Response Evaluation of Wind Tower Designed with Composite Structure

K. Fujishita[(1)], W. Sekine[(2)], Y. Yagi[(3)] , A. Iwasaki[(4)] , M. Kimura[(5)]

[(1)] *Kozo Keikaku Engineering, Japan, kazuhiro-fujishita@kke.co.jp*
[(2)] *Kozo Keikaku Engineering, Japan, sekine@kke.co.jp*
[(3)] *FUJI P.S CORPORATION, y.yagi@fujips.co.jp*
[(4)] *FUJI P.S CORPORATION, a.iwasaki@fujips.co.jp*
[(5)] *Kozo Keikaku Engineering, Japan, madoka@kke.co.jp*

Abstract

The high-rise wind turbines are designed for obtaining a greater amount of power generation in stable wind speed of high elevation in these days. Supporting tower of wind turbines are generally designed with pre-fabricated steel tube, however, it is difficult to design the bottom part of the high-rise wind turbines because of the upper limit of the diameter of the steel tube.

In Europe, the wind turbines designed by composite structure with lower concrete part and higher steel part have been constructed. About the tower, the lower part section is designed by assembling several pieces of precast concrete member. For the vertical direction of lower concrete part, the pre-stressed steel cable is installed inside the tower for resisting against the large bending moment of lower part. The bottom concrete part can be designed without restriction of member size to realize the high-rise wind tower.

Recently, the application of same structural system is considered in seismic area like Japan, however, the safety of the structure under the seismic condition has not been fluently confirmed. There is the high possibility of the response excitation of the top steel part compared to ordinary building structures because of the stiffness discontinuity, furthermore, the installed tension force by the pre-stress cable of lower part may also contribute to enhance this condition. Therefore, the excitation under the seismic condition should be evaluated by the time-history analysis including the effect of pre-stress member.

Also, there is the response spectrum method for evaluating maximum seismic response without time-history analysis. The applicability of the method for the proposed composite structure should be confirmed.

In this paper, the tower design is conduct with the GA-based optimal design system under the Japanese guide-line and the time-history analysis of wind tower models including the tension member is carried out to observe the pre-tension effect of cable. Also, the easy response evaluation method is required for the cost evaluation of different type design, therefore, the applicability of the response spectrum method for the proposed structural system is discussed.

Keywords: Pre-stress Concrete Structure, Time-history Analysis, Response Spectrum Method

1. Introduction

Recently, the increment of the demand of renewable energy makes investors construct large scale wind turbines to generate energy more efficiently. In Europe, the rotor scale and the height of wind turbines became larger year by year to achieve the target reduction of greenhouse gas emissions. Wider rotor diameter at higher height contributes to increase the amount of power generation, therefore, the height of recent wind tower becomes over 100m and rotor diameter is over 120 m, then the power generation capacity reaches near 10 MW [1]. However, the required structural and fatigue potential of support structure of wind turbine are also increased and the large size requirement of member may interfere with transportation limitation. The previous paper mentions that transportation limitation of steel tube diameter is about 4.5m on ordinary steel tubular tower [1]. Many construct sites of wind turbines in Japan are at mountain district, so this limit may be stricter in Japan. If the high-rise wind tower is focused, it is difficult to satisfy it on reasonable construction cost, because of the thickness of steel material must be larger to resist the large overturning moment at the tower bottom within the transport limitation. Therefore, it is required to propose the new structural system of supported tower satisfying these restrictions and to investigate the dynamic response under the severe seismic condition.

Based on the above, the various structural systems of high-rise wind towers are constructed by several tower designers [2]. The lattice structure with small steel members [3], the pre-stress concrete structure [4] and so on have been constructed as the support tower of wind turbine. On the lattice wind tower, the small diameter members consisting lattice structure are easily transported on the standard logistic. And precast concrete tubular tower can be split into small segments and they can be reassembled by the pre-stress cable for vertical and horizontal direction, therefore, these structural systems can overcome the transport limitation. Accordingly, these structural technologies make it possible to construct high-rise wind towers.

However, the construction costs of these structural system is higher than that of the normal steel tubular tower because of the increment of assemble cost on site, therefore, it is reasonable to apply this system at only the lower part of tower structure which needs large amount of member to resist large overturning moment under design horizontal force on the design of high-rise wind tower. The concepts of hybrid system of upper steel tubular tower and lattice structure or concrete tubular tower on the lower part of tower have been also introduced on previous papers [5] and it can be considered that the application of the concept against the design of high-rise wind tower over 120m height is effective.

The structural design under the seismic load is also required on the construction in seismic area. Wind tower structure expects few structural damping, therefore, the dynamic response of wind turbine under seismic event may show unstable dynamic behavior compared to ordinary building structures. There are several researches about the investigation of seismic response of the pre-stressed concrete tower [6]. If the composite tower which is consisted of steel tubular tower and pre-stress concrete structure is considered it may shows different vibration shape compared to ordinary wind tower because of material discontinuity for the vertical direction. This structural system is consisted of upper soft part (steel member) and lower rigid part (concrete member), its uneven distribution of shear and bending stiffness may cause the vibration amplification of upper soft part and the increment of shear force of lower part. And the pre-stress cable installed in parallel to reduce tension force of concrete member also may cause some additional effect against its dynamic response.

In this paper, the dynamic response of composite wind tower consisted of steel cylinder and pre-stress concrete structure is investigated. 100m-140m height tower models are constructed for observing the dynamic behavior of different height tower. Recently, the optimization method based on genetic algorithm (GA) [7] is tried to apply against the structural design of architecture and civil engineering structure. In this paper, the outline designs of pre-stress concrete tubular tower are designed with GA in order to realize reasonable structural design by minimizing the amount of concrete material against the static storm wind force in Japanese design guideline. As the dynamic analysis model, the multi-degree of freedom model (MDOF model) is constructed as basic model and its mass and stiffness are set at the height of each tower segment based on the outline tower design. Additionally, the analysis model whose PC cable elements are modeled in parallel with the concrete tower and the analysis model whose concrete member has non-linear property is investigated to clear up each factor affects the dynamic behavior of whole tower. The eigan-value

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analysis of these models is carried out and the dynamic response is analyzed. Finally, the response evaluation method based on the response spectrum is applied for the composite tower and the applicability is validated.

