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SEISMIC CONSIDERATION FOR HIGH RISE CONCRETE WIND TURBINE TOWERS

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ABSTRACT :

Globally increasing demand and cost of energy motivate the Shanghai Science and Technology Development Committee wind energy research program to develop a large wind turbine (LWT) technology that will allow wind systems to compete in regions of low wind speed. Most of current wind turbine powers on world are less than 1.0 MW supported by tubular steel towers and their tower height less than 80 meter. Earthquake load may not be significance to the tower design due to following reasons; first, wind turbine towers often are placed in wide-open fields and high wind gust areas, the wind load from the turbine and direct wind pressure on the tower usually governs the design of the tower; second, steel tubular tower structures usually are lighter than concrete structures, thus, they have less seismic inertial force than that of concrete tower. For large wind turbine towers with the turbine head weight getting heavier, the seismic load very likely becomes governing loading case for a pre-stressed concrete tower, especially in region of the high seismic zones. As tower getting taller and support large turbines, pre-stressed concrete tower solution becomes more competitive in overall cost in the latest study. In this paper a series height of the pre-stressed concrete towers segmental stacked and post-tensioned by tendons are studied and compared with steel tubular towers under prescribed seismic load under different design code including China GB, US IBC 2003 and Euro-code. The tapered tower fundamental dynamic properties are estimated by the Raleigh-Ritz method and compared with FEM results.

KEYWORDS: High rise structural systems; Wind turbine tower; Seismic design; Prestressed concrete tower

1. INTRODUCTION

As oil market price in NYMEX surpassed \$100 per barrel in early 2008, seeking alternative energy resources becomes more and more urgent in global wise by many governments. Wind energies are apparently one of the most popular alternative energy and widely used in many European countries. Most of wind turbine powers around word are less than 1.5MW and tower heights are less than 80m. In order to allow wind systems to compete in regions of low wind region, U.S. Department of Engery initiates several programs for Low Wind Speed Turbine (LWST) project partnered with industry. Under the WindPACT program and Next Generation Turbine program, the cost comparison study for steel tower, hybrid with concrete and full post-tension concrete tower with large wind turbine load was investigated by Berger/ABAM in 2004. The report also concluded that the cost of the post-tension concrete tower become more and more competitive to steel tower as the tower rising over 100m height with large wind turbine load. Today, wind power in China is developing rapidly and received strong support particularly from the government. European companies gain most of the share from China's wind power equipment market, only GE wind power from US stay active in china market. In 2007, In order to catch up with global trend of the LWST study for wind energy, Shanghai Science and Technology Committee sponsored the feasibility study for large wind turbine application in 2007.

More 80% of the wind turbine towers are designed by using welded steel tubular structures. As the wind turbine power getting large and wind turbine tower getting taller, the wall thickness for tubular steel structure need be thickened due to local buckling. It will increase the difficulties for welding and lower the fatigue behavior. Because the steel price soared in last several years, the cost of steel tower increased sharply. The pre-stressed concrete tower alternatives become more attractive for the low material cost and better fatigue properties. It can be a very efficient substitution for steel tubular towers. In current wind turbine tower design, earthquake load may not be significance to the tower design due to light weight with less seismic inertial force induced. For large wind turbine (LWT) towers with the turbine head weight getting heavier, the seismic load very likely becomes governing loading case for a pre-stressed concrete tower, especially for those towers in high seismic zones. This paper presents simple seismic design procedures based on ASCE-7-05 seismic load. A series height of the pre-stressed concrete towers segmental stacked and post-tensioned by tendons are studied and compared with steel tubular towers under prescribed seismic load under different design code including China GB, IBC 2003 and Euro-code. The tapered tower fundamental dynamic properties are estimated by analytical approximation method with modified Raleigh-Ritz method and compared with FEM results.

2. SEISMIC DESIGN LOAD

2.1 Lateral Force Load

The wind turbine tower lateral stiffness and mass distribution are symmetric in plan with respect to two orthogonal horizontal axes; in addition, the horizontal dimensions of the structure reduce gradually from base to the top, without abrupt changes. The wind turbine towers are categorized as flexible structures and the response are not significantly affected by the contributions of higher modes of vibration. According to Eurocode 8 4.3.2 and GBJ 135-90 3.4.3, the static lateral force method is appropriate for designing symmetric round shape turbine tower design.

