İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

WIND TURBINE CONICAL TUBULAR TOWER OPTIMIZATION BY USING GENETIC ALGORITHM

M.Sc. Thesis by
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Programme: Aeronautics and Astronautics Engineering

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SYMBOL LIST

B : Horizontal dimension of tower measured normal to wind direction
b, α : Constant
C_c : Material coefficient
C_f : Force coefficient
C_f : Force coefficient
$C_s(T)$: Base shear coefficient
D : Rotor diameter
E : Steel Young's modulus
$F_z(z)$: Lateral wind load pressure on tower
F_a : Site coefficient as a function of site class and short period MCE
F_v : Site coefficient as a function of site class and 1s period MCE
fa : Applied axial compression stress
Fa : Allowable compression stress
fb : Applied bending stress
Fb : Allowable bending stress
ft : Natural frequency in Hz
Fv : Allowable shear stress
Fy : Yielding stress
F(z) : Lateral distribution forces
G_f : Gust factor
g_v,g_Q, g_V : Constant for gust effect factor
g : Accelareation caused by gravity
H : Height of tower
I : Moment of the inertia of the tower cross section
I : Occupancy importance factor is equal to 1.0
I_z : Intensity factor of turbulence
K_d : 0.95 for a round cylinder tower
$K_z(z)$: Terrain exposure coefficient
K_{zt} : Topographic factor
K : Cantilever type of structure
Kh,Kr : Soil spring constants of translation and rotation
k0 : Exponent for the first mode profile.
L : Length of the tower
L_z : Integral length scale of the turbulence
max(ΔMx,yB) : Maximum moment range at tower base x or y direction
max(ΔMx,yT) : Maximum moment range at tower top x or y direction
m : Slope of the curve
MTx, y(z) :Tower overturning moment along the tower due to wind turbine load
Mz(z) : Overturning moment
Mf(z) : Moment produced by the fatigue DEL thrust along steel tower
n_1 : Tower natural frequency

q_z : Velocity pressure
Q : Background response
R : Reduction factor are equal to 1.
R : Resonant response factor
r : Radius of tower section
$S(z)$: Section modulus that varies along the height of tower
S_{D1} : Design spectral response acceleration at 1 second
S_{DS} : Design spectral response acceleration at short periods
S_{M1} : MCE spectral response acceleration at 1 second.
S_{MS} : MCE spectral response acceleration for short periods
$S_a(T)$: Spectra Acceleration
S_1 : Mapped MCE spectral response acceleration at a period of 1s.
S_s : Mapped MCE spectral response acceleration at short periods
T : Structural period
t : Wall thickness of steel tower
$V_{e1}(z)$: One year extreme wind speed
$V_{e50}(z)$: 50 years extreme wind speed
V_{gust} : Extreme operating gust magnitude (EOG)
V_z : Mean hourly wind speed (ft/s) at height z
V : Base Shear
$V(z, t)$: Wind speed shall be defined for a recurrence period of N years
$VDx(z)$: Direct load on x direction
VTx, y : Shear force on tower because of turbine load on either
$Vx, y(z)$: Total shear force along tower
$Vz(z)$: Shear force
$v(z)$: Wind distribution along the tower
W : Total weight of steel tower with Turbine Head
$w(z)$: Weight distribution as a function of height
W_t,Wtow : Weight of head mass and tower mass respectively
w_t : Estimated natural frequency of the tower
z_{hub} : Hub height
γm : Material factor
γsd : Failure factor
β : Damping ratio percent of critical h, B, L
β : Wind shear exponent
Δ_1 : Turbulence scale parameter
γ_{DL} : 1.20 for dead load
γ_{WL} : 1.60 for wind load on tower ASCE-7 Load Factor
$\Delta(z)$: Tower deflection
$\Delta\sigma_{rmax}$: Maximum allowable stress range at N_0 cycles(typically 10^4)
$\Delta Mx, y(z)$: Total moment range along the tower
$\tau(z)$: Overturning reduction factor
α_1 : Standard deviation
α_a : Maximum applied stress
α_u : Buckling stress
α_0 : Reduction coefficient for axial load
α_B : Reduction coefficient for bending load
γ_F : 1.35 for wind turbine loads (IEC) Partial Safety Factor
σ_{cr} : Elastic critical buckling stress

1. INTRODUCTION

1.1 Wind Turbine Towers

The cost of wind turbine towers can amount nearly 20-25% of the total investment cost for wind energy plant. Minimization of mass of wind turbine tower has become more crucial job for the last two decades. Most modern wind turbines are installed with tubular conical steel towers from the point of aesthetics. They are generally manufactured in 20-30 meters long welded sections. They are bolted each other on site.

Steel tubular conical towers are manufactured as the tapered steel or concrete. The steel towers could be welded or press together in sections in a factory or on the site. Transportation condition is limited for towers with base diameters of 4.4 m. Therefore, special transportation provisions are required or the sections must be segmented for shipment and then field assembled to complete circular tower sections.

1.2 Tower Design Criteria

The main target is to obtain a solution which will mitigate and the cost of the wind turbine tower by using genetic algorithm method. The optimum shell thickness along the tower from base to top is calculated as per international engineering standards in accordance with structural stability.

The tower of wind turbine gathers net loads from the tower head and transmits these loads to the foundation. The main load is the axial load on the rotor. On the other hand, dynamic loading is generated by wind turbulence and constantly by blade tower interaction.

The stiffness of tower is based on the tower top weight and the tower height. Additional design requirements have to be satisfied with adequate strength since admissible stresses are not exceeded. For conical towers, shell buckling must be prevented.

1.3 Design Standards

Load calculation of steel conical tower is carried out on the basis of wind turbine design requirements of the standard IEC61400-1 .Thrust force caused by rotor on the tower is taken as per WindPACT study design in NREL. Seismic and direct wind load on tower are calculated as per ASCE 7-98 and Eurocode 1 part 2.4 respectively. In addition fatigue strength analysis is designed in lieu of the Eurocode 3. The strength design criteria is evaluated by AISC-89.

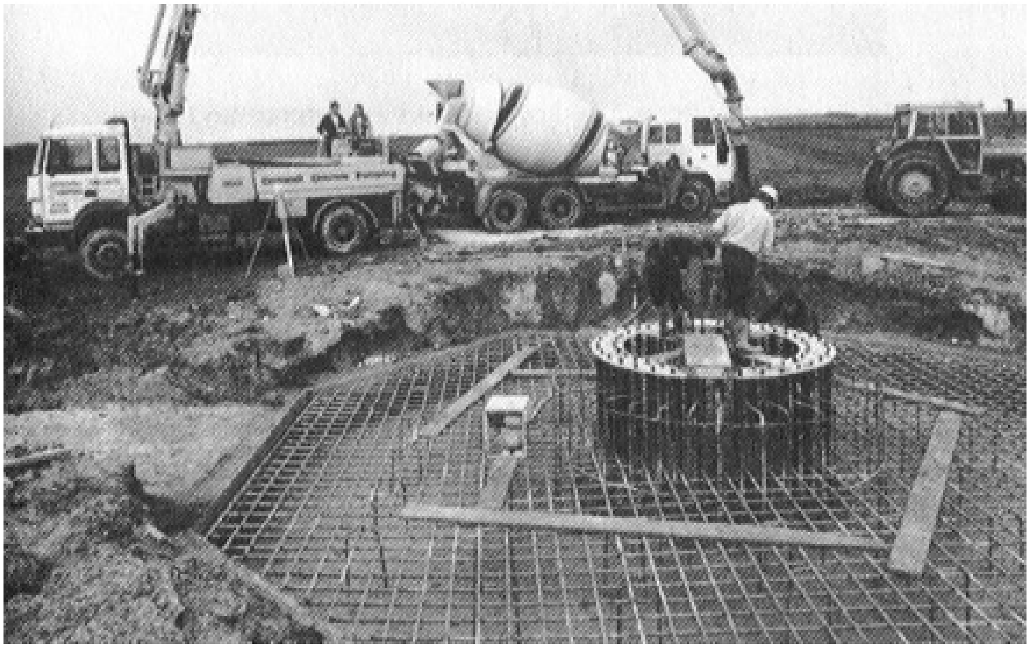

Figure 1.1 Wind Turbine Tower Foundation Work

2. APPROACH AND ASSUMPTION

2.1 Tower Size and Wind Turbine Capacity

The existing pre-sized tower is tackled to evaluate as per analysis and design conditions. Steel tower is assumed to be located in Balıkesir-Bandırma region with 52 m tower height and the 54.7 m tower hub height. The top diameter of tower is 2.56 m and the base diameter tower is 4.3 m. The power capacity of wind turbine is rated 1.5 MW. Wind turbine is provided by General Electric Wind Turbine Technology Company.

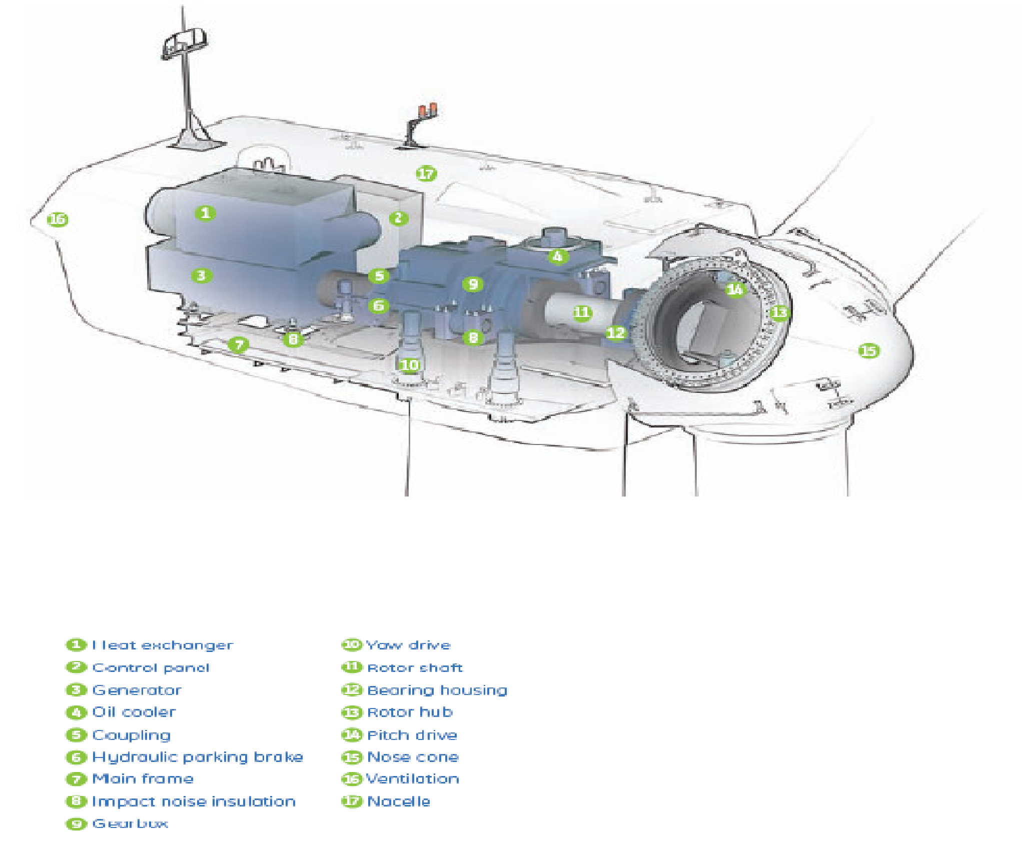

Figure 2.1 Wind Turbine Components

The design concept is to be considered conical shape of steel wind turbine tower with two sections flanged. For wind turbine steel tower, S355J0 material quality is used.

Table 2.1 Wind Turbine Specifications

Technical Data	1.5s	1.5se	1.5sl (50Hz only)	1.5sle	1.5xle
Operating data					
• Rated capacity:	1,500 kW	1,500 kW	1,500 kW	1,500 kW	1,500 kW
• Cut-in wind speed:	4 m/s	4 m/s	3,5 m/s	3,5 m/s	3,5 m/s
• Cut-out wind speed (10 min. avg.):	25 m/s	25 m/s	20 m/s	25 m/s	20 m/s
• Rated wind speed:	13 m/s	13 m/s	14 m/s	14 m/s	12,5 m/s
• Wind Class - IEC:	IIa	Ib	–	IIa ($V_{ave} = 5.5$ m/s)	IIIb ($V_{ave} = 8.0$ m/s)
• Wind Class - DIBt WZ:	II/III	–	II	–	II
Rotor					
• Number of rotor blades:	3	3	3	3	3
• Rotor diameter:	70,5 m	70,5 m	77 m	77 m	82,5 m
• Swept area:	3904 m²	3904 m²	4657 m²	4657 m²	5346 m²
• Rotor speed (variable):	12,0 – 22,2 rpm	12,0 – 22,2 rpm	11,0 – 20,4 rpm	11,0 – 20,4 rpm	10,1 – 18,7 rpm
Tower					
• Hub heights - IEC:	64,7 m	54,7/64,7 m	–	61,4/64,7/80 m	58,7/80/100 m
• Hub heights - DIBt:	64,7 m	–	61,4 to 100 m	61,4/64,7/80/85/100 m	58,7/80/100 m
Power control	Active blade pitch control	Active blade pitch control	Active blade pitch control	Active blade pitch control	Active blade pitch control

2.2 Wind Turbine Tower Design Objectives

The aim of this study is to evaluate and optimize thickness of conical shape of steel tower by using genetic algorithm optimization method in conformity of international standards which have been utilized. As a result of study, the lowest weight is obtained and the highest stiffness is enabled on steel tower structure.

A wind turbine tower is the main structure which supports rotor, control systems and blades. A more efficient structural design of the tower should ensure safety and cost-effective design for the complete wind turbine system.

The diminish in structural weight is advantageous in terms of manufacturing and installation cost. In this study the height and diameter of sections of the tower is assumed to be constant without change as initially sized dimensions. The thickness of each section of steel tower defines the main cost function in calculation mass of tower.

The main tower structure must have a sufficient strength. Maximization of strength is the main criteria to augment the overall structure stability and decrease the probability of fatigue failure against cracks and distortion on tower.

2.3 Analysis and Design Methodology

Wind turbines have been developing for a couple of decades in all over the world. Since the cost of wind energy has been becoming more lucrative and competitive when comparing to other energy alternatives and also demand for the investment of wind energy has decreased gradually in many countries especially in Denmark, Germany Spain, Netherlands and US and Eastern Europe countries as well. The fact that wind energy is renewable and has no direct pollution concerning environmental effects COx and NOx emissions, makes wind energy more attractive in all over the world.

Because of above development on wind energy, many wind turbine manufacturers and other structural providers are focused on optimization to reduce investment cost initially to compete other energy resources such as nuclear, coal, natural gas.

