Proceedings of The Twelfth (2002) International Offshore and Polar Engineering Conference
Kitakyushu, Japan, May 26−31, 2002

ISBN 1-880653-58-3 (Set); ISSN 1098-6189 (Set)

Dynamic Response Analysis of Onshore Wind Energy Power Units during Earthquakes and Wind

Osamu KIYOMIYA
Fellow of JACE, Professor, Department of Civil Engineering, Waseda University
Tokyo, Japan
Tatsuomi RIKIJI
Master student, Department of Civil Engineering, Waseda University
Tokyo, Japan
Pieter H.A.J.M. van GELDER
Senior Scientific Staff Member, Department of Civil Engineering, Delft University of Technology
Delft, the Netherlands

ABSTRACT

Load combinations of wind forces and earthquake forces for wind energy power units are investigated from a probabilistic point of view When large-scale earthquake events are taken into consideration in structural design, wind forces at the mean wind velocity during service life can be added to the earthquake forces. Dynamic response analysis is carried out to estimate sectional forces at the wind energy power units. It is made clear that the tower has fairly large seismic capacity when the tower is designed by wind force at strong wind velocities due to typhoons.

Key Words: Wind energy power unit, Dynamic response analysis, Wind force, Earthquake force, Sectional force, Load combination

INTRODUCTION

Design loads of wind energy power units on land are mainly dominant by the lateral wind force. When wind farms are constructed at earthquake activity areas such as in Japan, earthquake loads should be taken into consideration in the structural design. In general, two extreme events seldom happen at the same time and may be ignored in the structural design. However, how to combine these loads is still unclear, even though the wind energy power units are always subject to the wind forces. Therefore, in this paper, the combination of the wind load (not only strong wind but also weak wind) and the earthquake load for wind energy power units is studied by a probabilistic procedure. Dynamic response analysis by finite element method can be adopted to know the effect of the load combination caused by two simultaneous loads. In this research, the wind forces are assumed to be static and, on the other hand, the earthquake forces are considered to be time dependent.

OUTLINE OF WIND ENERGY POWER UNIT

An example of an offshore wind farm in Denmark is shown in Photo.1.

Photo. 1 Offshore wind energy power units.

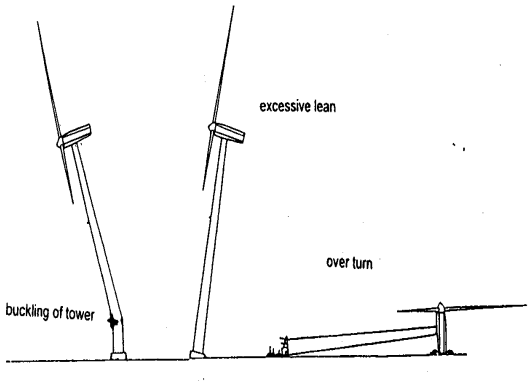

Fig.1 Collapse paterns of wind energy power units

For the wind forces and the earthquake forces, many different failure modes should be discussed for each component of the wind energy power units. Structural failure of the tower, foundation, the blades, the turbine, etc., as well as fatigue of parts of the machines are estimated for the performance of wind energy power units. For earthquakes, the wind energy power units should not overturn and collapse as shown in Fig.1. Also excessive lean of the tower gives low electric power and the possibility of contact of the blades with the tower. After a detailed investigation and repair if necessary, the wind energy power units should work again without reconstruction or large-scale repair for ordinary scale earthquakes.

LOAD COMBINATION

Outline of the load combination

The load combination problem consists of finding an equivalent loading system to present the effect of two or more stochastic loads (Sakamoto, 1998). An additive manner for the load combination is adopted in the conventional design. In general, wind loads are dominant in the structural design and earthquake loads may also be a dominant load in vigorous earthquake activity areas. The wind forces and earthquake forces should be chosen from the above various loads in Japan where typhoons are existing and earthquake activity is very vigorous. When wind energy power units are installed offshore, wave forces, tidal forces, dynamic water pressure during earthquakes, etc. should be considered. For the design method the following two combinations are adopted for wind energy power units. Temperature load is not considered at extraordinary load condition. The earth pressure for the foundation is related to design of the foundation and is not considered in the tower model part. When the wind energy power units are installed onshore or offshore, the wave force can be adopted.

