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Flexıble body dynamıcs of offshore wınd turbıne under extreme loadıng condıtıons

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Abstract

The flexible body of offshore wind turbine is exposed to self-weight of turbine together with wind, wave and earthquake induced by the external forces. In order to be able to carry out the dynamic analysis of this body, wind forces represented in accordance with the various standarts, wave forces expressed by using different wave theories in terms of water depth and different wave parameters, and the graphics or digital data on earth motion acceleration recorded during earthquake can be taken into account within unique model to be developed for each case.

In this study, maximum wind force by Turkish Technical Standarts (TSE498), wave force based on linear theory and earthquake acceleration recorded for Duzce province in western Turkey are used for solving the dynamic behaviour of offshore wind turbine. Unique model is also developed at the scope of this study. Steady wind and unsteady wave-induced oscillations along flexible body are analytically and numerically investigated. In analytical model, they are specified according to using the elastic curve equation for assumption of initial shape function needed and analyzing with the Single-degree of freedom system (SDOF). In case of numerical analysis, the Stress Analysis Program (SAP2000) based on the Finite Element Method is used.

Finally, the oscillations on the unique model of flexible body are expressed in analytical form by SDOF and in numerical form by SAP2000. The results obtained from both analyses are also compared. It is seen that the analytical model developed by shape function-based assumptions give us the spatial and temporal oscillating function within the tolerable convergence. On the contrary, the results of numerical analysis execute the displacements changing instantanously and positionally along flexible body except for functional expression.

Keywords: Offshore Wind Tower, The Single Degree of Freedom System, Dynamics of Wind Tower Column, Axially Loaded-Tapered Column under Extreme Loading, Application of SAP2000

Introduction

Wind turbine can be technically possible installed on the land or marine. Especially, offshore tower with the except of generator, blades and their energy efficiencies equipment can be designed as cantilever column anchored by circumferencial bolting on the gravity-type concrete caisson or pile-like tower extension embedded in elastic soil in terms of foundation. In this case, a part of tower is influenced laterally by non-linear wind pressure and wave motion defined linearly or non-linearly. Besides, an earthquake motion applied to all tower is also considered at the stage of dynamic analysis that is carried out numerically and analytically to be taken as one basic reference method. Generally, the way to be preferred is to select one of the analytical methods based on the Single-Degree of Freedom system (SDOF) or the Multi-Degree of Freedom system (MDOF) and the numerical methods such as the Finite Difference Method and the Finite Element Method.

In this study, offshore wind tower due to earthquake motion, non-linearly varying wind pressure and wave motion based on the Linear wave theory is specially designed as tapered-section and cantilever column anchored by bolts. The dynamical analysis are carried out by means of both of the SDOF system analytically and the Time History Analysis numerically based on the FEM. The time-varying displacements at two reference points of the highest and mid points of tower are analyzed and compared with the considered analytical and numerical methods.

Methodology

a. Loads Acting on Offshore Wind Turbine

Tower are due to earthquake, wave and wind effects. Loads acting on it are composed of vertical self-weight of tower with generator and blades, and horizontal earthquake, wind, and wave motions.

Non-linearly varying wind pressures can be commonly defined as Eurocode, DnV (Det Norske Veritas) and TSE (Institute of Turkish Standarts) demonstrated as in Fig.1.

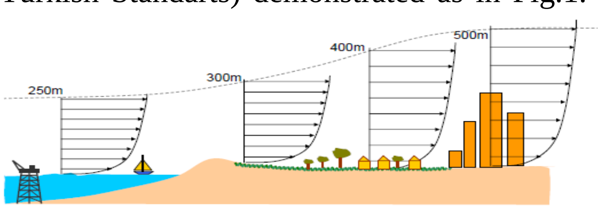

Figure 1: Non-linear wind velocity profiles varying by heights.