Figure 2.2 Wind Turbine Installation Work

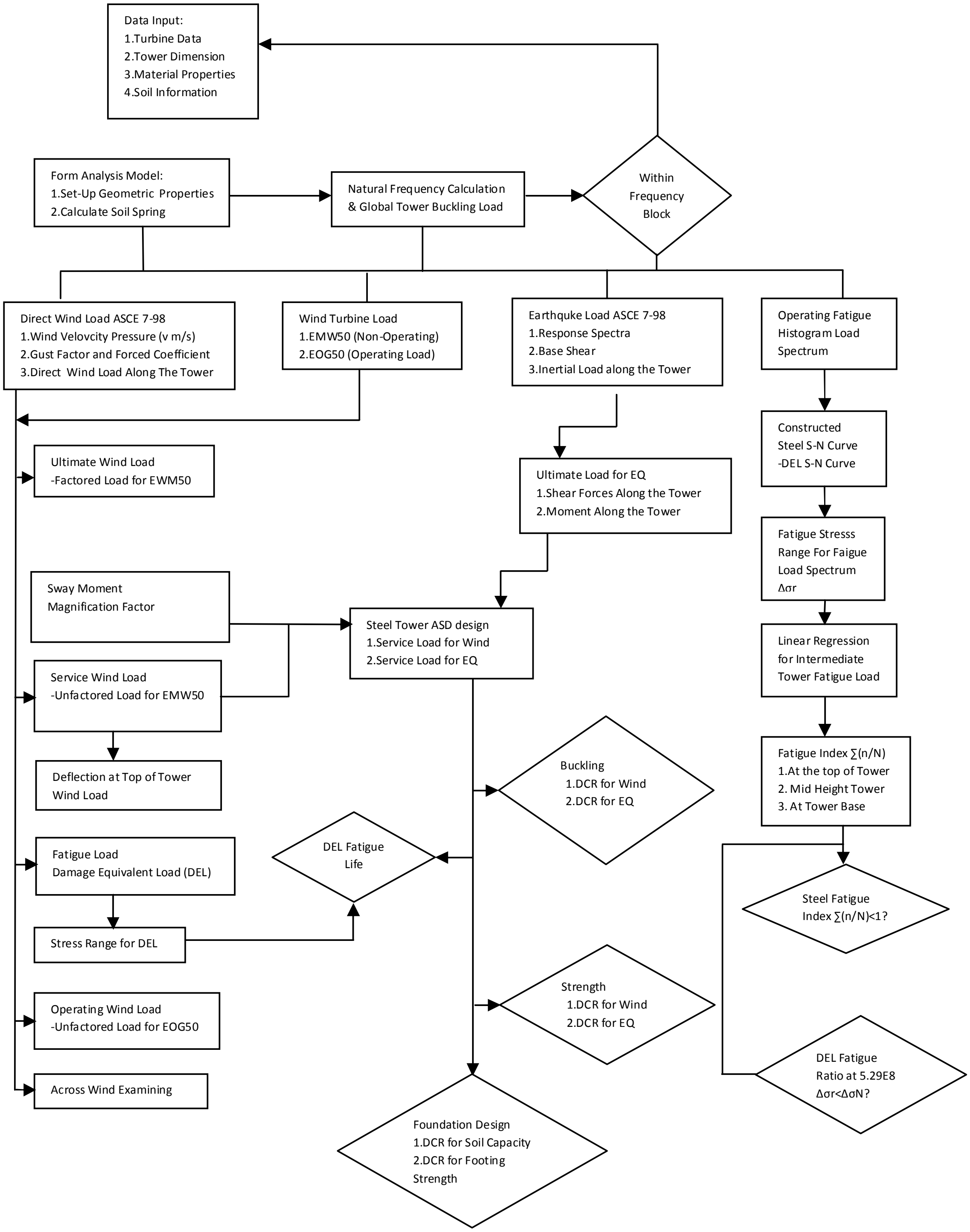

Figure 2.3 Flow Chart of Load Calculations [6]

3. DESIGN OF CONICAL STEEL TOWER

3.1 Design Considerations

The most widely used tower type currently for wind turbines is the cantilever conical shape steel tube. Typical tubular steel towers are constructed as tapered conical tubes in which the diameter and wall thickness abates from the base of the tower to the top. The typical tubular steel towers are costly installed with prefabricated conical tubes delivered to the site from a workshop. The dimensions and shapes of the steel towers depend firstly on maximum strength, stiffness, local buckling, and fatigue strength requirements.

3.1.1 Allowable Stress Design (ASD)

The steel towers are primarily designed and sized to meet the AISC strength design criteria. Allowable stress design method (ASD) is used in lieu of AISC-89 for the steel tubular tower design.

The load combination method for the service load (characteristic load) condition is carried out with reference to ASCE-7-98. A 0.7 load factor is executed for the earthquake load accordingly. The allowable bending stress Fb for noncompact section is 0.6 Fy, in which the yielding stress Fy of the steel tubular structure is typically 345 MPa (50 ksi). The allowable shear stress Fv is 0.4 Fy. The allowable compression stress Fa is represented by the following formula [6]:

$$\mathrm{F} = \frac{\left[1 - \frac{(\mathrm{K}\frac{\mathrm{L}}{\mathrm{r}})^2}{2.\mathrm{C_c}^2}\right].\mathrm{Fy}}{\frac{5}{3} + \frac{3(\mathrm{K}\frac{\mathrm{L}}{\mathrm{r}})}{8\mathrm{C_c}} - \frac{(\mathrm{K}\frac{\mathrm{L}}{\mathrm{r}})^3}{8\mathrm{C_c}^3}} \tag{3.1}$$

Where, $(\mathrm{K}\frac{\mathrm{L}}{\mathrm{r}})$ is the slenderness ratio of steel tower. K is 2 for the cantilever type of structure, and L and r are the length of the tower and radius of section, respectively. The material coefficient Cc is calculated by:

$$C_c = \sqrt{\frac{2.\pi^2.E}{Fy}} \tag{3.2}$$

Where E is the steel modulus. When KL/r is greater than Cc, the allowable compression stress Fa shall be recalculated by:

$$Fa = \frac{12.\pi\hat{}2.E}{23.(K\frac{L}{r})^2} \tag{3.3}$$

Typically, the ratio of the applied axial compression stress fa to the allowable compression stress Fa of the steel tower is less than 0.15. The combined stress for the applied bending stress fb acting on the steel tower shall be satisfied with interaction equation.

$$\frac{fa}{Fa} + \frac{fb}{Fb} \leq 1 \tag{3.4}$$

3.1.2 Fatigue Design

For the steel tower design, the fatigue loading in addition to stiffness and strength of tower is critical due to the large number of cyclic repetitive loads from the wind turbine’s routine operation. Fatigue is a local flaw on material resulted from variations of stresses or strains. There are two type of fatigue cycles which are low-cycle and high-cycle fatigue. Low-cycle fatigue is related to non-linear material and geometric behavior. High-cycle fatigue is mainly governed by elastic behavior.

Wind turbines are subject to fluctuating winds and hence fluctuating forces. Metal fatigue is a well known problem in many industries. Metal is therefore generally not favored as a material for rotor blades. When designing a wind turbine it is extremely important to calculate in advance how the different components will vibrate both individually, and jointly. It is also important to calculate the forces involved in each bending or stretching of a component.

The stress-range frequency distributions or stress–range spectra may be transformed to fatigue-damage equivalent constant-amplitude stress-range spectra using Miner's rule, into DEL .The safety verification for fatigue at the limit state equal to the total fatigue loading over the life of the tower may be carried out by:

The DEL method facilitates to determine the steel tower preliminary dimensions in any circumstances which fatigue load histogram data does not exist. The SN curve for the DEL method can be expressed in the following [6]:

$$\log[\Delta\sigma s(n)] = \log(80\ \mathrm{MPa}) + \frac{2x10^{6} - n}{m} \tag{3.5}$$

The number of cycles corresponding to the withstand limit along the tower height z can be calculated by using DEL method.

$$N(z) = \frac{Mf(z)}{\Delta\sigma_{rmax} S(z).\sqrt[m]{N_0}} \tag{3.6}$$

Where:

$Mf(z)$	is the moment produced by the fatigue DEL thrust along steel tower
$S(z)$	is the section modulus that varies along the height of tower
$\Delta\sigma_{rmax}$	is the maximum allowable stress range at N_0 cycles (typically 10^4)
m	is the slope of the curve
n	is the number of cycles.

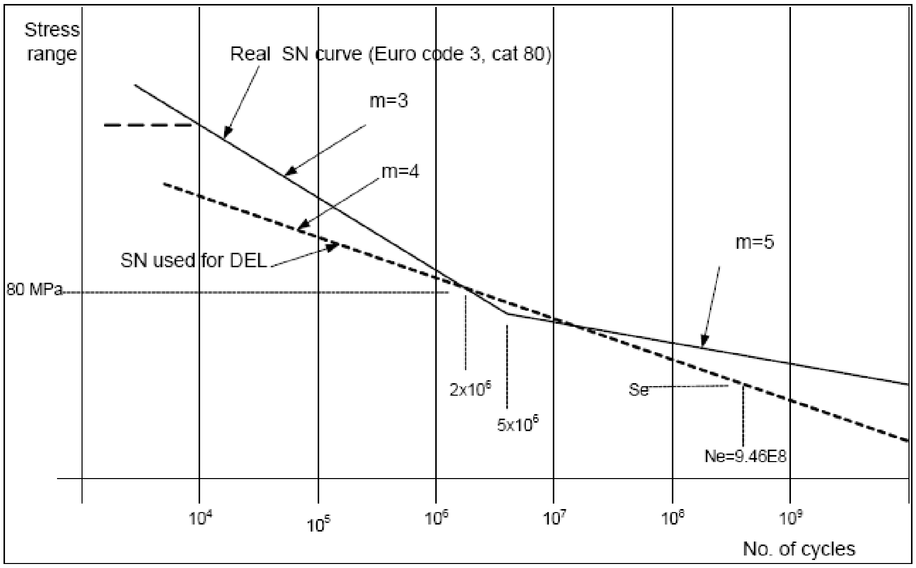

Figure 3.1 Damage Equivalent Load for Steel Tower

3.1.2.1 Operational Wind Fatigue Design

For the fatigue design of steel towers WindPACT loads was reviewed in accordance with Damage Equivalent Load (DEL) approach to reach accurate results. For the wind turbine steel tower, the effects of direct wind load are not taken into account in lieu of the current industry study. Fatigue was reviewed at the base of tower and midpoint of the tower.

3.1.2.2 Damage Equivalent Load for Steel Tower

There are many fatigue calculation methods. One of which is DEL method in most cases where the full histogram of fatigue cycles is available but only a DEL specified. The DEL is added by a value of SN slope (m=4 used in this circumstance and a number of cycles (Ne)).

Total moment range along the tower is calculated as follows:

$$\Delta \mathrm{Mx}, \mathrm{y}(\mathrm{z}) = \frac{[\max(\Delta \mathrm{Mx}, \mathrm{yT}) - \max(\Delta \mathrm{Mx}, \mathrm{yB})]\mathrm{z}}{\mathrm{h}} + \max(\Delta \mathrm{Mx}, \mathrm{yB}) \qquad (3.6)$$

max(ΔMx,yT)=Maximum moment range at tower top x or y direction

max(ΔMx,yB)= Maximum moment range at tower base x or y direction

Safety Factor of DEL is 1.0

Consequence failure factor and material factor:

γsd.γm=1.15x1.1=1.265 [6]

Number of cycles $5.29\mathrm{x}10^8$ for 1.5 MW turbine this represents a 20 year lifetime.

3.1.3 Local Buckling Stress

The strength of the tubular steel tower in axial compression is the lesser of the yield strength and the elastic critical buckling stress σ_{cr} is calculated by:

$$\sigma_{cr} = 0.605\ \mathrm{E}.\frac{\mathrm{t}}{\mathrm{r}} \qquad (3.7)$$

Where, r is the cylinder radius and t is the wall thickness. However, the presence of imperfections, particularly those introduced by welding, will significantly reduce the tower wall resistance to buckling. As per steel tower design, the reduction coefficient α_0 for axial load is found by:

$$\alpha_0 = \begin{cases} \dfrac{0.83}{\sqrt{1+0.01.\mathrm{r/t}}} & \text{if } \dfrac{\mathrm{r}}{\mathrm{t}} < 212 \\ \dfrac{0.70}{\sqrt{0.1+0.01.\mathrm{r/t}}} & \text{if } \dfrac{\mathrm{r}}{\mathrm{t}} > 212 \end{cases} \tag{3.7}$$

The reduction coefficient αB for bending load is calculated as follows:

$$\alpha_B = 0.1887 + 0.8113\alpha_o \tag{3.8}$$

The buckling stress σu can be computed in terms of the yielding stress Fy:

$$\alpha_u = \begin{cases} Fy\left[1 - 0.4123\left(\dfrac{Fy}{\alpha_B.\sigma_{cr}}\right)^{0.6}\right] & if \quad \alpha_B.\sigma_{cr} > Fy/2 \\ 0.175.\alpha_B.\sigma_{cr} & if \quad \alpha_B.\sigma_{cr} < Fy/2 \end{cases} \tag{3.9}$$

The maximum applied stress σa combined with normal stress and shear stress is calculated by σa :

$$\sigma_a = \sqrt{(fa+fb)^2 + 3fv^2} \tag{3.10}$$

The unity strength ratio check for combined stresses is found as follows. Now that the steel tower is liable to combined stress with axial compression and bending moment, the steel tower is designed to satisfy the combined stress check. This check named unity check interaction equation is carried out in accordance with the AISC manual (ASD 9th Edition).

$$\frac{fa}{Fa} + \frac{fb}{Fb} \leq 1 \quad \text{for } fb \leq 0.15Fb \tag{3.11}$$

Where

fa is the applied compression stress

Fa is the allowable stress

fb is the applied bending stress

Fb is the allowable bending stress

4. LOADS ON TOWER

4.1 Earthquake Load

This section is based on ASCE 7-98 Earthquake Load Specification. Even though earthquake load seems to be not much significant effect on design of steel tower because of the fact that wind turbine towers are placed in low seismic areas, earthquake load should be taken into account so as to be more precise in designing of steel tubular tower.

Steel tubular tower structures particularly weigh lesser than concrete structures, they therefore are subjected to less inertial force than concrete towers.

4.1.1 Site Class Parameter

The definitions presented here below apply to upper 100ft (30m) of site profile. Profiles encompassing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom in which there are a total of n distinct layers in the upper 100 ft (30 m) [5].

Vs is the shear wave velocity in ft/s

N is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/ft as directly measured in the field without corrections [5].