2.2 Earthquake Load

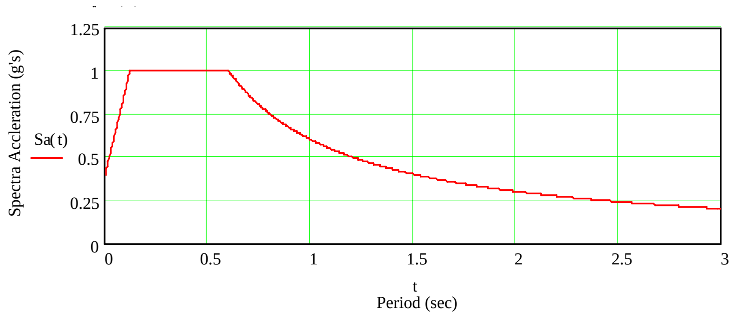
In most of current steel tubular wind turbine tower analysis and design, earthquake load may not be significance to the tower design due to following reasons; first, wind turbine towers often are placed in wide-open fields and high wind gust areas, the wind load from the turbine and direct wind pressure on the tower usually governs the design of the tower; second, steel tubular tower structures usually are lighter than concrete structures, thus, they have less seismic inertial force than that of concrete tower. For large wind turbine (LWT) towers with the turbine head weight getting heavier, the seismic load very likely becomes governing loading case for a pre-stressed concrete tower, especially along the high seismic zones. Seismic analysis and design have to conform to local seismic specifications and building codes. In this paper, ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures,* static equivalent earthquake load method is used for earthquake analysis. In this paper, a LWT tower presumably located in the moderate seismic region is used in illustration of the design earthquake load. From the earthquake geographic map, the Maximum Considered Earthquake (MCE) ground motion for soil site category "B" with 5% damping is **1.5g** (S_S) for 0.2 sec and **0.6g** (S_1) for 1 sec. The wind turbine towers are typically located in wide-open fields with very low occupancy. Therefore, the occupancy importance factor is equal to **1.0**. For post-tension prestressed concrete tower, no reduction factor for tower is employed and soil category "**D**" is assumed. The design earthquake spectral acceleration at short period, S_{DS} and one second period S_{D1} are determined by

$S_{DS} = 2/3\ F_a\ S_S = 1.0\ g$

$S_{D1} = 2/3\ F_v\ S_1 = 0.6\ g$

Where the site factor for $F_a = 1.0$ and $F_v = 1.5$ from table 9.4.1.2 of ASCE 7-05. Design response spectra $S_a(T)$ can be expressed and plotted as follows:

$$S_a(T) = \begin{cases} \dfrac{S_{D1}}{T} & if \quad T > T_s \\ S_{DS}(0.4 + 0.6\dfrac{T}{T_0}) & if \quad T < T_0 \\ S_{DS} & otherwise \end{cases}$$



ASCE 7 Design Response Spectra

Where the $T_S = S_{D1}/S_{DS}$ and $T_0 = 0.2\ T_S$.

2.2 Design Earthquake Load on Tower

The earthquake lateral load distributes along the tower height (h) according to its weight distribution. Define axis (z) along the tower height and weight distribution w(z) as function of height. The total weight of tower (W) with turbine head weight;

$$W = \int_0^h w(z)dz + W_{HeadMass} \tag{2.1}$$

The base shear coefficient $Cs(T) = Sa(t)\ I/R$ is function of response spectra where the importance factor (I) and reduction factor (R) are equal to 1.0 and earthquake base shear $V = Cs(T)\ W$.