Logarithmic velocity profile change with increasing height and von Karman coefficient (k) as in Eq.1. (Barltrop et al, 1991)

$$U_{LOG} = (U_* / k) \times ln(Z / Z_o) \quad (1)$$

Here are shear force on surface (τ_0), air density (ρ), characteristic wind velocity on surface ($U_*=(\tau_0/\rho)^{½}$), Project area (A_r), the design height on structure (h), Project gross area (A_T) and shape coefficient $Zo=(A_R/A_T)(0,5h)$, height of structure Z, von Karman coefficient k=0,4. Corrected logarithmic wind velocity profile is expressed by height gradient (Z_G) and dimensionless parameter (a) in Eq.2. (Barltrop et al, 1991).

$$U_{DLOG} = \frac{U_*}{k}\left[ln\left(\frac{(Z-D)}{Z_0}\right) + 5,75a - 1,88a^2 - 1,33a^3 + 0,25a^4\right] \quad (2)$$

This equation explaines as angular velocity of the world (Ω), degree of lattitude (λ) wind frequency $f_C=2\Omega sin\lambda$, height gradient $Z_G= U_*/6f_c$, diameter of tower D and dimensionless parameter $a=(Z-D)/Z_G$. Eurocode velocity profile is written by taking earth coefficient k_T as follows;

$$U_{EU} = U_{BAS} \times k_T \times ln(Z / Z_0') \quad (3)$$

This represents the velocity in the elevation which is 100m above the sea surfaceere U_{BAS} and earth roughness Z_0'. Velocity profile on DnV standart is defined by considering wind magnitude factor at instant α and height factor β) as in Eq.4. (DnV,2010)

$$U_{DnV} = \alpha \times V_{1hr10} \times (Z / 10)^{\beta} \quad (4)$$

Here, V_{1hr10} is the average wind velocity blowing during an hour in 10-m height.). The wind velocity profile by TSE varies with wind speed U_{TS} and pressure p as seen in Table 1. (TSE,1997).

Table 1: Height, wind speed and pressure.

Height h(m)	Wind Speed U_{TS}(m/sn)	Wind Pressure p(KN/m^2)
0≤h≤8	28	0,5
8≤h≤20	36	0,8
20≤h≤100	42	1,1
100≤h	46	1,3

Lastly, the maximum value of the above mentioned wind speed profiles is taken as design wind speed.

$U=max\{U_{LOG},U_{DLOG},U_{EU},U_{DnV},U_{TS}\}$

In this study, the design wind speed is taken as a basis among the presented profiles in Fig.2.

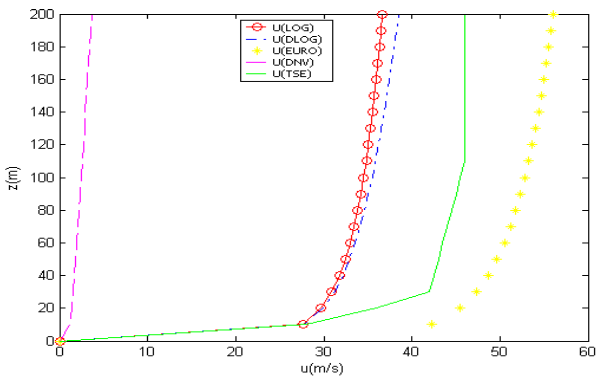

Figure 2: Wind speed profiles varying heights.

Lateral wind pressure from either generator or blades by TSE can be stated by wind pressure (p) and aerodynamic force coefficient (C') as in Eq.5.(Dyrbye et al,1997)

$$Q_P = p \times C' \times A_R \times U^2 \qquad (5)$$

Lateral wind force for tower diameter D can be re-written as follows;

$$q_K = 0,613 \times C \times U^2 \times D \qquad (6)$$

Here C is shape coefficient by earth roughness.