$$Ltotal = Lrwind + Lrwave + Leq \quad (1)$$

Where,

Ltotal is the total force, *Lswind* is the wind force during storm (this gives the maximum wind force) and *Lswave* is the wave force. *Lrwind* is the wind force at the wind velocity for the serviceability condition, *Lrwave* is the wave force for the serviceability condition and *Leq* is the earthquake force. However, how to determine the wind forces during the earthquakes has not been discussed. By rough approximation, the maximum load X of two loads X_1 and X_2 is derived from Turkstra's rule in Eq. (2):

$$max\ X = max[(max\ X_1 + X'_2) : (X'_1 + max\ X_2)] \quad (2)$$

Load Modelling by Statistics

The data of an observed wind time history are best fitted by a two-parameter Weibull distribution as:

$$f_V(V) = (\beta/\alpha)(V/\alpha)^{\beta-1} EXP(-(V/\alpha)^{\beta}) \quad (3)$$

Where,

$f_V(V)$: Probability of occurrence of wind speed V

V : the wind speed

α and β : scale parameter and shape parameter (≈2.0), respectively.

Fig.2 shows the probability density of occurrence of the mean wind speeds at observation sites at the North Sea (Coelingh et al, 1996).

Th database at the North Sea contains records for three hours periods of every winter and summer from 1998 until 1995 (Coelingh, 1996), and (Repko et al, 2000). The mean wind velocities during the observation are about 8-10m/s. The α-factor is 11.0 and β-factor of Weibull distribution is about 2.1. Input ground motion of earthquakes in the design is based on past observations that give information on acceleration wave forms and amplitudes of the acceleration waves (Nozu et al., 1997).

For determining the peak value and the wave form in design work, there are two ways. For a level 1 earthquake, a probabilistic analysis is adopted. Maximum acceleration amplitudes of each large earthquake at a certain site is fitted to a Weibull distribution.

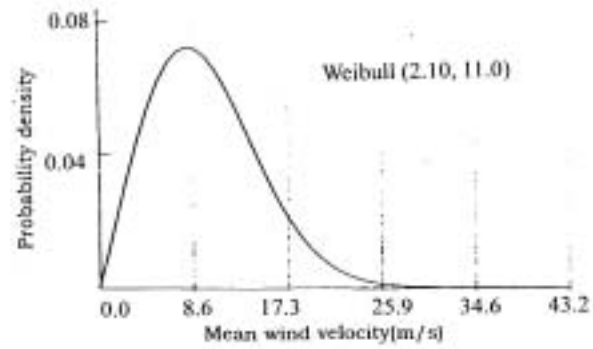

Fig.2 Weibull distribution of the wind speed at North Sea

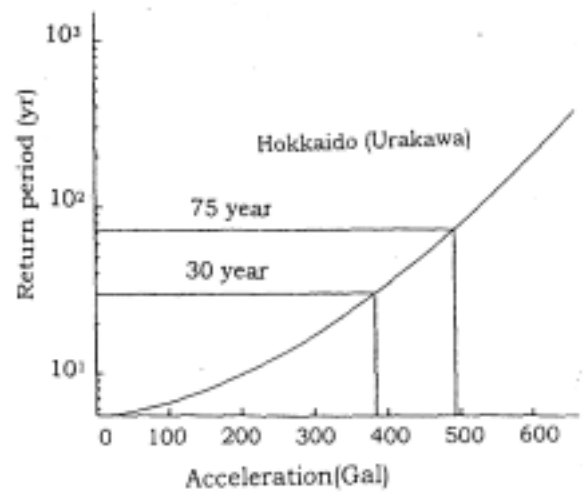

Fig.3 Relationship between return period and acceleration of the earthquakes at Hokkado

Maximum acceleration at a return period of 75 years during a 50 years life time of the wind units is estimated from the Weibull distribution. The Weibull distribution function (*f(x)*) is here calculated by 40 events at Hokkaido which is located in the northern part of Japan. α=1.66 and β=279 are estimated by a fitting calculation of the Weibull distribution. From the cumulative function ($F(x)$) obtained by annual maximum data series, the return period (T_R) can be estimated.

$$F(x) = 1 - EXP(-(x/\beta)^{\alpha}) \quad (4)$$

$$T_R = T_{ob} / N_{ob}\ (1 - F(x)) \quad (5)$$

Where,

T_{ob} ; the observation term (years)

N_{ob} ; No. of occured earthquake events.

