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Performance-based Wind-resistant Design for High-rise Structures in Japan

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Abstract

This paper introduces the current status of high-rise building design in Japan, with reference to some recent projects. Firstly, the design approval system and procedures for high-rise buildings and structures in Japan are introduced. Then, performance-based wind-resistant design of a 300 m-high building, Abeno Harukas, is introduced, where building configuration, superstructure systems and various damping devices are sophisticatedly integrated to ensure a higher level of safety and comfort against wind actions. Next, design of a 213 m-high building is introduced with special attention to habitability against the wind-induced horizontal motion. Finally, performance-based wind-resistant design of a 634 m-high tower, Tokyo Sky Tree, is introduced. For this structure, the core column system was adopted to satisfy the strict design requirements due to the severest level of seismic excitations and wind actions.

Keywords: High-rise buildings, Performance-based design, Wind tunnel test, Aerodynamic instability, Habitability, Damping devices

1. Introduction

The design of buildings in Japan is based on the Building Standard Law of Japan (BSLJ), which specifies the minimum building design requirements based on Performance Based Design (PBD). The wind load provisions of the current BSLJ are similar to those of the Recommendations for Loads on Buildings (RLB) published in 1993 by the Architectural Institute of Japan (AIJ), although the AIJ-RLB was revised in 2004. The BSLJ requires designers to check building performance for two wind load levels as follows. 50-year-recurrence wind loads (Level 1) requires the main resisting system to be within allowable stress and the components/cladding not to fall off. 500-year-recurrence wind loads (Level 2) require the main resisting system not to collapse. It is also required that design of buildings higher than 60 m shall be approved by the Minister of Land, Infrastructure and Transport (MLIT), where time-domain dynamic response analyses are obligatory for seismic design. This design approval procedure is carried out by designated organizations such as the Building Center of Japan on behalf of MLIT. Each designated organization has formed special committees consisting of experts in various building engineering fields from universities and the Japan Structural Consultants Association. As AIJ-RLB is not a law, structural designers are essentially required to comply with the BSLJ. However, AIJ-RLB has been widely used or consulted by structural designers requiring more sophisticated building designs or for compensating parts not covered by BSLJ.

Very strong seismic excitations such as the 2011 off the Pacific coast of Tohoku Earthquake on March 11, 2011, Magnitude 9.0, maximum recorded ground acceleration 2,933 cm/s^2, have to be considered for structural and building design in Japan. It is also true that Japan has very strong tyhpoons, e.g., Tyhpoon Maemi passed over Miyakojima Island on September 10 and 11, 2003, and a 3 s gust wind speed exceeding 90 m/s was recorded (Cao et al., 2009). For seismic actions, buildings should be lightweight and flexible, but for wind actions, buildings should be massive and rigid. Thus, opposite design requirements apply for seismic and wind actions, and very high levels of both seismic actions and wind actions have to be considered in Japan. In general, the dominant external design load is seismic load for the majority of high-rise buildings, say lower that 200 m high. Therefore, they are basically light-weight and flexible, thus making them vulnerable to winds, and habitability to building vibrations induced by daily winds is enevitably an important issue in Japan.

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Mixture of low-rise, medium-rise and high-rise buildings is another feature of the landscape of Japanese cities, and effects of construction of a high-rsie building on environmental conditions in the surrounding area can be very significant. Thus, environmental assessment of pedestrian-level winds is strongly required for high-rise building construction.

Since the early '70s, unique and significant development has been made in Japan for structural performance against external actions, evaluation of habitability to building vibrations, and wind environmental assessment. Based on these studies, several relevant recommendations and guidelines have been issued and have been utilized by designers. Guidelines for the Evaluatuion of Habitability to Building Vibration published by AIJ (AIJ-GEHBV, 2004) has been commonly used for checking livability or comfort performance of high-rise buildings during daily winds (Tamura et al., 2006). To satisfy target criteria for building habitability, application of damping devices is one of the feasible solutions, and many high-rise buildings in Japan have been equipped with auxilliary damping devices. For environmental assessment, the Environmental Effects Assessment Municipal Bylaw (EEAMB) has been enforced by the Metropolis of Tokyo since October 1981. The EEAMB requires wind environmental assessment based on an appropriately conducted wind tunnel study or CFD analysis for buildings higher than 100 m and having a total floor area of over 10^5 m^2. The EEAMB also recommends two assessment methods for wind environmental evaluations, i.e., Murakami et al. (1983) and WEI (1989). More interestingly, full-scale measurements of pedestrian level winds should be conducted one year before and after construction in order to validate the assessment made in the design stage.

From various wind resistant design aspects, suppression of wind-induced responses is an important issue. As is very well known, crosswind response due to periodic Karman vortex shedding is predominat over along-wind or torsional responses for high-rise or super-high-rise buildings. Therefore, aerodynamic means to prevent formation of Karman vortices, to reduce their intensity and periodicity, and to minimize spatial correlation of shed vortices along the vertical axis are useful. Recently, many high-rise and super-high-rsie buildings with unconventional configurations, such as Burj Khaliha and Shanghai Tower, have been constructed around the world. One reason for their curious and complicated configurations is the advantageous aerodynamic characteristics, especially for the crosswind component (Tanaka et al., 2012).

This paper introduces design principles and special features of three recent examples of high-rise buildings constructed in Japan.

2. Project Example -1:

Performance-Based Wind Resistant Design for a 300 m High Building; Abeno Harukas

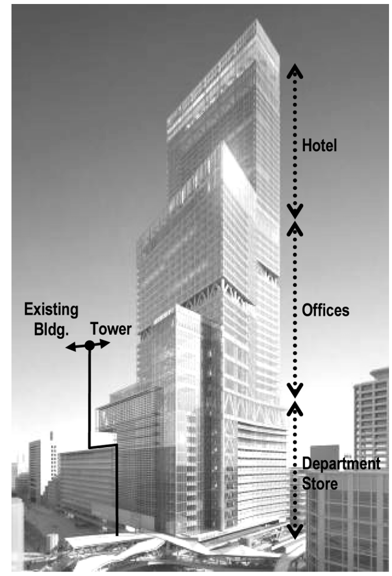


Figure 1. Northwest view rendering.

