main structures are intact, it is possible that striped-off and scattered roofing and other external claddings and components could cause enormous economic loss.

Related to wind resistance design in the Building Standard Law of Japan, return periods of approximately 50 years are supposed. It can be said that damage caused by strong wind can be mitigated, depending on the importance level of a facility, by assigning incremental wind loads to external members that are based on levels surpassing those in the Law and then verifying the wind pressure resistance of these members.

Currently, the wind force of tornadoes is not taken into account in common wind resistance design, but it is considered that the concepts mentioned above can, to a certain degree, mitigate tornado-induced building damage as well.

• Sharing of Information about Member Applications

Because the performance appraisal of external cladding and components is commonly entrusted to construction companies and the manufacturers of these members, how to assign appropriate roles in any structural verification is liable to be unclear. Therefore, it is imperative that information about the strength and other properties of these external members be adequately shared among the designers, construction companies and member manufacturers throughout the process from design to construction.

Further, studies of recent damage illustrates that the damage is often caused by secular deterioration of structural members and a subsequent loss of their strength and by the adoption of inappropriate repair methods. These examples suggest that appropriate maintenance of external cladding and components and proper repairs are indispensable in mitigating damage to members caused by strong wind. ■

Reference

Japan Metal Roofing Association and Japanese Society of Steel Construction: Standard of Steel Roofing, SSR 2007 (published in 2008)

Photo 4 Example of strength test for the connection of folded roofing (SSR2007)

Wind Load Provisions in the Building Standard Law of Japan

by Yasuo Okuda, National Institute for Land and Infrastructure Management

Introduction

Article (1) Purpose of the Law and System of the Building Standard Law of Japan states, “The objective of this law is to establish minimum standards regarding the site, structure, facilities, and use of buildings in order to protect the life, health, and property of the nation, and thereby to contribute to promoting the public welfare.” As stated, the Law regulates the construction of every type of building in Japan, and provides the minimum standard to be observed in building construction.

In 2000, the Enforcement Order of the Building Standard Law [Law] and its Notifications were widely revised, and the wind load-related provisions in the Enforcement Order and its Notifications were also widely revised based on *Recommendations for Loads on Buildings* (1993) [hereafter, *Recommendations*] issued by the Architectural Institute of Japan. The wind load values adopted with the establishment of the Law in 1950 were uniform throughout the nation and went unchanged for 50 years, but in the revision of the Law in 1998 and of the Enforcement Order and its Notifications in 2000, the wind load values were changed so as to take into account local and ancillary conditions. Further, in 2007, the Enforcement Order of the Building Standard Law was also revised to require the submition of a structural calculation document for exterior members at the time of building confirmation, a step that had formerly been exempted.

Meanwhile, the Architectural Institute of Japan has been revising *Recommendations* nearly every 10 years since it was first issued in 1981, and the latest version is scheduled for publication in February 2015.

In discussing the standards and specifications for wind-resistant design of buildings in Japan, an outline of the wind load specifications prescribed in the Building Standard Law and in *Recommendations* is introduced in this article. Also introduced is an outline of diverse guidelines conforming to the wind load provisions of the Building Standard Law as prepared by the respective industry organizations.

Wind Load Provisions of the Building Standard Law

While *Recommendations* has been revised nearly every 10 years by the Architectural Institute of Japan to reflect the latest advances in research, the wind load provisions of the Building Standard Law have not been so frequently revised since the Law’s establishment in 1950. However, following the revision of the Law in 1998 (introduction of performance-based design in building standards), widely-ranging revisions and newly-established requirements were made to the Law’s Enforcement Order and to related notifications in 2000. As this was happening, the wind load provisions of the Law were also widely revised based on *Recommendations* (1993). As regards the wind load values that were uniformly enforced nationwide following the establishment of the Law in 1950, it has recently become possible to prescribe more rational wind loads that reflect local and ancillary conditions and the structural characteristics of individual buildings. Among the specific approaches to determine more rational loads are:

- Clarification of separate wind loads for structural framing and exterior members
- Introduction of a standard wind velocity Vo
- Introduction of ground roughness classifications
- Introduction of gust loading factors
- Settlement of two load levels of damage criterion and safety criterion in the calculation of response and limit strength
- Adoption of SI units
- Enhancement of wind force coefficients, etc.

Difference between the Building Standard Law and *Recommendations for Loads on Buildings*

• Basic Principles Applied in the Building Standard Law and *Recommendations for Loads on Buildings*

Although the wind load provisions of the Law currently in use are based on *Recommendations* (1993), there are fundamental differences between them. Because the Building Standard Law has binding legal force, any judgment contrary to the Law would not be legally permissible. Further, while the minimum standard load level is settled, any design that uses a load level lower than the minimum standard prescribed in the Law is impermissible, but designs that use load levels surpassing the minimum standard are permissible. On the other hand, *Recommendations* itself has no legally binding force and shows its concept and parameters required to conduct structural design to structural designers so that it has become possible for the structural designer to select the necessary load level (the basic wind load is settled to meet a strong wind having a return period of 100 years, and the structural designer can select his optional load level obtained by use of the conversion coefficient more than the level of the Building Standard Law).

As stated above and in contrast to the Building Standard Law, *Recommendations* is not a legally binding document. Nevertheless, *Recommendations* is frequently referenced when specific evaluation methods in the Law are not applicable, such as wind force coefficients for buildings with a special architectural configuration, increased wind velocity caused by landforms, or a vibration response characteristic of high-rise buildings taller than 60 m. It can be said that *Recommendations* serves to complement the wind load provisions prescribed in the Law.

• Specific Differences in Wind Load Specifications between the Building Standard Law and *Recommendations for Loads on Buildings*

—Clarification of separate wind loads for structural framing and exterior members

While partially-common wind force coefficients were applied for both structural framing and exterior members in the Building Standard Law before its revision in 2000, wind loads were not so clearly distinguished between the two categories. But after revision of the Law in 2000, wind loads for structural framing and exterior members were more clearly distinguished in conformance with *Recommendations*, which were included in the Enforcement Order and related Notifications.

The wind load for structural framing is the wind force that works on an entire building structure, and differs depending on the wind direction. The load on exterior members is the wind force that works on roofing materials and other exterior members (area: about 1~5 m^2), and shows maximum and minimum values in all wind directions. Accordingly, the wind pressure per area has a relation: the load on exterior member ≥ the load on structural framing.

—Introduction of standard wind velocity Vo

Before its revision in 2000, the Building Standard Law defined velocity pressure q as 60 √h and made wind load uniform throughout the nation. Following the 2000 revision, velocity pressure q has been determined using the standard wind velocity Vo, the vertical distribution of wind velocity based on ground roughness classification, the gust effect factor and other influences and, further, takes into account local and ancillary conditions and the structural characteristics of individual buildings.

Fig. 1 shows the standard wind velocity Vo in Japan, which is settled at "30~46 m/s depending on the rate of occurrence of wind damages based on the recorded history of past typhoons and associated wind properties." The value of 30~46 m/s is obtained by converting the annual maximum wind velocities recorded by meteorological offices nationwide to wind velocities with a return period of 50 years (10-minute average wind velocity at a height of 10 m over ground with a surface roughness classification II). The figure shows respective standard wind velocities in the cities, towns and villages of Japan in 2000 that were divided into nine areas classified by wind velocity level. The standard wind velocity thus obtained has allowed the dominant wind characteristics of each area to be reflected in the design wind velocity.