75 year return period is adopted for a 50-year service life for ordinary facilities, and a 30-year return period may be adopted here for 20 years for the wind energy power units in point of same occurrence probability as ordinary facilities. Fig.3 shows relationships between return periods and maximum ground accelerations. Maximum acceleration (980 Gal = 980 cm/s^2) in Hokkaido is 490 Gal for a 75 year return period and 380 Gal for a 30-year return period, respectively.

Results of combining loads

Exceeding probability at a certain value for the earthquake events or the wind events is obtained from its joint probability density. The occurrence probability is simply calculated from multiplication of both probability densities. Fig. 4 shows relationships between the mean wind velocities and return periods of two events (equivalent return period: earthquake of 30 years return period is considered wind events). When the mean wind velocity is 0 m/s, the return period of two events is 30 years which is the same as that of only earthquake events. When mean wind velocities increase, the equivalent return periods become longer. When the mean wind velocity is about 8 m/s, the equivalent return period is estimated to be about 50 years.

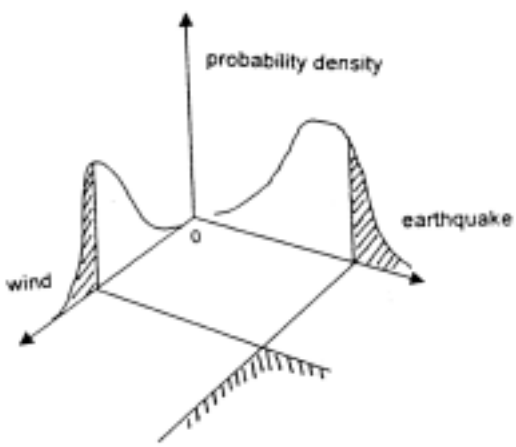

Fig.4 Concept of simultaneous occurrence Probability

In this mean wind speed range, this figure shows that the probability of two events occurring at the same time is fairly small.

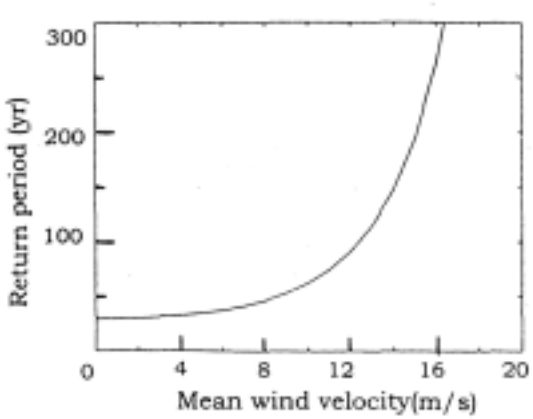

Fig. 5 Return periods of mean wind velocities (given a 30-years earthquake event).

This means that a 50 years equivalent return period of the combined events is the same as a 50 years earthquake event. The equivalent return period becomes very large when the mean wind velocities are over 25 m/s. It can be said that superposition of two events should not be taken into consideration in structural design work.

STRUCTURAL DESIGN METHOD

Model of wind force

The wind forces act on the tower, the nacelle and the blades. The wind forces are chosen for blade's conditions according to the wind speeds as follows:

- cut-in wind speed: the blades are not rotating (idle) for very low wind speeds until 3-4 m/s
- resonance: the blades are rotating and the tower is vibrating by induced vortexes of low wind speeds of several m/s
- rated wind speed: the blades are rotating with full electric power energy in high wind speeds of 11-15m/s
- cut-off wind speed: blade rotation is stopped compulsorily due to change of the blade angle to stall the blades at a wind speed of about 25 m/s
- wind speed at storm: the blade rotation is stopped and the nacelle side faces wind direction. Yaw system possibly does not work by breaking. The wind speed at storm is determined by the design wind speed for a 50 years return period.

A wind velocity *(V)* at z m height is presented by Eq. (6). The wind velocity distribution is presented by an adiabatic profile.

$$V(z)=1.4\ V_{ref}\ (z/z_{hub})^{0.11} \qquad (6)$$

Where,

V_{ref} : the ultimate wind velocity for 0.02 annual probability (50 yr. return period) at the site

z_{hub} : height of the hub location.

Estimation of the wind force on the blades is not easy because the blades are rotating with a constant speed due to fluctuation of the wind speeds and with aero-elastic behaviour of the blades. The simplest estimation of a static horizontal airflow load *(F_0)* is adopted here and is obtained by the next equation.

$$F_0=300A \qquad (7)$$

Where,

F_0 is in unit of N (Newton)

$A=\pi\eta R^2$: sweep area of the rotor

R : rotor radius (m)

η : nominal efficiency<0.9

This value (η) is sometimes proposed by the fabrication company of the wind energy power units.