S is undrained shear strength in psf(kPa) not to exceed 5000 psf (240 kPa)

Site Coefficients and adjusted maximum considered earthquake spectral response acceleration parameters:

The maximum considered earthquake spectral response acceleration for short periods ($\mathrm{S_{MS}}$) and at 1 s ($\mathrm{S_{M1}}$) adjusted for site class effects, should be determined by:

$$S_{MS} = Fa.Ss \tag{4.1}$$

$$S_{M1} = Fv.S_1 \tag{4.2}$$

Where:

S_1 = Mapped maximum considered earthquake spectral response acceleration at a period of 1s as determined in accordance with Section 9.4.1 (ASCE 7-89)

S_s = Mapped maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1 (ASCE 7-89).

F_a and F_v are defined in Tables 9.4.1.2.4a and b respectively in accordance with Section 9.4.1 (ASCE 7-89). According to the ASCE 7-89 9.4.1.2.5 design spectral response acceleration at short periods, S_{DS} and at 1 s period S_{D1}, shall be determined from equations 9.4.1.2.5-1 and 9.4.1.2.5-2 respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \tag{4.3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{4.4}$$

From the earthquake geographic map, the maximum considered earthquake (MCE) ground motion for soil site Category B with 5% damping is 1.5 g (S_s) for structures with a short period of 0.2 s and 0.6 g (S_1) for structures with a period of 1 s. The wind turbine towers are typically located in open areas away from population centers with very low occupancy. Because, the occupancy importance factor (I) is equal to 1.0. Site Classification D is assumed for Balıkesir-Bandırma/Marmara Region. Site Classification D is typified by stiff soils with shear velocity (Vs in soil) typically 600–1,200 fps (183–366 m/s). For an actual site specific design, the soil category will be determined from the results of a geotechnical investigation [5].

$$S_{DS} = \frac{2}{3} F_s S_S = 1.0g \tag{4.3}$$

$$S_{DI} = \frac{2}{3} F_v S_I = 0.6g \tag{4.4}$$

F_a and F_v can be defined according to the ASCE 7-98 Table 9.4.1.2.4a-4b respectively.

F_a is the site coefficient as a function of site class and short period MCE

F_v is the site coefficient as a function of site class and a 1 second period MCE

g *is* the acceleration caused by gravity.

Table 4.1 Site Classifications Determination in accordance with ASCE 7-98 [5].

Site Classificitation	Vs	N or Nch	Su
A-Hard Rock	>5000 fps (>1500 m/s)	N/A	N/A
B-Rock	2500 to 5000 fps (760 to 1500 m/s)	N/A	N/A
C-Very dense soil and soft rock	1200 to 2500 fps (760 to 1500 m/s)	>50	>2000 psf (>100 kPa)
D-Stiff Soil	600 to 1200 fps (180 to 370 m/s)	15 to 50	1000 to 2000 psf (50 to 100 kPa)
E-Soil	<600 fps (<180 m/s)	<15	<1000 psf (<50 kPa)
F-Soils requiring site specific evaluation			
1-Soils vulnarable to potential failure or collapse			
2-Peats and/or highly organic clays			
3-Very high plasticity clays			
4-Very thick soft/medium clays			

$$S_a(T) = \begin{cases} \dfrac{S_{DI}}{T} & if \quad T < T_s \\ S_{DS}\left(0.4 + 0.6\dfrac{T}{T_0}\right) & if \quad T < T_0 \\ S_{DS} \;\; Otherwise & \end{cases} \tag{4.5}$$

$$T_S = S_{DI}/S_{DS} \tag{4.6}$$

$$T_0 = 0.2.T_s \tag{4.7}$$

Design spectral response acceleration, T is the structural period.

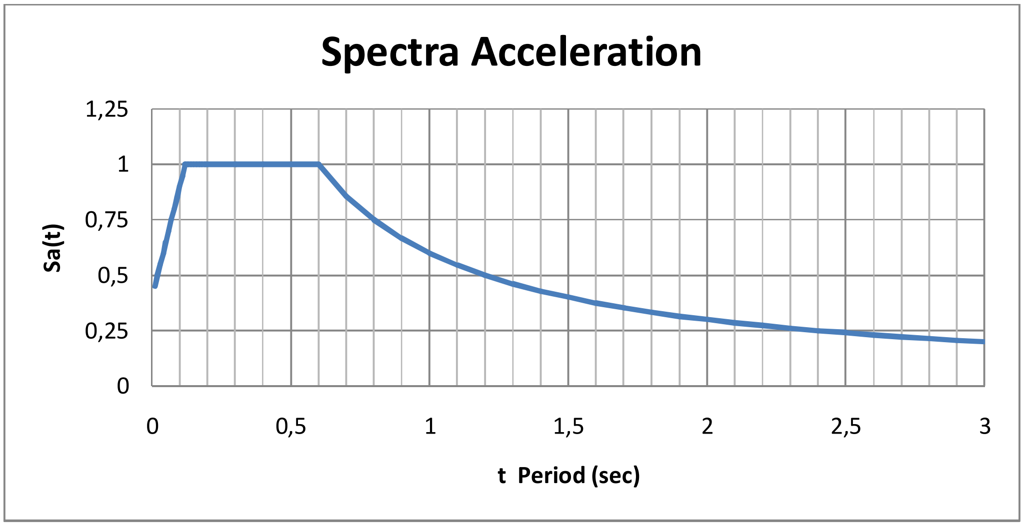

Figure 4.1 Earthquake Spectra Acceleration

4.1.2 Design Earthquake Load

The earthquake lateral load affects the whole tower height h as per its weight distribution.

$$W = \int_z^h w(z)dz + W_{Head\ Mass} \tag{4.8}$$

w(z) is weight distribution as a function of height

W is the total weight of steel tower with Turbine Head

Base shear coefficient;

$$C_s(T) = Sa(t).\frac{I}{R} \tag{4.9}$$

I is the importance factor. R is the reduction factor. Both R and I are equal to 1 [5].

Base Shear is $V = C_s(T)W$ (4.10)

The period of the tower T can be estimated by $Ta = Ct.h^{0.75}$ (4.11) where Ct =0.02 structural coefficient, Ta approximate fundamental period of the tower structure as per ASCE 7-98.

The lateral distribution forces F(z) can be determines as here below :

$$F(z) = \frac{w(z)z^{k0}}{\int_z^h w(z)z^{k0}dz + W_{Head\ Mass}h^{k0}}V \tag{4.12}$$

$$k_0 = \begin{cases} 1 & if\ T < 0.5\ s \\ 2 & if\ \ T > 2.5\ s \\ 0.5T + 0.75 & Otherwise \end{cases} \tag{4.13}$$

k_0 is the exponent for the first mode profile.

$$Ft = \frac{W_{Head\ Mass}h^{k_0}}{\int_0^h w(z)z^{k_0}dz + W_{Head\ Mass}h^{k_0}}V \tag{4.14}$$

Shear force Vz(z) and overturning moment Mz(z) along the tower :

$$Vz(z) = \int_z^h F(x).dx + Ft \tag{4.15}$$

$$Mz(z) = \tau(z).\left[\int_z^h F(z).(x-z).dx + Ft.(h-z)\right] \tag{4.16}$$

Where, the overturning reduction factor $\tau(z)$ is determined as follows:

For the top 10 stories $\tau(z) = 1.0$, for the 20th story from the top and below $\tau(z) = 0.8$ and for stories between the 20th and 10th stories below the top , a value between 1.0 and 0.8 determined by a straight line interpolation according to the ASCE 7-98 [5].

Tower deflection can be calculated via below formula:

$$\Delta(z) = \int_0^z \frac{Mz(x)}{E(x).I(x)}.(z-x)dx + \frac{V_{zo}}{Kh} + \frac{M_{zo}}{Kr}.z \tag{4.17}$$

Kh and Kr are soil spring constants of translation and rotation.

4.2 Wind Load

In most cases, the loads on a wind turbine can be classified as follows [15]:

-Aerodynamics blade loads
-Gravity loads on the rotor blades
-Centrifugal forces and Carioles forces due to rotation
-Gyroscopic loads due to yawing
-Aerodynamic drag forces on tower and nacelle
-Gravity loads on tower and nacelle

The forces that induce on the rotor and hub that are transmitted to the tower and ultimately to the foundation is due to the effects of wind mass and aero-elastic forces. Wind turbine design loads consist of inertia, mass and aerodynamic forces acting on the rotor.

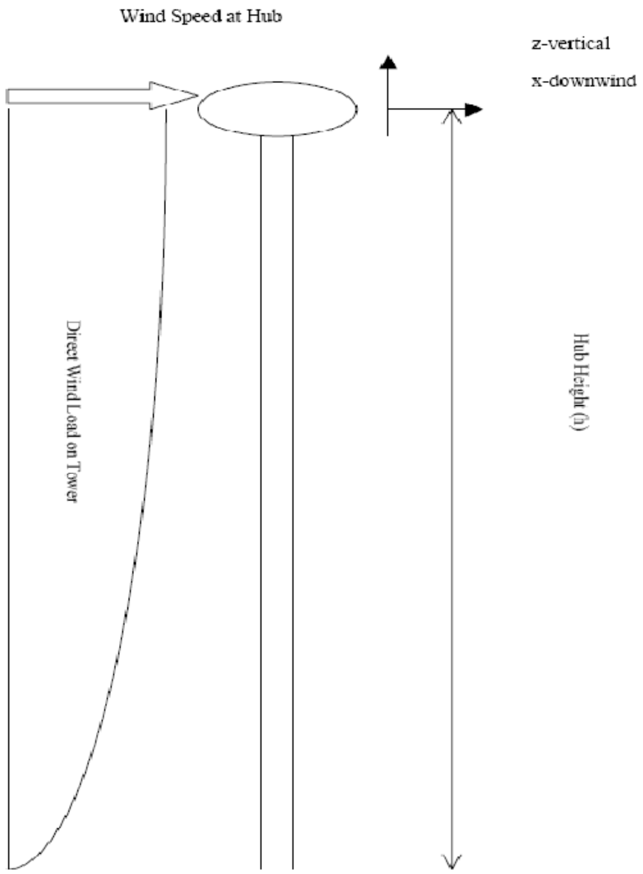

Figure 4.2 Wind Turbine Tower Parameters

4.2.1 Direct Wind Pressure on Tower

Direct wind load on the towers differs for the tower dimension. Wind load effect on the tower is designed according to ASCE 7 -98.The partial safety factor determined by IEC 61400-1 standard for wind turbine design is 1.1DL+1.35WL for normal and extreme load, for ASCE 7-98 load factor is 1.2 DL+1.6WL [6].

ASCE load factors for direct wind load on the tower structure were used because they were more consistent with the code method used to calculate the direct wind load on the tower. The steel towers are particularly less stiff and remarkably not heavier than the concrete towers for the identical same tower top turbine loads.

According to the Extreme Wind Speed Model (EWM), the 50 years extreme wind speed Ve50 and the one year extreme wind speed Ve1 shall be based on the reference wind speed. For the Wind Turbine Generator System designs, Ve50 and Ve1 are determined in the following as per IEC 61400-1 [4]:

$$V_{e50}(z) = 1.4V_{ref}(\frac{z}{z_{hub}})^{0.11} \tag{4.18}$$

$$V_{e1}(z) = 0.75V_{e50}(z) \tag{4.19}$$

For the extreme non-operating condition IEC recommends EWM50 and likewise for the extreme operating condition, IEC recommends EOG50.

z_{hub} is the hub height (from base to the centre of nacelle).

z is the tower height (from base to the top of tower).

Extreme operating gust magnitude (EOG) V_{gust} for a recurrence period of N years shall be represented by the following formula;

$$V_{gustN} = \beta\left[\frac{\alpha_1}{1 + 0.1(\frac{D}{\Delta_1})}\right] \tag{4.20}$$

Where:

α_1 is the standard deviation according to the equation

$$\alpha_1 = \frac{I_{15}\left(15\frac{m}{s} + aV_{hub}\right)}{a+1} \tag{4.21}$$

$\varDelta_1$ is the turbulence scale parameter ,according to below equation

$$\varDelta_1 = \begin{cases} 0.7\ z_{hub}\ for\ z_{hub} < 30\ m \\ 21m\ for \quad z_{hub} \geq 30m \end{cases} \tag{4.22}$$

D is the rotor diameter

β =4.8 for N=1 and β=6.4 for N=50 in accordance with ASCE 7-98.

Values for I_{15} and a are given in table 1 in IEC 61400-1.

The wind speed shall be defined for a recurrence period of N years by the equation:

$$V(z,t) = \begin{cases} V(z) - 0.37V_{gustN}\sin\left(\frac{3\pi t}{T}\right)\left(1-\cos\left(\frac{2\pi t}{T}\right)\right) & for\ 0 \leq t \leq T \\ V(z) & for\ \ t < 0\ and\ \ t > T \end{cases} \tag{4.23}$$

Where $V(z)$ is defined in equation $V(z) = V_{hub}(\frac{z}{Zhub})^{0.2}$ (4.24)

T=10.5 s for N=1 and

T=14 s for N=50

For the extreme direct wind acting on the tower related to IEC non-operating EWM50 ,a wind shear exponent β=0.1 and for operational wind speed related to IEC extreme operating condition EOG50 , a wind shear exponent β=0.2 is used .

Accordingly, wind distribution along the tower is $v(z) = v\_hub(\frac{33ft}{z})^{\beta}$ (4.24)

The towers are assumed to be located in flat unobstructed area for direct wind exposure Category D where wind flows over the open water and flat terrain.

Importance factor is 1.0 for low occupancy concerning the wind turbine erection and installation.

The velocity pressure

$$q_z = 0.613K_zK_{zt}K_dV^2\ \left(\frac{N}{m^2}\right) \tag{4.25}$$

Where:

The topographic factor K_{zt} is 1.0 for the flat area.

K_d is 0.95 for a round cylinder tower in accordance with Table 6-6 in ASCE 7-98.

The terrain exposure coefficient is determined as per Table 6-5 of ASCE 7-98 or by the following formula:

$$K_z(z)\begin{cases} 2.01(\frac{15ft}{z_g})^{\frac{2}{\alpha_1}} & if\ z < 15\,ft \\ 2.01(\frac{z}{z_g})^{\frac{2}{\alpha_1}} & otherwise \end{cases} \tag{4.26}$$

Where z_g the nominal height of the atmospheric boundary layer is 213 m and α_1 is 11.5 for exposure D category in accordance with ASCE 7-98 [5].

4.2.2 Direct Wind Load on Tower

The direct wind load on the tower is based on not only the direct wind pressure on the tower but also on the gust factor G_f and the force coefficient C_f [5].

