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Behaviour–Finite Elements Analysis–Modeling of Hyperbolic Cooling Towers under Static and Vortex Wind Forces

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Abstract

Aim of this paper is software package utilized towards a practical application by considering problem of natural draught hyperbolic cooling towers. This paper deals with the study of hyperbolic cooling tower of 120 m high above ground level. This cooling tower has been analyzed for wind load using ANSYS by assuming fixity at the shell base. For this analysis a single case of the tower with alternative 'I' and 'V' supports is taken up. The wind load on this cooling towers has been calculated in the form of pressure by using the circumferentially distributed design wind pressure coefficients as given in IS: 11504-1985 code along with the design wind pressures at different levels as per IS: 875 (Part 3)-1987 code. The analysis has been carried out using and 4-noded shell element. The vertical distribution of membrane forces along and the circumferential distributions at base, throat and top levels have been studied for the cooling tower.

Keywords: Natural draught • Hyperbolic cooling tower • Finite element analysis • Equivalent plate thickness • Radial deformation

Introduction

The algorithms utilized in the analyses of shell [1] are employed towards demonstration of their applicability to an important practical problem. For this, the Natural Draught Hyperbolic Cooling Tower is considered. The towers in practice are supported either by I column system or V–type column system. In reference [2], a tower of 160 m height has been considered with this alternative supporting system. It is obvious that by taking up the investigation of these towers an additional benefit occurs in the manner of comparison of the relative effectively of these alternative support systems. In view of this, the data pertaining to these towers has been used herein for investigations.

Materials and Methods

Aerodynamic model–tubing response study

For the measurement of time-varying pressures in models subjected to random wind, small pressure transducers having sufficiently high frequency response are widely used. A pressure measurement tubing system of inadequate frequency response can lead to significant errors, particularly in peak pressures. In full scale, a value of 1 Hz is considered an appropriate upper limit of fluctuations in moderate winds. Based on reduced frequency scaling and wind tunnel model scales it is possible to obtain the upper frequency limit for the model pressure frequency system. For example, for a 1:500 scale model tested in a wind tunnel with a velocity scale of 1:1, the scaled model frequency limit would be

$(n\ L / U)\ m=(n\ L / U)\ p$ …… (1)

$n\ m=n\ p\ (L\ p / L\ m)\ (U\ m / U\ p)=1\ (500 /1)\ (1 / 1)=500$ Hz …….(2)

Where, (nL/U) is the reduced frequency, n is the frequency, U is the mean velocity at building height and L is a representative length dimension. Subscripts 'm' and 'p' refer to model and prototype, respectively. Pressure measurement systems used for wind tunnel studies have frequency responses typically from 50 to 500 Hz. In this range, there is a possibility for significant loss of fluctuating pressure signal and consequent attenuation of peak pressures while acquiring pressure data. The distortions are function of tubing length and diameter, the path through any pressure-scanning switch, and the internal volume of the transducer. As these distortions are dependent on frequency, the response characteristics of any given tubing system are calculated using a standard frequency response function approach. The tubing system under consideration has a total length of 150 mm and 1.2 mm internal diameter with restrictor located at 70 mm from the end closer to the model port and is used to acquire the data. Restrictors which are used to distort the pressure signal had an inner diameter of 0.4 mm and a length of 25 mm. considering the volume of the pressure transducers, only one restrictor has been used for each tube to achieve flat frequency response up to 500 Hz, approximately. The signals are collected through pressure transducer and the data are analyzed to obtain spectra corresponding to simulated frequencies using Fast Fourier Transform (FFT). The response of the tubing system under consider at ion is assessed through the ratio of spectral amplitudes as given below:

Ratio of Spectral Amplitude=(Spectral Amplitude of the Tubing System under Test)/(Spectral Amplitude of the response of reference short tube (30 mm long)………. (3)

The response of the shorter tube is referred as reference response. When the ratio of the responses of reference short tube to tubing system under consideration is 1.00, there is no necessity of any correction to the transfer function up to that level of frequency response, while measuring the pressures (mean or peak) on the building models under test. That is, the tubing system is assumed to record the pressures without any distortion, and the output traces can be used for further analysis without any correction to the signal, up to that frequency limit. The characteristic of the tubing is further validated theoretically by a standard frequency response function approach. The theoretically obtained frequency response and the corresponding phase shift are shown in Figures 1 and 2. Based on these figures, the tubing configuration chosen for the present study can be used for a scale up to 1:500, as the response is found to be almost flat and the phase shift is linearly varying up to 500 Hz.

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The following are the Wind Co-efficient used while using Wind Analysis:

C_{pe} (Z,) = P_e (Z,)/ 0.5 (U. Z^2)

And C_{pi} (Z,) = P_i (Z,)/ 0.5 (U. Z^2) ……. (4)

where, (z,) is the mean pressure for coordinates z and θ, suffix 'e' and 'i' are correspondingly for external and internal pressures, p is the static pressure of the approaching flow, is the density of air and U(z) is the mean velocity at height z.