Fig. 1 Drawing of Standard Wind Velocity Vo

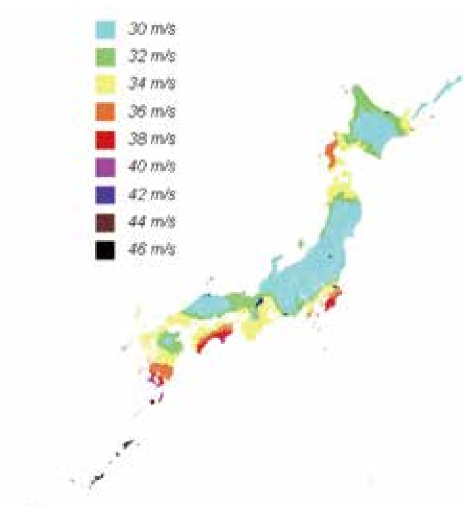


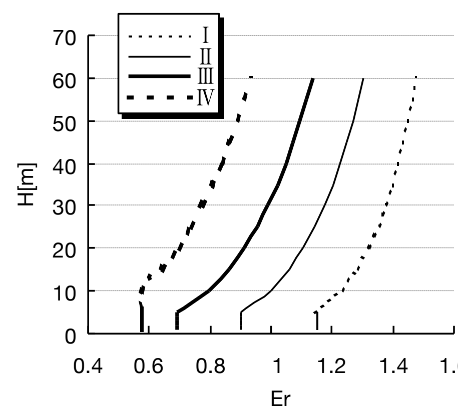
Table 1 Ground Roughness Classifications and Ground Conditions in *Recommendations for Loads on Buildings*

Classification of ground roughness	Ground conditions at peripheral areas	Representative examples
I	Flat area with nearly no obstacles	Coastal zone
II	Area with obstacles such as agricultural products, area where tress and low-rise buildings lie scattered	Rural zone
III	Area with dense trees and low-rise buildings, area where medium- and high-rise buildings (4~9 stories) lie scattered	Forest zone Industrial zone Housing zone
IV	Area with dense medium- and high-rise buildings (4~9 stories) in wider range	Medium- and high-rise urban zone
V	Area with dense high-rise buildings (10 or more stories)	High-rise urban zone

—Introduction of ground roughness classifications

In *Recommendations*, the specified ground roughness is selected by the structural designer from among the five classifications and photos shown in Table 1 based on his judgment. On the other hand, the Building Standard Law adopts a vertical distribution of wind velocity (Fig. 2) that is similar to that in *Recommendations*, but in the Law, ground roughness is clearly divided to four classifications depending on the specified area (Table 2) in order to eliminate as much vagueness in the classification as possible. Because ground roughness classifications I and IV are settled by specified administrative agencies based on the regulations, classifications II and III are to be adopted in most areas (refer to Table 2).

Fig.2 Vertical Distribution of Wind Velocity Prescribed by the Building Standard Law

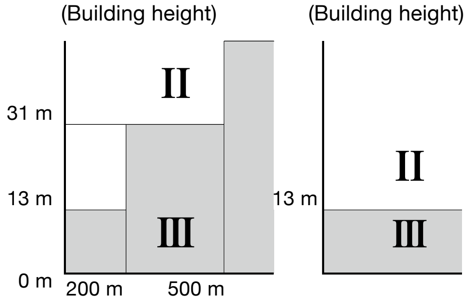


—Introduction of gust loading factor

The gust loading factor G_f was introduced in the 2000 revision of the Law and conforms to *Recommendations*. The numerical value of the gust loading factor G_f is settled according to the ground roughness classification and the building height while taking into account wind turbulence and building scale and structural characteristics. On the other hand, in the method adopted in *Recommendations*,

Table 2 Classification of Ground Roughness Prescribed by the Building Standard Law

Ground roughness classification	Inside urban planning area	Outside urban planning area
I		◎
II	○	○
III	○	○
IV	◎	



(Distance from sea coastal line or lake coastal line)*
*Sea coastal line or lake coastal line is to be set to have a distance of 1,500 m or more from the line to the opposite shoreline.

Inside the urban planning area   Outside the urban planning area

the structural designer finds the value of gust effect factor by taking into account wind turbulence, building scale and structural characteristics and by using the calculation formula.

—Settlement of two load levels of damage criterion and safety criterion in the calculation of response and limit strength

Before being revised in 2000, the Building Standard Law used allowable stress calculations and retained horizontal strength calculations to settle load levels; but, after its revision in 2000, the Law introduced critical strength calculations, too. In critical strength calculations, two criteria—damage criterion and safety criterion—were provided and their respective loads were settled. For the wind load, two load levels having 50-year and 500-year return periods respectively were settled, and loads conforming to the safety criterion were set at 1.6 times the loads conforming to the damage criterion.

—Adoption of SI (International System of Unit)

Before the Building Standard Law was revised in 2000, it had employed an engineering unit system, but following the adoption of SI (International System of Unit) in the Japanese Industrial Standards in 1991, SI was also adopted in the Law. In the system of engineering units that had been in use, both mass (kg) and force (kgf) were employed, but they were barely distinguishable and sometimes caused misunderstanding. However, in the SI system, mass (kg) and force (N) are clearly distinguished, and the force relationship is 1 kgf = 1 kg × g (gravitational acceleration) ≒ 9.8 N. As a result, wind pressure that had been expressed using kgf/m^2 is now expressed in SI units as N/m^2, and a numerical value of about 9.8 times the conventional value is adopted in the SI unit system. Meanwhile, SI units were introduced in *Recommendations* in 2004.

—Enhancement of wind force coefficients, etc.

In the Building Standard Law before its revision in 2000, the wind coefficient and wind pressure coefficient were diagrammatically expressed using a two dimensional building section, but after the Law's revision in 2000, the two-dimensional section was changed to a three-dimensional expression. Further, from 2008, the building standard improvement subsidy project began implementing wind tunnel and other tests to derive wind force coefficients for hip roofs, rooftop advertising plates, porch handrails and other members. In 2013 it became possible for structural designers to reference these coefficients.

Wind-resistant Design Guidelines of Various Industrial Organizations

Structural designers have been obliged to submit a structural calculation document for exterior building members (roofing materials, exterior walls, openings, etc.) at the time of building confirmation, but often the design and installation of these members are trusted to people specializing in the particular structural members. Given such a situation, the industrial organizations of the exterior members industry have independently prepared the guidelines shown below. These guidelines aid structural designers, project owners and supervisors in confirming that the wind resistance of exterior members conforms to the wind load provisions of the Building Standard Law.

Roofing materials

- Japan Roof Tile Industry Association and others: Guideline for Tile Roof Standard Design and Installation (2001)
- NPO Japan Exterior Furnishing Technical Center: Guideline for Decorative Slate Covering for Housing Roof and Roof Wind-resistant Design and Installation (2002)
- Japan Metal Roof Association and Japanese Society of Steel Construction: Steel Sheet Roof Structure Standards SSR2007
- Japan Copper Development Association: Copper Sheet Roof Structural Manual (revised in 2004)
- Architectural Institute of Japan: Japanese Architectural Standard Specification JASS12, Roofing Work (2004)

Exterior walls

- Architectural Institute of Japan: Japanese Architectural Standard Specification JASS27, Dry Exterior Wall Work (2004)
- Japan Fiber Reinforced Siding Manufacturers Association: Fiber Reinforced-type Siding and Standard Execution (2nd version 2009), Improvement of Housing Quality and Durability and Exterior Wall Ventilation Structure (2001)
- Japan Metal Siding Industry Association: Execution Manual of Japan Metal Siding Industry Association (2008)
- Extrusion Cement Plate Association: Standard Specifications for ECP Execution (2010)
- Architectural Institute of Japan: Japanese Architectural Standard Specification JASS21 ALC Panel Work (2005)
- Autoclaved Lightweight Aerated Concrete Panel Association: ALC Panel Structure Design Guideline (2004), ALC Thin Panel Design and Construction Guideline (October 2002), ALC Attachment Structure Standards (2004)
- Architectural Institute of Japan: Japanese Architectural Standard Specification JASS14 Curtain Wall Work (1996)
- Curtainwall Fire Window's Association: Curtain Wall Performance Standards (2006)
- Precast Concrete System Association: Guidance for Design, Precast Curtain Wall Calculation Examples (Temporary revised version)

Openings (door, window glass, etc.)

- Japan Rolling Shutters & Doors Association: Wind Pressure-resistant Strength Calculation Standards for Shutters and Overhead Doors (2003)
- Architectural Institute of Japan: Japanese Architectural Standard Specification JASS17 Glass Work (2003)

* * * * *

An outline of the wind loads on buildings adopted in Japan is introduced by comparing the wind load provisions in the Building Standard Law and the guidelines in *Recommendations for Loads on Buildings* of the Architectural Institute of Japan. Further, the guidelines for wind-resistant design pertaining to exterior building members, prepared by related industrial organizations, were introduced. ■

Evaluation of Habitability Related to Wind-induced Building Vibration

by Osamu Nakamura, Wind Engineering Institute Co., Ltd.

It is known that the wind can cause buildings to vibrate. When wind-induced vibration occurs, it causes discomfort, queasiness, seasickness and other adverse effects that can lead to complaints and to building deterioration. Further, in cases when a building vibrates more violently, panic can occur and lead to a state of chaos. How vibrations are felt differs depending on social environment and personal sensitivity, and cannot be objectively evaluated; further, criteria related to building vibration differ in different countries. Meanwhile, when people recognize that a building is safe even when it does vibrate, their uneasiness is mitigated and their sensitivity to the vibrations decreases.