2. Structural Design of Composite Tower

2.1 Outline

In this chapter, the structural design method of the geometry of composite tower by structural optimization with GA is explained briefly. Composite tower is consisted of two parts, lower pre-stress concrete tower with high strength precast concrete and upper steel tower shown as Fig.1. The design area of optimal design is lower concrete tower supporting the ordinary steel tubular tower which is attached on the top of the lower concrete tower. The design parameter is organized firstly and the enormous design parameters are investigated on the optimization system with GA to search the most cost effective design solution satisfying design restriction shown as Fig.1.

The design parameters of the lower concrete tower are also shown in Fig.1. The geometry of concrete member is decided based on a quadratic curve defined by Lagrangian interpolation formula, which expresses the tower outer geometry by following four parameters; the tower diameters of top and bottom piece and the coordinate of the middle point on the two dimensional coordinate. It is based on the assumption that optimal tower shape can be represented by the quadratic curve and it is reasonable because the structure of support tower of wind turbine is similar to the cantilevered beam distributed the wind force in the perpendicular direction. The thickness of tower is constant on each segment in the paper, the tower outline is decided by five parameters. And the applying pre-tension force per one pre-stress cable is decided as 2300 kN, the number of installed pre-stress cable is also design parameter. These six parameters are optimized with GA based optimization procedure.

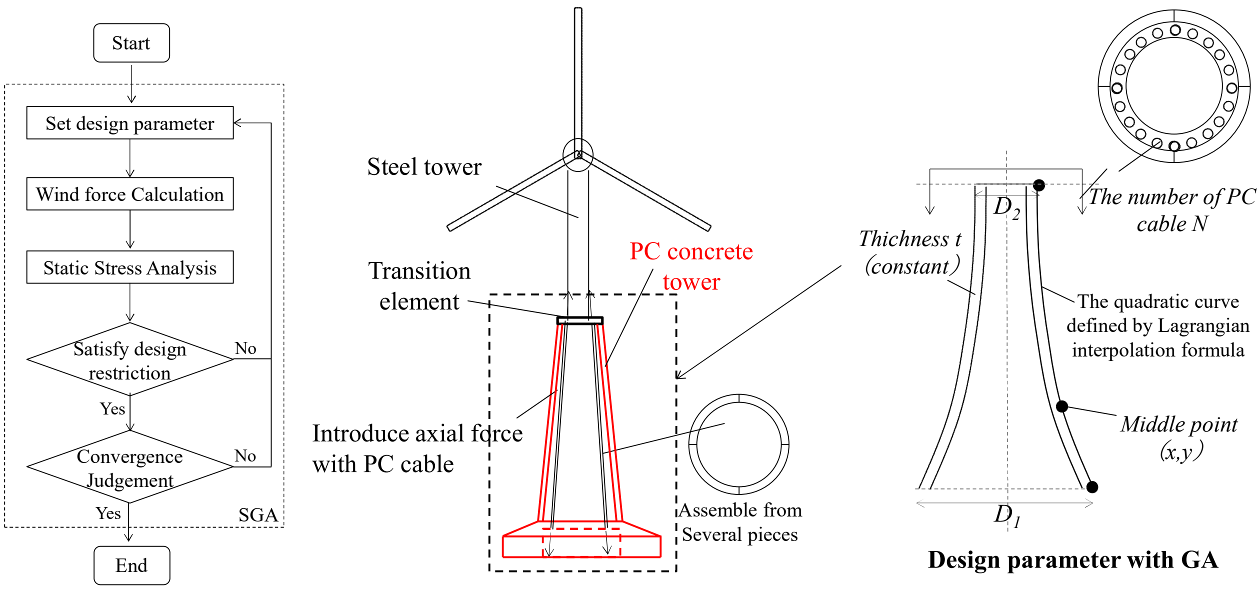

Fig. 1 – The concept of optimal design of composite tower

2.2 Design Load

The wind force employed for the tower design is defined from the Japanese wind turbine design guideline [8]. The wind property and the parameter of wind turbine are shown in Table.1 and Table. 2, respectively. The rotor diameter of wind turbine is 51m, whose output capacity is 2MW. The horizontal wind force of the tower is changed for the elevation area during the section design, the effect is considered for the wind force calculation. The calculated bending moment at each height is shown in Fig.2. h_{pc} means the height of lower concrete tower and the bending moment of four different height tower is plotted inside the graph.

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Table 1 – Assumed wind property

The relative roughness degree	Division II
Gust Response Factor	2.10
Reference wind speed V_o	34

Table 2 – Parameter of wind turbine

	Coefficient of wind force	**Area (m^2)**
Blade	1.3	99.7×2
Rotor + nacelle	1.11	45.7
Steel tower	0.6	
PC concrete tower	1.0	