The tower in the calculation model can be separated from the foundation model and the base of the tower is assumed to be a fixed connection. Almost all large wind energy power units are fabricated by tubular pipes, which are manufactured in sections of 20-30m length with flanges at either end. The tubular pipes are bolted together at the construction site. The tower is of conical shape and its diameter increases towards the tower base. According to the bending moment distribution, the diameter of the tubular pipe and thickness of the pipe are determined and the base part has a large diameter and thickness of the pipe.

The wind force from the blades gives mainly four force components at the top of the tower as shown in Fig.6 (DNV 2001). Only the

horizontal force(F_{yt}) is considered in this paper because this component mainly gives the large sectional forces. Sectional loads in the tower at arbitrary height (h) can be calculated from the loads applied to the nacelle location and the horizontal wind forces applied to the tower.

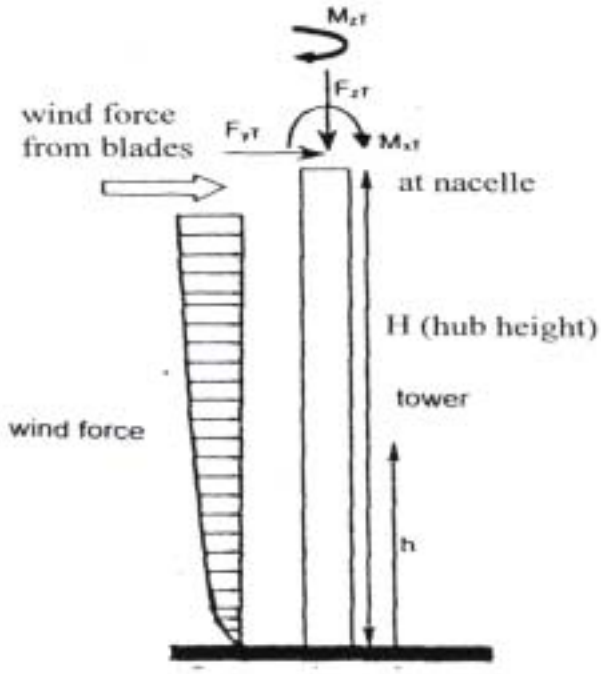

Fig.6 Calculation model of wind energy power unit

When the wind energy power units are constructed at open field such as onshore, grass land, etc. the power in Eqn. (6) is taken as 0.11 and Gust factor as 1.8-2.0, respectively in the Japanese building code. Gust factor is 1.8 above a height of 40m and 2.0 at a height of 10m. For heights between 10m and 40m, *Gr* is obtained by interpolation (Lungu et al., 1996). *C (z)* depends on the Reynold's number and is adopted to be 1.2 for the nacelle. The drag coefficient for the tower is also determined by its configuration and dimension. The tower is divided into several sections and the drag coefficient for each section is determined as a cylindrical configuration. The drag coefficient for the tower is adopted to be 0.5-0.9.

DYNAMIC RESPONSE ANALYSIS

Calculation model

Fig. 7 shows the outline of a calculation model by the finite element method for a wind energy power unit supported by the pile foundation (Rikiji, 2001). In this research, the wind energy power unit is assumed to be constructed at shore. Therefore wave force is nit considered. The model consists of a wind energy power unit, pile foundation and surface layer. A structural model with a 1650kW wind energy power unit is chosen. Height of the tower (hub height; from ground surface to the nacelle axis) is 60m. The tower is fabricated with steel tubes. The diameter of the lower part of the tower is 4.025m and that of upper part is 2.31m. Thickness of the steel tube is 24mm at base part and its thickness decreases along height to 10mm at the nacelle connection part. The steel material is assumed to be SM400 and its yield stress is 231N/mm^2. The yield moment of a whole section of the tube at base of the tower is 68000kN·m. The yield moment at top of the tower is 9500kN·m. Maximum yield shearing force at the base of the tower is 40250kN and at top of the tower 9640kN, respectively. Three blades with diameter of 66m are made of light metal and the projected area of the blades is 167.70m^2. The weight of the blades is 225.56kN and that of the nacelle 559.00kN, respectively. The weight of the tower is 892.24kN.