The main topics of discussion in this article are: how a building vibrates when subjected to the wind, how people feel wind-induced vibration, how wind-induced vibration is treated in different countries, and what countermeasures against wind-induced vibration are taken in Japan.

Wind-induced Building Vibration

When the wind causes a building to vibrate, the vibrations that move in the along-wind direction are dominant in low-rise buildings, but in high-rise buildings, the vibration pattern is more complicated and is composed of translational vibrations consisting of two horizontal components (along-wind direction and across-wind direction) and rotational vibration caused by torsion. Thus, because such random vibrations are the result of external forces that change irregularly in terms of both time and space, they consist of responses of various frequency components and, therefore, cannot be evaluated using only a single frequency.

However, in common high-rise buildings, the primary natural frequency component of the translational vibrations appears prominently. In particular, the time history of the acceleration, which is the target for evaluation in this article, can be deemed to be the harmonic vibration of just the primary natural frequency that is accompanied by random and comparatively gentle amplitude modulations, as shown by the example in Fig. 1. In practice, vibrations occur that are composed of bi-directional translational vibrations and torsional vibration. However, these major types of vibration appear alternately, and it is seldom that the maximum response to the translational bi-directional vibrations and the torsional vibration appear simultaneously.

Accordingly, even when any maximum acceleration of the translational bi-directional vibrations and the torsional vibrations is considered to be in close agreement with the maximum acceleration composed of the translational and torsional vibrations, there is not a large difference in the evaluation results. Further, when the natural frequency in the vibration-direction differs, the level of one's perception of vibration differs depending on the frequency, and accordingly an evaluation of habitability can be made based on the vibration direction.

Perception of Vibrations

While a person's perception of and sensitivity to vibration are closely tied to displacement, velocity, acceleration, jerk and other factors, it is not truly known which of these factors is the most significant element for the perception of and sensitivity to vibration. A person's perception of and sensitivity to vibration differs according to the amplitude targeted in the evaluation, but, when evaluating habitability related to wind-induced vibration, acceleration is the frequently adopted factor.

While there is no firm reason why acceleration is frequently adopted, if anything, it can be said that a person's perception of vibration is greatly affected by one's physical response to floor vibration. However, building vibrations that take habitability into account can be accepted as single-period vibrations in most cases, and displacement, velocity, acceleration and jerk are in a proportional relation via frequency. Accordingly, when either of these is used, the evaluation results are substantially identical.

Fig. 2 shows the discussion results pertaining to the relation between acceleration and the average threshold at which vibrations are perceived. The figure is based on major survey results obtained from laboratory tests and of actual buildings subjected to strong winds; in both cases, the questionnaire results were statistically evaluated. The plot indicates the average values of the vibration perception threshold with personal de-

Fig. 1 Acceleration Time History at Top of the Building

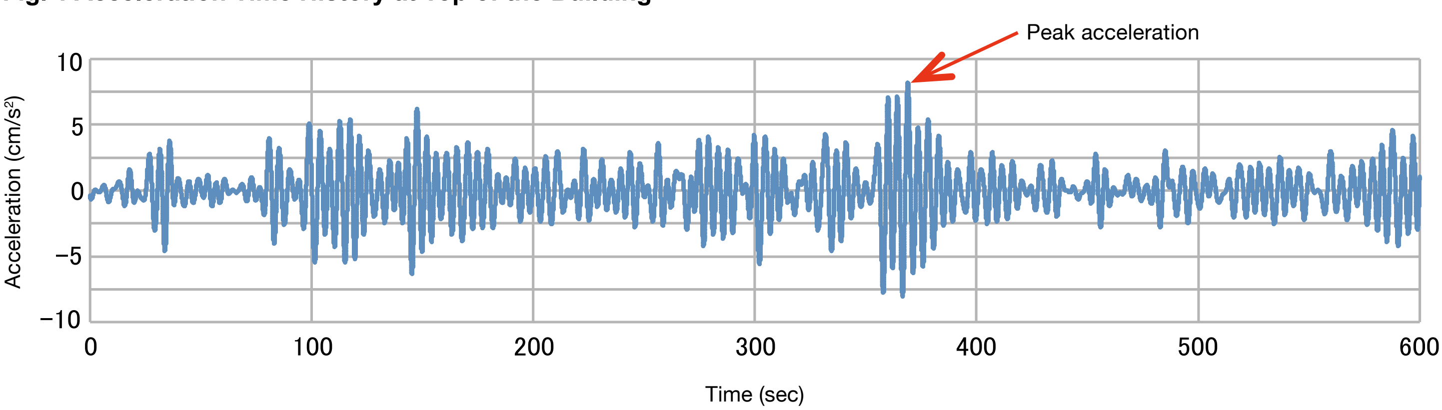
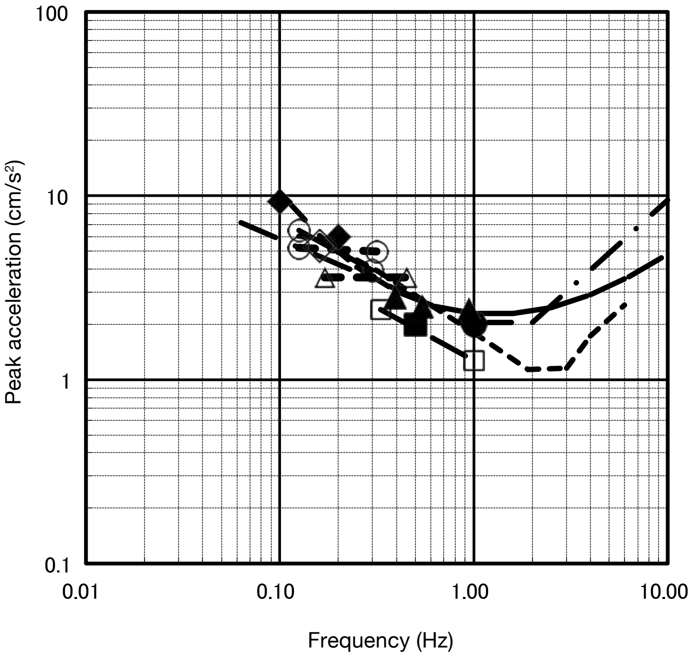


Fig. 2 Average Vibration Perception Thresholds



viations, or a perception probability of 50%. Specifically, for example, a peak acceleration of about 5 cm/s² in the neighborhood of a frequency at 0.2 Hz is shown in Fig. 2, which means that 50% of people perceive vibrations when subjected to a frequency at 0.2 Hz having a peak acceleration of 5 cm/s².

It is found in the figure that the average vibration perception threshold shows differences (different symbols in the figure) depending on the examining organization; but the same trend in vibration perception threshold having relatively few deviations is shown in the figure, in spite of survey results obtained from different organizations. That is, the perception of vibration depends on frequency, and the tendency is shown in which the vibration is most highly perceived in the neighborhood of frequencies of 1~3 Hz.

Occupant Serviceability Criteria in Different Nations

To cope with issues relevant to wind-induced building vibration, occupant serviceability criteria have been standardized in several nations. Fig. 3 shows a comparison of representative criteria for certain nations. In some nations, the standard deviation for acceleration is adopted instead of the peak value for acceleration, but in this figure, the criteria are shown replacing the peak values.

These criteria are standardized so that the peak acceleration during their respective return periods does not surpass the value for a building's natural frequency. However, the Japanese criteria show levels at 10, 30, 50, 70 and 90% of perception probability, and determination of the level of perception probability to which a return period of 1 year is set is entrusted to the judgment of the structural designers. Buildings in Japan are commonly designed targeting a perception probability of 50%. In the Australian criteria, plural return periods are given.