The tower period (T) can be estimated either using the empirical formula; $Ta = Ct\ h^{3/4}$ where $Ct = 0.02$ and $T = 1.4\ Ta$ or calculated more accurately by finite element method (FEM) or other analytical methods, (see dynamic properties section). The lateral distribution forces F(z) can be defined as

$$F(z) = \frac{w(z)\cdot z^{k0}}{\int_0^h w(z) z^{k0} dz + W_{HeadMass} h^{k0}} V \tag{2.2}$$

where k0 is the exponent for the first mode profile

$$k0 = \begin{cases} 1 & if \quad T < 0.5s \\ 2 & if \quad T > 2.5s \\ (0.5\cdot T + 0.75) & otherwise \end{cases} \tag{2.3}$$

The head mass concentrated force at the top of tower (Ft)

$$Ft = \frac{W_{HeadMass} h^{k0}}{\int_0^h w(z) z^{k0} dz + W_{HeadMass} h^{k0}} V \tag{2.4}$$

The shear force - Vz(z) and overturning moment Mz(z) along the tower height can be calculated from;

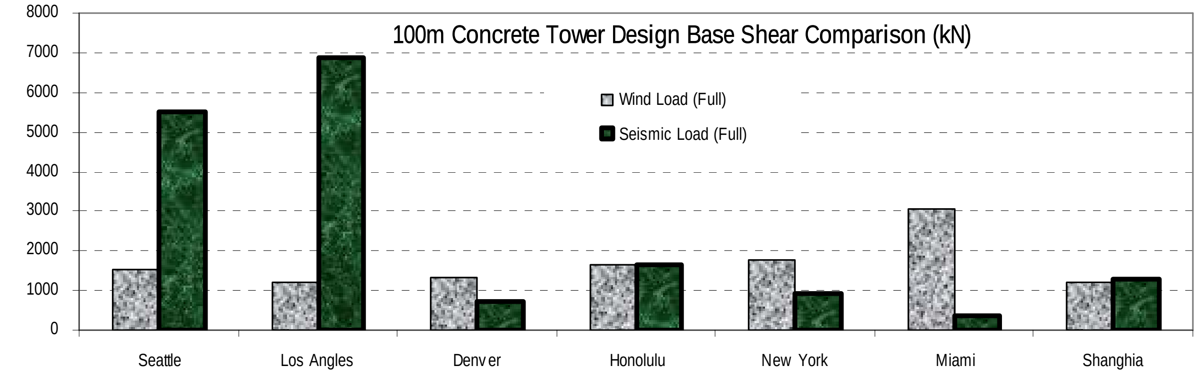
$$Vz(z) = \int_z^h F(x)\cdot dx + Ft \tag{2.5}$$

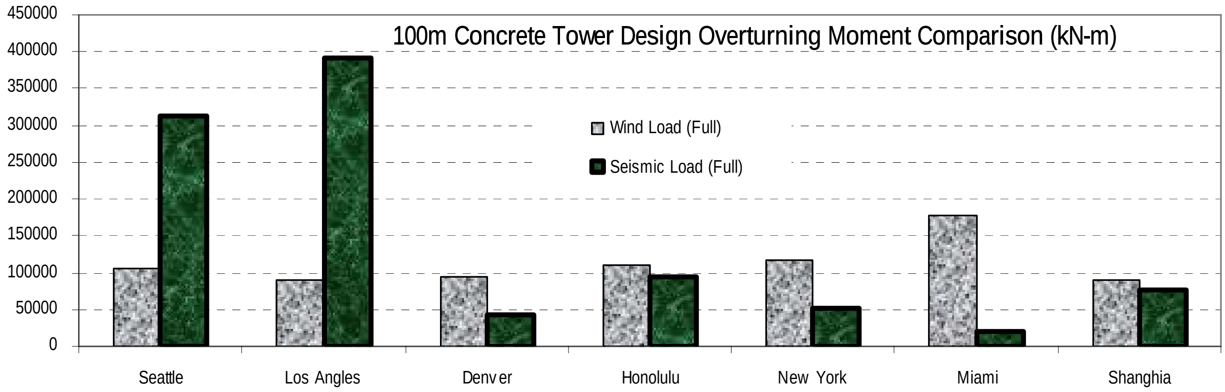
$$Mz(z) = \tau(z)\cdot[\int_z^h F(z)\cdot(x-z)\cdot dx + Ft\cdot(h-z)] \tag{2.6}$$

where the overturning moment reduction factor $\tau(z)$ is *1.0* for the tower top *30m* and *0.8* for the tower bottom *30m* or linear interpolation for the height between the top most and bottom most. Accordingly, tower deflection $\Delta(z)$ along the height can be calculated by following formula;

$$\Delta(z) = \int_0^z \frac{Mz(x)}{E(x)\cdot I(x)}\cdot(z-x)\cdot dx + \frac{Vz(z=0m)}{Kh} + \frac{Mz(z=0m)}{Kr}\cdot z \tag{2.7}$$

where Kh and Kr are the soil spring constants of translation and rotation. The followings charts are the comparison of the base shear and overturning moment with factored wind load and seismic load for 1.5MW 100m towers in various cities in United n States.