Wave-induced lateral force is determined by considering the Morison Equation composed of drag and inertia forces at the depth of pipe center and the Linear wave theory which satisfies the particle wave velocity and acceleration by taking the design parameters; wave period, height and lenght with the design water depth. Regular and sinusoidal wave, its transformation up to design depth is taken as a basis. Water particle velocities (Eq.7) and accelerations (Eq.8) on Linear wave theory representing a regular and harmonic wave ; (Dean et al,1991).

$$u(x,y)=\frac{H}{2}g\frac{T}{L_d}\frac{\cosh(kx)}{\cosh(kd)}\cos(-wt) \qquad (7)$$

$$\dot{u}(x,y)=g\pi\frac{H_0}{L_d}\frac{\cosh(kx)}{\cosh(kd)}\sin(-wt) \qquad (8)$$

Here are ω: angular frequency, t: time and k:wave number. The external forces on flexible turbine body together with its elastic curve are shown in Fig.3.

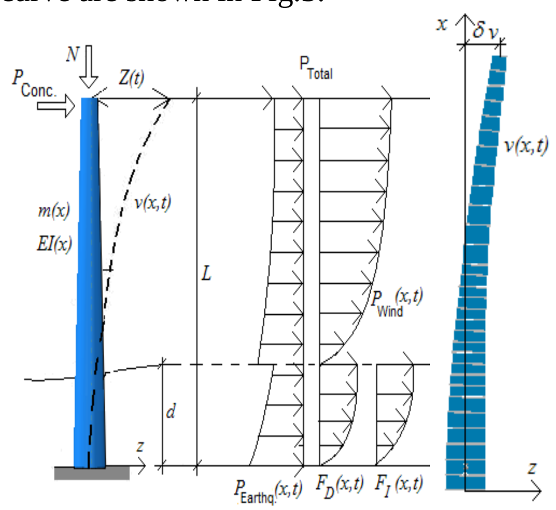

Figure 3: a. Tower and acting forces
b. Elastic curve of cantilever tower

The drag (F_D) and inertia (F_I) forces by the Morison equation the means of design wave parameters from the Linear wave teory are expressed as in Eq.9. (CERC,2002) (Rahman,1998).

$$p(x,t)=F_I(x,t)+F_D(x,t) \quad F_I(x,t)=C_M.\rho.\frac{D^2}{4}\dot{u}(x,t) \quad F_D(x,t)=\frac{1}{2}.C_D.\rho D u(x,t).|u(x,t)| \tag{9}$$

Here they are represented that *D is* the design depth of tower, ρ is water density, *u(x,t)* and *a(x,t)* are respectively wave velocity and acceleration, C_M and C_D are the drag and inertia hydrodynamic coefficients.

Earthquake-induced lateral force is considered by earthquake acceleration varying time and tower weight. In this study, the earthquake records occured in Düzce province in Northwestern of Turkey in 1999 is supposed as the design eathquake acceleration.

b. Governing Equation of Flexible Body Movement

By discretizating the time and coordinate dependent displacement function *v(x,t)* of motion of tower, it is written *analytically* as coordinate-dependent shape function $\psi(x)$ assumed as the statically-loaded elastic curve of tower and time-dependent displacement *Z(t)* as in Eq.10. (Clough et al,1993)

$$v(x,t)=\psi(x)Z(t) \tag{10}$$

The generalized SDOF system in non-linear form can be expressed for forced external forces induced by earthquake, wind and wave motion.

$$m^*\ddot{Z}(t)+c^*\dot{Z}(t)+\overline{k^*}Z(t)=P^*_{Top}(t) \tag{11}$$

Here, $Z(t)$, $\dot{Z}(t)$, $\ddot{Z}(t)$, $P_{Top}(t)$, (m^*),(c^*) and ($\overline{k}^*$) detailed in Eq.12 to Eq.16, .the are respectively displacement, velocity, acceleration, total external force in Eq.17 to Eq.20, generalized mass, damping and stiffness. (Clough et al,1993) (Celep,1996)

$$m^*=\int_0^d\left(m(x)-\frac{\gamma_{su}}{g}\right)\psi(x)^2dx+\int_d^L m(x)\psi(x)^2dx$$