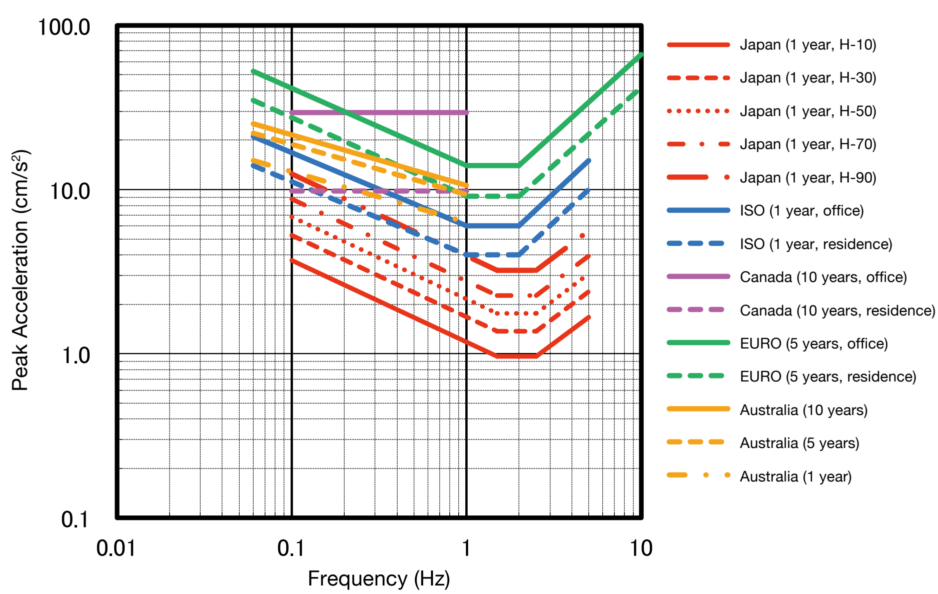
In the case of short return periods, how to deal with routine vibration is to be taken into account, and in the case of long return periods, how to handle great vibrations that occur only rarely is to be taken into account; but, the determination of return period will differ depending on the social environment and the concept of different nations.

Countermeasures against Wind-induced Building Vibration

In order to mitigate wind-induced building vibration, the method that improves building rigidity is used. However, as can be seen in Figs. 2 and 3, in the low frequency zone of 1 Hz or less, even if vibration is mitigated by improving rigidity, this results in an increase in natural frequency which, in turn, increases the likelihood that vibration will be perceived and that habitability will not be improved in the end. To correct this, methods that enhance vibration-damping performance are adopted in most cases. In this case, because a low acceleration level is usually targeted in order to improve habitability, it is necessary to adopt a damping system that is effective even in the low-acceleration zone. Currently, many high-rise buildings have been constructed in which vibration-damping devices are installed.

On the other hand, from an architectural planning perspective, there is a method to plan the floor and plane position according to a building's expected use and frequency. Further, in order to prevent both visual and auditory perception of vibration, measures are adopted that will prevent creaking in partition walls and other secondary structural members and that will prevent the vibration of window shades and pendant lights. ■

Fig. 3 Comparison of Occupant Serviceability Criteria for Tall Buildings



Performance-based Wind-resistant Design for 300 m Vertical City

by Kiyoaki Hirakawa, Takenaka Corporation

Outline of Building and Structure

ABENO HARUKAS (hereinafter HARUKAS) is Japan's tallest skyscraper, standing at 300 meters, which was completed in March 2014 (Photo 1).

The building site is situated in Abeno, Osaka, which is a city representative of Japan and the world's seventh largest metropolitan area. This area has been growing fast and drawn the most attention in recent years.

HARUKAS is a superhigh-rise vertical city with the gross floor area of approx. 212,000 square meters. Rising 60 stories above the ground and 5 underground stories, this tower incorporates diverse functions: a terminal station, a department store, an art museum, offices, a hotel, an observatory, parking spaces and more. No other building of this scale has been built above a station in any place of the world.

Japan is one of the most earthquake and typhoon-prone countries in the world. Details of the performance-based seismic design for this building are summarized in Ref. 1. This paper focuses on the performance-based wind-resistant design for HARUKAS as below.

When a strong wind blows on a building, a Karman vortex is generated on the leeward of the building and lets the building shake in the direction orthogonal to the wind. The illustrations as indicated in the Fig. 1 show the visualized statuses of Karman vortexes, which indicate that the effect of a Karman vortex is minimized in the case of HARUKAS (below right) compared with a rectangular solid building (below left).

Fig. 1 Statuses of Karman Vortexes

Photo: SUZUKI hisao

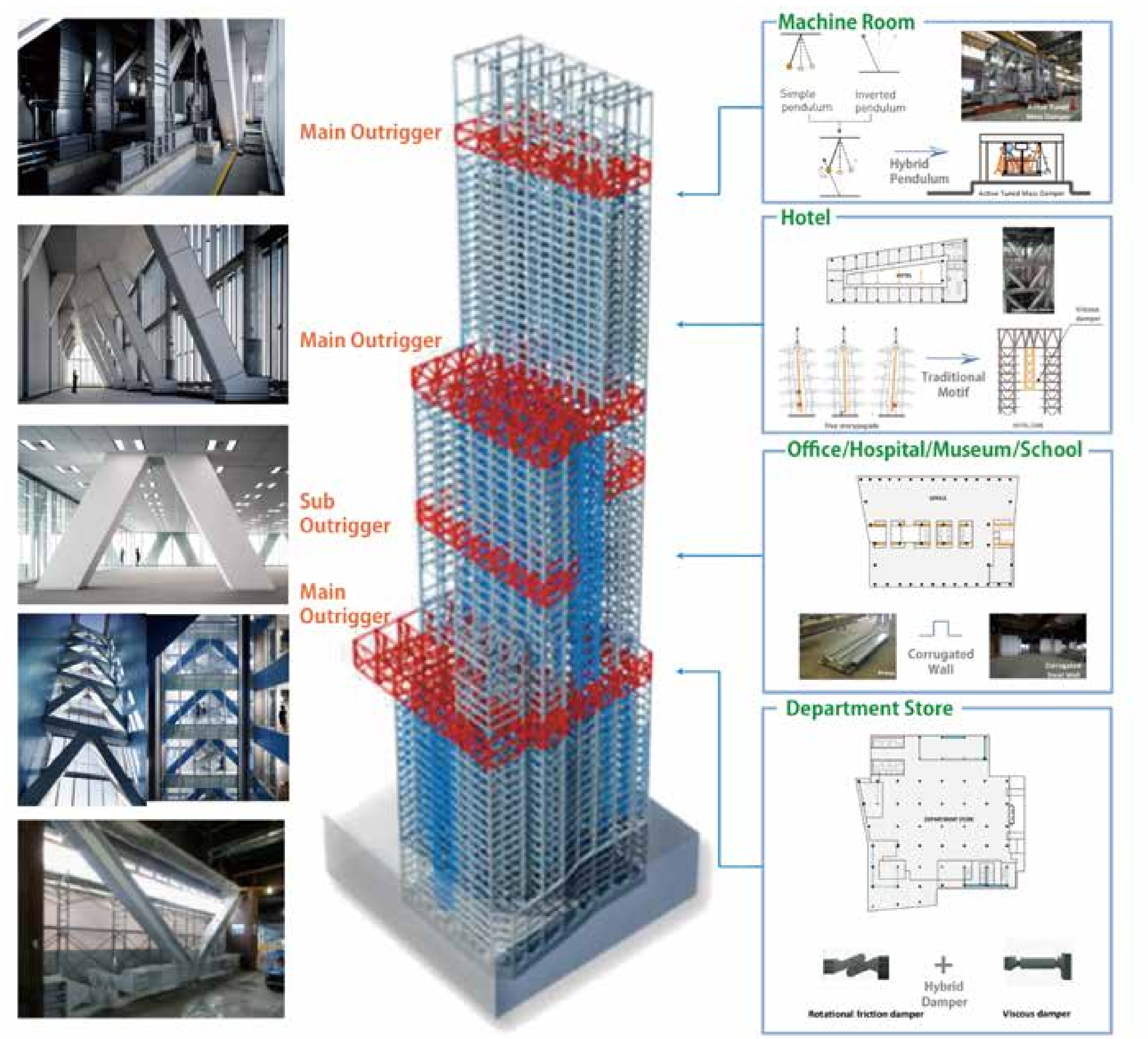
Photo 1 Northwest view

The quality of aerodynamic characteristics is extremely critical in wind-resistant design of a skyscraper as high as 300 meters. A "setback" type building like HARUKAS is a building shape with excellent aerodynamic performance that efficiently reduces overturning moment acting on a building affected by a Karman vortex.

As shown in Fig. 2, the superstructure is composed of three "blocks" having setbacks on the north side. The lower block is for the department store, the middle one for offices and the upper one for a hotel. The upper block has a large atrium in the center. Located between the blocks and at the top of the upper one are transfer-truss floors. In order to enhance horizontal and torsional rigidity against strong earthquakes and wind excitation, outrigger mega-trusses are placed in the transfer floors and the middle block.