G_f is calculated by the following equation :

$$G_f = 0.925\left(\frac{1 + 1.7I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7g_v I_z}\right) \tag{4.27}$$

Where:

The intensity factor of turbulence $I_z = 0.15(33ft/z)^{1/6}$ (4.28)

The background response Q and the resonant response are given in accordance with Eq.6.4 of ASCE 7-98.

$$Q = \sqrt{\frac{1}{1 + 0.63(\frac{B+h}{L_z})^{0.63}}} \tag{4.29}$$

Where B is the horizontal dimension of tower measured normal to wind direction, L_z is the integral length scale of the turbulence at the equivalent height given by $L_z = l(z/33ft)^{\epsilon}$, l and € are constants listed in Table 6.4 of ASCE 7-98.

g_R and g_Q shall be taken as 3.4 and g_V is given by:

$$g_v = \sqrt{2\ln\left(3600n_1\right)} + \frac{0.577}{\sqrt{2\ln\left(3600n_1\right)}} \tag{4.30}$$

R the resonant response factor is given by:

$$R = \sqrt{\frac{1}{\beta}R_n R_h R_B (0.53 + 0.47R_L)} \tag{4.31}$$

$$R_n = \frac{7.47N_1}{(1 + 10.3N_1)^{5/3}} \tag{4.32}$$

$$N_1 = \frac{n_1 L_z}{V_z} \tag{4.33}$$

$$R_l = \frac{1}{\acute{\eta}} - \frac{1}{2\acute{\eta}^2}\left(1 - e^{-2\acute{\eta}}\right) \quad for \quad \acute{\eta} > 0 \tag{4.34}$$

$$R_l = 1 \quad for \quad \acute{\eta} = 0$$

Where

n_1 = tower natural frequency;

$R_l = R_h$ settings $\acute{\eta} = 4.6\frac{n_1 h}{V_z}$

$R_l = R_B$ settings $\acute{\eta} = 4.6\frac{n_1 B}{V_z}$

$R_l = R_L$ settings $\acute{\eta} = 15.4\frac{n_1 L}{V_z}$

β = damping ratio percent of critical h, B, L

V_z is mean hourly wind speed (ft/s) at height z determined from below equation:

$$V_z = b\left(\frac{z}{33ft}\right)^{\alpha} V\left(\frac{88}{60}\right) \tag{4.35}$$

Where b and α are constants listed in Table 6.4 of ASCE 7-98 and V is the basic wind speed in mph.

The force coefficient C_f is determined as per Table 6-10 of ASCE 7-98 for the ratio of height to diameter of 12, C_f is approximately selected 0.64 with the interpolation method in accordance with values in the Table 6-10 for the moderately smooth round cylinder tower with $D\sqrt{q_z} > 2.5$ where D is average diameter of tower [5].

Lateral wind load along the tower is calculated by the direct pressure on the projected area which differs with respect to diameter d(z). $F_z(z)$ is determined in the following equation.

$$F_z(z) = q_z G_f C_f d(z) \tag{4.36}$$

Accordingly, wind shear force and overturning moment Vz(z) and Mz(z) respectively along the tower are calculated with the below formula:

$$Vz(z) = \int_z^h F_z(x)dx \tag{4.37}$$

$$Mz(z) = \int_z^h F_z(z).(x-z)dx \tag{4.38}$$

The tower deflection is along the tower can be calculated:

$$\Delta(z) = \int_0^z \frac{M_z(x)}{E(x)I(x)}(z-x)dx \tag{4.39}$$

Where E is the elasticity modulus of structural material. I is the moment of the inertia of the tower cross section.

4.3 Load Factors and Load Combinations for Ultimate Design Wind Load

For more ultimate strength design, many factors are incorporated to existing design load combinations. For instance, in conformity of the ASCE recommendations structures, components and foundations shall be designed so that their ultimate design strength equals or exceeds the effects of factored loads in the following combinations.

Partial Safety Factor :

$\gamma_F = 1.35$ for wind turbine loads (IEC) [4]

ASCE-7 Load Factor:

$\gamma_{WL} = 1.60\ for\ wind\ load\ on\ tower$

$\gamma_{DL} = 1.20\ for\ dead\ load$

Combination of EWM50 and EOG50 (Unfactored Wind Load)

$$DL + WL_{direct} + TWL\_turbine \qquad (4.40)$$

Factored Load Combination for EWM:

$$\gamma_{DL}.DL + \gamma_{WL}.WL_{direct} + \gamma_F.TWL\_turbine \qquad (4.41)$$

Fatigue Wind Load Combination:

$$DL + WL_{direct} + \Delta TWL\_turbine(fatigue\ load) \qquad (4.42)$$

Summary of load conditions as follows [6]:

- 1.4DL (4.43)
- 1.2DL+(1.35TWL+1.6WL) (4.44)
- 1.2DL+EQ (4.45)
- 0.9DL-(0.35TWL+1.6WL) (4.46)
- 0.9DL-EQ (4.47)
- 1.0DL+ ΔWL turbine fatigue load (4.48)
- 1.0DL+1.0TW+1.0WL (4.49)

Where:

DL is dead load

TWL is the wind-induced turbine load

WL is direct wind load on the tower

EQ is earthquake load

Ultimate design wind load=Extreme wind load effects(with safety factors)+Factored direct wind load on tower

Wind load combination as in the following:

Shear distribution along the tower on x or y direction

The total shear force along tower:

$$Vx, y(z) = VTx, y + VDx(z) \tag{4.50}$$

Where:

VTx, y is shear force on tower because of turbine load on either x or y direction

$VDx(z)$ is direct load on x direction only along differed along the tower

Moment distribution along the tower

Tower overturning moment MTx,y(z) along the tower due to wind turbine load can be calculated by the linear interpolation method.

$$MTx, y(z) = \frac{[max(Mx, yT) - max(Mx, yB)]z}{h} + max(Mx, yB) \tag{4.51}$$

Where, max(Mx,yT) is maximum moment of x or y direction at top of the tower due to wind turbine load.

max(Mx,yB) is maximum moment of x or y direction at tower base due to wind turbine load.

Moment distribution along the tower on x and y direction

Total moment along the tower is $Mx, y(z) = MTx, y(z) + MDx(z)$ (4.52)

Where MTx, y (z) *is* overturning moment due to wind turbine load

MDx(z) is overturning moment due to direct wind load on x direction

Wind load direction combination

Shear $V(z) = \sqrt{Vx(z)^2 + Vy(z)^2}$ (4.53)

Moment $M(z) = \sqrt{Mx(z)^2 + My(z)^2}$ (4.54)

PT =load applied at the tower top in the z direction (along vertical axis of the tower).

VT =load applied at the tower top in the horizontal x (downwind) direction.

MT = moment applied at the tower top.

PD = tower dead load, not including the turbine head weight.

VD = tower base shear resulting from the effect of direct wind on the tower.

MD = tower base moment resulting from the effects of direct wind on the tower.

P = total load applied at tower base, including tower dead load and turbine head weight.

V = total tower base shear, including direct wind and turbine load effects.

M =total tower base moment, including direct wind effects appropriately combined with turbine load effects [6].

Service wind load is regarded as the controlling unfactored case either EOG50 or EWM50. As a result, the unfactored service design wind load is simply found as follows:

Service Wind Load = Unfactored EOG50 or EWM50 wind load + Unfactored direct load on the tower.

4.4 Dynamic Behavior of Steel Tower

Main key consideration in wind turbine design is the avoidance of resonant tower oscillations excited by rotor thrust fluctuations at rotational or blade passing frequency. The damping ratio may be only 2-3 percent for tower oscillations and order of magnitude less for side-to-side motion, so unacceptably large stresses and deflections could develop if the blade passing frequency and tower natural frequency were to coincide. Rotational frequency is less of a concern because cyclic loadings at this frequency only arise if there are geometrical differences between blades.

Wind turbine towers are customarily classified as per the relationship between the tower natural frequency and exciting frequencies. Natural frequency of tower greater than the blade-passing frequency is said to be stiff on the other hand towers natural frequency between rotational frequency and blade passing frequency are regarded as soft [9].

$$w_t = 1.75\sqrt{\frac{EIg}{H^3(W_t + 0.25Wtow)}} \tag{4.52}$$

Where:

w_t is the estimated natural frequency of the tower

H is the height of tower

E and I are elastic modulus and moment of inertia of the tower

W_t and $Wtow$ are the weight of head mass and tower mass respectively

The natural frequency in Hz is calculated by:

$$ft = w_t/2\pi \tag{4.53}$$

5. OPTIMIZATION OF WIND TURBINE TOWER

5.1 Genetic Algorithm

Throughout 1960-1970, the psychology and computer science expert, John Holland is the first person who has studied in Genetic Algorithms. Holland who researched about Machine learning, by being inspired from Darwin's theory, he considered to execute the process of life of organism as a model in software area. Instead of improving skill of learning of machine structure, Holland envisaged reproduction, crossover, and mutation of the colony which has formed in such structures successfully and could be created individuals. At the end of his study, He published a book whose name is Genetic Algorithms. In addition, in 1985 David E.Goldberg , civil engineer who is a PhD student of Holland , submitted thesis and then he released his book in 1989 related to Genetic Algorithm.

5.2 Definition of Genetic Algorithms

Genetic Algorithms is one of the methods used in optimization problems. Particularly, it is based on natural selection. Genetic Algorithms is dependent on that the best generation has to live in nature. Although many genetic algorithms have been said with different structures, mostly GA comprises of three basic operations.

Genetic Algorithm uses reproduction, crossover and mutation operators to define fitness and to create new solutions. Reproduction is simply a process to make decision which strings should remain and how many copies of them should be produced in the pool. The decision is made by comparing the fitness of each string. The fitness indicates survival potential and reproduction efficiency of the string in the next generations. For an optimization problem, the fitness function is the objective function of optimization problem .Another specialty of Genetic Algorithm is involved in single-group solution. By means of this skill, in a lot of solutions, the best is selected and the worst can be eliminated. One of the most important discrepancy of Genetic Algorithms from other algorithms is to be able to select. In GAs suitability of fitness results in increasing the probability of selection itself but never guarantees. Selection is also randomized like the creation of initial population. However in this randomized selection, fitness of solution determines the probabilities of its selection.

5.3 Advantages of Genetic Algorithm

- GA can optimize by using continuous and discrete parameters.
- It does not need knowledge of derivative.
- It can search by using a lot of parameters.
- Local minimums can be eliminated easily.
- Not only single-solution but also it can represent the list of optimum parameters.
- Data which is created numerically can be worked by means of experimental data or analytical functions.

5.4 The Principals of GA

The work of GA can be summarized as below:

- Probable solutions coded is formed a solution group,
- Each chromosome is found how much to be suitable,
- In order to create new population, chromosomes are carried out reproduction and crossover operators,
- In order to create new chromosomes, old chromosomes are eliminated,
- The fatnesses of the created chromosomes in new population are recalculated,
- Unless generation time is over, new population is subject to some operations as applied older population,
- The best chromosome is the result up to that time which has been found.

Before the application of GA ,the number of individuals in population has to be determined foremost.

The number of individuals is recommended between 100-300 intervals. Population is created randomly. Afterwards, the fitness function is determined. This function is operated as per chromosomes values. Fitness function comprises of GA engine. In most cases, success of GA is based on whether chromosomes work efficiently or not.

Reproduction of chromosomes is applied in accordance with values of fitness function. So as to select this reproduction roulette wheel or tournament method can be used.

Crossover operation in population leads to diversity. This operation facilitates the meetings of the best chromosomes in the population. In that way, the best generation can be obtained as a result of this operation. Mutation in GA means that a part of chromosome replaces with another part. For double arrays, crossover can be executed randomly by the change of any bit. The lowest probability of crossover in population results in remove of some specialties. This prevents from obtaining the best solutions in optimization problem. There is no strict coefficient for the probability of Crossover and Mutation. It is recommended for Mutation 0.01-0.001 and for Crossover 0.5-1.0 intervals respectively.

After all these operations, by eliminating old chromosomes, it is fulfilled in a constant population value. All of the chromosomes created in new population is re-calculated and obtained its quality. GA is continuously functioned .There occur a lot of populations recalculated. During the process of computing populations, since the best individuals save till that time. It means that the best solution is the result of problem.

We can summarize how GA works in short as shown in the figure. GA initiates by generating a randomized population. Solutions represent chromosomes. Afterwards the fitness function evaluating values of chromosomes are determined. And each chromosome is also found how much it is optimum for the solution. To create a new population, aforementioned chromosomes are subjected to crossover operation. Crossover process enables diversified individuals in the population. Mutation means that any part of chromosome is changed from outside. The old chromosomes are eliminated to make future chromosomes effective in new population. All of the chromosomes located in new generation are recalculated to find new population's efficiency. In this way, GA works in recursion.

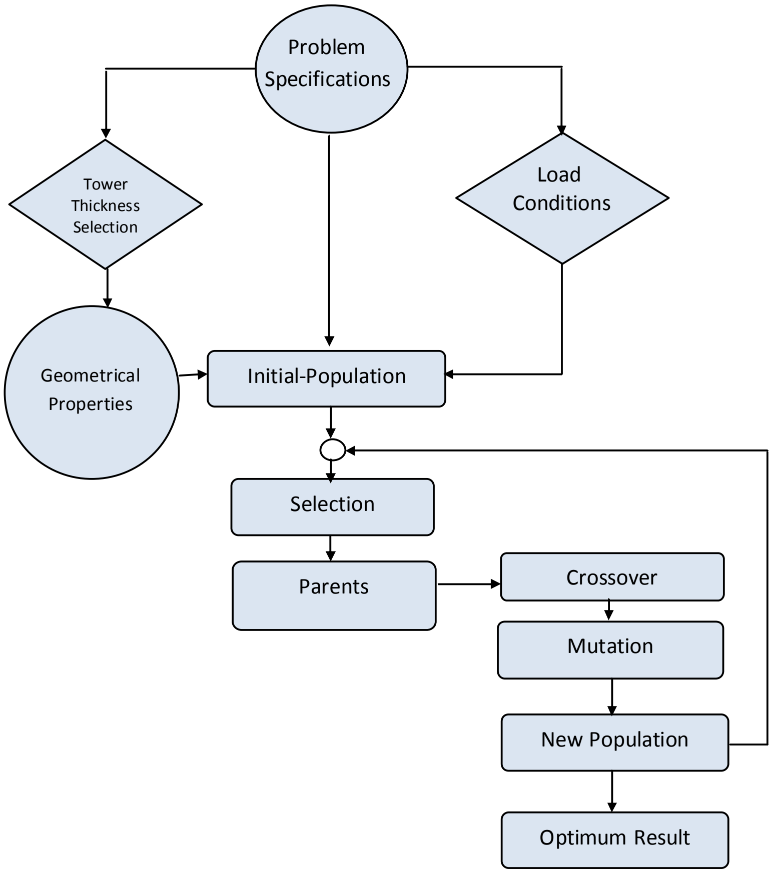

Figure 5.1 GA Operation Flow Chart

5.5 Binary Strings Genetic Algorithm Differences

1. GA is involved in codes of parameters. As long as parameters are coded.

2. GA searches the best solution not only in local area but also in Global area.

3. GA does not cover data what it needs to do but knows how it does. Therefore it is such a blind-search method.

4. GA works as per probability rule. And It cannot be estimated how much good GA program will function and give the best solution.