(12)

$$c^*=a_1\int_0^L E.I(x).\psi^{u}(x)^2dx \tag{13}$$

$$k^*=\int_0^L E.I(x).\psi^{u}(x)^2dx \tag{14}$$

$$k_G{}^*=N\int_0^L\psi^{\prime}(x)^2dx \tag{15}$$

$$\overline{k^*}=k^*-k_G^* \tag{16}$$

In these equations, they are represented that m(x) and I(x) are mass and inertia moment of the tapered tower, E is the modulus of the elasticity, N is the axial compressive force, a_1 is the damping coefficient, k^* and k_G^* are the generalized elastic and geometric stiffnesses.

$$P^*_{Earthq.}(t)=-\ddot{v}(t)\left(\int_0^d\left(m(x)-\frac{\gamma_{su}}{g}\right).\psi(x)dx+\int_d^L m(x).\psi(x)dx\right)$$

(17)

$$P^*_{Wind}(t)=\sum p_i(t)\psi_i(x) \tag{18}$$

$$P^*_{Wave}(t)=\int_0^d p(x,t).\psi(x).dx \tag{19}$$

$$P^*_{Total}(t)=P^*_{Earthq}(t)+P^*_{Wind}(t)+P^*_{Wave}(t) \tag{20}$$

By solving analytically the generalized SDOF system, the displacement function Z(t) is obtained. The Stress Analysis Program (SAP 2000) is taken as basis for *numerical* analysis.

The acceleration earthquake in Düzce excluding Tsunami, wave velocity and acceleration based on Linear wave theory and Morison equation and wind pressure by TSE are loaded to the SAP 2000 and solved by using THA based on the FEM. Comparison of the displacements obtained from the mentiond analytical method based on the

SDOF system and numerical method from the SAP 2000 is realized and discussed

c. Solving The Governing Equation by Numerical Analysis Technique

In this study, the tapered wind tower column which has 6m-long, 4m-base and 1.50m-top diameter and steel trunk and fixed-end foundation. Extreme wind loading among the mentioned standarts is determined in Turkish Standart TS468. With respect to this, the considered wind profile is transformed to the concentrated loads at certain intervals.

Dead load including total weight of generator and wings weights and lateral resultant wind load are taken respectively as 83tons and 10.65tons (Krough,2004). Other wind turbine features and marine environmental parameters are respectively presented in Table 2 and Table 3.

According to the maximum lateral and vertical loading, the lateral displacements on cantilever-type column beam-like are calculated by using SAP2000 software and then, the results to be normalized are assumed as shape function presented in Eq.21.

$$\psi(x) = 0.037\left(\frac{x}{L}\right)^3 + 0.982\left(\frac{x}{L}\right)^2 - 0.016\frac{x}{L}$$
$$\{\psi(x) = 0,1\} \qquad (21)$$

Mass and inertia moment functions of tapered column sections are determined by using the Curve Fitting Technics as in Eq.22.

$$m(x) = 0.2 - 0.125\frac{x}{L}$$
$$I(x) = -0.12\left(\frac{x}{L}\right)^3 + 0.59\left(\frac{x}{L}\right)^2 - 0.93\left(\frac{x}{L}\right) + 0.49 \qquad (22)$$

Here, the origin point of x-axis is taken as fixed-end of tower. The concentrated external forces generated from design wind profile together with their acting points on flexible turbine body are presented schematically in Fig.4 and then numerically in Table.4. External drag force (P_4) on blades and rotor as dealt with in litetatures is concentratedly loaded at the top point. (Durukan,2007) (Krogh,2004).

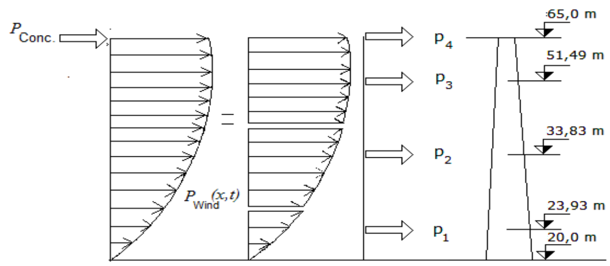

Figure 4: Wind concentrated loads and acting points on tower.

Table 2: Tower material and geometric features.