2.1. Outline of building and structure

"Abeno Harukas" is to be the first building that reaches as high as 300 m in the seismic-prone country of Japan. It is currently under construction, and scheduled to open in 2014. Situated in Abeno, Osaka, the building will accommodate 60 stories above ground and 5 basement floors, and will become a new landmark. See Figs. 1 and 2 for the north-west view rendering and the latest under-construction view, respectively.

The superstructure is composed of three "blocks" having setbacks on the north side. The lower block is for the Kintetsu Department Store, the middle one for offices and the

Figure 2. Latest view under construction.

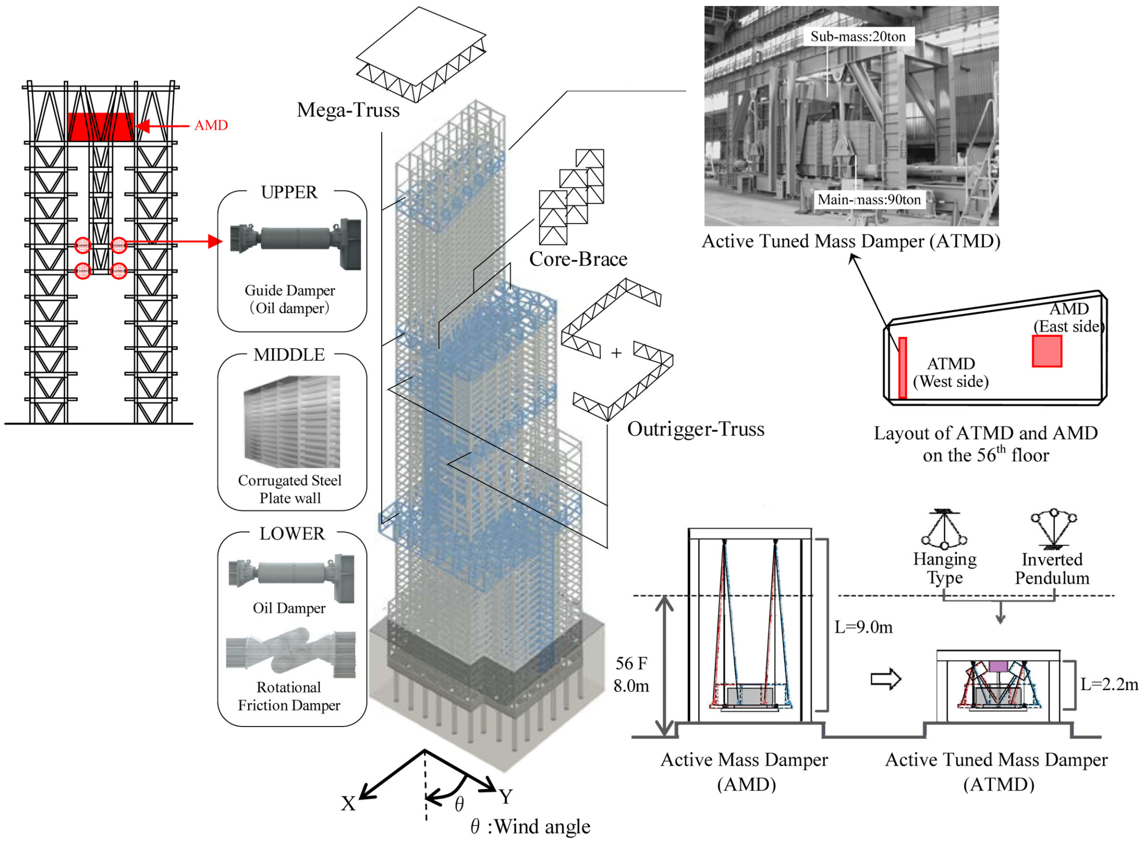


Figure 3. Overall composition of superstructure and damping devices.

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, as shown in Fig. 3.

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 dampers (AMD and ATMD) are installed on the 56th floor in order to improve the habitability mainly of the hotel in the upper block (Fig. 3). Evaluation of habitability against wind load will hereinafter be des-

Table 1. Study items for wind-resistant design

Study item		Design wind speed*	Wind load		Design criteria
			Average load	Fluctuation component	
Safety	Structural framework	V_0 = 34 m/s (Level 1) V_0 = 42.5 m/s (Level 2) (to comply with Notification**)	Wind tunnel test (measured wind pressure)	Spectrum modal; Load combination to comply with Guide.	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 less

*An average wind speed with an interval of 10 minutes at 10 m above ground level.
**Notification related to the Building Standard Law (MLIT-BSLJ, 2007), and the Recommendations for Loads on Buildings (AIJ-RLB, 2004).

cribed in detail.

Details of the performance-based seismic design for this building are summarized in the references (Hirakawa et al., 2011; Nakai et al., 2012). This paper focuses on the performance-based wind-resistant design for “Abeno Harukas” as below.

2.2. Outline of wind-resistant design

Table 1 shows the design wind speeds, criteria and other items studied in developing the performance-based wind-resistant design for this building.

2.3. 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 (Fig. 4). Approximately 600 measuring points were embedded in an acrylic model to measure the wind pressures. Wind direction for wind tunnel tests is defined as shown in Fig. 5.

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. 6. 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 (Fig. 6(b)).

2.4. Calculation of wind loads

The wind loads on all the stories when the base shear is largest at wind angles 175 degrees and 85 degrees for X-direction and Y-direction, respectively, are shown in Fig. 7 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.

2.5. 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 (66.6m/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 degrees.

Nevertheless, aerodynamic vibration experiments were

Figure 4. Wind tunnel test model.