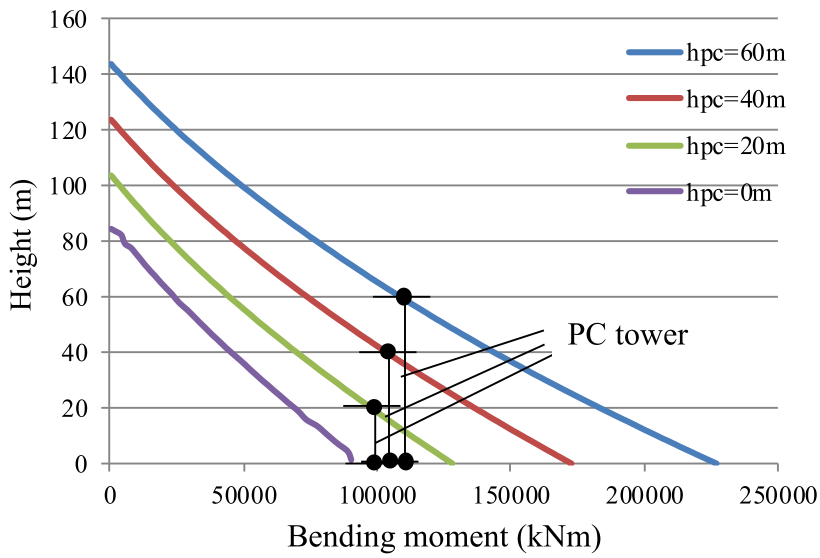

Fig. 2 – The bending moment of each height of wind turbine under the storm condition

2.3 Optimal Design Method with SGA

GA based optimization method is widely used on the structural design field because it is not required the mathematical expression of design variable for the optimization [7]. GA is the algorithm modeling the biological evolution with selection, crossing over and mutation. Although it is difficult to formulate design variable on many practical structural design cases, GA can deal a large variety of optimization problem if the objective function for the optimization can be calculated. The neighborhood solution of optimal solution can be obtained easily with GA algorithm, therefore, it is very usable tool for the practical structural design works. In this paper, SGA which deals single objective function is applied for the optimization.

The optimal design is carried out with SGA based on the several constraint conditions of member stress. The compression and tension stress of each concrete segment is investigated on the case of sustained loading and temporary loading under the storm condition in the linear stress analysis. It is confirmed that the member stress is under the allowable stress on each case shown in Table. 3. The assumed concrete strength F_c is

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50N/mm^2 in this case. The resistance force against tension force by the main reinforcement of the concrete section is not considered on design phase, all tension stress by the overturning moment is supported by only concrete material. PC cable is installed inside the hollow of concrete tubular member and the pre-stress force and additional force from the movement of tower is considered as the existing stress of PC cable.

The constraint conditions related to the member arrangement are also considered on the optimal design. The minimum thickness of concrete member is defined as 220 mm in the design guideline. The arrangement interval of PC cable is also decided from the perspective of the installing PC cable. These constraints are taken into account on the decision of the tower geometry and the arrangement of PC cable.

The design parameters minimizing objective function are searched with SGA based on above constraint conditions. Galapagos [9], which is the generic solver of GA included in Grasshopper is employed for solving this optimization problem. The weight of concrete tower is simply defined as the objective function representing the construction cost of composite tower on the optimization, although the material amount of concrete may not be directly related to it. The penalty method is employed to considering constraint conditions and the large amount of number is added to the objective function if the solution is not satisfied with the constraint conditions shown as Eq. (1). If the objective function becomes large number, the solution is eliminated from the candidate of the optimal design solution during the search with GA. The optimal calculation is continued until the convergence of objective function.

$$\text{Minimize } f(x)+\sum P$$

$$\text{If } g(x_i)>0 \quad P(x_i)=1000000 \tag{1}$$

Table 3 – Allowable stress of pre-stress concrete

Case	Compression force	Tension force
Sustained loading	$1/3F_c$	0
Temporary loading	$1/2F_c$	Bend and crack strength [10]

2.4 The Optimal Design

The design shapes and the stress verification values of optimized composite towers with SGA are shown in Fig. 3. The tendency of optimized shape is almost similar on different height models and the curvature of a quadratic curve defining tower diameter is large at the bottom part of tower. The tower diameter of lower part is determined from the stress limit of concrete tension force due to the large overturning moment. However, the shape is not simply decided based on the moment diagram of cantilever beam. The tower shape of upper part is determined from the stress limit of compression force because the large initial axial force introduced by pre-stress cable causes large compression stress on the tower top section which has smaller section area compared to the bottom section. Therefore, the design procedure of composite tower during structural optimization is assumed as follows. The bending moment under the storm force can be calculated based on the initial tower geometry. The required pre-tension force is estimated to decrease the tension force of the tower bottom section to the stress limit of concrete and the number of pre-stress cable is designed. The minimum requirement of the arrangement pitch of pre-stress cable on the top of tower is decided based on the construction and it defines the upper limit of pre-tension force. If the tension stress of the tower bottom is still over the stress limit, the tower top diameter is sometimes made larger to install more PC cable and to resist large compression force by the increment of design pre-tension force. Tower bottom diameter is also modified based on the recalculated tension force in parallel. The design parameter of composite tower is repeatedly updated like this to obtain the optimal solution satisfying all criteria and these design procedures numerically realizes the rational structural design.

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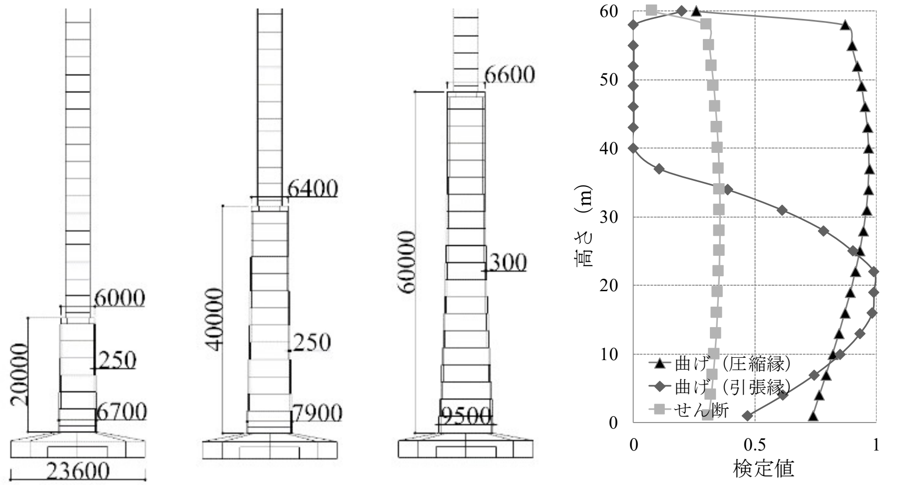