A total of four types of dampers, both viscous and hysteresistic, are placed mainly at the four corners in the lower block, around the central core in the middle block and around the atrium in the upper block in order to absorb energies input by earthquakes or wind. In addition, two kinds of mass damp-

Fig. 2 Structural Planning



ers (AMD and ATMD) are installed on the 56th floor in order to improve the habitability mainly of the hotel in the upper block. Evaluation of habitability against wind load will hereinafter be described in detail.

The robust structure using outriggers and shear studs helps reduce the natural period of the building to prevent occurrence of an aerodynamic unstable vibration (phenomenon of shaking causing greater shaking) which is likely to be generated by a soft building with a longer period. Moreover, use of vibration control dampers enhances damping performance to restrain the building shaking caused by a strong wind and settle it down in a short period of time.

Outline of Wind-resistant Design

Table 1 shows the design wind speeds, crite-

Table 1 Study Items for Wind-resistant Design

Study item		Design wind speed	Wind load		Design criteria
			Average load	Fluctuation component	
Safety	Structural framework	Vo=34 m/s (Level 1) Vo=42.5 m/s (Level 2) (to comply with Notification)	Wind tunnel test (measured wind pressure)	Spectrum modal; Load combination to comply with *Guidelines for Loads on Buildings* (Architectural Institute of Japan)	Structural members shall be within elastic range and story drifts shall be 1/100 or less for Level 2.
	Aerodynamic unstable vibration	1.2 times wind speed for structural framework	–	Aerodynamic vibration test using MDOF model	No aerodynamic unstable vibration shall occur at not more than 1.2 times Level 2 design wind speed.
	Exterior claddings	1.10 times wind speed specified in Notification	Wind tunnel test (measured wind pressure)	–	Glass shall not be broken.
Comfort	Habitability study	V=17 m/s (recurrence interval of one year)	Wind tunnel test (measured wind pressure)	Spectrum modal; (frequency of wind direction to be considered)	"H-30" (about 30% of habitants feel quakes) or more

ria and other items studied in developing the performance-based wind-resistant design for this building.

Outline of Wind Tunnel Tests

Wind pressure measurement tests were conducted to determine the wind pressures acting on this building. The scale of the wind tunnel test model for that purpose was 1/500, and the modeling range was a radius of 700 meters (Photo 2). Approximately 600 measuring points were embedded in an acrylic model to measure the wind pressures.

Base shears were calculated by spectrum modal response analyses taking only the first mode into consideration. The relationship between the base shears at the wind speed for "Level 2" corresponding to the return period of 500 years and wind angles are shown in Fig. 3. The maximum base shear in the north-south (Y) direction, a narrow side of the building, appears at wind angle 85°, which is nearly the east-west (X) direction.

Calculation of Wind Loads

The wind loads on all the stories when the base shear is largest at wind angles 175°and 85°for X-direction and Y-direction, respectively, are shown in Fig. 4 in comparison with the seismic loads for "Level 2."

The seismic loads exceed the wind loads on all stories in the X-direction and almost all stories except for a few lower stories in the Y-direction. Loads that incorporate both types of loads were established as the external loads for sectional design.

Studies of Aerodynamic Unstable Vibration

The wind speed at which the frequency generated by Karman vortex calculated by the wind pressure measurements coincides with the building's natural frequency (0.169 Hz) in the Y-direction is 97.9 m/sec., which is more than 1.4 times the wind speed (66.6 m/sec.) with the recurrence interval of 500 years.

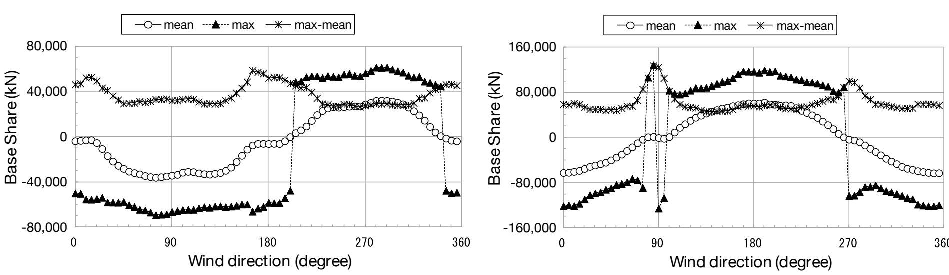
It seems that this building has a configuration in which aerodynamic unstable vibration is unlikely to occur, because the building width varies with building height in the Y-direction with a larger wind pressure area corresponding to the orthogonal directions for wind directions of 90° and 270°.

Nevertheless, aerodynamic vibration experiments were conducted considering that the upper block is thin and possibly vulnerable to torsional vibration. The experiments used a 5-lumped-mass 3D model which has the same mass, eigenvalue and damping (0.03 for translational mode and 0.014 for torsional mode) as the design values (Photo 3). As a result, it is confirmed that aerodynamic unstable vibration does not occur at less than 1.2 times the design wind speed with the recurrence interval of 500 years, as shown in Fig. 5.

Photo 2 Wind tunnel test

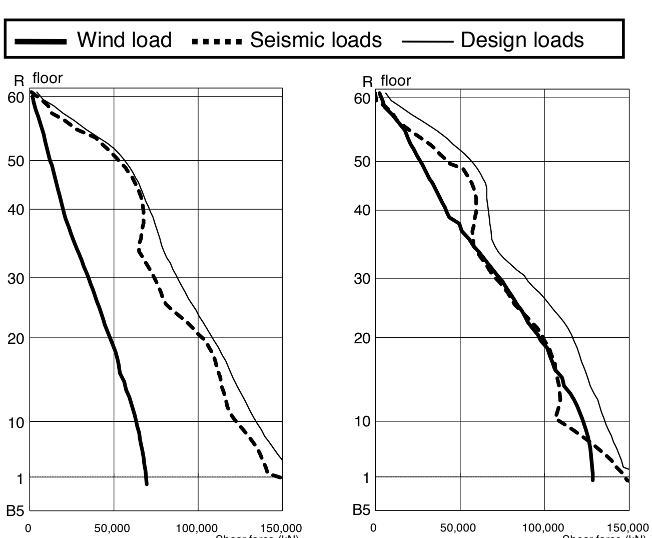
Photo 3 Aerodynamic vibration experiment

Fig. 3 Relations between Base Shears and Wind Angles



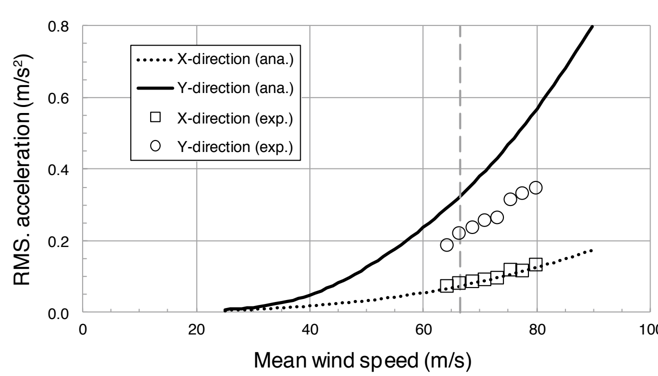
(a) X-direction (b) Y-direction

Fig. 4 Comparison between Wind Loads and Seismic Loads



(a) X-direction (b) Y-direction

Fig. 5 Aerodynamic Vibration Test



Evaluation of Habitability

There will be a hotel in the upper block of this building (see Photo 4), for which comfortable habitability has to be provided by keeping the response accelerations less than approximately 3 cm/sec^2, at Class H-30 (about 30% of occupants present perceive tremor) with the recurrence interval of one year. For that purpose, two kinds of active mass dampers were installed on the 56th floor to reduce response accelerations in case of strong winds.

Two active mass dampers work only when their period is synchronized with the natural period of the building, which is as long as about 6 seconds. One active mass damper (AMD) at the east side is a conventional pendulum. The other active tuned mass damper (ATMD) at the west side is a conventional suspended pendulum combined with an inverted pendulum so as to minimize the suspended length (2.2 m) and avoid exceeding ceiling height as shown in Fig. 6.

Habitability in the hotel rooms is improved with mass dampers for the narrow side (north-south; Y-direction) of the building as shown in Fig. 7. However, the vibration in the wide side (east-west; X-direction) is sufficiently small without mass dampers.

The building ensures a high level of habitability by reducing the acceleration of shaking in a short direction to about a half when a strong wind blows, which is as frequent as approximately several to ten-odd times a year.