3. TOWER DYNAMIC PROPERTIES

Dynamic magnification effects impact directly on the fatigue loads to be considered in the design of the tower. It is very important to design the tower frequency to avoid the excitation of resonant oscillators resulting from rotor thrust fluctuations at the blade-passing frequency or, to a lesser extent, at the blade rotational frequency. The conventional methods for estimation of tower natural frequency are either using finite element computer modeling (FEM) or approximation analytical method (AAM). The FEM methods are usually easy and accurate to use with aid of many commercial FEM software. However, the FEM also shows time consuming for sizing the tower by trial-error. For the preliminary tower dynamic design, an AAM is developed in explicit form. The simple equations can allow designer to quickly and easily reach the solution for determining the section of the tower. For a simple straight cylindrical steel tower, the first natural tower bending frequency (in rad/s) is estimated by Harrison, Hau and Snel, refer to "*Large Wind Turbines Design and Economics (2000)*" as:

$$\omega_t = 1.75\sqrt{\frac{E \cdot I \cdot g}{H^3 (W_t + W_{tow}/4)}} \tag{3.1}$$

where ω_t is the estimated natural frequency of tower; H is the height of tower; E and I are elastic modulus and moment of inertia of tower; W_t and W_{tow} are the weight of head mass and tower mass respectively. Obviously, for the more complicated hybrid tower with a tapered section and with consideration of flexible foundation, this simple AAM formula is not sufficiently accurate. An analytical method based on energy method (Rayleigh approximation) is introduced below to estimate the fundamental frequency for the hybrid and tapered tower.

3.1 Rayleigh's Method for Approximating the Fundamental Frequency

Rayleigh approximation method is based on the energy conservation principle. Assume that the deflection curvature function of the member is given,

$$y(x,t) = Y(x) \cdot \sin(\omega \cdot t + \alpha) \tag{3.2}$$

where *Y(x)* is the assumed deflection curvature function of along the tower *(x)* under simple harmonic motion. The maximum kinetic (T_{max}) energy is

$$T_{\max} = \frac{\omega^2}{2}\int_0^h m(x)\cdot Y(x)^2 \cdot dx \tag{3.3}$$

where *m(x)* is mass distribution along the tower at height of *h*, and the maximum strain energy (U_{max}) is

$$U_{\max} = \frac{1}{2}\int_0^h E(x)\cdot I(x)\cdot [Y''(x)]^2 \cdot dx \tag{3.4}$$

where *Y''(x)* is the second order derivative of *Y(x)*, *E(x)* and *I(x)* are varied modulus and moment of inertia along the tower. According to energy conservative principle, $T_{max} = U_{max}$, the natural frequency is

$$\omega^2 = \frac{\int_0^h E(x)\cdot I(x)\cdot [Y''(x)]^2 \cdot dx}{\int_0^h m(x)\cdot [Y(x)]^2 \cdot dx + \sum_i m_i \cdot Y(x_i)^2} \tag{3.5}$$

where $\sum m_i\ Y(x_i)^2$ is the summation of concentrated mass at *i* location. From above equation, the mass - *m(x)*, modulus – *E(x)* and moment of initial – *I(x)* can be varied along the height of tower. The accuracy of the calculated natural frequency of the tower largely depends on assumed defection function.