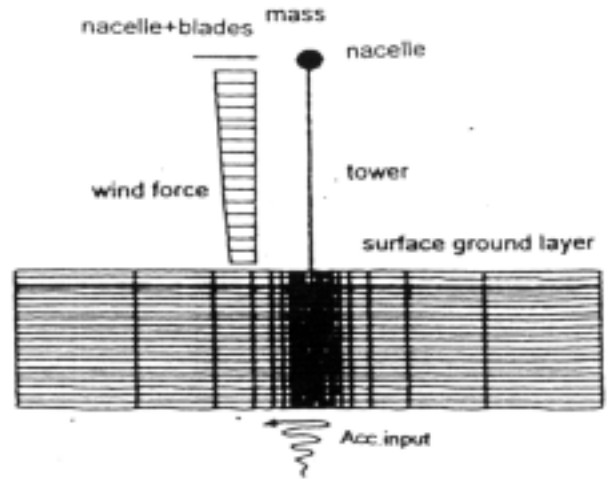

Fig.7 A dynamic response model with finite elements

In the model, the tower and the pile foundation are replaced by beam elements. Number of beam elements of the tower is 21. The nacelle and blades are replaced by a concentrated mass with 80.05ton at the top of the tower. Damping of the tower is assumed to be 2%.

Model of surface layer

The soil layer to -14m from surface is fairly soft with average standard penetration N-value of 2-4. The surface layer above -40m is replaced by 420 plane strain elements with rectangular shape. Material non-linearity of the soil is represented by the Ramberg-Osgood relationship that can express the hysteresis pattern of the soil. Base rock surface (engineering orient base) is located at –32m depth.

Yield strain of the Ramberg-Osgood relationships is obtained from 50% decrease of the initial shear modulus at relationships between shear modulus and strain for each soil layer.

Model of foundation

Foundation of the wind power energy unit in the model is pile type made of reinforced six concrete piles with 1.2 m diameter and 30m length. The piles were cast into a very hard sand layer at -32m depth. 28 longitudinal reinforcement bars with 29 mm diameter are arranged and stirrups with 16mm diameter at an interval of 150 mm are also arranged in concrete piles to -6.4m depth. No. of reinforcement bars is decreased at the lower part of the piles. The ordinary type concrete is cast and its compressive design concrete strength is 24N/mm^2. Bending yield moment at top of a pile is 1588 kN·m and the ultimate bending moment is 2478 kN·m respectively. Non-linear property of the pile is represented by the Takeda model that can express deterioration and hysteresis pattern of the piles during earthquakes. However, the strength of a pile foundation is fairly larger than that of the tower in this model.

Calculation procedure

Static wind forces and an acceleration wave form are used for input of the external forces to the model of the wind energy power unit. An acceleration wave form is input for the base rock location at – 40 m in the dynamic response analysis model. The Port Island wave form as shown as Fig.8 is adopted for the input acceleration record. The maximum acceleration amplitude of the wave form is ranging from 0 Gal to 679 Gal with an interval of 100 Gal. This input acceleration record was obtained at a depth of -82m at Port Island during the Hyougoken-Nanbu Earthquake in 1995 and this wave form is recommended to use for the dynamic response analysis in the new design code of the port and harbour facilities (Japan Port 1999).
The dynamic response of both the structures and the surface ground is obtained through integration of Eq. (8) in the time domain during 15 s.

$$m\ddot{y} + c\dot{y} + ky = -m\ddot{a} + W \qquad (8)$$

Where,

$\ddot{y}$:acceleration
$\dot{y}$:velocity
y : displacement
$\ddot{a}$: input acceleration
W : static wind force
M : mass
C : damper
K : rigidity

The numerical integration procedure of Newmark-β is adopted to integrate the equation.

In this study, wave force is not considered because the wind energy power unit is assumed to be constructed at back side of the breakewater caissn or on the breakwater caisson.