L(m)	**D_t(m)**	**D_u(m)**	**e (m)**	**E kN/m^2**	**a_1**	**N (kN)**	**γ_P kN/m^3**
65.0	4.00	1.50	0.02	2.1×10^8	0.005	830,0	78.34

Table 3: Design wave parameters.

H (m)	**T (s)**	**g (m/s^2)**	**d (m)**	**C_M**	**C_D**	**γ_W kN/m^3**
1,8	5	9,81	20,0	2,4	0,7	10.30

Here, e is the thickness of cylindirical tower.

Table 4. Equivalent concentrated wind loads.

Wind loads	**P_1**	**P_2**	**P_3**	**P_4**
Magnitude in kN	5,91	12,81	26,74	106,5
Acting points x(m)	23,93	33,83	51,49	65,0
Shape function ψ(x)	0,576	0,6819	0,8262	1,0

Other external force is induced by the recorded earthquake acceleration of Duzce province in Western Turkey demonstrated in Fig.5. External design forces composed of wind force, horizontal wave force and lateral earthquake force are calculated as in Eq.23a.b.c.

$$P^*_{Wind} = 140,73\ kN$$

$$P^*_{Wave}(t) = -43,501\sin(1,26\ t) + 3,754\cos(1,26\ t).\left|\cos(1,26t)\right|$$

(23.a.b.c)

$$P_{Earthq.}(t) = -22,465\ \ddot{v}_g(t)$$

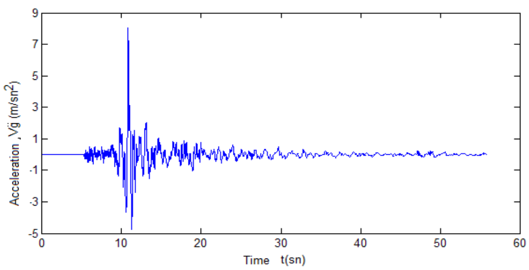

Figure 5: Earthquake acceleration record ($\ddot{v}_g(t)$) for Düzce/Turkey.

However, the total load varying with time is defined as in Eq.24;

$$P^*_{Total}(t) = -22,465\ \ddot{v}_g(t) + 140,73 - 43,501\sin(1,26\ t) + 3,754\cos(1,26\ t).\left|\cos(1,26t)\right|$$

(24)

This expression will be used as external forcing in the SDOF system analysis preferred for analytical method. The generalized mass, damping and rigidity are defined as in Eq.25;

$$m^* = 0,186\,L \qquad c^* = 0,798\frac{a_1 E}{L^3} \qquad k^* = 0,798\frac{E}{L^3} \qquad k^*_G = 1.365\frac{N}{L}$$

(25)

The generalized non-linear equation of motion can be expressed in the following equation;

$$0,187\,L\,\ddot{Z}(t) + 0,798\frac{a_1 E}{L^3}\dot{Z}(t) + \left(0,798\frac{E}{L^3} - 1,365\frac{N}{L}\right)Z(t) =$$
$$-22,465\,\ddot{v}_g(t) + 140,73 - 43,501\sin(1,26t) + 3,354\cos(1,26t).\left|\cos(1,26t)\right|$$

(26)

In this expression, buoyancy force is substracted from the weight of submerged tower in air by decreasing axial compression load at the sea level. In solving inhomogenous equation of motion, the initial condition is taken as;

$$t = 0 \rightarrow Z(0) = 0, \qquad t = 0 \rightarrow \dot{Z}(0) = 0 \quad (27)$$

By using the Maple software, the numerical analysis is performed and then the displacement-time relation of turbine body is found out as in Fig.6. (Maple V:Learning Guide)

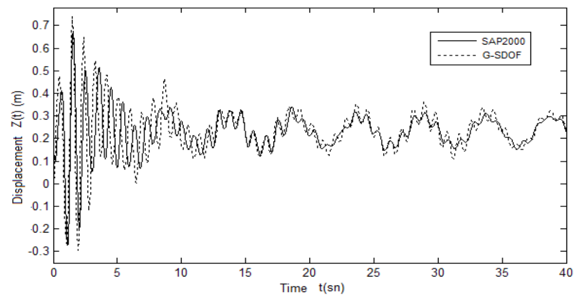

Figure 6: Dispacement-time variations on tower due to externally forcing.