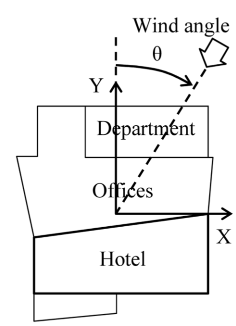
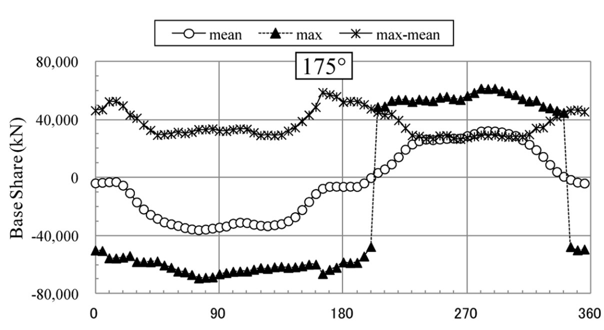
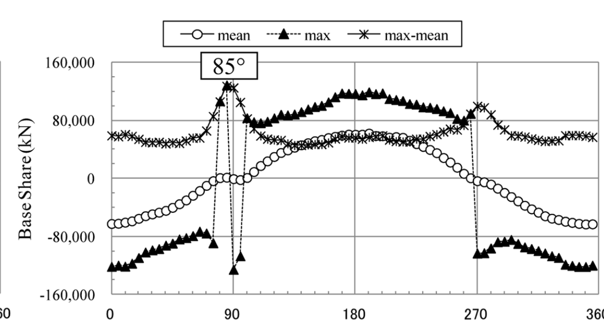


Figure 5. Definition of wind direction.

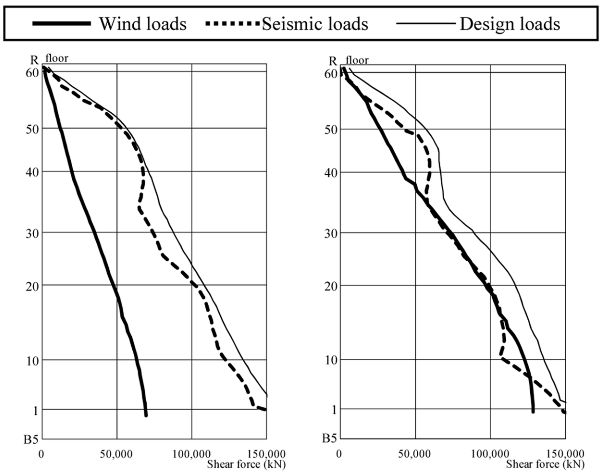


(a) X-direction



(b) Y-direction

Figure 6. Relations between base shears and wind angles.



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

Figure 7. Comparison between wind loads and seismic loads.

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 (Fig. 8). 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. 9.

2.6. Evaluation of habitability

There will be a hotel in the upper block of this building, for which comfortable habitability has to be provided by keeping the response accelerations less than approximately 3 cm/sec^2, at Class H-30 (AIJ-GEHBV, 2004) (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 59th floor to reduce response accelerations in case of strong winds, as shown in Fig. 3.

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 with a suspended length of 9.0 m. 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.

Figure 8. Aerodynamic vibration test using MDOF model.

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. 10. However, the vibration in the wide side (east-west; X-direction) is sufficiently small without mass dampers.

2.7. Conclusion on performance-based wind resistant design

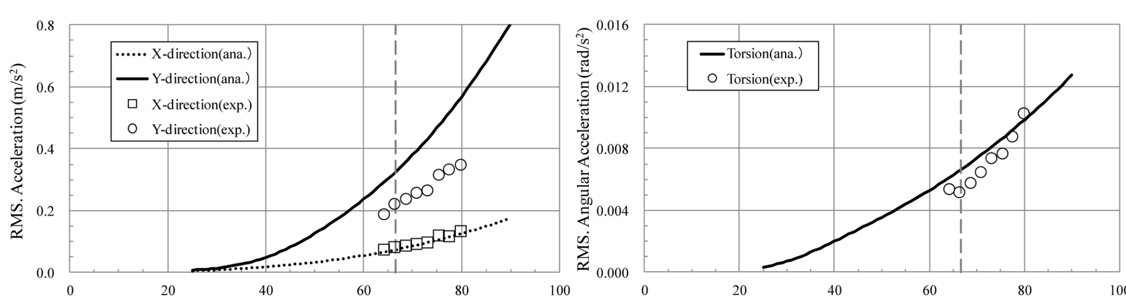
This section introduces the performance-based wind resistant design of the first 300 m-high building in Japan. The building configuration, superstructure systems and various damping devices are sophisticatedly integrated to ensure a higher level of safety and comfort against wind load.

3. Project Example -2:

Habitability Grade against Wind-Induced Horizontal Motion of a 213 m-High Building

3.1. Introduction

The habitability of buildings from the viewpoint of en-



(a) Translational mode (b) Torsional mode

Figure 9. Relation between average wind speeds on building top and acceleration on 57th floor.

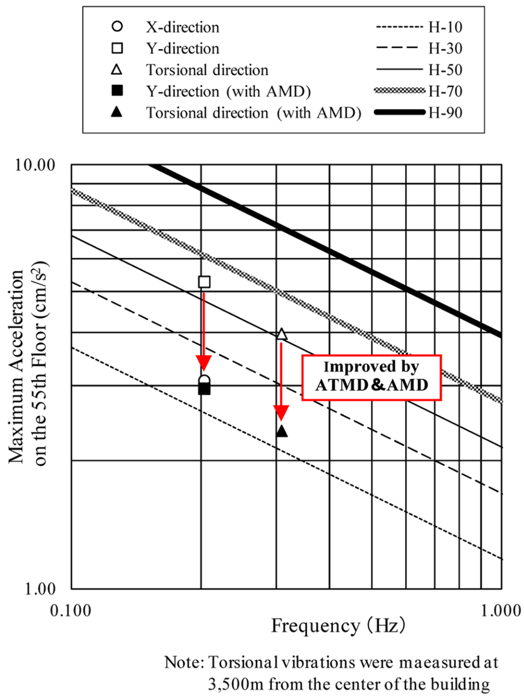


Figure 10. Habitability evaluation of hotel guest room on 55^{th} floor.