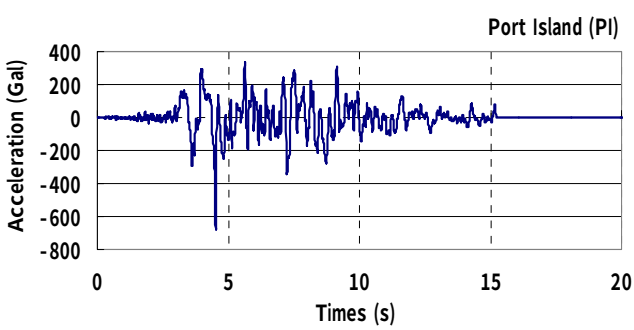

Fig.8 Input wave form at Port Island in 1995

CALUCULATION RESULTS BY DYNAMIC RESPONSE ANALYSIS

Response of structure and ground surface

From an eigen value analysis of the finite element model, the first order of vibration of the wind energy power tower unit including the surface layer is 0.41Hz, 2nd is 1.32Hz and 3rd is 1.82 Hz, respectively. Comparatively a low frequency is dominant in the calculation model.
Fig. 9 shows relationships between the input maximum acceleration amplitudes at the base rock and the maximum accelerations at the top of the tower for each calculation. The maximum accelerations calculated at the tower by the dynamic response analysis are nearly proportional to the maximum input accelerations. This is reason why the rigidity of the foundation is fairly large and the tower does not show non linear region of the materials.

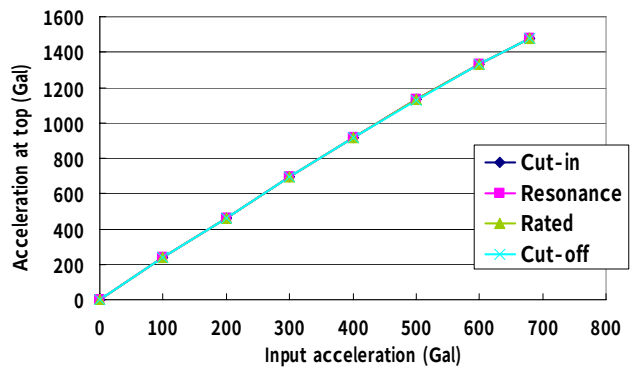

Fig.9 Relationships between accelerations of the tower and input accelerations

An acceleration of more than 1000 Gal is calculated at the top of the tower when the input accelerations exceed over 450 Gal. Acceleration of the tower is not effected by the static wind conditions in this calculation procedure.

Fig. 10 shows the acceleration wave form at the top of the tower for an input maximum acceleration of 679 Gal. About 1500 Gal is calculated at the top of the tower. Waves with frequency components of 0.4 Hz and 1.3 Hz dominate at the top of the tower. These frequencies are the same as those by the eigen value analysis of the calculation model in Fig. 8.

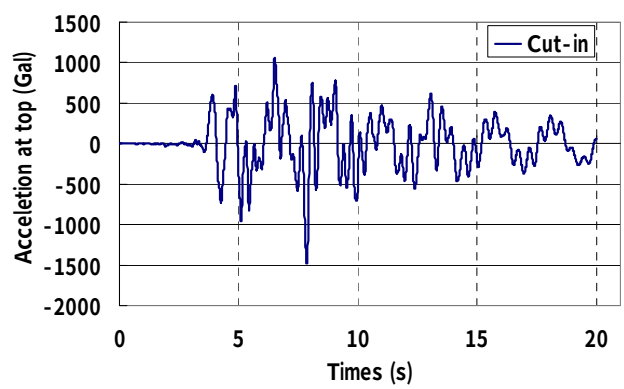

Fig.10 Acceleration at top of wind energy power unit

Fig.11 shows relationships between the input maximum acceleration amplitudes at base rock and the relative maximum displacements at the top of the tower. The displacements of the tower are almost proportional to the input acceleration amplitudes because non-linearity of the materials is not large.. The displacement at the top of the tower is about three times of that at the base of the tower. The displacements by the wind forces include the displacements at the top of the tower in this figure.

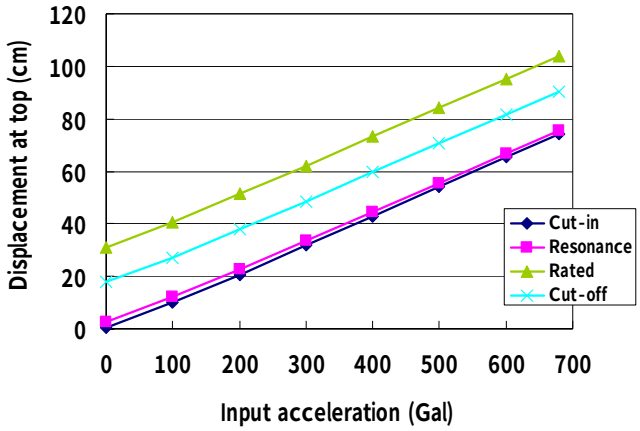

Fig.11 Relationship between acceleration and displacement at top

In general, wind force at cut-off wind speed is smaller than that at rated wind speed because the blade is stalled over cut- off wind speed.Fig. 12 shows a relative displacement wave form at the top of the tower for the maximum input acceleration of 679 Gal. The waveform has a sine shape, in which the low frequency of 0.5 Hz dominates. The maximum relative displacement is about 75 cm at the top of the tower. Inclination of the tower caused by vibration is assumed to be within 1% and this value may therefore give no effect on the blade rotation. Within 5%, rotated blades do not contact to the tower even though flexible material is used for the blades..