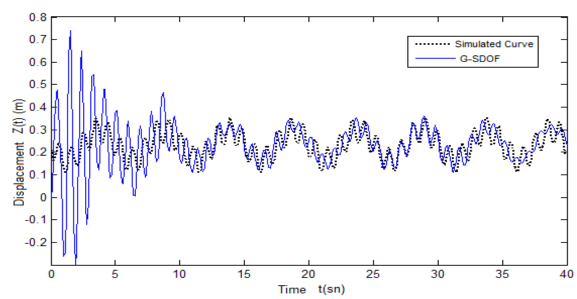

Figure 7: Comparison of analytical solution by SDOF and numerical solution by SAP 2000.

The displacement at the top point is recorded as maximum 0.70m at the beginning of oscillations. This is damped to 0.23m just after 15s to 20s. In Fig.7., the displacement-time relation by G-SDOF is simulated to analytically expressed form in Eq.28.

$$Z(t) = 0,23 + 0,066\sin(1,26t + 3,07) + 0,057\sin(6,64t - 2,72)$$

(28)

$$v(x,t)=\left(0,037\left(\frac{x}{L}\right)^3+0,982\left(\frac{x}{L}\right)^2-0,016\frac{x}{L}\right)\times\left(0,23+0,066\sin(1,26t+3,07)+0,057\sin(6,64t-2,72)\right)$$

(29)

By taking the time-dependent displacement equation *Z(t)* and coordinate-dependent elastic curve equation $\psi(x)$, both of time and coordinate-dependent equation is defined as in Eq.29;

Results from numerıcal analysıs

By considering the extreme combination of the earthquake, wave and wind loads, the displacements at the top of tower results from solving dynamically the generalized SDOF system relating to and numerically the Time History Analysis based on the Finite Element Method relating to the SAP 2000 software are illustrated in Table.5 and Table.6. Both results are matched up with each other.

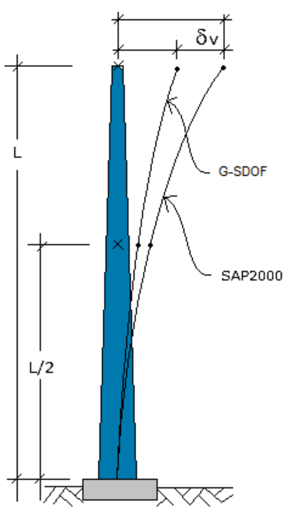

Figure 8: Comparison of both numerical methods

Conclusıon and recommendatıons

Some differences in amplitudes of vibration at top point are determined as %3 in wind, 0.8% in earthquake, 8.7% in wave and 1.2% in total loading. These differences at the mid-point of towers between analytical and numerical methods rise 12% in total loading. In literature, this value will be able to reach to the 22% level between theoretical and actual cases (Clough et al,1993).

Table 5: Displacements of Flexible Body and their differences.

Top (L=65m)	SDOF	SAP2000	δυ(m)
Wind(m)	0,2431	0,2362	0,0069
Eathquake(m)	0,5009	0,5051	0,0042
Wave(m)	0,0759	0,0693	0,0066
Combined (m)	0,8199	0,8106	0,0093

Table 6: Comparing the body displacements induced by different loadings.

Location	Top-point (L)	Mid-point (L/2)
SAP2000 (m)	0,8199	0,2228
TSDS (m)	0,8106	0,1962
δυ (m)	0,0093	0,0266

The SDOF system gives us the time and coordinate-dependent function of oscillation with respect to the numerical solution and the motion can be characterized in the SDOF system. However, it may be said that the assumption of shape function based on the elastic curve function corresponding to the maximum loaded system gives rise the mentioned displacement differences. The shape function as assumed in this study can be taken as either first of itertive process or one of the other movement functions of flexible turbine body.

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