5.6 Initializing Population

IPOP=(hi - lo) x random{ Nipop x Npar } + lo

hi = up value of parameters

lo = low value of parameters

Nipop x Npar = Randomized Population-Chromozomes Matrix

In the created population, the values of chromosomes vary. Their values are defined depending on evaluation of fitness function. Chromosomes in initialized population are very crucial to define which chromosomes are capable of creating new generation. All genes are ordered according to the fitness values. And the best of Npop is saved for the next generation on the other hand other is removed. This natural selection is repeated at each step of genetic algorithm. It can be reached the best solution at the end of process. At this point, the number of population is Npop. Each chromosome is not capable of being a parent. The Best numbers of Ngood individuals are saved and the remaining Nbad individuals are removed.

5.7 Crossover

For crossover operation, two chromosomes are selected.

$Parent_1=[p_{m1}, p_{m2}, p_{m3}, p_{m4}, p_{m5}, p_{m6}, \ldots, p_{mNpar}]$

$Parent_2=[p_{d1}, p_{d2}, p_{d3}, p_{d4}, p_{d5}, p_{d6}, \ldots, p_{dNpar}]$

In the end of crossover operation, parameters exchange as belows:

$Child_1=[p_{m1}, p_{m2}, p_{d3}, p_{d4}, p_{m5}, p_{m6}, \ldots p_{dNpar}]$

$Child_2=[p_{d1}, p_{d2}, p_{m3}, p_{m4}, p_{d5}, p_{d6}, \ldots p_{mNpar}]$

Up to the this point , since the applied strategy above is more suitable to be coded in accordance with binary system, above method cannot result in good solutions with finite parameters. Afterwards, the parameter value of individual to be created can be computed by:

$P_{new}=\beta p_{mn}+ (1-\beta)p_{dn}$

Where:

β : [0, 1] random number between 0 and 1

p_{mn}: mother chromosome nth parameter ,

p_{dn} : father chromosome nth parameter , In case β is equal to 1, the contribution of father chromosome is zero to new chromosome. In case β is equal to 0, the contribution of mother chromosome is zero to new chromosome. In case β is equal to 0.5, the contribution of parents is equal.

In order to produce all parameters pertaining of linear combinations of parents, at most the equal number of Npar can be mixed. Mixing ratio for each parameter is selected similar and different as well. With such these applied methods, created parameters do not exceed the pre-defined boundaries in general. For instance simply:

$p_{new1} = 0.5\ p_{mn} + 0.5\ p_{dn}$

$p_{new1} = 1.5\ p_{mn} - 0.5\ p_{dn}$

$p_{new1} = -0.5\ p_{mn} + 1.5\ p_{dn}$

As shown examples above are linear crossover methods. As a result, new individual can be written with heuristic approach in the selection of β between 0 and 1 values .

$p_{new} = \beta\ (p_{mn} - p_{dn}) + p_{dn}$

In Heuristic approach method, some values can exceed the boundary condition sometimes. In this case new individual is eliminated and algorithm is carried on using with a new β value.

In accordance with values of new parent, α coefficient to define new interval based on mixed crossover (BLX-α) was firstly claimed by Eshelman and Shaffer in 1993.

5.8 Mutation

Change of parameters at slight rates located in new generated chromosomes is known as mutation. If probability is selected at high value, searching algorithm diverts to random operation. Mutation operation impede algorithm to obstruction of local minimums.

If we consider a binary string with a length of six is used to code the real variable and the population size is set to be four. Using a random process, four starting points 011111, 111000, 001000 and 100001 are selected. The four strings represent the real values 31.0, 56.0 8.0 and 33.0 respectively and the corresponding numbers of copies these strings receive are theoretically, 0.74, 2.36, 0.06 and 0.84. There will be one copy of 011111, two copies of 111000, one copy of 100001 and no copy of 001000 in the mating pool. Practically reproduction is done at random. A range is created according to the fitness of each individual. Thus, a better string will occupy a bigger portion in the range and consequently has more probability to be selected into the mating pool. To perform one and two point crossovers one and two crossing sites along the string are chosen at random in the following [7]:

For one-point crossover:

011 | 111

111 | 000

For two-point crossover:

0 | 111 | 11

1 | 110 | 00

5.9 Optimization Problem

Optimization problems are generally expressed as given in the following:

Minimize $f(X)$
Constraints

$$gk(X) \leq 0$$

$$x_j^1 \leq 0 \leq x_j^u \qquad j = 1, \ldots \ldots \ldots n$$

f(x) is the objection function , $gk(x)$ is constraints set and $X = \{x_1, x_2, x_3 \ldots \ldots x_n\}$ is real variables set.

The objective function is determined for the steel tubular tower in the followings. Because of the fact that objective function is related with the mass of tower, it is directly involved in cost. Design constraints are calculated by penalty functions as below. Each penalty function is zero as long as values are inside allowable ranges.

p1(x) : Margins of safety combined stress (bending stress and shear stress for torsion) because of the wind load effect.

p2(x) : Combined stress (bending stress and shear stress) because of the earthquake load effect.

p3(x) : Natural frequency for the 1st mode bending.

p4(x) : Fatigue stress

Mtower : Mass of turbine tower

Minimize f(x) = Mtower(x) . (1 + p1(x) +p2(x) + p3(x) + p4(x))

Constraints $12 \leq t_x^i \leq t_x^{i+1} \ldots.. \leq t_x^{i+51} \leq 26$ the thicknesses of sections

i=1 ,2 ,........................,51

$\text{U\_C\_WL(x)} \leq \text{DCRsw(x)}$

Unity check of critical combined buckling stress ratio due to wind effect load against combined stress ratio:

$\text{U\_C\_EQ(x)} \leq \text{DCRsq(x)}$

Unity check of critical combined buckling stress ratio U_C_EQ(x) due to earthquake effect load against combined stress ratio DCRsq(x) :

$$w_t < w_r$$

$w_t = \text{Natural frequency of tower 1st mode}$

$w_r = \text{Operation frequency } of\ turbine$

Design flow is shown as follows. This GA structure minimizes tower mass subject to general dimensions, design loads and some design restrictions. Load calculation depends on wind turbine design requirements of the standard IEC61400-1 and ASCE 7-98. All extreme loads of tower sections are calculated by the load combination. Fatigue loads also are calculated by DEL method.

Input Data

1) General Specifications: Tower height, diameters of tower base and top , turbine mass.
2) Material Characteristics: Mass density, SN curve allowable and yield stresses, Young's modulus.
3) Structural Parameters: Height of segments, Thickness range.
4) Load Conditions: Basic wind speed, direct wind pressure and force on tower, Aerodynamic loads, Seismic loads.
5) GA Parameters: Initial population, crossover, mutation operators.
6) Design Loads: ASCE 7-98, IEC, Eurocode load calculations.
7) Safety Factors: Dead load, Wind load and Partial safety factors are used for load combination.

Optimization Program

1) Natural Frequency: 1st mode of natural frequency is found as per equation and is avoided to be subject to resonance of tower.
2) Extreme Loads: Extreme wind loads are evaluated for each section.

3) Fatigue Damage: It is designed to DEL (Damage Equivalent Load).
4) Fitness Function: Each gene has information of wall thickness of each 1 m long tower segment. Main target is to find minimum values of wall thicknesses for fitness function given above.

Output Parameters

Best generation is found for the thickness of sections of towers along the height of tower.

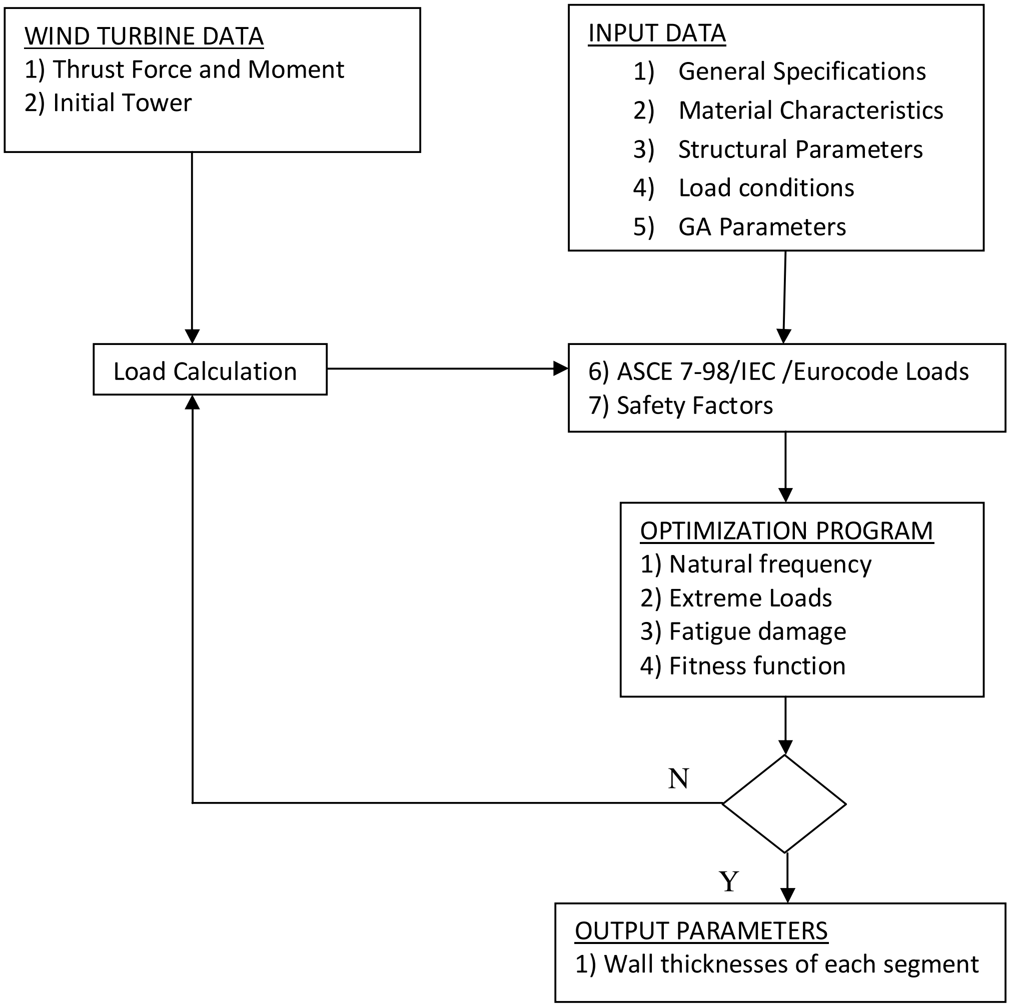

Figure 5.2 Design Flow [13]

6. CONCLUSION

Each 1m section along tower represented a chromosome in GA. Each section was evaluated step by step in terms of buckling strength in GA. An objective function was flourished by using a genetic algorithm. It optimizes the thickness of steel tower ranging from top 12 mm to base 26 mm in the distribution of pattern. These structures are regarded as a tapered tower. As a result, the thicknesses of tower were evaluated separately in each 1 m section along the height of the tower. For the best solution, the weight of tower was obtained 63000 kg with a type of S355J0 material quality. And it gives results for the best solution as indicated above in Figure 12. Furthermore, the upcoming studies can be developed more as long as stiffness is obtained.