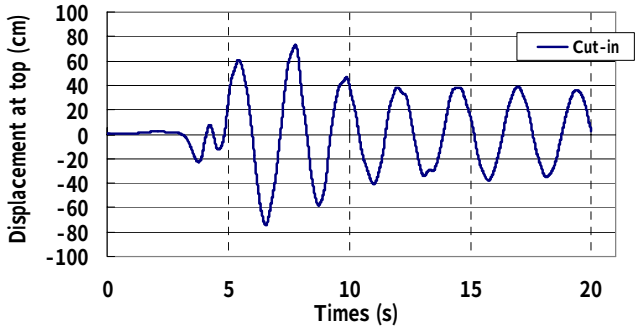

Fig.12 Displacement wave form at top

Calculated sectional forces

Fig.13 shows the relationships between the input accelerations and the bending moments at the base of the tower. The maximum bending moments at the base of the tower are nearly proportional to the input acceleration amplitudes. The bending moments due to earthquakes are added to those by wind forces in the figure. When the input acceleration is 0 Gal, only the wind forces are estimated at the base of the tower. Fairly large values of the bending moments are calculated at the tower when the input accelerations are large.

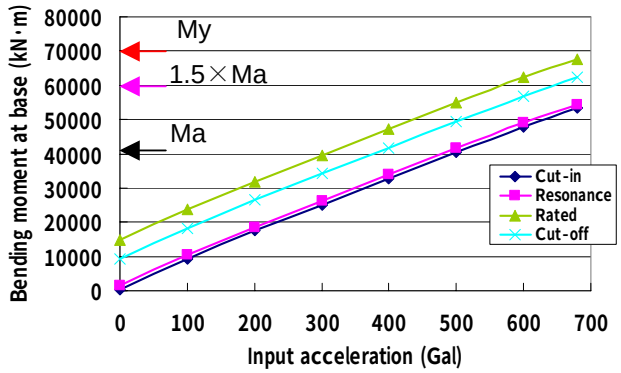

Fig.13 Relationships between bending moments at base of the tower and input accelerations

My in the Fig.13 shows yield moment at base of the tower. The maximum bending moment in the calculation is not larger than the yield moment. Ma shows allowable bending moment at the tower. Ma is calculated from allowable stress of the steel for ordinary loads combination. Allowable bending moment at the earthquake is 1.5 time of that of the ordinary condition. This tower is designed for the strong wind condition by allowable stress method. When input acceleration is larger than 600 Gal at rated wind condition, calculated bending moment is larger the allowable bending moment.

Fig.14 shows time histories of the bending moment at the base of the tower for an input acceleration of 679 Gal when the wind velocity is in cut-in condition.

The maximum bending moment by the calculation is 53000kN·m and a frequency of 0.43 Hz dominates in this wave form.

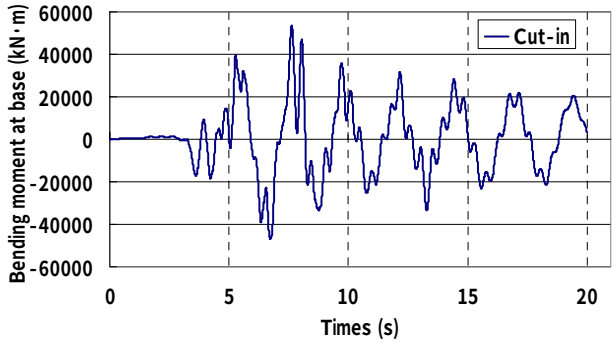

Fig.14 Wave form of bending moment at base of the tower

Considerations

The mean wind velocity during earthquakes should be adopted in view probabilistic considerations (Turkstra's rule). The rated wind velocity and cut-off wind velocity conditions are not necessary to be considered in the structural design when earthquake force is considred. At Hokkaido in Japan, the maximum acceleration amplitude at the base rock for level 1 earthquakes is 380 Gal for a 30 year return period and 490 Gal for a 75 year return period respectively. If a level 2 earthquake is considered in the design, the maximum acceleration value may be taken to be about 570 Gal at the base rock. The bending moment at the base of the tower is calculated to be about 50000 kN·m for such a level