(a) PC 20m (b)PC 40m (c)PC 60m (d) Stress safety ratio (PC60m)

Fig. 3 – Design shape of various height model of composite tower

3. Dynamic Analysis of Composite Wind Tower

The seismic response of pre-designed composite tower under the Japanese seismic design wave (BCJ-L2, The Building Center of Japan) is investigated with non-linear time history analysis in this chapter. Two types of analysis model are constructed to investigate the effect of pre-stress cable against seismic response. Non-linear modeling of the concrete member is also carried out to track the detail dynamic characteristic of concrete member. The vibration characteristic is analyzed based on the modal analysis and the response evaluation accuracy of target tower based on the modal analysis is also discussed.

3.1 Analysis Model

The two type of seismic analysis model of composite tower is constructed as shown in Fig. 4. These models are constructed on the non-linear time-history analysis program Resp-F3T [11]. The wind tower structure is modeled as multi mass model which simulates the mass and the stiffness at each tower segment, which is flexural shear model. These models are two-dimensional and the seismic wave is inputted for one direction. The basement boundary condition is fixed-base condition. Initial compression load caused by PC cable is modeled as initial beam load of concrete tower. The fiber beam element is employed to simulate non-linear characteristic of lower concrete tower on the dynamic analysis, although the member of steel tower is linear characteristic on the non-liner concrete model. It is consisted of the concrete elements divided as 12 squares and rebar elements shown as Fig. 4. The bi-linear characteristic whose yield stress is 390N/mm^2 is employed as the non-linear characteristic of steel rebar. The ratio of main reinforcement p_g against all section area is set as about 1.0% on this analysis model. The logarithmic function model [12] is employed as the non-linear characteristic of concrete element and the standard strain ε_c is assumed as 0.002.

The pre-stress cable is additionally modeled as the truss element between the top of the concrete tower and the ground on model B to investigate the effect of modeling of PC cable for the seismic response of whole structure. The property of PC cable element whose yielding strength is 0.7 times of the initial tension force is selected by considering design margin as shown Table. 4. PC cable introduced inside concrete tower is modeled as two equivalent truss elements as shown Fig. 4 on time-history analysis model and the property of equivalent element is also shown in Table.4. The additional nodes are constructed at the end points of PC elements on the top and bottom level of concrete tower. The coordinate of each node is decided based on the center of figure of the planar arrangement of all PC cables and these additional nodes are connected to the tower beam element by rigid beam element. The initial tension stress existed on the PC cable is installed at the truss element of analysis model and it is also simulated on non-linear characteristic of truss element. The

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member characteristic of PC cable is usually simulated as the linear element working for only tension side. If initial tension force of PC cable is considered, it is considered that the starting point of the non-linear characteristic should be shifted from the origin of *xy*-coordinate. The non-linear characteristic of the PC elements is modeled as the bi-linear model for both tension and compression side by considering this shifting in this paper. The difference between the yield strength and the initial axial force of PC cable is considered as the dummy tension strength and the initial axial strength is considered as the dummy compression strength on the non-linear characteristic of truss element, respectively.

Rayleigh damping is employed against both steel tower and concrete tower and the structural damping ratio is 0.5%, which is the composite tower with steel and concrete. *h*=0.2% is employed for the analysis of steel wind tower on the paper [13] and *h*=1.0% is the average measured value of damping ratio of concrete chimney [14].