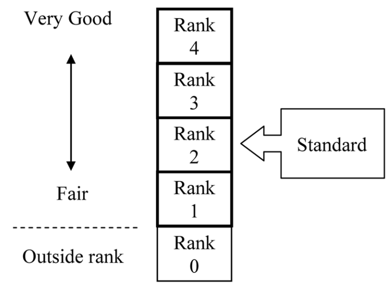


Figure 11. Performance Rank.

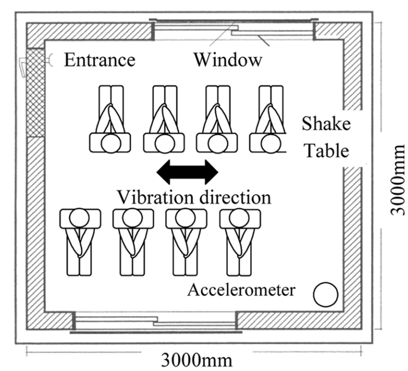


Figure 12. Experimental Situation.

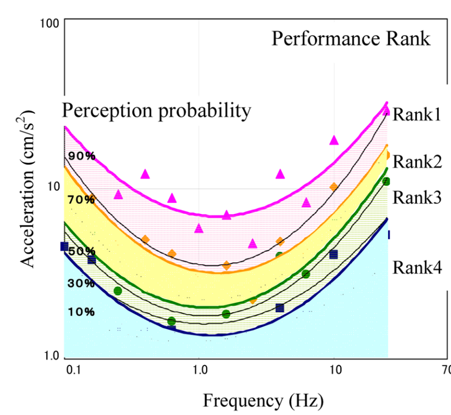


Figure 13. Results of Questionnaire.

vironmental vibration can be evaluated by plotting a performance evaluation curve that is based on perception probability, as explained in the "Guidelines for Evaluation of Habitability to Building Vibration (AIJ-GEHBV, 2004)". However, the design target value and the permissible value are generally assigned by the designer according to his/her judgment, and the quality of buildings might vary depending on the designer. Therefore, to provide a performance rank that links the extent of horizontal vibration to the quality of the living environment on the basis of the perceptions of building occupants, the authors executed vibration tests and carried out an attitude survey to gather data for estimating the performance rank. The results enabled the authors to provide a concrete definition of the performance rank in terms of a standard level and a preferable level on the basis of occupants' perceptions (Yoshida et al., 1991; Okuda et al., 2000; Noda et al., 2010).

3.2. Performance rank

Figure 11 shows the concept of performance rank. The rank is shown by five stages. Rank 0 is considered beyond the scope of daily vibration levels. Rank 1 is Fair, Rank 2 is Standard, Rank 3 is Good, and Rank 4 is Very Good.

3.3. Vibration test

A sensory test was conducted with forty subjects to investigate the relationship between habitability grade and physical amount of horizontal vibration of a building. The subjects answered a questionnaire focusing on views on habitability grade of environmental vibration before and after the test. Figure 12 shows the experimental situations. A 3 $m^2 \times 3$ m-high simulated habitable room was built on a shaking table. The input vibrations were basically a horizontal sine wave applied in a left-to-right direction relative to the subject. 14 frequency options were set ranging from 0.1 to 40 Hz and acceleration maximum values of 1.6~400 cm/s^2. The subjects were asked to respond to 11 questions.

3.4. Habitability grade

The results of the sensory test and the consciousness survey show that the habitability grade of horizontal vibration can be set on the basis of perception probability. The authors propose a new habitability grade of horizontal vib-

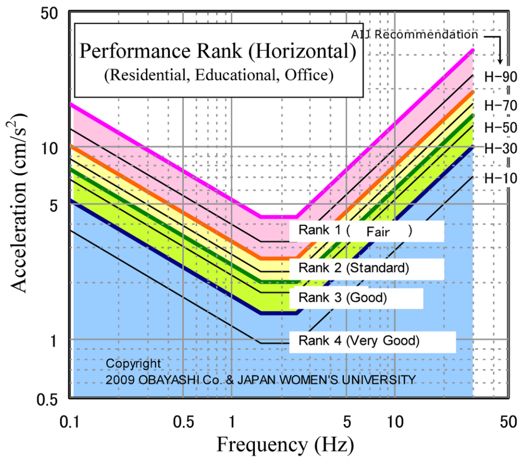


Figure 14. Habitability Grade.

ration based on the AIJ Guidelines (AIJ-GEHBV, 2004) in consideration of this relationship between habitability grade and perception probability. Figure 13 shows the performance rank based on the results of the questionnaire survey. The performance rank was evaluated based on the accumulation answer probability of 84.13%. Rank 2 is perception probability 60~80%. Rank 3 is perception probability 30~60%. Rank 4 is perception probability 30% or less. Figure 14 shows the habitability grade. The habitability grade is shown on the same curve as "Guidelines for Evaluation of Habitability to Building Vibration (Architectural Institute of Japan)(AIJ-GEHBV, 2004)".

3.5. Outlines of building and device

Figure 15 shows a Hybrid Mass Damper (HMD) installed in a building. Two HMDs were installed in the new DENTSU INC. Head Office building in 2002. The Dentsu tower has a unique floor plan shape, which looks like a crescent or a boomerang. Frames spanning 15.9 m are set at 7.2 m intervals along the approximately 140 m-long crescent shape and create a boomerang-shaped office floor on the south side with a service space inside. At the north side, rigid frames around shuttle elevators are connected to the boomerang frames by two-story-high reverse V-braces at the five floors that the shuttle elevators serve.