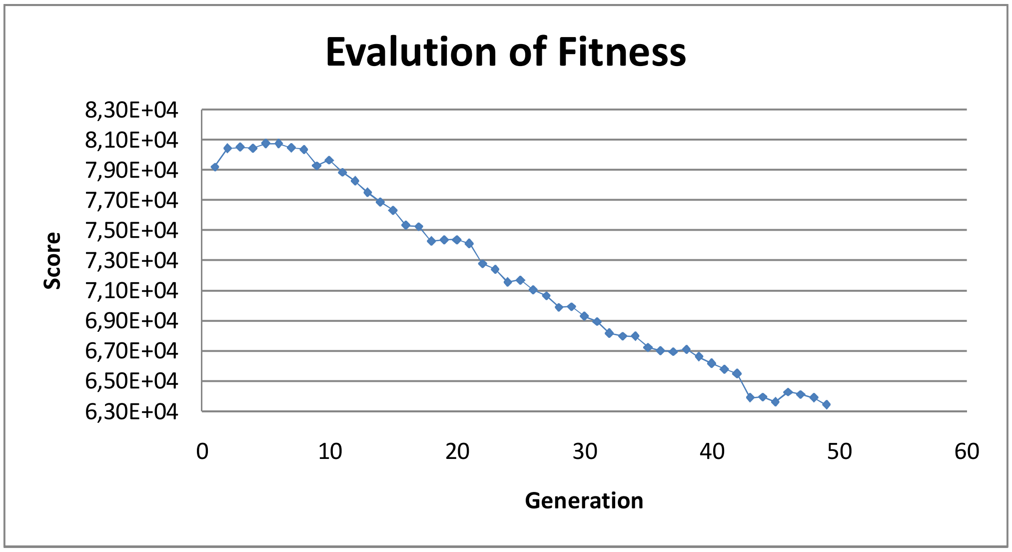

Figure 6.1 Evaluation of Fitness Function

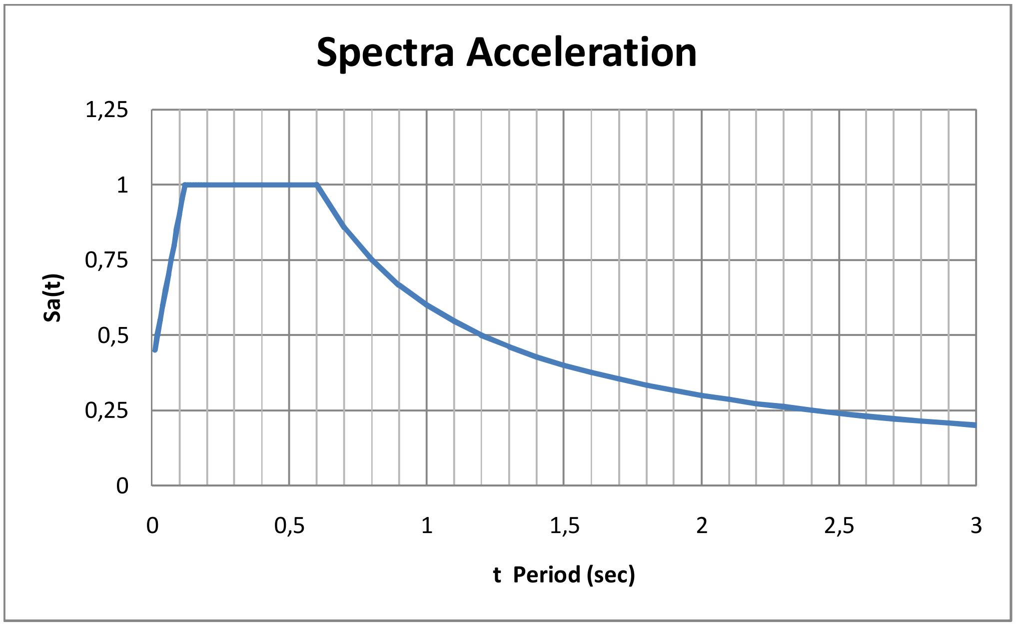

Figure 6.2 Spectra Acceleration

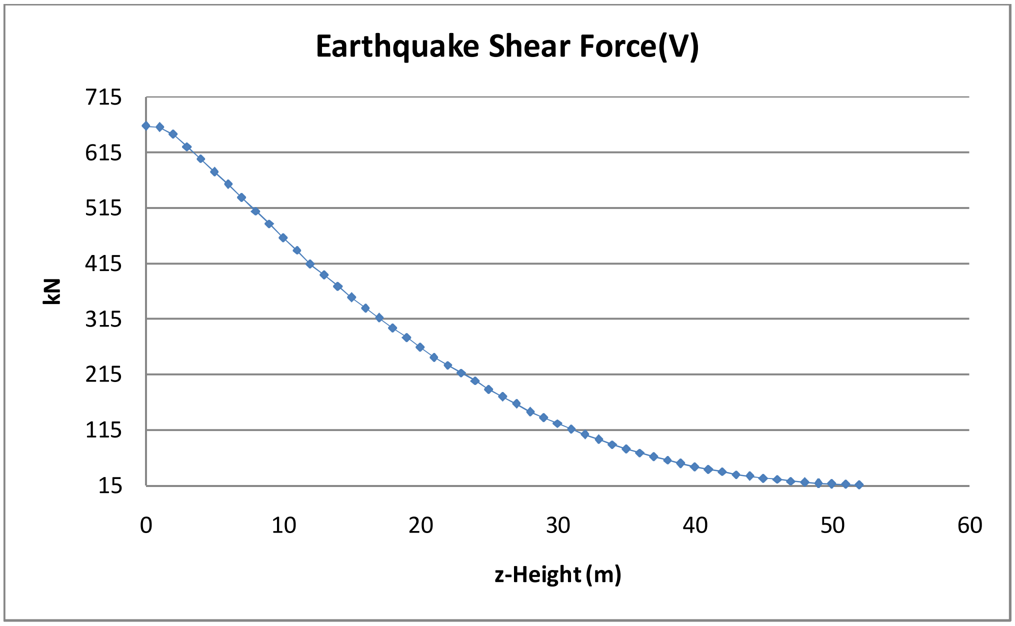

Figure 6.3 Earthquake Shear Force

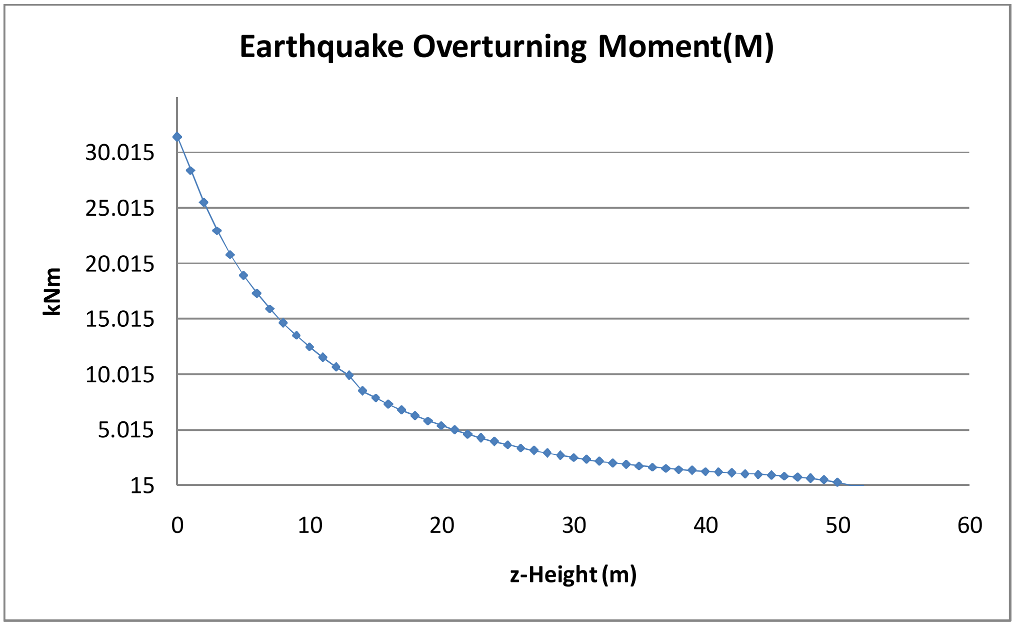

Figure 6.4 Earthquake Overturning Moment

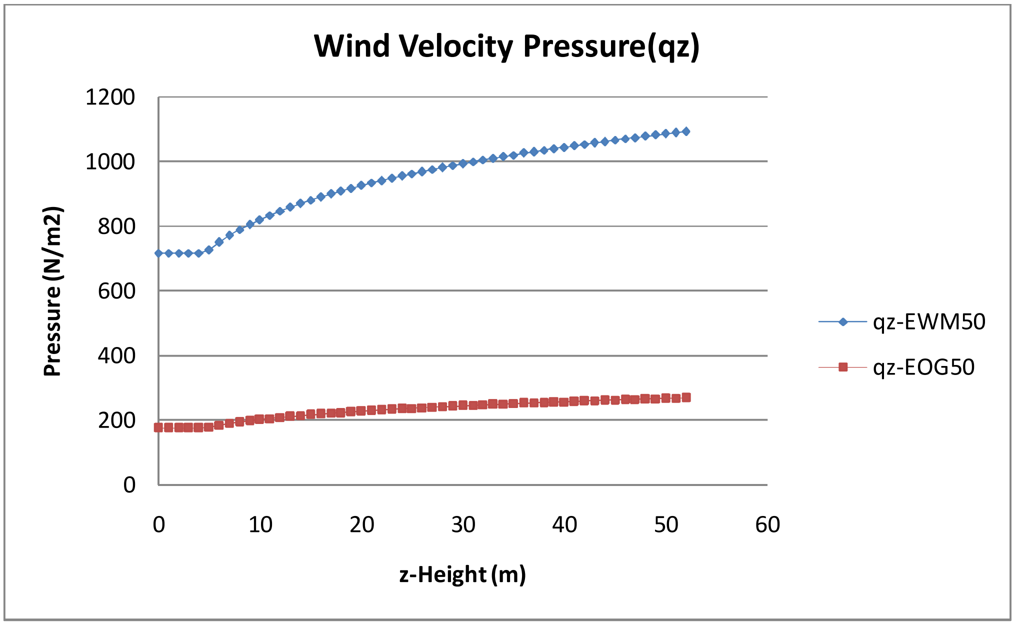

Figure 6.5 Wind Velocity Pressure on Tower

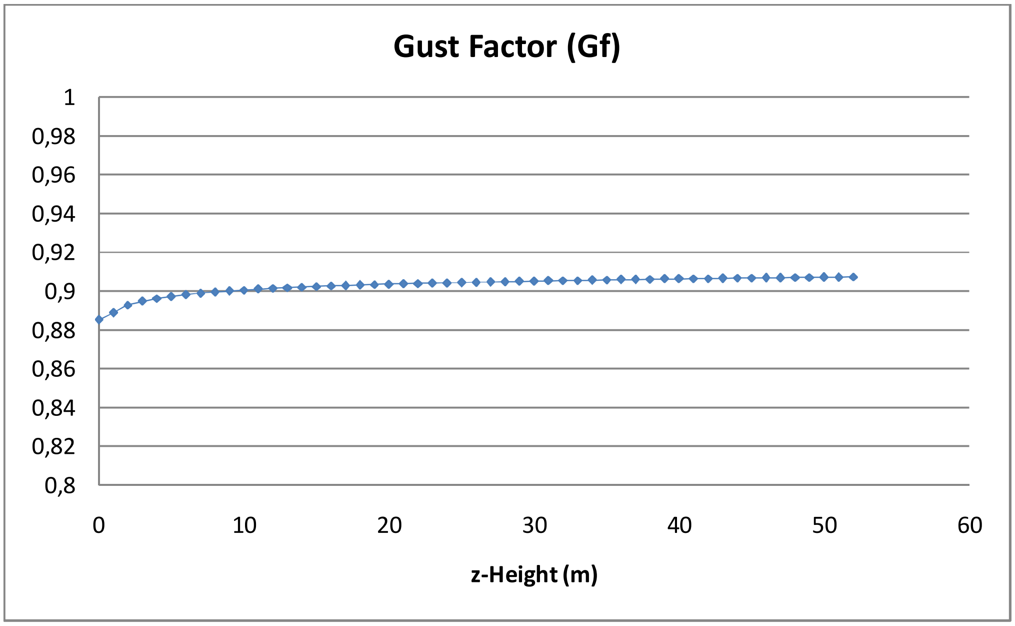

Figure 6.6 Gust Factor

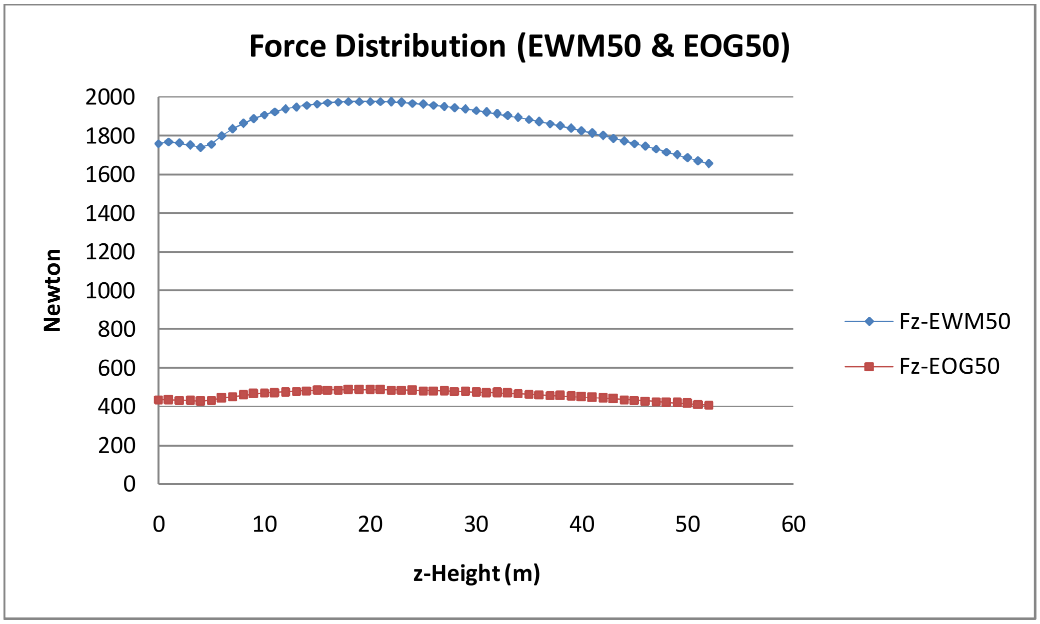

Figure 6.7 Force Distribution Because of Direct Wind Effect on Tower

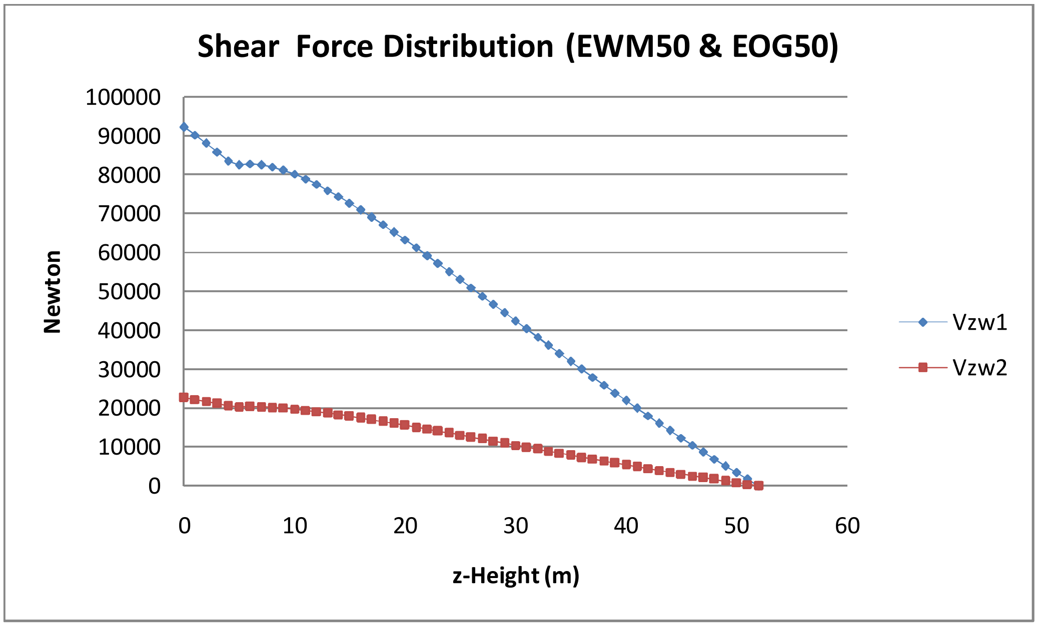

Figure 6.8 Wind Shear Force Along Tower

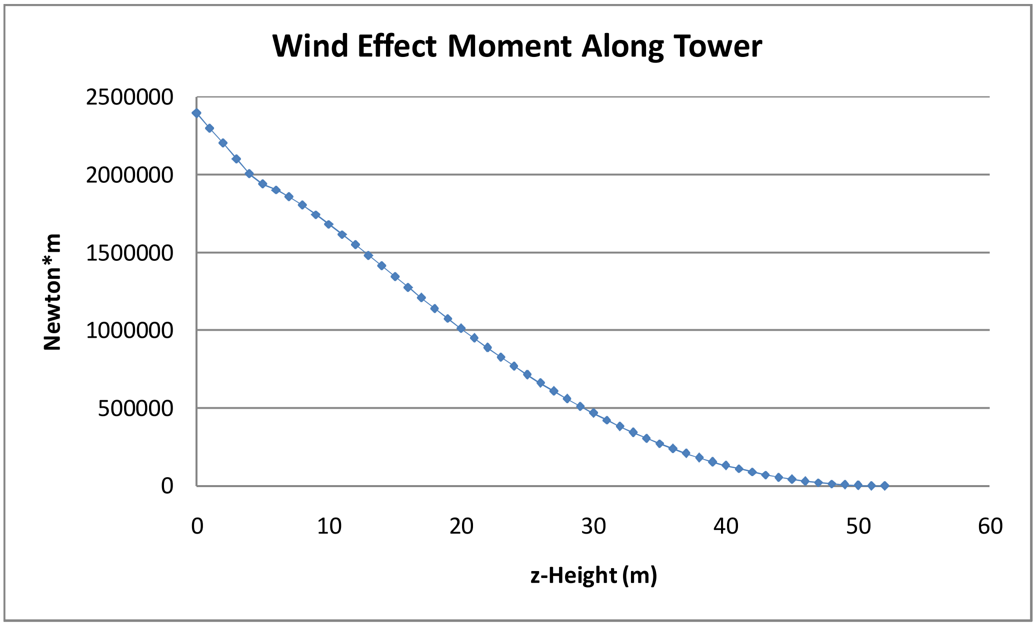

Figure 6.9 Wind Effect Moment Along Tower

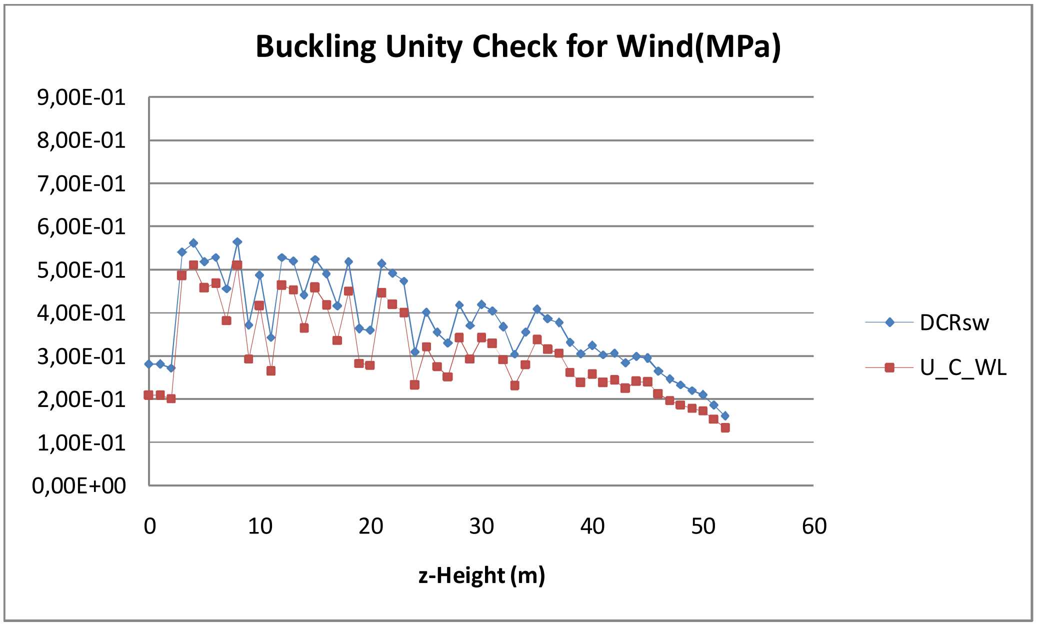

Figure 6.10 Buckling Unity Check for Wind Turbine Effect and Direct Wind Load

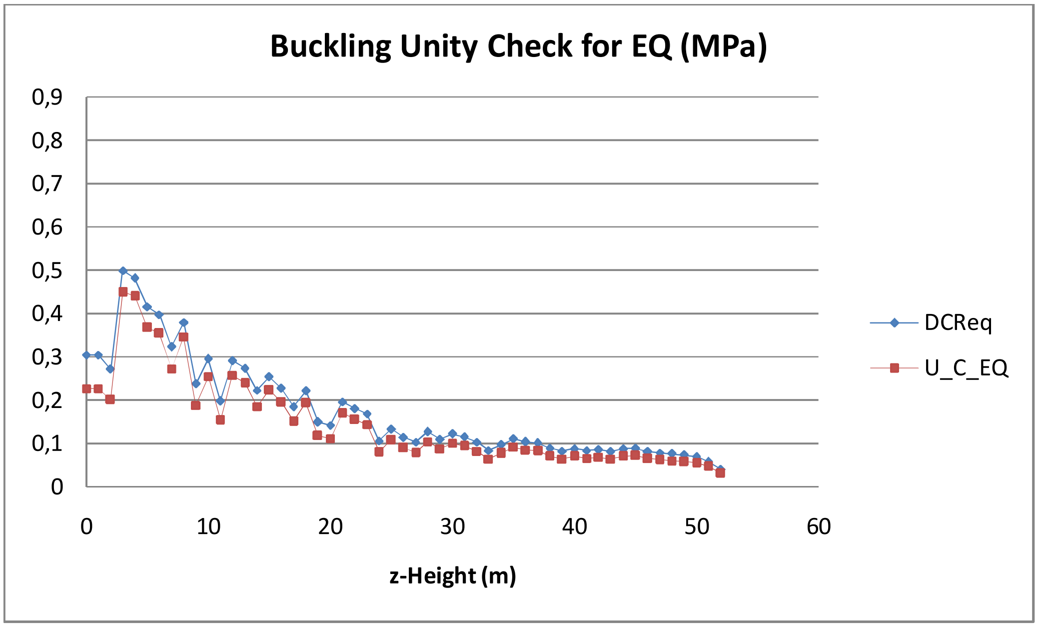

Figure 6.11 Buckling Unity Check for Earthquake Load

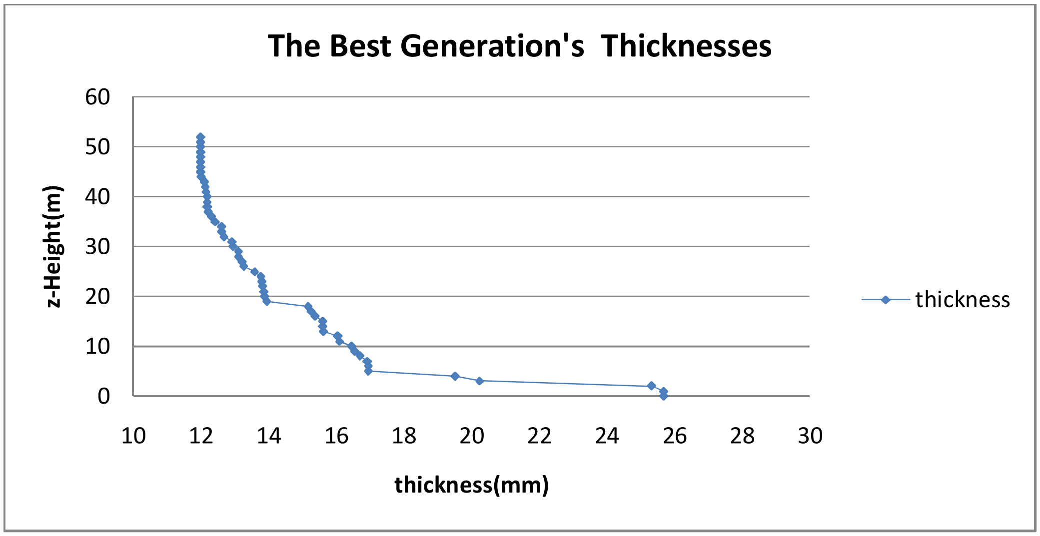

Figure 6.12 Thickness Distribution

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APPENDIX

MATLAB 7.0 Source Codes

```
low = [12] ;          % Thickness constraints
up  = [26];
nvar = 52;            % Number of variables
genNr = 5 ;           % Number of generations
global minval
% EARTHQUAKE LOAD
clc;
Db=4.3;                                  % m  diameter base
Dt=2.564 ;                               % m  diameter top
Density=7850;                            % kg/m3
E=196501*10^6 ;                          % Pa- Young modulus
h=52;                                    % m  Tower height
hhub=54.7 ;                              % m  hub height
headmass=84800;                          % kg  turbine and rotor height
Ss=1.5 ;                       % the mapped MCE spectral response at short periods
S1=0.6;                     % the mapped MCE spectral response at 1 second period
Fa=1.0;    % the site coefficient as a function of site class and short time periods
Fv=1.5;   % the site coefficient as a function of site class and a 1 second period MCE
Sds=(2/3)*Ss*Fa;  % The design earthquake spectral acceleration at short period
Sd1=(2/3)*S1*Fv; % The design earthquake spectral acceleration at 1 second  period
Ts=Sd1/Sds;
T0=0.2*Ts;
```

```
R=1; %Reduction factor
I=1 ; % Importance factor
Ct=0.02; % coefficient for steel as per ASCE
Beta=0.75;
Ta=Ct*(3.28*h)^Beta ;
T=1.4*Ta;  % Structural period
Cs=(Sd1*I)/(R*T);  % Seismic response
k0=0.5*T+0.75 ;  %mode shape factor
Ssqb=1.035 ;%moment magnification factor
for k=1:nvar+1;
   diameter(k)=(Dt-Db)*k/h+Db; %diameter distribution along the tower
end
% Parameters for genetic algorithm:
npop = 100;                    % Size of the population
crossProb = 0.7;               % Probability of crossover
mutProb = 0.1;                 % Probability of mutation
p_tour = 0.7;                  % Tournament probability
mut_scale = 0.01;              % Scale for mutations
n = genNr;                     % Number of runs
mytime = cputime;
ga_ok = 0;
initpop = zeros(npop,nvar);
% Initialize the population
for i = 1:npop ,
   x = (low + (up-low).*rand(1,nvar));
   %   display(i);
   %   display(x);
```

```
%     descending_x=sortrows(sort(x,2))
   initpop(i,:) =sort(x,2,'descend');
end
katsayi = 0;
pop = initpop ;                  % Initial population
for i = 1:n,
   [npop nvar] = size(pop);   % Number of individuals and variables
    for opr=1:npop
      totalwtower = 0;
      for k=1:nvar;
         katsayi = pop(opr, k);
  katsayi = katsayi * (0.001 * pi * (diameter(k) + diameter(k + 1)) * 0.5 * 1 * 7850);
         totalwtower = totalwtower + katsayi;  % kg Tower mass
      end
      for j=1:nvar;
         % m = 0;
                wtower = 0;
        for k=j:nvar;
            katsayi = pop(opr, k);
   katsayi = katsayi * (0.001 * pi * (diameter(k) + diameter(k + 1)) * 0.5 * 1 * 7850);
            wtower = wtower + katsayi
         end
  V=Cs*9.81*(headmass+totalwtower)*R*0.001;% kN  Base Shear
Vzq(j)=V*(((h-j)*(wtower*j^k0)/(h*totalwtower*j^k0+headmass*h^k0))+(headmass*h^k0/(headmass*h^k0+h*totalwtower*j^k0)));       %  kN shear force along the tower
to=0.9;
```

```
Mzq(j)=to*V*( ((wtower*j^k0)/(h*totalwtower*j^k0+headmass*h^k0))*(0.5*h^2-
j*h-(0.5*j^2-j*j)) + (h-j)*(headmass*h^k0)/(headmass*h^k0+h*wtower*j^k0) ) ;
%kNm over turning moment along the tower

% Direct Wind Load on Tower

        V_ref=29.27 ;    %m/s

V_EWM50=1.4*V_ref*(h/hhub)^0.11; % m/s wind speed at hub

alfa1=0.18*(15+2*V_ref)/(2+1); %standard deviation value as per IEC-61400-1

        delta1=21 ; % turbulance scale parameter as per IEC-61400-1


% m/s wind speed at hub %Hub height  gust magnitude

V_gust=6.4*(alfa1/(1+0.1*(Db/delta1)));


%The 50 year extreme wind speed

        V_EOG50=V_ref-0.37*V_gust*sin(3*pi*10/14)*(1-cos(2*pi*10/14)); %m/s


        V1=V_EWM50*(33/(h*3.28))^0.1 ;% Design wind speed m/s IEC-61400-1

        V2=V_EOG50*(33/(h*3.28))^0.2 ;% Design wind speed m/s IEC-61400-1

        Im=1 ;  % importance factor

        Kzt=1 ; % coefficient for flat area from ASCE 7-98

        u=2.5 ; % coefficient for flat area from ASCE 7-98

        v=1.5 ; % coefficient for flat area from ASCE 7-98

a1=11.5;zg=700;c=0.15;b=0.8;a=0.111;e=0.125;l=650 ; %Terrain Exposure
Constants for D Exposure (Table 6-4 of ASCE 7-98)

Kd=0.95 ; % wind direction factor

gq=3.6 ; gv=3.6 ; %gr=(2*log10(3600*n1))^0.5 + 0.577/((2*log10(3600*n1))^0.5)

if j<5

Kz(j)=2.01*(15/zg)^(2/a1) ; % terrain exposure coefficient

        else
```

```
        Kz(j)=2.01*(3.28*j/zg)^(2/a1) ;

      end

  qzV1(j)=0.613*Kz(j)*Kzt*Kd*V1^2 ; %wind pressure on tower as per V1