3.2 Tapered Cylindrical Tower with Different Structures

The tower may consist of different type of structures, such as steel tapered tower sitting on high concrete pedestal platform as lower part of tower. The diameter - *d(x)* of the tapered section along the tower height can be described as;

$$d(x) = \begin{cases} \dfrac{Dct - Dcb}{hc}\cdot x + Dcb & \text{if } x \le hc \\ \dfrac{Dst - Dsb}{h - hc}\cdot (x - hc) + Dsb & \text{if } hc < x \le h \end{cases} \tag{3.6}$$

where D_{cb} is the base diameter of concrete tower, D_{ct} is the top diameter of the concrete tower at height of *hc*. D_{sb} is the base diameter of steel tower and D_{st} is the top diameter of the steel tower at height of *h*. Similar definition of the tower wall thickness - *t(x)* along the tower height can be expressed as;

$$t(x) = \begin{cases} \dfrac{Tct - Tcb}{hc}\cdot x + Tcb & \text{if } x \le hc \\ \dfrac{Tst - Tsb}{h - hc}\cdot (x - hc) + Tsb & \text{if } hc < x \le h \end{cases} \tag{3.7}$$

where *T* is the wall thickness, thus, the moment of inertia *I(x)* and section area *A(x)* are;

$$I(x) = \frac{\pi}{64}[d(x)^4 - (d(x) - 2\cdot t(x))^4] \tag{3.8}$$

$$A(x) = \frac{\pi}{4}[d(x)^2 - (d(x) - 2\cdot t(x))^2] \tag{3.9}$$

the mass *m(x)* along the height and elastic modulus *E(x)* are accordingly written as

$$m(x) = \begin{cases} A(x)\cdot \rho c & \text{if } x \le hc \\ A(x)\cdot \rho s & \text{if } hc < x \le h \end{cases} \tag{3.10}$$

$$E(x) = \begin{cases} Ec & \text{if } x \le hc \\ Es & \text{if } hc < x \le h \end{cases} \tag{3.11}$$

where ρ_c and ρ_s are the density of concrete and steel; E_c and E_s are the elastic modulus of concrete and steel respectively. The deflection curvature function assumes – *y(a1,a2,x)* with the parameters *a1* and *a2;*

$$y(a1,a2,x) = \begin{cases} a1\cdot[1-\sin(\dfrac{\pi}{2}\cdot\dfrac{x+hc}{hc})] & \text{if } \quad x \le hc \\ a2\cdot[1-\sin(\dfrac{\pi}{2}\cdot\dfrac{x+h-2\cdot hc}{h-hc})] + a1\cdot[1+\dfrac{\pi\cdot(x-hc)}{2\cdot hc}] & \text{if } hc < x \le h \end{cases} \tag{3.12}$$

where *a1* and *a2* are modification parameters of defection function in units of the length. The first derivative *y'(a1,a2,x)* and second derivative *y''(a1,a2,x)* can be easily differentiated accordingly; the natural frequency of the tower can be described as

$$f(a1,a2) = \frac{1}{2\cdot\pi}\sqrt{\frac{\int_0^h E(x)\cdot I(x)\cdot [y''(a1,a2,x)]^2 \cdot dx}{\int_0^h m(x)\cdot [y(a1,a2,x)]^2 \cdot dx + \dfrac{Wh}{g}\cdot y(a1,a2,h)^2}} \tag{3.13}$$

where *Wh* is the total turbine head weight, the frequency *f = min[f(a1,a1)]* can be obtained the by plotting as function of parameters of *a1* and *a2*

3.3 Flexibility Foundation Effect on Tower Fundamental Frequency

The flexibility of the foundation has significant impact on the tower dynamic behavior. In order to include this factor, we here assumed that tower foundation has the horizontal spring constant - *Kh* and rotation spring constant – *Kr*. The deflection function *y(a1,a2,x)* can be revised as:

$$ya(a1,a2,a3,x) = y(a1,a2,x) + a3 \cdot [\frac{y(a1,a2,h) \cdot h^2}{Kr} \cdot \frac{x}{h} + \frac{y(a1,a2,h)}{Kh}] \quad (3.14)$$

where the parameter *a3* is the third factor and had the same units as spring constant *(Kh)*. The strain energy and kinetic energy shall be rewritten;

$$U_{\max}(a1,a2,a3) = \frac{1}{2}\int_0^h E(x) \cdot I(x) \cdot [y''(a1,a2,x)]^2 \cdot dx + \frac{[a3 \cdot y(a1,a2,h)]^2}{2 \cdot Kh} + \frac{[a3 \cdot y(a1,a2,h) \cdot h]^2}{2 \cdot Kr} \quad (3.15)$$

$$T_{\max}(a1,a2,a3) = \frac{\omega^2}{2}[\int_0^h m(x) \cdot ya(a1,a2,a3,x)^2 \cdot dx + \frac{Wh}{g} \cdot ya(a1,a2,a3,h)^2] \quad (3.16)$$