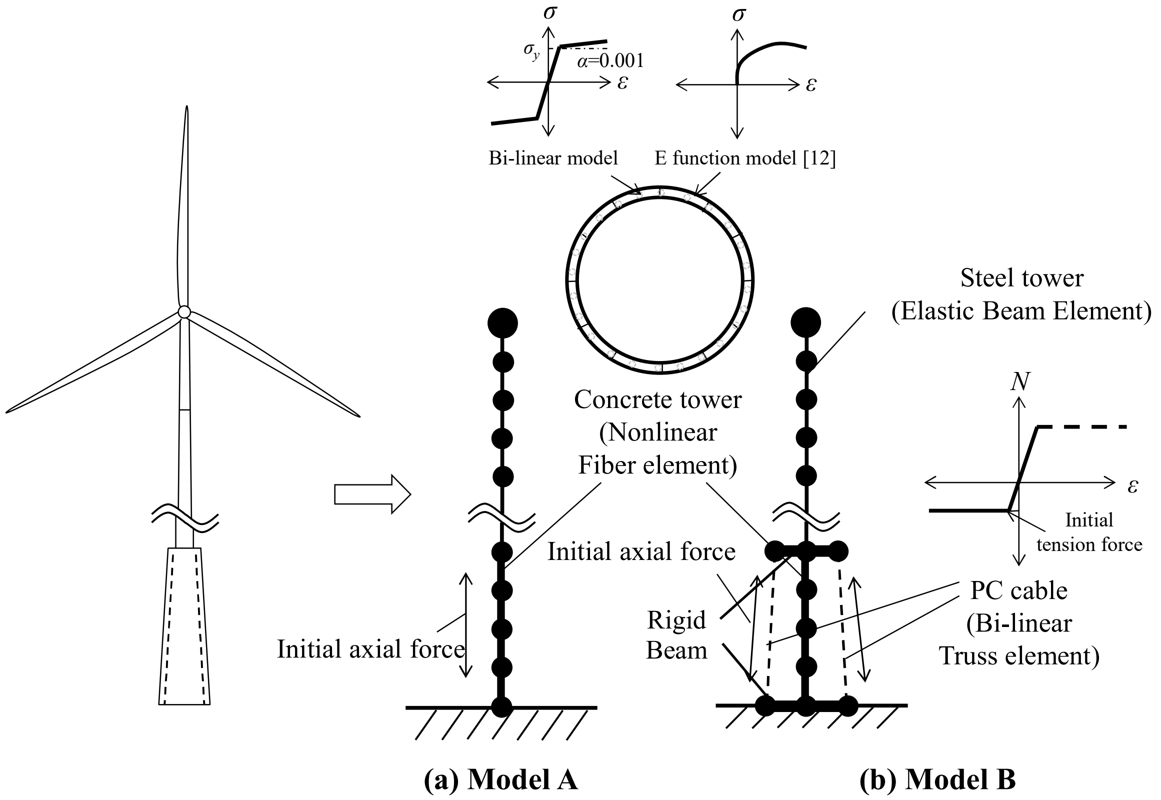

Fig. 4 – Seismic analysis model of composite tower

Table 4 – The property of PC cable

Model	*E* (N/mm²)	Section area per 1cable (mm²)	Initial tension force per 1cable (kN)	Allowable tension load per 1cable (kN)	Number of introduced PC cable	Equivalent area of truss element (mm²)
PC 20m	190000	1875.5	2300	3500	32	30008
PC 60m					40	37510

3.2 Analysis Result

Maximum response on the time history analysis is shown in Fig. 5 and 6. On model A (Not including PC element) and model B (including PC element), both linear and non-linear characteristic are applied as the

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characteristic of concrete member. Firstly, it is found that the maximum response of model A and B indicates almost same value in linear analysis, therefore, the modeling of PC cable element not clearly affects the linear dynamic response. The time-history of the stress of PC cable element of the concrete non-linear model of model B is shown in Fig. 6. It displays only tension force is existed in the PC cable during the analysis and it is smaller than the allowable tension stress of PC cable. Accordingly, PC cable element is totally elastic condition during whole dynamic analysis. Next, the analysis results of the linear and non-linear modeling of concrete member are compared. The modeling of PC cable element slightly affected the dynamic response especially in 60m model if the non-linear modeling of concrete member is carried out. The maximum story drift angle of PC tower exceeds 1/100rad on all non-linear models as shown in Fig.5 and moment hysteresis of concrete beam shown in Fig. 6 displays the stiffness reduction. As the result, the maximum overturning moment of non-linear model is smaller than that of linear model. The concrete member is obviously in non-linear area during seismic analysis. The time-history of rooftop displacement of PC tower is shown in Fig. 7 and the displacement phase is not shifted from the axis line during seismic wave. Therefore it can be said that the non-linearity of concrete member under seismic condition may acceptable to a certain extent because of small residual displacement after seismic event and it is able to decrease the design seismic force of composite tower if the allowable displacement is decided. Finally, if the maximum response of PC20m and PC60m model are compared, the graphs shows that the effect of higher vibration mode is significantly emerged on the maximum response of PC60m model especially on linear model.