Figure 16 shows the outline of the composition of the HMD. The HMD consists of an Active Mass Damper (AMD) installed on a Tuned Mass Damper (TMD) supported by four Multi-Layer Laminated Rubber Bearing systems, two tensile spring systems and four compressive spring systems. The compressive springs work to correct the non-linear stiffness of the Multi-Layer Laminated Rubber Bearings. The passive part of the HMD is equipped with an anti-twist mechanism composed of a linear bearing and a braking system that works under large external forces such as large earthquakes. The HMD system acts as a TMD in the right-angle direction, because the Multi-Layer Laminated Rubber Bearing in the HMD has coil

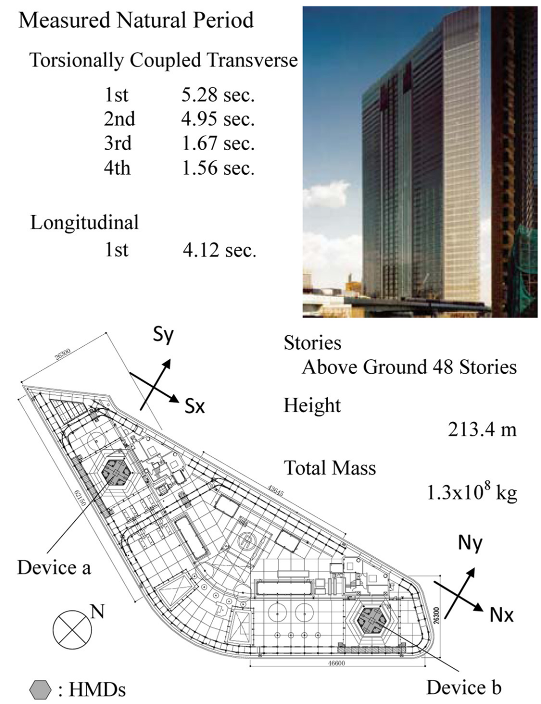
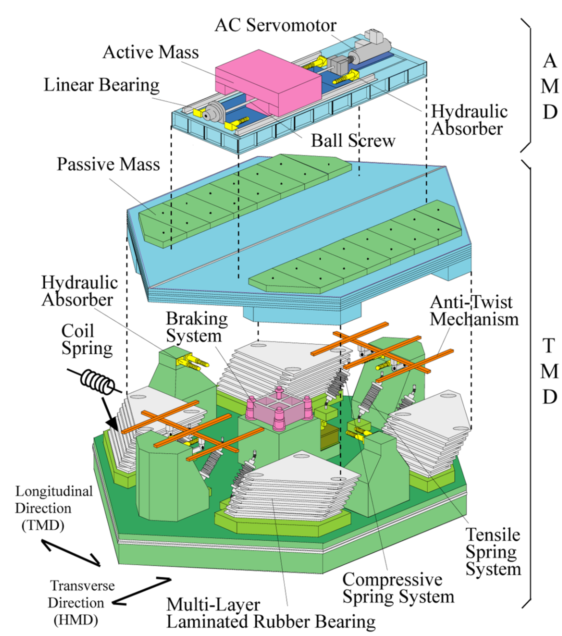


Figure 15. Location of HMDs installed in building.



Mass	AMD:2.0x10^4 kg , TMD:2.0x10^5 kg
Maximum Stroke	AMD:105.0 cm , TMD:70.0 cm

Figure 16. Schematic and Specification of HMD.

springs between the elements of rubber bearings in the right-angle direction to the active one for tuning the period of its direction.

Table 2 shows the damping performance obtained by the free vibration test. It is concluded that the performance of the controlled building meets the design criteria.

Table 2. Damping Performance

	Torsionally Coupled Transverse				Longi-tudinal
	1st Mode	2nd Mode	3rd Mode	4th Mode	1st Mode
Without Control	0.77%	0.92%	0.85%	0.83%	1.15%
With Control	12.7%	9.14%	3.17%	2.99%	3.59%

Table 3. Wind-induced Response Measurements

Year	Acceleration (cm/sec^2)				Mean Velocity (m/s)
	Sx 0.24 Hz	Sy 0.19 Hz	Nx 0.24 Hz	Ny 0.20 Hz	
2002	0.45	4.57	0.45	2.58	12.22
2003	0.59	4.41	0.46	3.24	14.02
2004	0.46	4.58	0.42	3.33	19.27
2005	0.43	3.50	0.37	2.23	15.73
2006	0.73	4.43	0.72	4.18	16.91
2007	1.58	4.21	1.56	4.35	18.22
2008	0.58	4.56	0.57	3.28	15.12
2009	1.22	4.52	1.19	3.58	21.92
2010	0.45	4.57	0.42	2.46	19.48

3.6. Results of evaluation

Table 3 shows accelerations of wind-induced response measurements and mean velocity (2002~2010). Figure 17 shows the results of evaluation based on Habitability Grade. These accelerations showed a performance rank of 3~4 and a perception probability of 10~50%. Thus, the design objective was satisfied. Moreover, the vibration control device (HMD) works effectively.

3.7. Conclusions

This study determined the specific meaning of the habitability grade of horizontal vibration based on residents' consciousness, such as a standard level and the requested level of a majority of people.

The high-rise building performance was verified by using the habitability grade. As a result, it has been demonstrated that the building performance was very good.

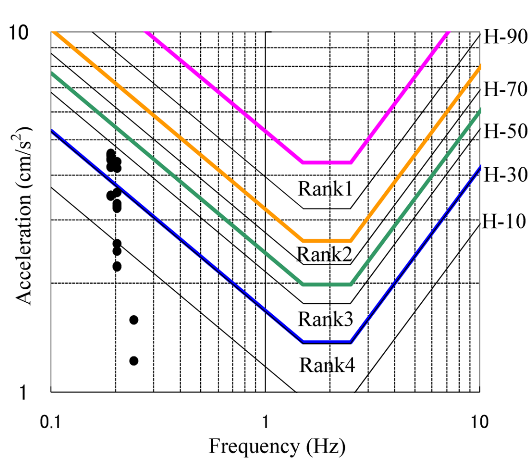


Figure 17. Result of Evaluation.