2 earthquake. The dimension of the wind energy power unit in this calculation model is determined by the allowable stress of a steel plate (140N/mm^2) for a design mean wind velocity of 34 m/s. The bending moment (*Ma*) for the allowable stress is 41400 kN·m in storm conditions and 62160 kN·m in earthquake conditions (x 1.5 times the storm condition). The yield bending moment (*My*) at the base of the tower is 68380 kN·m, therefore the bending moment at the base of the tower in the calculation is within the allowable bending moment and within the yield bending moment for even a level 2 earthquake. Even though the rated wind condition is considered for a dynamic response analysis, the tower may not be damaged. Therefore, it can be said that the wind energy power units have fairly large safety for even large-scale earthquakes of both level 1 and level 2. In other words, sectional dimensions of the tower are mainly determined by strong wind conditions such as typhoons and so on according to the calculations in this paper.

CONCLUSIONS

The results of this study are summarized below.

(1) Mean wind velocities and large-scale earthquake events are simulated from a Weibull distribution. The occurrence probability of the mean wind velocities and large-scale earthquake events can be calculated from these Weibull distributions. The probability of the simultaneous occurence of storms and large-scale earthquakes is extremely small.

(2) When large-scale earthquake events are adopted for structural seismic design of wind energy power units, the mean wind speed of the Weibull distribution, at most, is a reasonable value to combine with the earthquake events. Inversely, earthquake forces can be ignored when wind speed during storm conditions is considered for the structural design.

(3) The bending moment at the base of the tower does not reach the yield moment for the Port Island wave with an acceleration amplitude range until 679 Gal for the resonance wind condition.

(4) The wind energy power unit that is designed for very strong winds such as typhoons has a fairly large seismic capacity for both level 1 and level 2 earthquakes.

Further research is needed to obtain a more rational design work of the wind energy power units when both the wind forces and the earthquake forces are taken into consideration, in particular:

•Dynamic analysis is required to consider a dynamic behaviour of not only the earthquake but also the wind.

•Other types of foundations beside the pile foundation should be checked whether the seismic capacity is enough or not. With a gravity type foundation, inclination or movement of the tower may occur during large earthquakes.

•Sea wave forces should also be considered for the load combination in a structural design of offshore wind farms.

REFERENCES

A. Nozu, T. Uwabe, H. Sato, and T. Shinozawa (1997). "Relation between Seismic Coefficient and Peak Ground Estimated from Attenuation Relations",*Technical Note,893 of Port & Harbour Research Institute*, Ministry of Transport (in Japanese), pp5-30.

A. Repko, P.H.A.J.M. van Gelder, H.G. Voortman, J.K. Vrijling (2000). "Bivariate Statistical Analysis of Wave Climates" , *Proceedings of the 27th ICCE*, Vol.1, Coastal Engineering, Sydney, Australia, pp583-596.

DNV(221) (2001). "Guidelines for design of wind turbines,1st,edition," *DSI Grafisk Service*, Denmark, 253p.

Japan Port and Harbour Association (1999). "Technical Standards for Port and Harbour Facilities",(in Japanese).

J.P.Coelingh, A.J.M.van Wijk, A.A.M.Hotslag (1996). "Analysis of wind speed observation over North Sea". *Journal of Wind Engineering and Industrial Aerodynamics 61*, pp51-79.

June Sakamoto and Yasuhiro Mori(1998). "Two classes hazard intensity based on order statistics". *Structural safety and reliability*, Rotterdam, pp533 -540.

Lungu, D., Van Gelder, P.H.A.J.M., and Trandafir (1996). R., "Comparative study of Euro code 1, ISO and ASCE procedures for calculating wind loads." *Basis of design and actions on structures*, IABSE Colloquium; pp345-354.

M. Rikiji and O.Kiyomiya (2001). " Dynamic response analysis of wind energy unit subjected to wind and earthquake."*56th annual JSCE general meeting, Ist section* (in Japanese).

Robert E.Melchers (1999). "Structural Reliability Analysis and Prediction". *John Wiley & Sons*, pp184-250.