Conclusion on Performance-based Wind-resistant Design

This section introduces the performance-based wind-resistant design of the first 300 m-high building in Japan. The building configuration, superstructure systems and various damping devices are sophisticatedly integrated to ensure a higher level of safety and comfort against wind load. ■

Fig. 6 Mechanism of Active Tuned Mass Damper

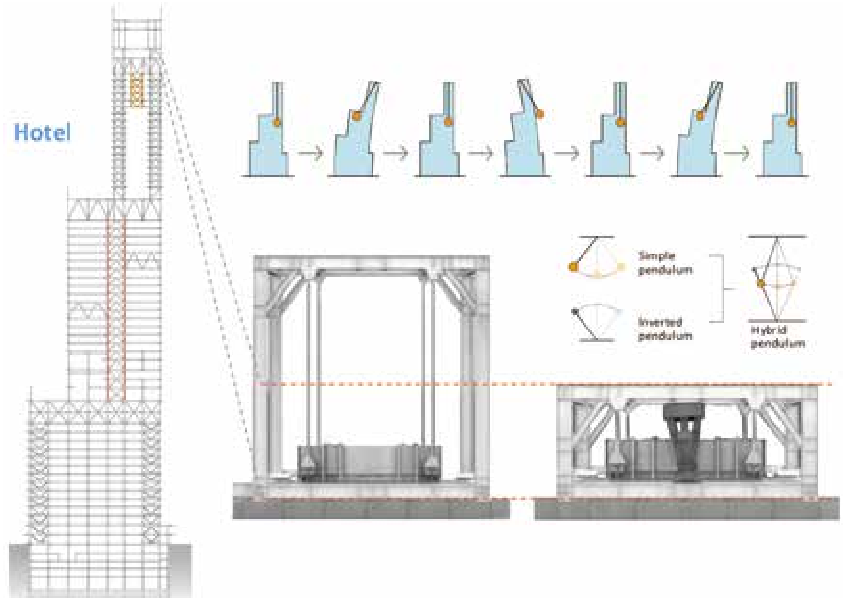
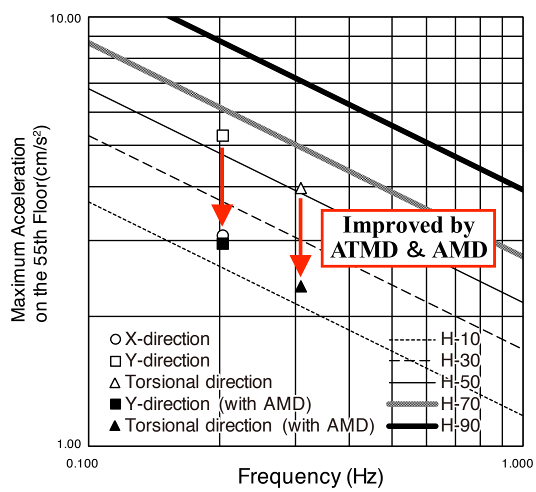


Fig. 7 Habitability Evaluation of Hotel Guest Room at 55th Floor



Note: Torsional vibrations were maeasured at 3,500 m from the center of the building.

Photo 4 Top of HARUKAS

Reference

1) Nakai, M., Koshika, N., Kawano, K., Hirakawa, K., and Wada, A. (2012). "Performance-Based Seismic Design for High-Rise Buildings in Japan" International Journal of High-Rise Buildings.

Basic Details about High-strength Bolt Joining

The Japan Iron and Steel Federation

In joining steel-frame members with high-strength bolts, two methods are commonly used: friction joining and tension joining. The two types of high-strength bolts in primary use are: high-strength hexagonal bolts and high-strength torque shear bolts. Depending on the application environment, in addition to bolts manufactured using general-purpose ordinary steel materials, hot-dip galvanized bolts and bolts manufactured using fire-resistant steel, weathering steel and stainless steel are available. Nominal diameters of commonly applied bolts are M16, M20, M22 and M24. In addition, some bolt makers offer bolts having larger diameters, such as M27 and M30.

Currently, the grade of high-strength bolts most commonly used in the building construction in Japan is F10T (tensile strength: 1,000 N/mm^2), and the grade of high-strength hot-dip galvanized bolts is F8T (tensile strength: 800 N/mm^2). In the past, F13T bolts (tensile strength: 1,300 N/mm^2) were manufactured but were later prohibited due to the occurrence of delayed fractures. Further, regarding Japanese Industrial Standards (JIS) grades F8T, F10T and F11T (tensile strength: 1,100 N/mm^2), the application of F11T bolts was prohibited for the most part. Further, F11T is not described in *Guidebook on Design and Fabrication of High Strength Bolted Connections* issued by the Architectural Institute of Japan.

Table 1 shows the high-strength bolts that, currently, are commonly applied in Japan.

Kinds of High-strength Bolts

• High-strength Hexagonal Bolts

The high-strength hexagonal bolts used to join ordinary steel products are specified in JIS: JIS B 1186 (Sets of high strength hexagon bolt, hexagon nut and plain washers for friction grip joints) and 1 set is defined as containing 1 bolt, 1 nut and 2 washers (see Photo 1). The reason why the bolts are specified in sets is to guarantee the mechanical properties, shape and dimensions of the bolt, nut and washers that constitute a set and, further, to specify the introduction of axial force.

Three kinds of hexagonal bolts are available according to the mechanical properties of the set to which they belong: Type 1 (F8T), Type 2 (F10T) and Type 3 (F11T). In Type 1 (F8T) sets, the joining efficiency is bad and there are not any JIS-certified production plants; and in the Type 3 (11T) sets, delayed fractures have occurred. Because of this, these two bolt sets are no longer in use. Currently, only Type 2 (10T) sets produced at JIS-certified production plants are in use.

Further, high-strength hexagonal bolts are classified as Type A or B according to the torque coefficient of the bolt set.

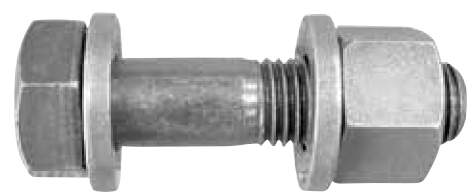


Photo 1 Set of high-strength hexagonal bolt

Table 1 High-strength Bolts Commonly Applied in Japan

Name	Standard	Grade	Nominal diameter	Bolt product	Tensile strength (N/mm^2)	Hardness (HRC)	Reference
High-strength hexagonal bolt	JIS B 1186	F10T	M12~M30	Bolt	1,000~1,200	27~38	JIS-certified productionplant
				Nut	–	16~35	
				Washer	–	35~45	
High-strength torque shear bolt	JSS II 09	S10T	M16~M30	Bolt	1,000~1,200	27~38	Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	35~45	
High-strength hot-dip galvanized bolt	Conformingto JIS B 1186	F8T	M16~M30	Bolt	800~1,000	18~31	Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	25~45	
High-strength fire-resistant steel bolt	JIS B 1186 JSS II 09	F10T-FR S10T-FR	M16~M24	Bolt	1,000~1,200	27~38	JIS-certified production plant S10T-FR: Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	35~45	
High-strength hot-dip galvanized fire-resistant steel bolt	Conforming to JIS B 1186	F8T-FR	M16~M24	Bolt	800~1,000	18~31	Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	25~45	
High-strength weathering steel bolt	JIS B 1186 JSS II 09	F10T-W S10T-W	M16~M24	Bolt	1,000~1,200	27~38	JIS-certified production plant S10T-W: Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	35~45	
High-strength stainless steel bolt	SSBS 301	10T-SUS	M12~M24	Bolt	1,000~1,200	27~38	Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	16~35	
				Washer	–	35~45	

Table 2 Examples of Chemical Composition of Steel Materials Used for High-strength Bolts, Nuts and Washers (%)

		C	Si	Mn	P	S	Cu	Ni	Cr	B	Al	Ti	Mo	Nb
High-strength ordinary steel bolt	Bolt	0.21	0.12	0.83	0.021	0.009	–	–	0.38	0.0016	0.032	0.021		
	Nut	0.34	0.17	0.75	0.030	0.021	–	–	–	–	–	–	–	–
	Washer	0.22	0.27	1.04	0.020	0.006	–	–	–	0.0016	–	0.022	–	–
High-strength fire-resistant steel bolt	Bolt	0.22	0.24	0.82	0.018	0.013	0.01	0.02	1.04	–	0.025	–	0.36	–
	Nut	0.22	0.20	0.78	0.016	0.016	–	–	1.09	–	0.035	–	0.36	–
High-strength weathering steel bolt	Bolt, nut, washer	0.22	0.19	0.74	0.014	0.024	0.30	0.37	0.63	0.0018	0.042	–	–	–
High-strength stainless steel bolt	Bolt, nut, washer	0.07	0.30	0.84	0.037	0.006	3.30	4.23	15.67	–	–	–	–	0.37

The steel materials that are commonly used to manufacture bolt products are: low-carbon steel with added chromium (Cr) and boron (B) for the bolts; carbon steel for the machine structure for the nuts; and carbon steel, or low carbon steel with added manganese (Mn) or B, for the machine structure for the washers (Table 2).