4. Project Example -3:

Performance-Based Wind Resistant Design for a 634 m-High Tower: Tokyo Skytree

4.1. Introduction

Tokyo Skytree (Fig. 18) is a new core facility for digital broadcasting for the Tokyo metropolitan area of Japan. It is 634 m (2,080 ft) high and is the highest tower in the world for broadcasting, and was completed in 2012. It is expected to be a tourist attraction, a base for broadcasting and telecommunications, and a quasi-disaster prevention centre of the Tokyo metropolitan area.

The requirements for structural designs in Japan are extremely severe, because several typhoons strike every summer and big earthquakes occur with high probability. Consequently, Tokyo Skytree was required to adopt high criteria, exceeding the building regulations in Japan, because of its heavy public responsibility to send valuable information to victims in a big disaster. Furthermore, the structural characteristics of this tower are different from those of other domestic structures, so a new design method had to be invented especially for earthquake and wind resistant design.

The Core Column System, unique system for vibration control, was invented for this tower to satisfy the requirements for structural design. Generally, steel towers have poor damping capacity, and improvement in damping ability was demanded for this tower. The Core Column System uses a core shaft of an emergency staircase comprising a reinforced concrete tube wall as a weight, using the theory of TMD (tuned mass damper).

4.2. Structural planning

This tower varies in silhouette, according to alteration of plan shapes: the bottom floor is triangular and the observatory floor is circular (Fig. 19). Steel structures com-

Figure 18. Tokyo Skytree.

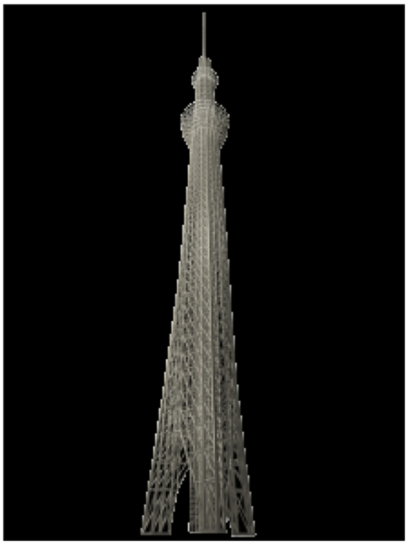


Figure 19. Superstructure.

prising pipe trusses were adopted to decrease weight and area presented to the wind that contributes to decrease power generated by the wind and pressure of residents around who always sense vaguely to the massive structure. Circular sections have fabrication and welding advantages compared with box sections, and make possible a roundish silhouette.

The maximum pipe strength is 630 N/mm^2, the maximum diameter is 2,300 mm, and the maximum thickness is 100 mm (Table 4). The frequency of each member was designed to be large enough to prevent vortex induced vibration up to strong wind "L3": 2000-year return period in Table 5.

4.3. Assumed disturbance

The in-service period for disturbance of the structural design of this tower is 100 years, which is longer than that for an average building in Japan, because this tower is expected to be a quasi-disaster prevention centre of the Tokyo metropolitan area. In addition, this tower has "L3" level criteria defined by the return period of a disturbance that the building regulations in Japan don't require, and that ensure that the tower will resist an unexpected big disaster (Table 5). The "L3" level assumes an earthquake resulting from the activity of hidden faults. Many faults have already been investigated in Japan, but a small earthquake under M6.9 doesn't leave a track on the ground surface. This criterion assumes the existence of such a hidden fault immediately under this site. This assumption is offered by the Japanese government, and geological survey has verified that no fault exists immediately under this site.

The structural safety limit according to the "L2" disturbance (in Table 5) for this tower is almost no damage, and it is the criteria to continue broadcasting and to support revival of victims in a big disaster, and the "L2" disturbance is the maximum level that the building regulations in Japan require for domestic buildings.

The regulations for the vibration velocity in frequent wind were established for the Gain Tower; the top of the tower and the broadcasting antennae.

4.4. Characteristics of aerological wind

The structural design of this tower was based on wind induced response rather than seismic response. It is most important for wind resistant design to define the wind profile of average wind velocity from the ground to the top of this tower. However, the development of boundary layer wind depends on the surface roughness of windward side ground, as shown in Fig. 20. Thus, observation of aerological wind over this site was an essential condition to determine the wind characteristics and to carry out wind resistant design (MC-BCJ-WFHBD, 2002). A wind profile was inferred from previous studies to define with power

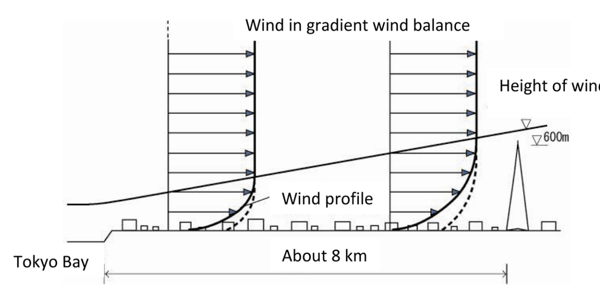


Figure 20. Notion of boundary layer.

Table 4. Maximum size of steel pipe (high performance steel)

Type	Strength (N/mm^2)	Maximum Diameter (mm)	Maximum thickness (mm)
630 N/mm^2 Class	630	1200	80
500 N/mm^2 Class	500	2300	100
400 N/mm^2 Class	400	1900	60

Table 5. Design Criteria

Level	Standards of domestic law	Specification of design for disturbance	Structural safety limit
L1	Rare	Strong wind : Return period = 100 years Earthquake : middle	No damage
L2	Very rare	Strong wind : Return period = 1350 years Earthquake : Big	Almost no damage
L3	Unexpected	Strong wind : Return period = 2000 years Earthquake : Hidden faults	Elastic behaviour

Figure 21. Balloon-launching system.

Figure 22. Observation of wind with GPS Sonde.

law, but no one knew the height of wind in gradient-wind-balance over this site.