The fundamental frequency of the tower can be calculated as

$$f = \min[\frac{1}{2 \cdot \pi}\sqrt{\frac{U_{\max}(a1,a2,a3)}{T_{\max}(a1,a2,a3)}}] \quad (3.17)$$

Another alternative method is simply combined the frequencies with different motion in a series of springs. The rigid body rotation frequency *(fr),* translation frequency *(ft)* and tower flexure frequency *(f(a1,a2))* can be obtained from following relation;

$$\frac{1}{f^2} = \frac{1}{fr^2} + \frac{1}{ft^2} + \frac{1}{f(a1,a2)^2} \quad (3.18)$$

3.4 Rayleigh's Method for Critical Buckling Load of the Tower

Another application for Rayleigh energy method is to estimate the critical buckling load for the tower. For any possible deformations of the tower which satisfy with its boundary conditions, the total potential energy *(Π)* shall be zero. Therefore, for designated deflection of the tower, the global buckling load of the tower is the minimum of all possible axial compression loads. Based on this principle, the axial load work *(W_{max})* including the self-weight can be determined by;

$$W_{\max}(a1,a2) = \frac{1}{2}[Pcr\int_0^h y'(a1,a2,x)^2 \cdot dx + Wh \cdot \int_0^h y'(a1,a2,x)^2 \cdot dx + \int_0^h m(x) \cdot g \cdot x \cdot y'(a1,a2,x)^2 \cdot dx] \quad (3.19)$$

The first term is the work done by the tower critical load *(Pcr)* or simplified as **$Pcr\ \lambda_{max}$**. The second and third terms are the work done by the turbine head weight and the tower self weight or simplified as **Ws_{max}** According to the minimum potential energy principle;

$$\Pi = U_{\max} - W_{\max} = 0 \quad (3.20)$$

For prescribed deformation shape of the tower, the total potential energy is always above zero. The less of potential energy has, the more accurate deformation shape of tower will be. Thus, the buckling load can be solved by minimizing the potential energy. Assume that the strain energy *(U_{max})* is approximated without considering the flexibility of the foundation;

$$U_{\max}(a1,a2) = \frac{1}{2}\int_0^h E(x) \cdot I(x) \cdot [y''(a1,a2,x)]^2 \cdot dx \quad (3.21)$$

The critical buckling load can be determined by

$$Pcr = \min(\frac{U_{\max} - Ws_{\max}}{\lambda_{\max}}) \quad (3.22)$$

3.5 Remarks on Rayleigh's Method

The Rayleigh method for estimating the natural frequency of tower is very simple and computationally easy without requiring a large amount of iteration as compared with the FEM method from SAP2000 commercial FEM software program. The accuracy of the fundamental frequency calculation from the Rayleigh's method depends on building an accurate deflection shape function. Also, it is limited to relatively simple structures and its fundamental frequency. The comparison between the FEM and the Rayleigh's method for the tower with fixed restraint at base are listed for comparison.

Tower Natural Frequency Comparison for FEM and Rayleigh's Method with Fixed at Base

Tower	1.5MW			3.6MW (EQ)			3.6MW (Wind)			5.0MW		
Type	SAP	Rayleigh	Diff.	SAP	Rayleigh	Diff.	SAP	Rayleigh	Diff.	SAP	Rayleigh	Diff.
Hybrid	0.484	0.503	3.93%	0.528	0.561	6.25%	0.385	0.395	2.60%	0.768	0.814	5.99%
Conc	0.617	0.621	0.65%	0.61	0.685	12.30%	0.442	0.501	13.35%	0.855	0.963	12.63%
Steel	0.412	0.461	11.89%	0.364	0.405	11.26%				0.385	0.43	11.69%