• High-strength Torque Shear Bolts

High-strength torque shear bolts are specified by the Japanese Society of Steel Construction in JSS II 09. One bolt, 1 nut and 1 washer are specified as 1 set (Photo 2), and only a single set is specified in JSS II 09, namely, Type 2 (S10T). High-strength torque shear bolts are denoted as grade S10T and are distinguished from high-strength hexagonal bolts that are denoted as F10T.

Torque shear bolts feature a bolt configuration with a round bolt head and the provision of the pin tail at the bolt tip via the break-off groove (refer to Photo 2). These bolts also feature that the required axial force for high-strength torque shear bolts is obtained by fastening the bolt until when the pin tail causes fracture and accordingly finishing of bolting work can easily been confirmed. Meanwhile, the required fastening axial force of the bolt set is specified in the standards for high-strength torque shear bolts. Table 3 shows the tension required for fastening bolts set at room temperature.

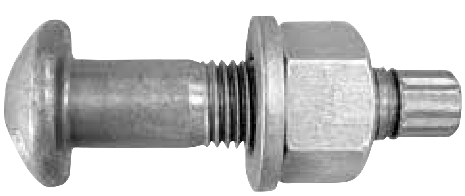


Photo 2 Set of high-strength torque shear bolt

Table 3 Introduction Tension for Sets of High-strength Torque Shear Bolts (Room Temperature)

Nominal diameter	Average value of bolt introduction tension per 1 production set(kN)
M16	110~133
M20	172~207
M22	212~256
M24	247~298
M27	322~388
M30	394~474

Because high-strength torque shear bolts are not standardized in JIS, bolt makers have obtained general approval from the Minister of Land, Infrastructure, Transport and Tourism to manufacture this type of bolt.

• High-strength Hot-dip Galvanized Bolts

In order to provide rust prevention or corrosion protection for steel-frame members, high-strength hot-dip galvanized hexagonal bolts (Photo 3) are used to join sections of steel frames manufactured using hot-dip galvanized products of ordinary steel. Because sets of high-strength hot-dip galvanized hexagonal bolts are not standardized in JIS and because the F value (strength rating) is not settled in the Building Standard Law of Japan, bolt makers have obtained general approval for "high-strength hot-dip galvanized bolt joining" from the Minister of Land, Infrastructure, Transport and Tourism, based on Article 37 of the Building Standard Law. To this end, high-strength hot-dip galvanized hexagonal bolts are manufactured conforming to the standard in JIS.

Galvanizing of bolts, nuts and washers is conducted at HDZ55 (coating mass: 550 g/m^2 or more), and over-tapping of the nut threads is conducted prior to galvanizing.

Conventionally, the strength rating of hot-dip galvanized bolts was set at the F8T level, taking into account the drop in strength and the occurrence of delayed fractures that are attributable to the galvanizing bath temperature being higher than the tempering temperature for F10T high-strength bolts. However, high-strength bolts having an F12T strength, that are manufactured using recently developed, ultrahigh-strength bolt technology, have been put into practical application.

The friction surfaces after galvanizing are given a slight blasting treatment to improve the surface roughness to 50 micron Rz or higher. When special surface treatments other than blasting are applied, a slip strength test is conducted to confirm surface friction.

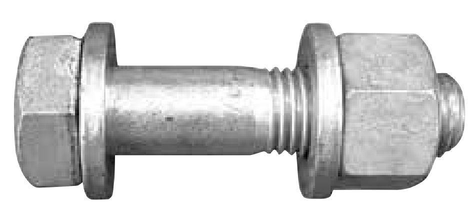


Photo 3 Set of high-strength hot-dip galvanized bolt

Design and Joining of High-strength Bolt Joints

• Allowable Strength of High-strength Bolts

The allowable strengths prescribed for the friction joining and tension joining of high-strength bolts (F10T and S10T) are stipulated in the Enforcement Order of the Building Standard Law, and the allowable strengths of high-strength hot-dip galvanized bolts (F8T) are specified according to the approval of the Minister of Land, Infrastructure, Transport and Tourism. The allowable strengths, listed by the nominal bolt diameter, are summarized in Table 4.

The allowable shear strengths of high-strength hot-dip galvanized bolts (F8T), shown in Table 4, are found from the formula: "allowable shear strength = 0.40 × Bo (design bolt tension)," in which the slip coefficient is set at 0.40. On the other hand, the allowable shear strength of high-strength hexagonal and torque shear bolts (F10T and S10T) is calculated using a slip coefficient set at 0.45. Meanwhile, the allowable shear strength and the allowable tensile force for F10T and S10T are similar to those prescribed in *Design Standard for Steel Structures* of the Architectural Institute of Japan.

• Fastening Operations

High-strength bolt fastening is undertaken by a group of bolts and in the following order: primary fastening → marking → final fastening. Meanwhile, the instruction of work procedure for ultrahigh-strength bolts is similar to that for high-strength bolts, but the primary torque for ultrahigh-strength bolts is different from that of high-strength bolts.

—Primary fastening

The primary fastening of high-strength hexagonal bolts, high-strength torque shear bolts and high-strength stainless steel bolts is undertaken using the primary fastening torque values shown in Table 5; high-strength hot-dip galvanized bolts are fastened using the primary fastening torque values shown in Table 6.

—Marking

After the primary fastening is completed, all the bolts, nuts and washers of all the bolt sets, as well as the structural members, are marked.

—Final fastening

After a bolt set has undergone primary fastening and marking, final fastening is conducted by rotating the nuts. High-strength hexagonal bolts are fastened using the prescribed torque to obtain the standard bolt tension. High-strength torque shear bolts are fastened using a dedicated electric wrench until the break-off groove fractures.

Final fastening of high-strength hot-dip galvanized bolts and high-strength stainless steel bolts is performed by rotating the nut 120° from the mark made at the completion of primary fastening and marking.

—Inspection

After final fastening, any excess bolt length and nut rotation are visually inspected to confirm whether or not fastening is normally undertaken. Meanwhile, for high-strength hexagonal bolts and high-strength torque shear bolts that are fastened by means of the torque method to the standard bolt tension, or bolt elastic range, the degree of nut rotation at the time of final fastening will show a slight change depending on the lev-

Table 4 Allowable Strength of High-strength Bolts (Sustained Load)

Grade according to mechanical property of bolt	Nominal diameter	Design bolt tension (kN)	Standard bolt tension (kN)	Allowable shear strength (kN)		Allowable tensile force (kN)
				1-surface friction	2-surface friction	
F8T F8T-FR	M16	85.2	93.7	22.7	45.4	50.3
	M20	133	146	35.4	70.8	78.5
	M22	165	182	44.0	88.0	95.0
	M24	192	211	51.2	102	113
	M27	250	275	66.6	133	143
	M30	305	335	81.3	163	177
F10T (S10T)and F10T (S10T)-FR (M16~M24) F10T (S10T)-W(M16~M24) 10TSUS (M12~M24)	M16	106	117	30.2	60.3	62.3
	M20	165	182	47.1	94.2	97.4
	M22	205	226	57.0	114	118
	M24	238	262	67.9	136	140
	M27	310	341	85.9	172	177
	M30	379	417	106	212	219

Table 5 Primary Fastening Torque for High-strength Bolts

Grade	Nominal diameter	Fastening torque (N・m)
F10T S10T	M16	Approx. 100
	M20	Approx. 150
	M22	Approx. 150
10T-SUS (M16~M24)	M24	Approx. 200
	M27	Approx. 300
	M30	Approx. 400

Table 6 Primary Fastening Torque for High-strength Hot-dip Galvanized Bolts

Grade	Nominal diameter	Fastening torque (N・m)
F8T	M16	Approx. 100
	M20	Approx. 150
	M22	Approx. 150
	M24	Approx. 200
	M27	Approx. 200
	M30	Approx. 250

el of primary fastening, but the change will remain within a few tenths of a degree.