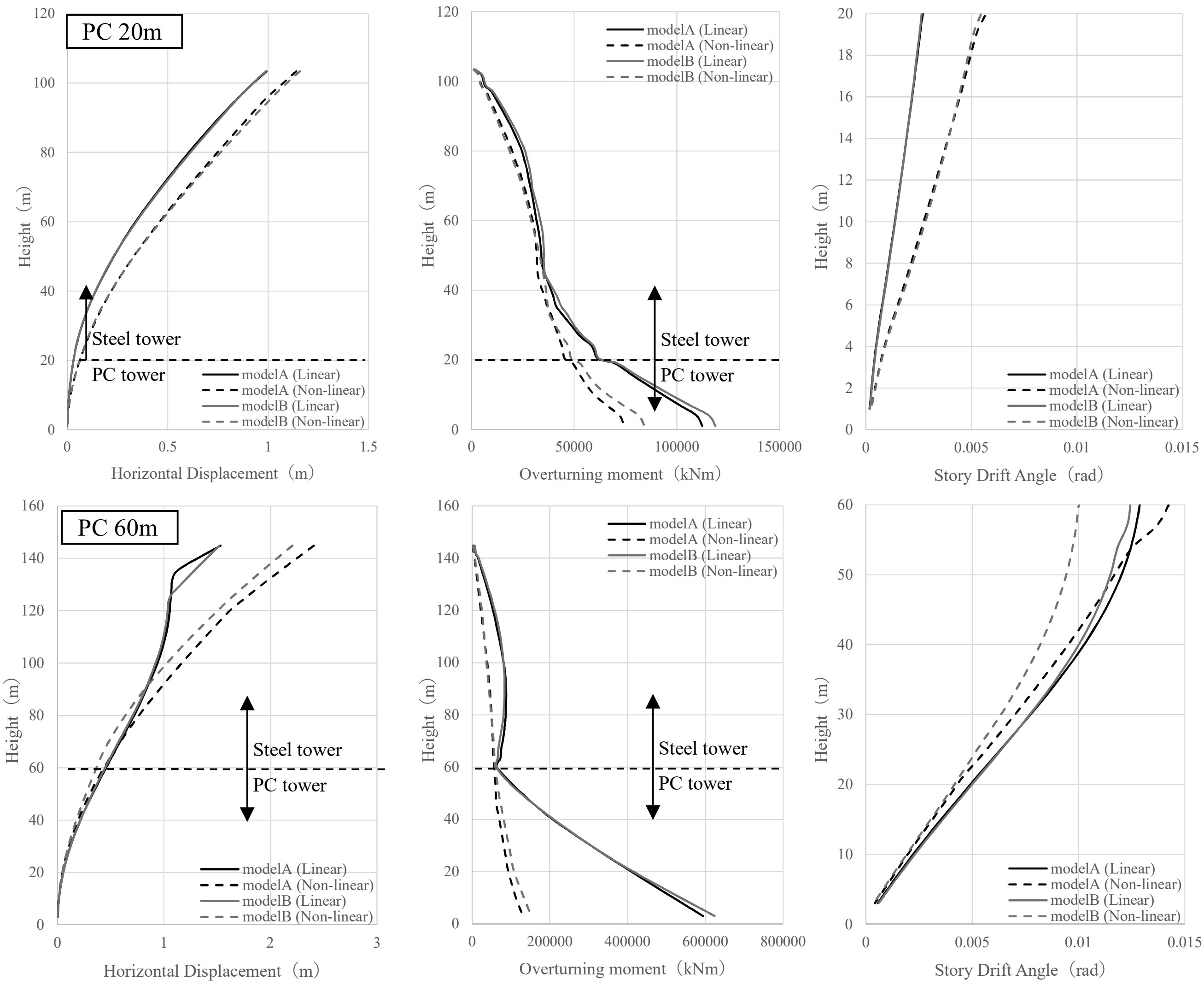

Fig. 5 – Maximum response

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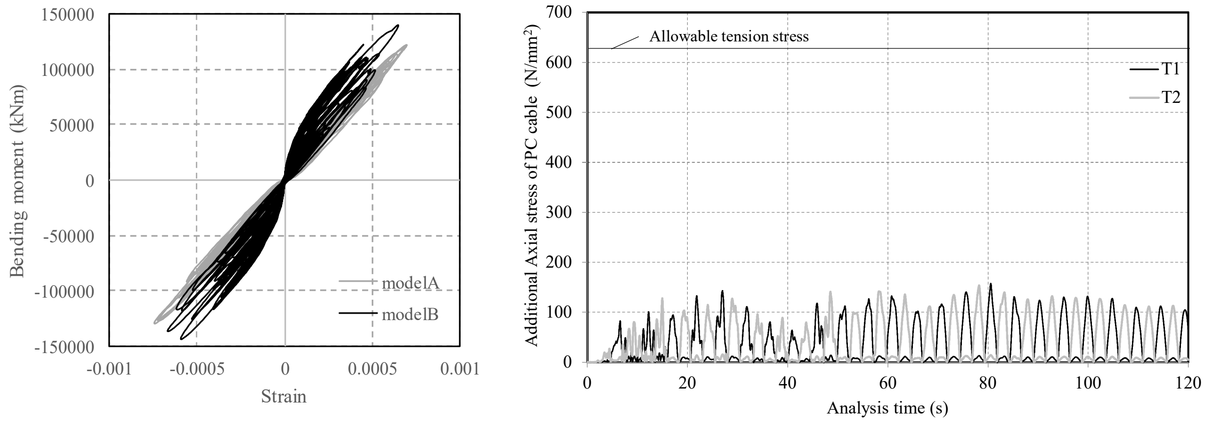

(a) Concrete member of the tower bottom (b) Equivalent element of PC cable

Fig. 6 – Time history of stress of the structural member (PC 60m model (Non-linear analysis))

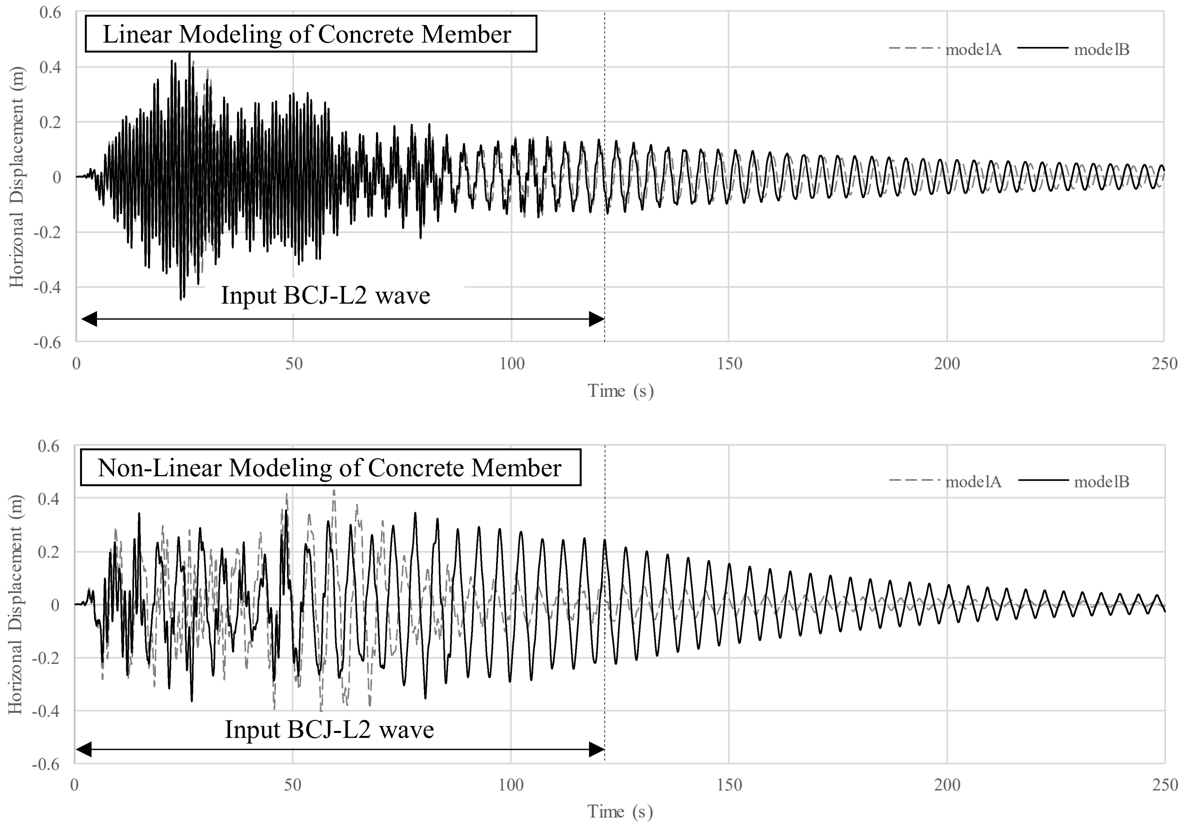

Fig. 7 – Time history of rooftop displacement of PC tower (PC 60m model)

3.3 Response Evaluation with Response Spectrum Method

In this chapter, the evaluation of the maximum displacement response of composite tower with modal analysis is carried out and the evaluation accuracy for the linear dynamic response of target structure is discussed. The wind tower has small damping, therefore, it is important how to decide the spectrum curve for evaluation. The damping correction factor for applying response spectrum method against small wind turbine structure has been proposed [15], therefore, the effectiveness of the method is validated in this paper.