  qzV2(j)=0.613*Kz(j)*Kzt*Kd*V2^2 ;%wind pressure on tower as per V2

      Iz(j)=0.15*(33/(3.28*j))^(1/6) ; %intensity of turbulance

      Lz(j)=l*(3.28*j/33)^e ; %Integral length scale

naturalfrequency=(1/(2*pi))*1.75*((1.96501*10^11*pi*(1/64)*((0.5*(Db+Dt))^4-
((0.5*(Db+Dt))-19*0.001*2)^4))/(hhub^3*(headmass+totalwtower*0.25)))^0.5 ; %
Hz

      n1=naturalfrequency ;

       Vz(j)=(1/9)*(j/33)^(1/11.5)*2.236*V1*(88/60) ;

      N1(j)=n1.*(Lz(j)./Vz(j)) ; %ASCE 7-98

      Rn(j)=7.47*N1(j)/((1+10.3*N1(j))^(5/3)) ; %ASCE 7-98

Rh(j)=(1/(4.6*n1*52/Vz(j))) - (1/(2*(4.6*n1*52/Vz(j)))^2) * (1-exp(-
2*(4.6*n1*52/Vz(j)))) ;

      Rb(j)=Rh(j) ; %ASCE 7-98

      RL(j)=(1/(15.4*n1*0.5*(Db+Dt)/Vz(j)))-
(1/(2*(15.4*n1*0.5*(Db+Dt)/Vz(j))^2))*(1-exp((-2*(4.6*n1*52/Vz(j))))) ;
%ASCE 7-98

%ASCE 7-98 resonant response factor

 Rtotal(j)=((1/0.02)*Rn(j)*Rh(j)*Rb(j)*(0.53+0.47*RL(j)))^0.5 ;

Q(j)=(1/(1+0.63*(((Db+Dt)*0.5+hhub)/Lz(j))^0.63))^0.5 ; % the background
response

Gf(j)=0.925*((1+(1.7*Iz(j)*(3.4^2*Q(j)^2+5.49^2*Rtotal(j)^2)^0.5))/(1+1.7*3.4*Iz(
j)));  %gust factor

      Cf=0.65; % roundness factor according to (Table 6-10 ASCE 7-98)


Fz1(j)=qzV1(j)*Gf(j)*Cf*diameter(j);% Force distribution along the tower

Fz2(j)=qzV2(j)*Gf(j)*Cf*diameter(j);% Force distribution along the tower
```

```
Vzw1(j)=Fz1(j)*h-Fz1(j)*j; % kN Shear force for wind effect
Vzw2(j)=Fz2(j)*h-Fz2(j)*j ;% kN Shear force for wind effect

Mzw1(j)=Fz1(j)*(0.5*h^2-j*h+0.5*j^2);% kNmMoment along the tower
Mzw2(j)=Fz2(j)*(0.5*h^2-j*h+0.5*j^2);% kNm Moment along the tower
% TURBINE WIND LOAD ACCORDING TO THE WINDPACT DESIGN
% EMW50                               EOG50
%THRUST AT YAW BEARING
Fxt1=90                              Fxt2=173  ;    %kN
Fyt1=146;                            Fyt2=17    ;    %kN
Fzt1=395;                            Fzt2=360   ;    %kN
Mxt1=1613;                           Mxt2=384  ;   %kNm
Myt1=813;                            Myt2=504   ;  %kNm
Mzt1=480;                            Mzt2=100   ;  %kNm
%THRUST AT BASE
% EMW50                                 &EOG50
Mxb1=17335;                           Mxb2=2344 ;   %kNm
Myb1=5570;                             Myb2=1716 ;   %kNm
Mzb1=437;                             Mzb2=128;     %kNm
%Fatigue Thrust Range
Fxt=57; %kN
%Fatigue Applied Moment  Range
Mxt=121;    %kNm
Myt=554;    %kNm
Mzt=551;    %kNm
Mxb=2100;     %kNm
Myb=4000;    %kNm
```

```
        %*********Fatigue Linear Interpolation
        Mxft(j)=((Mxt-Mxb)*j/h)+Mxb; %kNm
        Myft(j)=((Myt-Myb)*j/h)+Myb; %kNm
        Mwf(j)=(Mxft(j)^2+Myft(j)^2)^0.5; %kNm
         %Moment Linear Interpolation
        %EMW50                                   EOG50
Mxwt1(j)=((Mxt1-Mxb1)*j/h)+Mxb1 ;      Mxwt2(j)=((Mxt2-Mxb2)*j/h)+Mxb2 ;
%kNm
Mywt1(j)=((Myt1-Myb1)*j/h)+Myb1;       Mywt2(j)=((Myt2-Myb2)*j/h)+Myb2;
%kNm
Mwt1(j)=(Mxwt1(j)^2+Mywt1(j)^2)^0.5 ; Mwt2(j)=(Mxwt2(j)^2+Mywt2(j)^2)^0.5 ;
%kNm
 Mzwt1(j)=((Mzt1-Mzb1)*j/h)+Mzb1 ;      Mzwt2(j)=((Mzt2-Mzb2)*j/h)+Mzb2 ;
%kNm
        Mxwt(j)=((Mxt-Mxb)*j/h)+Mxb ;  %kNm
        Mywt(j)=((Myt-Myb)*j/h)+Myb ;  %kNm
 %Moment (Thrust from Wind Turbine)
        %EMW50                            %EOG50
        MxwtF1(j)=Mxt1+Fyt1*(h-j);                MxwtF2(j)=Myt2+Fyt2*(h-j);
%kNm
        MywtF1(j)=Myt1+Fxt1*(h-j);                MywtF2(j)=Myt2+Fxt2*(h-j);
%kNm
MwtF1(j)=((MxwtF1(j))^2 + (MywtF1(j))^2)^0.5 ; MwtF2(j)=((MxwtF2(j))^2 +
                                                MywtF2(j))^2)^0.5;  %kNm
 MxwtF(j)=Mxt;                           MywtF(j)=Myt+Fxt*(h-j);     %kNm

 MwtF(j)=MywtF(j);  %kNm
        % ULTIMATE LOAD (ASCE 7-98 Load Combination)
        %load factor for ultimate load
```

```
          yDL=1.2 ;   %dead load factor

          yWL=1.6;    %wind load factor

          ySF=1.35;   % partial safety factor

          yf=1;       %Fatigue safety factor


          %Factored EWM50

PuT=(Fzt1-headmass*9.81*0.001)*ySF+yDL*headmass*9.81*0.001  ;

% kN   load applied at the tower top in the z direction(along vertical axis of the tower)

PuD(j)=yDL*wtower*9.81*0.001;

% kN   tower dead load not including the turbine mass


VuT=ySF*(Fxt1^2+Fyt1^2)^0.5;

% kN   load applied at the tower top in the horizontal x (downwind) direction

VuD(j)=yWL*Vzw1(j)*0.001;

% kN   tower base shear resulting from the effect of direct wind on the tower

 MuTx(j)=ySF*Mxwt1(j) ;% kNm  moment applied at the tower top

MuTy(j)=ySF*Mywt1(j)   ;% kNm  moment applied at the tower top

MuT(j)=(MuTx(j)^2+MuTy(j)^2)^0.5 ;% kNm  moment applied at the tower top

MuD(j)=yWL*Mzw1(j)*0.001 ;

% kNm  tower base moment resulting from the effects of direct wind on the tower

Pu(j)=PuT+PuD(j);

% kN   total load applied at the tower base including tower dead load amd turbine head mass

Vu(j)=((Fxt1*ySF+VuD(j))^2+(Fyt1*ySF)^2)^0.5 ;

% kN   total tower base shear including direct wind and turbine load effects

          Mu(j)=((MuTy(j)+MuD(j))^2+MuTx(j)^2)^0.5 ;

%SERVICE LOAD (Unfactored Wind Load)
```

```
        %UnFactored EWM50
        PsT(j)=Fzt1;
        PsD(j)=wtower*9.81*0.001;
        VsT=(Fxt1^2+Fyt1^2)^0.5;
        VsD(j)=Vzw1(j)*0.001;
 MsTx(j)=Mxwt1(j);
 MsTy(j)=Mywt1(j);
MsT(j)=(MsTx(j)^2+MsTy(j)^2)^0.5;
MsD(j)=Mzw1(j)*0.001;
Ps(j)=PsT(j)+PsD(j);
Vs(j)=((Fxt1+VsD(j))^2+Fyt1^2)^0.5;
Ms(j)=((MsTy(j)+MsD(j))^2+MsTx(j)^2)^0.5;
%Operation Load(Unfactored Wind Load)
%UnFactored EWM50
PoT=Fzt2 ;
PoD(j)=wtower*9.81*0.001;
VoT=(Fxt2^2+Fyt2^2)^0.5;
VoD(j)=Vzw2(j)*0.001;
MoTx(j)=Mxwt2(j);
MoTy(j)=Mywt2(j);
MoT(j)=(MoTx(j)^2+MoTy(j)^2)^0.5;
MoD(j)=Mzw2(j);
Po(j)=PoT+PoD(j);
Vo(j)=((Fxt2+VoD(j))^2+Fyt2^2)^0.5;