On the other hand, for high-strength hot-dip galvanized bolts and high-strength stainless steel bolts that are fastened using the nut rotation method to a point close to the bolt strength, the final nut rotation amounts to 120° (the prescribed rotation amount). Because the post-fastening relaxation of high-strength hot-dip galvanized bolts and high-strength stainless steel bolts is greater than that of high-strength hexagonal bolts and high-strength torque shear bolts, the required fastening bolt tension is increased to the bolt's yield area.

In cases when it is found by visual inspection that both the bolt and washer cause their co-rotation and axial rotation due to marking provided after primary fastening and that abnormality is observed in the nut rotation amount, the bolt set is replaced with the new one. In such cases, once a high-strength bolt has been used, it should never be reused.

—Fastening operation control

In order to confirm whether or not a friction surface has been correctly treated or if proper fastening has been undertaken, the Architectural Steel Framing Quality Control Organization of the Japanese Society of Steel Construction has issued instructions titled "Qualification System for Engineers for Architectural High-strength Bolt Joining Control" for high-strength bolts in general and "Qualification System for High-strength Bolt Execution Engineers" for high-strength hot-dip galvanized bolts and high-strength stainless steel bolts. Based on the instruction of work procedures described in these two qualification systems, engineers engaged in high-strength bolt joining operations can exercise good execution control.

Because these engineers are highly knowledgeable about the execution of high-strength bolt work, they can be expected to ensure a high degree of quality execution work when installing high-strength bolt connections.

Recent Developments in High-strength Bolts

As stated above, the strength rating of commonly used high-strength bolts has been increased to F10T grade (1,000 N/mm^2). This is attributable to the unavoidable risk of delayed fractures that are associated with the use of high-strength bolts of F11T or higher grades.

However, as the size and strength of steel-frame members have increased in recent building construction, the adoption of the commonly applied F10T bolts has led to an excessive increase in the size of joining members and in the number of bolts required. To cope with this situation, the need is growing for more compact bolt joints, or for stronger high-strength bolts. In answer to this need, several bolt makers have successively hurdled the problems associated with delayed fracture that is caused by higher bolt strength, and have developed and put into practical use ultrahigh-strength torque shear bolts having a tensile strength of 1,400 N/mm^2, as well as ultrahigh-strength hot-dip galvanized bolts having a tensile strength of 1,200 N/mm^2.

• Ultrahigh-strength Torque Shear Bolts

Ultrahigh-strength torque shear bolts are now in practical use thanks to the development of steel materials with high delayed-fracture resistance and to improved thread configurations that relax stress concentration (Photo 4). The basic configuration and dimensions conform to the standard JSS II 09 of the Japanese Society of Steel Construction. Ultrahigh-strength torque shear bolts offer a high design strength that is about 1.5 times that of conventional bolts (F10T), and the corresponding bolt joints are compact, about 2/3 the size of conventional bolt joints.

Because of these characteristics, users can obtain many advantages from the application of ultrahigh-strength torque shear bolts: reductions in the expense and term of construction work and high efficiency and labor savings of bolting work. Because of this, the application of these bolts is increasing in the construction of high-rise buildings that use large structural members, shopping centers that feature wider column-to-column spans, and production plants and warehouses with heavy floor weights. Table 7 shows examples of ultrahigh-strength torque shear bolts. ■

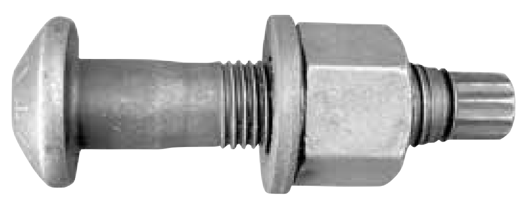


Photo 4 Set of ultrahigh-strength torque shear bolt

Table 7 An Example of Ultrahigh-strength Bolts

Name	Standard	Grade	Nominal diameter	Product	Tensile strength (N/mm^2)	Hardness (HRC)	Reference
Ultrahigh-strength torque shear bolt	–	Specified originally by respective bolt makers (Ex: SHTB, STCB, USSB)	M16~M24	Bolt	1,400~1,490	39~47	Approval by Minister of Land, Infrastructure, Transport and Tourism
				Nut	–	30~40	
				Washer	–	40~50	

Preparation of Reference Materials on Steel Construction Technologies in Japan

The Japan Iron and Steel Federation (JISF) has prepared a list of reference materials on steel construction technologies in Japan. In order to encourage wider use of Japanese steel construction technologies overseas and at the request of the Ministry of Land, Infrastructure, Transport and Tourism. JISF introduces in these reference materials 27 kinds of steel construction technologies and steel construction products that are widely applied in Japan in the fields of building construction and civil engineering.

These technologies and products help to improve the functions of buildings, port/harbor facilities and other infrastructures and are demonstratively effective in disaster prevention. These reference materials are now available at our website (http://www.jisf.or.jp/en/activity/sctt/index.html). The technologies and products introduced are:

- New Structural System Buildings Employing Innovative Steel Materials
- SN Steel - Structural Steels for Buildings
- Cold-formed Rectangular Steel Tubes for Buildings - BCR/BCP
- Low Yield-point Steel for Building Structures
- Buckling Restrained Brace - BRB
- Concrete-Filled Tube - CFT
- Thermo-Mechanical Controlled Process - TMCP Steel
- High Strength Steel for Buildings - SA440/H-SA700
- Port Renovation Method Employing Steel Materials
- Seismic Retrofitting of Quays, Seawalls and Breakwaters Employing Steel Materials
- Method of Reinforcing Existing Bridge Foundation with Steel Pipe, Sheet Piles and Steel Pipe Piles
- Lateral Flow Control for Revetment
- Steel Framed House
- Liquefaction Control Earthquake Resistance Measures Using Steel Sheet Piles
- Debris Flow Control Method Employing Steel Materials
- Landslide Disaster Control Method with Cribbing Structure (Steel Cribbing)
- Falling-stone Prevention and Slope Protection Work Using Steel Materials
- Steel Structure Disaster Protection Center Building
- High Tide and Tsunami Wave Control with Steel Materials
- Water Shield Revetment Using Steel Piles and Steel Pipe Sheet Piles
- School Facility Built with a Steel Structure (Multipurpose Institution)
- Floating Disaster Management Base (Mega-float)
- Fire Resistant Steel (FR)
- Ultra-High-Strength Bolt
- H-beam with Fixed Outer Dimensions
- Steels for Bridge High-performance Structure (SBHS)
- Weathering Steel for Bridges

Request for Participation in Survey of *Steel Construction Today & Tomorrow*

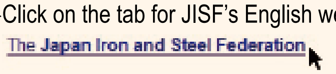
Steel Construction Today & Tomorrow, a joint periodical of the Japan Iron and Steel Federation (JISF) and the Japanese Society of Steel Construction, is published three times a year. It is the only English periodical that distributes technological information about steel construction in Japan to the worldwide construction community.

We are conducting a survey of the periodical's readership regarding publication of the three issues planned for fiscal 2014. The survey's major aim is to gain an accurate understanding of reader needs so as to enhance the usefulness of the publication. The survey forms are available as follows.

• At the JISF Website

→Enter "jisf" in the search window of your internet browser

→Click on the tab for JISF's English website



→Click on the tab for Steel Construction Today & Tomorrow

→Click the survey form

• Printed Form for Faxing

A survey form is enclosed in the magazines sent to our regular subscribers. Please answer the questions in the form and fax to +81-3-3667-0245.

Your positive participation in the readership survey will greatly help us to enhance the usefulness of *Steel Construction Today & Tomorrow*. This will benefit both your country and the Japanese steel industry. To attain this goal, we eagerly seek your ready cooperation in filling-out and returning the survey's "questionnaire". ■

STEEL CONSTRUCTION TODAY & TOMORROW

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Published jointly by

The Japan Iron and Steel Federation
3-2-10, Nihonbashi Kayabacho, Chuo-ku, Tokyo 103-0025, Japan
Phone: 81-3-3669-4815 Fax: 81-3-3667-0245
URL http://www.jisf.or.jp/en/index.html

Japanese Society of Steel Construction
Yotsuya Mitsubishi Bldg. 9th Fl., 3-2-1 Yotsuya, Shinjuku-ku, Tokyo 160-0004, Japan
Phone: 81-3-5919-1535 Fax: 81-3-5919-1536
URL http://www.jssc.or.jp/english/index.html

Edited by
Committee on Overseas Market Promotion, The Japan Iron and Steel Federation
Chairman (Editor): Keiji Ando

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