The natural period and the effective mass ratio of each analysis model on 1st - 5th mode based on the eigan-value analysis are shown in Table 5. The natural period of model B including the PC cable element is a little

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smaller than that of model A because of the additional stiffness by PC truss element. The total effective math ratio from 1st mode to 5th mode is over 85% on each model, therefore that it is possible to evaluate dynamic response by modal analysis based on the dynamic characteristic of 1st - 5th mode on the target structure [16].

It is not practical to directly use the spectrum of BCJ-L2 wave for the response evaluation. The design spectrum (Kokuji spectrum) is proposed in Japanese seismic design guideline and it is employed for the evaluation. The spectrum of BCJ-L2 wave and the Japanese design spectrum (Kokuji spectrum) ×1.25 of several damping ratios are compared in Fig. 8. The shape of acceleration response spectrum of BCJ-L2 is correspond to the Kokuji spectrum×1.25 on h=5%. However, the shape of spectrum is unstable under the small damping like h=0.5%. The damping correction factor estimating the response spectrum of the structure with small damping ratio has been proposed as following equation by modifying damping correction factor in Eurocode in paper [15].

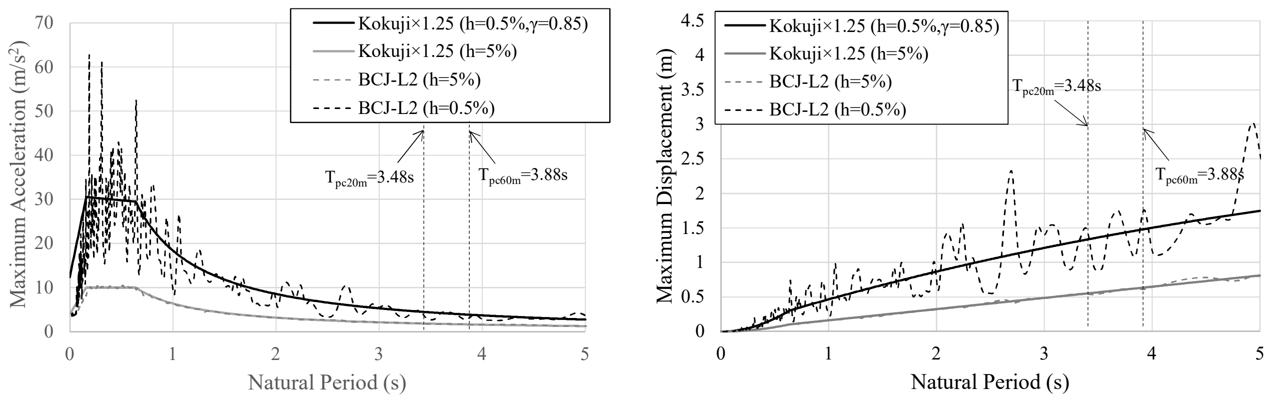
$$F_{\xi}(\xi, T)=(7/(2+100\xi))^{-0.07T+0.7\gamma+0.5} \qquad (\xi < 0.05) \tag{2}$$

The concept of reliability level is additionally introduced as the coefficient γ for evaluating unstable spectrum in this paper. γ=0.85 is recommended to assure the design reliability in paper [15], therefore, it is considered in this paper. The spectrum of γ=0.85 mostly captures the tendency of the median value of the spectrum of BCJ-L2 in Fig. 7. Accordingly, the maximum response evaluated based on the spectrum and the maximum response of each mode is summed up with SRSS (Square-Root-of-Sum-of-Squares) method.

The comparison of linear dynamic maximum displacement response and the evaluation value of response spectrum method are shown in Fig. 9. If γ=0.85 is assumed, the spectrum method slightly overestimates the maximum displacement response on the linear dynamic analysis, however, the evaluation value captures the tendency of displacement distribution and the higher mode excitation of PC60m model. The result assumed γ=0.5 is also shown in Fig. 10 and the evaluation accuracy is higher than the case of γ=0.85 in this case. The reasonable γ for the response evaluation should be discussed more by carrying out more parametric study on future study.

Table 5 – Natural period and effective mass ratio of analysis model

Type	Model	T_1 (s)	T_2 (s)	T_3 (s)	T_4 (s)	T_5 (s)	M_1	M_2	M_3	M_4	M_5	$M_{1\text{-}5}$
PC 20m	A	3.48	0.43	0.18	0.10	0.06	0.39	0.15	0.17	0.10	0.04	0.85
	B	3.47	0.43	0.17	0.10	0.06	0.39	0.15	0.17	0.10	0.04	0.86
PC 60m	A	3.88	0.79	0.33	0.16	0.11	0.23	0.35	0.10	0.11	0.05	0.84
	B	3.80	0.77	0.33	0.16	0.11	0.22	0.35	0.10	0.10	0.05	0.84