Mo(j)=((MoTy(j)+MoD(j))^2+MoTx(j)^2)^0.5;
%Fatigue combine Load
Pf(j)=9.81*0.001*(wtower+headmass);
```

```
Vf(j)=Fxt;

t(j)=pop(opr,j);

A(j)=pi*0.25*(diameter(j)^2-(diameter(j)-t(j)*0.001*2)^2);

I(j)=pi*(1/64)*(diameter(j)^4-(diameter(j)-t(j)*0.001*2)^4);

S(j)=2*I(j)/diameter(j);

r(j)=(I(j)/A(j))^0.5;

%Applied stress for Wind

fa(j)=0.001*Ps(j)/A(j); %mpa axial stress

fb(j)=0.001*Ms(j)/S(j) ;%mpa bending stress

fv(j)=0.001*Vs(j)/(pi*diameter(j)*0.5*0.001*t(j));%mpa shear stress

fvt(j)=0.001*Mzwt1(j)*diameter(j)/(4*I(j));%mpa shear stress for torsion

fab(j)=fa(j)+fb(j)       ;   %mpa max normal stress

fvvt(j)=fv(j)+fvt(j)   ;   %mpa max shear stress

aw(j)=((fa(j)+fb(j))^2 + 3*(fv(j)+fvt(j))^2)^0.5 ;% mpa Combined stress

%APPLIED STRESS FOR EARTHQUAKE

faq(j)=(10^-6)*(9.81*wtower+9.81*headmass)/A(j);  %mpa axial stress

fbq(j)=(10^-3)*(0.7*Mzq(j)/S(j)) ;           %mpa bending stress

fvq(j)=(10^-3)*(0.7*Vzq(j)/(pi*diameter(j)*0.5*0.001*t(j))); %mpa shear stress

faqbq(j)=faq(j)+fbq(j);                      %max normal stress Mpa

aq(j)=((faq(j)+fbq(j))^2+3*(fvq(j))^2)^0.5;  % combined stress    Mpa

Fy=345; %mpa

K=2;

Cc=(2*pi^2*E*(10^-6)/Fy)^0.5;

Fb(j)=0.6*Fy; %allowable bending stress Mpa

Fv=0.4*Fy; %allowable shear stress Mpa

 Fa(j)=((1-(K*1/r(j))^2/(2*Cc^2))*Fy)/(5/3+ (3/8)*(K*1/r(j))/(Cc)-(1/8)*(K*1/r(j))^3/(Cc^3)); % mpa Allowable compression stress
```

```
%Unity Check for Wind Load
X(j)=fa(j)/Fa(j);
Y(j)=(fv(j)+fvt(j))/Fv;
Z(j)=fb(j)/Fb(j) ;
J(j)=(fa(j)/Fa(j))+(fb(j)/Fb(j)) ;
DCRsw(j)=fa(j)/Fa(j) + 1.042*fb(j)/Fb(j) ;% CONSTRAINT
DCRsq(j)=faq(j)/Fa(j) + 1.042*fbq(j)/Fb(j); % CONSTRAINT
DCRfat(j)=0.78;
t(j)=pop(opr,j);
LL(j)=212;

%Elastic tube buckling stress
acr(j)=0.605*(10^-6)*E*0.001*t(j)/(0.5*diameter(j));
 if  0.5*diameter(j)/((0.001)*t(j))< LL(j)
 ao(j)=0.83/(1+0.01*0.5*diameter(j)/(0.001*t(j)))^0.5;
 %reduction coefficient for axial load
        else
 ao(j)=0.70/(0.1+0.01*0.5*diameter(j)/(0.001*t(j)))^0.5 ;
%reduction coefficient for axial load
        end
 aB(j)=0.1887+0.8113* ao(j) ;
 %reduction coefficient for bending moment
        Fy(j)=345; %mpa
 if (aB(j)*acr(j))>(Fy(j)/2)
       au(j)=Fy(j)*(1-0.4123*(Fy(j)/(aB(j)*acr(j)))^0.6);%combined buckling stress
        else
          au(j)=0.175*aB(j).*acr(j); %combined buckling stress
```

```
        end
        %UNITY CHECK FOR WIND AND EQ LOAD
 U_C_WL(j)=aw(j)/ au(j);   %combined buckling stress ratio %
 U_C_EQ(j)=aq(j)/au(j);     %combined buckling stress ratio %
% *************** UNITY CHECK *****************************
%*************FATIGUE UNITY CHECK***********************
        N1(j)=5.29*10^8; % Design Cycle
        yss=1.265;
        Delta_y=301; % Mpa
        N0=10000 ;  % Initial Cycle without Degradation
        m=4 ; % slope rate
        Mfs(j)=Delta_y*S(j)*10^6*10^-3; %Yielding Moment
        Mss(j)=Mfs(j)*N0^(1/m);%Extrapolated Yielding Moment
        Mf(j)=5500; %kNm
        N(j)=(Mwf(j).*yss/Mss(j)).^(-4);%Number of cycle at applied moment
        Fatigue_rate(j)=N1(j)/N(j);
        DCRfat(j)=0.78
        operation_frequency=0.342 ; % Hz
        % OBJECTIVE FUNCTION
       wind(j)=0;
         if DCRsw(j) <= U_C_WL(j)
           wind(j) =U_C_WL(j)  * 0.01;
        end
        earthquake(j)=0;
        if DCRsq(j) <= U_C_EQ(j)
           earthquake(j) = U_C_EQ(j) * 0.01 ;
        end
```

```
            frequency(j)=0;
            if naturalfrequency<=operation_frequency;
                frequency(j) = naturalfrequency * 10 ;
            end
            fatigue(j)=0;
            if DCRfat(j)<=Fatigue_rate(j)
                fatigue(j)= Fatigue_rate(j)*0.01;
            end
   GlobalFitness = totalwtower*( 1 + wind(j)+earthquake(j)+frequency(j)+fatigue(j));
  end
  values(opr) = GlobalFitness;
 end
end
minval = min(values)
bestgen = pop(find(values == minval),:)
for ik = 1:genNr,
stats(ik,:) = [min(values) mean(values) std(values)];
end
oldpop = pop;
oldvalues = values;
d = up - low;
% Recombine individuals
for j = 1:npop,
 % Select two parents
for k = 1:2,
 k;  % Pick individuals for the tournament
 idx = []; idx = 1 + floor(rand(2,1)*npop);
```

```
 tour_val = oldvalues(idx)
 % Select a parent
 if (rand < p_tour)
            cc=rand;
            tmp = find(tour_val == min(tour_val));
        else
            ccx=rand;
            tmp = find(tour_val == min(tour_val));
        end
        pnt(k) = idx(tmp(1));
    end
    parent1 = oldpop(pnt(1),:) ;
    parent2 = oldpop(pnt(2),:) ;
    child = parent1;
    mask = rand(1,nvar) <= crossProb;
    mm=rand;
 idx = []; idx = find(mask);
    if (length(idx) > 0)
        length(idx);
        coeff = 2*rand(size(idx))-0.5;
        child(idx) = (1-coeff).*parent1(idx) + coeff.*parent2(idx);
    end
end
% Mutations
for ii = 1:nvar,
    if (rand <= mutProb)
        d(ii) = up - low;
```

```
        chil=child(ii);

        child(ii) = child(ii) + mut_scale*d(ii).*randn;

%           d(ii)=d(ii);

        mut_scale=mut_scale;

        xxrandn=mut_scale*d(ii)*randn;

        ff=chil+xxrandn;

    end end

for f=1:52

    low(f)=12;

    up(f)=26;

    child(f) = min([min([child(f); low(f)]); up(f)]);

end

pop(j,:) = child;

bestgen ;
```

RESUME

Serdar Yıldırım was born in İstanbul on the 3rd of May, 1981. He obtained his B.Sc. degree from Sakarya University , Department of Mechanical Engineering in 2003. After 1 year , he attended M.Sc in Aeronautical and Astronautical Engineering at Institute of Science and Tecnology of ITU , in 2004. He has been working since September 2005 as a Mechanical Design Engineer in ENKA TEKNİK construction company performing industrial facilities mostly abroad.