First, the observation of aerological wind was planned by a wind profiler and GPS Sonde (Figs. 21 and 22). A wind profiler is an instrument for observing aerological wind velocity using a sound wave, but the sound is too loud to use downtown. Thus, there was no alternative to using only the GPS Sonde. With this method, balloons are released in wind and transmit their position every second by GPS to a base, enabling wind velocity to be easily determined. 50 balloons were launched from the roof of a building near the site, and it was observed that the average wind velocity was constant from 1,000 m to 1,300 m. It is difficult to define the height of wind in gradient-wind-balance because there were too few observations for accurate estimation. However, from this research it was decided to accept a power law under a height of 634 m, the top of the tower, to define the wind profile for wind resistant design.

This site is located in downtown Tokyo, but the surface roughness for wind resistant design is the same as that for the bay area. Enough distance from the coast is needed to develop a boundary layer up to the top of this tower, and this site is only 8 km from the coast. The turbulence effects within the atmospheric boundary layer were extrapolated from previous studies.

Figure 23. Entire wind tunnel test.

Figure 24. Wind tunnel test for portion.

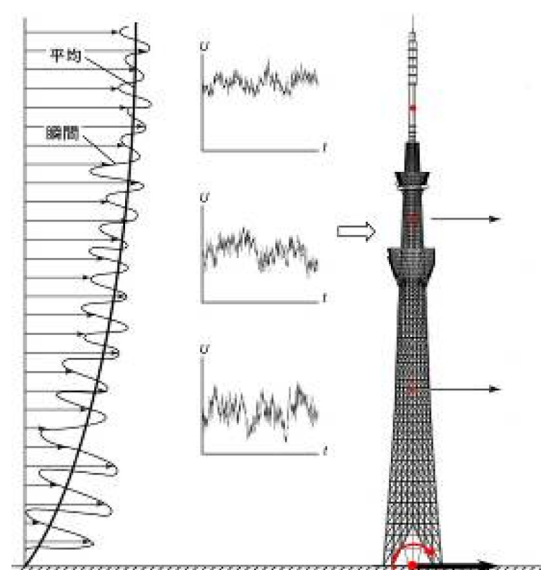


Figure 25. Time history response analysis with artificial wind fluctuation data.

4.5. Verification of structural safety for wind response

Boundary layer wind tunnel simulations were executed that simulate behaviors of this tower against airflow generated as natural wind observed over this site, and the wind

(1) Set characteristics of natural wind (Verification with statistical methods)

・Height of wind in gradient wind balance over this site is defined according to observation of aerological wind. Power law is adopted to define wind profile under height of 634m.

・Turbulence effects within atmospheric boundary layer are extrapolated from previous studies.

・These processes are verification by statistical methods

⇩

(2) Boundary layer wind tunnel simulations (Physical simulation)

・Boundary layer wind tunnel simulations are executed in airflow generated with observation over this site. Aerodynamic force balance tests, aerodynamic vibration tests, full elastic model tests, etc. are executed to verify wind response of tower by physical simulation.

・Materials to create artificial wind fluctuation data are secured from these tests.

⇩

(3) Time history response analysis in strong wind (Logical simulation)

・Time history response analysis is executed to verify wind response, safety of members, effect of vibration control system, fatigue failure of welding, etc. by logical simulation.

Figure 26. Flow of wind resistant design.

response was thus directly verified (Figs. 23 and 24). The stability and wind response were analyzed by time history response analysis with artificial wind fluctuation data that simulated the wind tunnel test results.

Artificial wind fluctuation data were created, targeting the power spectral density of fluctuation components obtained by the overturning moment of the base level in wind tunnel experiments, and this was one of the Monte Carlo simulations (Fig. 25). It is possible with this analysis to verify the safety of members, the effect of the vibration control system, the fatigue failure of welding, etc. The procedure of the wind resistant design developed for this tower is shown in Fig. 26.

4.6. Outline of vibration control system

The structural design of this tower, for example the decision on member sections, is decided from wind induced response rather than seismic response. But it was clarified in basic study that acceleration during an earthquake is too large to operate the instrument for broadcasting unless damping is added as a vibration control system.

As unique systems for vibration control, the Core Column System was invented for this tower to satisfy the severe requirements. Generally, steel towers have poor damping capacity, and improvement in damping ability was demanded for this tower. The core column system uses the core shaft of the emergency staircase built with a reinforced concrete tubular wall as a weight applying the theory of TMD (tuned mass damper).

4.6.1. Tuned Mass Damper on top

The Gain Tower, top of this tower, has to control wind response to ensure reliability of broadcasting. Specifically,

Figure 27. TMD on top.

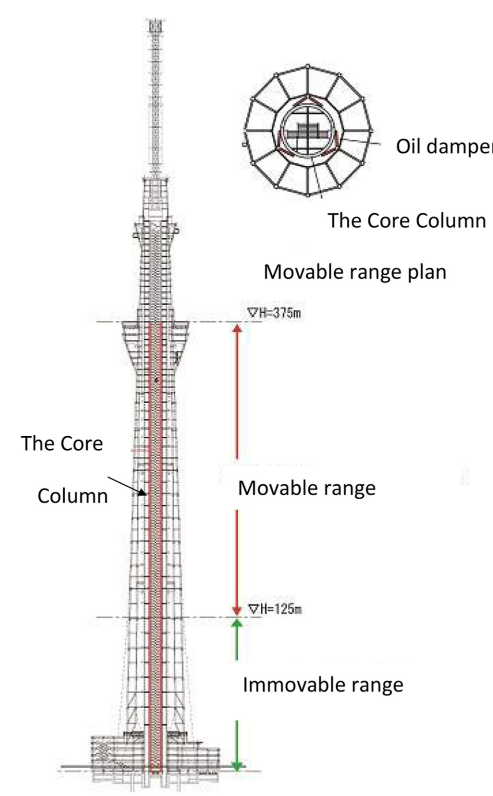


Figure 28. Notion of response control system with the Core Colums.