(a) Acceleration response spectrum (b) Displacement response spectrum

Fig. 8 – Response spectrum of BCJ-L2 wave

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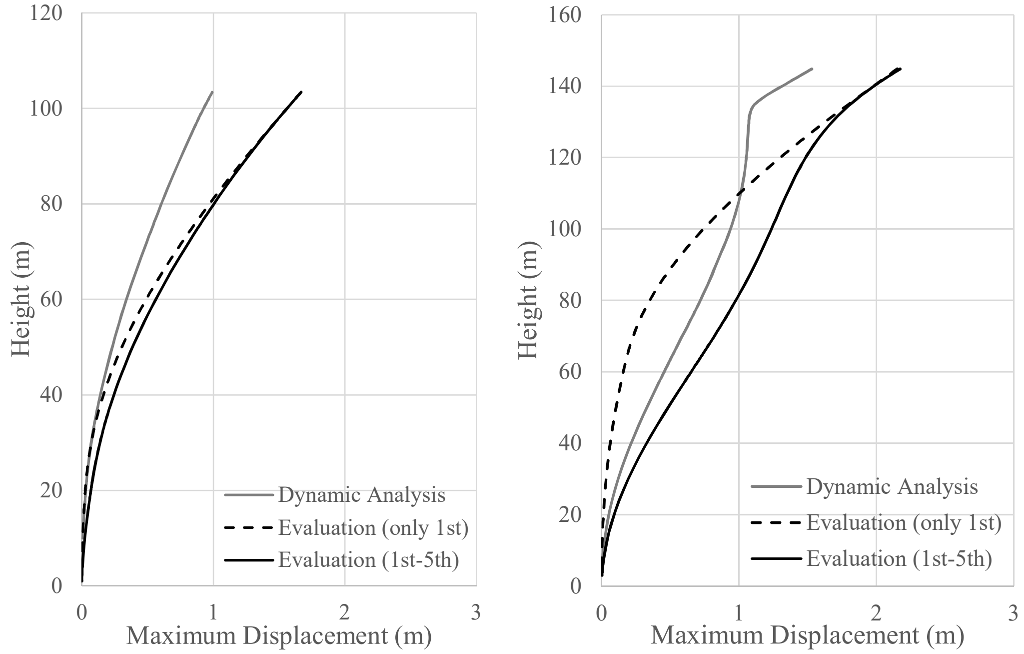
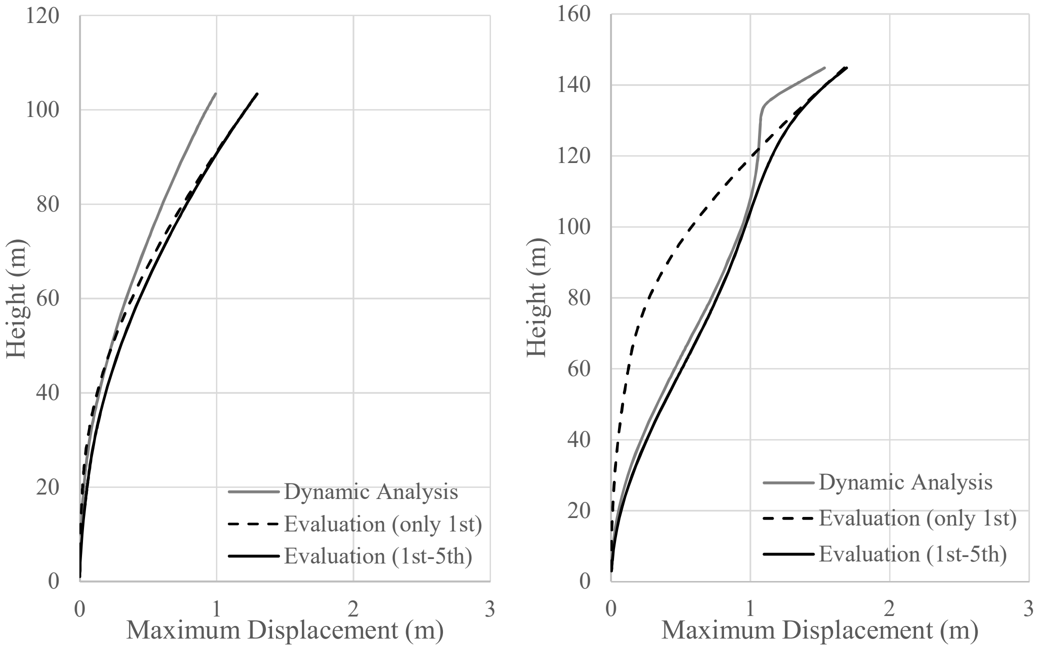


(a) PC 20m (b)PC 60m

Fig. 9 – The evaluation of maximum displacement (γ=0.85)



(a) PC 20m (b)PC 60m

Fig. 10 – The evaluation of maximum displacement (γ=0.5)

4. Conclusions

This paper presented the design method of composite tower for wind turbine consisted of steel and pre-stress concrete member with GA optimization method. The dynamic response of composite tower was investigated with the different type of analysis model. Finally, the response evaluation method based on the design

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spectrum was applied for the composite tower and the applicability was validated. The following summarizes are the main conclusion in this paper.

1) The reasonable design of composite tower from the aspect of the balance between introduced PC tension force and concrete member size to resist large overturning moment caused by the horizontal wind load can be obtained with the optimal design method with SGA based on the static stress analysis.

2) The modeling of PC cable on the dynamic analysis is not affected its dynamic response if the concrete member is modeled as linear structure. If the stiffness decline and the hysteresis damping of concrete member are simulated on the analysis, the maximum shear force and bending moment of concrete tower is decreased and the introduction effect of PC cable against seismic response is slightly emerged. It is confirmed that the stress of PC cable under seismic condition is lower than yielding stress of PC cable.

3) The effective mass ratio of 1st to 5th mode exceeds 85% on target wind tower and the evaluation of maximum displacement response based on the 1st to 5th mode with response spectrum method [15] slightly overestimates the response of composite tower, however, it can be roughly captures the maximum displacement distribution under the seismic condition.

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