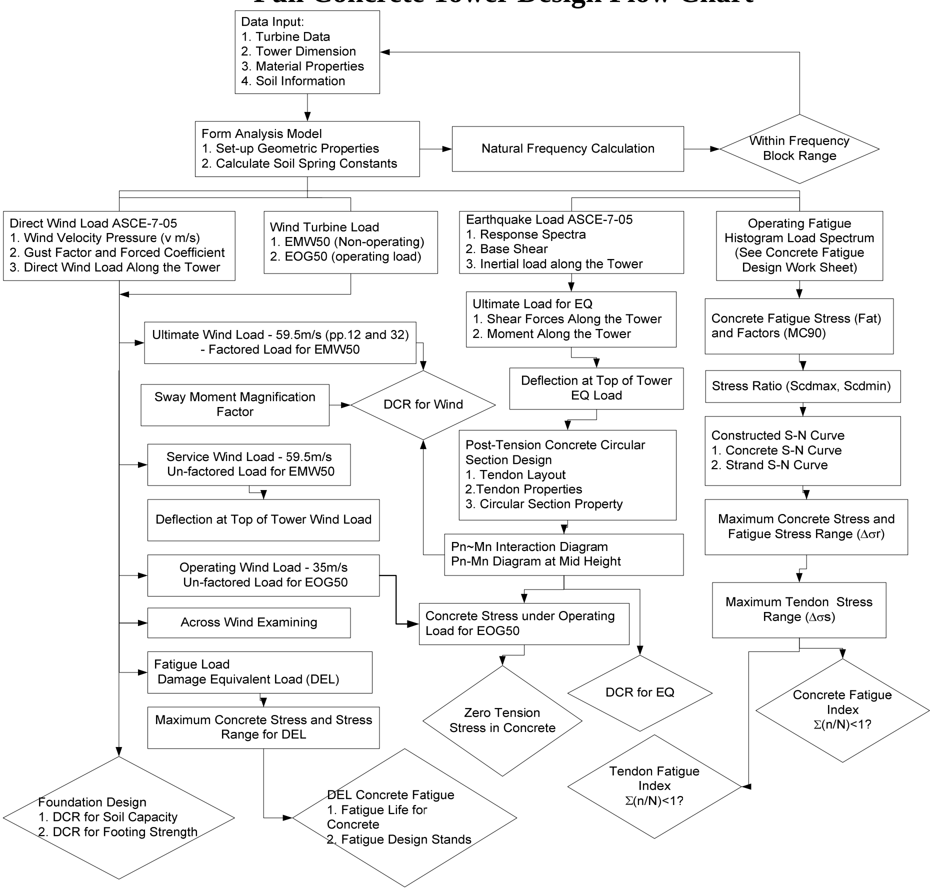
The results of frequency given by the Rayleigh's method are consistently higher than those given by the FEM method. This is because the prescribed flexure curves are generally stiffer than actually are. According to minimum energy principle, exact solution shall have the most minimum energy. This character also applies for the tower critical buckling load calculation. The results show that the Rayleigh's method has good approximation for the tapered tower frequency estimation. It reduces a large amount of time to target the right size of the tower by the trial-error method by using the finite element method.

4.0 Wind Turbine Design Procedure

Wind turbine tower strength design should meet following design criteria:

1. Wind turbine tower dynamic properties should be designed to avoid resonant frequency of the turbine excitation frequencies for turbine bladder rotations.
2. Structural tower should meet design criteria for operation wind load and extreme wind load.
3. Wind turbine tower should be examined for earthquake load.
4. Fatigue load should be included for tower design during the normal operation condition.

Full Concrete Tower Design Flow Chart



5. CONCLUSION

This is still in early stage for large wind turbine concrete tower preliminary design. Dynamic analysis with soil interaction for wind turbine tower should be studied according to site specific soil condition. From our study, seismic design will be most likely governing the concrete tower design for large turbine tower over 80m high due to heavier head mass and tower inertial forces especially in those moderate and high seismic region. Static lateral seismic analysis is appropriate and conservative comparing with seismic response spectra method. Further study is required for investigating seismic modification factor between segmental type and conventional post-tension concrete tower. Since the turbine tower design shall satisfy strength, dynamic properties and fatigue requirement simultaneously, Rayleigh's method is proved to be simple and easy to approximate dynamic properties even for tapered tower sections.

6. ACKNOWLEDGEMENT

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