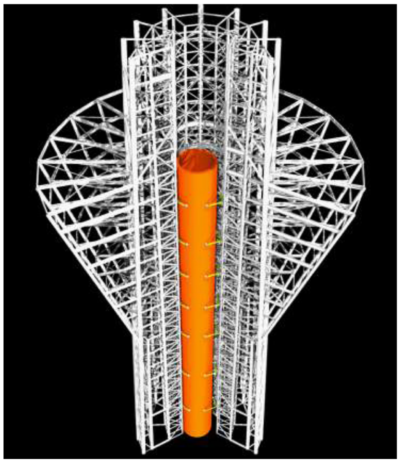


Figure 29. Section of the Core Column.

Figure 30. The Core Column.

velocity response against daily wind has to be controlled under a constant level required for a new digital broadcasting tower. Two TMD systems were installed at the top of this tower: the upper one weighed 25 Mg (25 metric tons) and the lower one weighed 40 Mg (40 metric tons) (Fig. 27).

4.6.2. Response control system with core column

The Core Column System, a unique vibration control system using a core shaft as an added mass, was developed for this tower (Figs. 28, 29 and 30). This column comprised a circular cylinder of reinforced concrete, and had a diameter of 8.0 m, a thickness of 600 cm, and a height of 375 m. It was free from the main steel frame of the tower. The upper half was connected with oil dampers (Fig. 31)

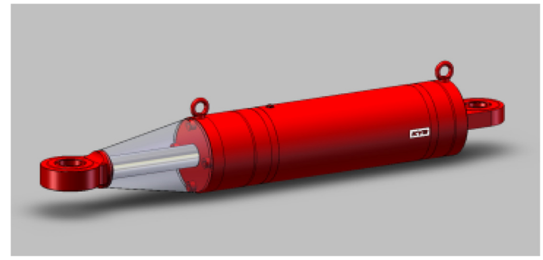


Figure 31. Oil damper.

and the lower half was connected with steel members. Therefore, it is a column but it is independent of the tower and doesn't support the tower's weight. This vibration control system is effective over a wide range of earthquakes. It can reduce the acceleration response during an earthquake by a maximum of 50%, and a that during strong wind by a maximum of 30%.

4.7. Conclusions

Tokyo Skytree is a new core facility for digital broadcasting for the Tokyo metropolitan area of Japan, and it requires strict design criteria because of its heavy public responsibility to send valuable information to victims of a big disaster.

The maximum disturbance for the structural design of this tower is the strong wind of 83 m/s for 10 minutes mean value at its top, and the structural safety limit for the disturbance is elastic behavior.

The Core Column System, unique system for vibration control, was invented for this tower to satisfy these requirements. The Core Column System uses the core shaft of the emergency staircase comprising a reinforced concrete tubular wall as a weight applying the theory of TMD (tuned mass damper). This vibration control system is effective over a wide range of earthquakes. It can reduce the acceleration response during an earthquake by a maximum of 50% and that during strong wind by a maximum of 30%.

5. Conclusions

In this paper, the design approval system and procedure for high-rise buildings and sructures in Japan were firstly introduced. Then, design principles, special features and performance-based wind-resistant designs of two kinds of high-rise buildings and world's tallest broadcasting tower constructed in Japan were summarized.

References

AIJ-GEHBV. (2004). "Guidelines for the Evaluation of Habitability to Building Vibration." *Architectural Institute of Japan*, Maruzen, pp. 132. (in Japanese)

AIJ-RLB. (1993, 2004). "Recommendations for Loads on Buildings." *Architectural Institute of Japan*, Maruzen, pp. 651. (The English version of AIJ-RLB-2004 was published in

2006)

Hirakawa, K., Maeno, T., and Saburi, K. (2011). Performance Based Design of 300 Meter High Building, *Proc. Structural Engineers World Congress 2011*, High Rise Buildings, ID29, April 2011, Como, Italy.

MC-BCJ-WFHBD. (2002). “The Wind Force for Hyper-Building Design.” *Ministry of Construction*, *The Building Center of Japan*, *Hyper-Building Laboratory* (in Japanese)

MLIT-BSLJ. (2007). “The Building Standard Law of Japan.” *Ministry of Land, Infrastructure, Transport and Tourism (MLIT)*, *Japan*.

Murakami, S., Iwasa, Y., and Morikawa, Y. (1983). “Investigation of statistical characteristics of wind at ground level and criteria for assessing wind-induced discomfort.” *Transactions of the Architectural Institute of Japan*, 325, pp. 74~84 (in Japanese)

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*, 1(3), pp. 155~167.

Noda, C. and Ishikawa, T. (2010). “Habitability grade of horizontal vibration based on residents' consciousness.” *Journal of Environmental Engineering* (Transactions of AIJ), 75(648), pp. 131~137.

Okuda, H. and Kageyama, M. (2000). “Experimental study on the optimum vibration control of AMD with variable gain under stroke constraint and with spill-over prevention.” *Journal of Structural and Construction Engineering*, AIJ, No. 532, pp. 87~94 (in Japanese)

Tamura, Y., Kawana, S., Nakamura, O., Kanda, J., and Nakata, S. (2006), Structures and Buildings, *Proc. of the Institution of Civil Engineers*, 159(SB5), pp. 283~293.

Tanaka, H., Tamura, Y., Ohtake, K., Nakai, M., and Kim, Y. C. (2012). *Journal of Wind Engineering and Industrial Aerodynamics*, 107-108, pp. 179~191.

WEI. (1989). *New Guide to Wind Environment*, Edited by Wind Engineering Institute, Kajima-Shuppan. (in Japanese)

Yoshida, K., Suzuki, T., Kageyama, M. and Nobata, A. (1991). “Active vibration control using dynamic vibration absorber driven by servo motor.” *Colloquium on Control of Structures*, JSCE, Part. B, pp. 299~308.