To understand the mechanism of vortex shedding based on the experimental studies on the cooling tower model under study, Griffin universal Strouhal number, 'G', is also derived [3]. The concept of a universal Strouhal number is that same size vortex sheet may be expected to originate from different bodies, when proper scaling for reference length and velocity is used, besides the vortex shedding frequency. Griffin demonstrated that the product of the wake Strouhal number and wake drag coefficient tends to be a constant equal to 0.073 0.005 based on experimental results on a circular cylinder under uniform flow conditions. The Griffin's Universal Strouhal number is given by:

G = S. C_{fx}/ K^3 ……. (6)

where, Cfx is the mean drag coefficient at 0 degree angle of wind incidence; 'k' is the wake parameter and is given by:

K = $(-C_{pb})^{0.5}$ ………… (7)

Where, C_{pb} is the base pressure.

Description of towers

The geometry configuration of cooling tower shell is defined by Bangash, MYH [3].

r = Δr+a

Where, r is radius of tower shell at a height 'z' (m). The parameters a, b, Δr are, as per given below (Table 1).

Accordingly, the profiles of the towers are as shown in below (Figure 3). All the elevation details i.e. height of tower, indicated in the following Figure 3, are in meters.

Material properties of concrete considered are: E=3.4 x 107 kN/m^2, ν=0.167, γconc=23 kN/m^3

Finite element idealizations

The finite element idealization for both the towers is developed by employing both 4 nodded plate elements [4]. In this, 32 elements in hoop direction and 30 elements in meridional direction are provided. The height is 175 m and the thickness of the shell changes from 105 cms at the lintel level through 20 cms at the top of tower. In the meridional direction, the model has the mean radii and the shell thicknesses at various elevations as shown in below (Table 2). (All dimensions are in 'm').

There are 16 column supports supporting the alternative 4 nodded plate elements at the base of the tower. The c/s of the columns is 90 cms x 90 cms. The idealization of columns is carried through 4 segments of two no elements. These details for 'I' type and 'V' type supports are presented in (Figure 1). For both the models the base nodes of columns i.e. from node numbers 1 to 16 are kept fixed for all the six degrees of freedom (u, v, w, θx, θy, θz) and the models the structural system has been analyzed for its self-weight and also along with that it has been analyzed for the effect of wind load [5]. Its intensity has been calculated by using IS 875-III, which is especially for the Code of Practice for Design Loads (Wind Loads).

Table 1. Basic data for cooling towers.

Height (z)	9.17 m-125 m	125 m-175 m
A	51.9644	0.2578
B	113.9896	8.0293
Δr	-15.3644	36.3422

Table 2. Elevational mean radius and thickness details (design data).

		I Supports	V Supports
Displacement in m due to wind load at extreme top level	Column	0.167	0.292
	-15.3644	0.167	0.29

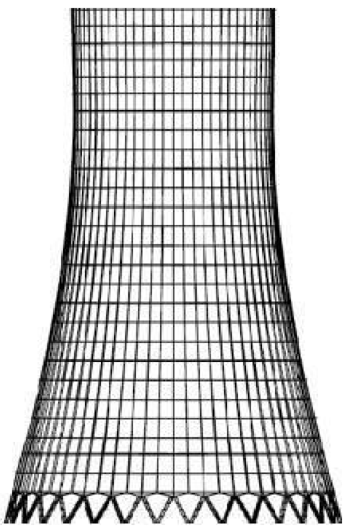

Figure 1. Idealization schemes showing column and equivalent plate details for 'V' type column supports.

Linear elastic response-concept of equivalent plate thickness

As pointed out above the linear elastic response for the towers is derived in respect of application of the self-weight and the wind load. In view of the fact that the soft-wares employed for the cylindrical shells deal exclusively with plate elements 8, a concept is developed wherein the column supports have been transformed into equivalent shell elements, so as to treat the complete tower system as a shell structure. The equivalent plate thicknesses for the column supports are based upon a consideration that the vertical deflection at the top of the tower remains same as the once due to column support wherein only the influence of the self-weight is considered.

As the complete development of the software for analysis of various types of elements is employing exclusively the plate elements therefore it was considered more practical to transform the column supports in the towers for equivalent plates. For this the influence of the self-weight was considered by analyzing the tower structures with columns and plate combinations. The vertical displacement at the top was determined through this analysis. For deriving the equivalent plate thickness 4 nodded plates were considered with the idealization now taking the format as shown in Figure 1. With this kind of idealizations numbers of trials are taken to arrive at the plate thickness which would produce the same vertical deflection as was found out for the column plate systems. In this manner the equivalent thickness" for the 'I' column supports was derived equal to 0.040m and for 'V' column support it was derived equal to 0.037 m.

It may be noted that as far as different types of supports are concerned the equivalent thicknesses are quite close to each other. To ascertain the validity of this kind of alternative formulation for carrying out the further kind of analysis the influence of the wind loads is examined for both the column plate system and the equivalent plate system. The results for the sway suffered by the systems are indicated in Table 3. In the graph presented below y axis indicates elevation height in meters, while x axis indicates the displacement due to wind load in meters.

It is possible to conclude that; the deflected profile is almost identical in case of equivalent structure as compared to the original structure.

1. The present investigation is planned to compare relative behaviour of 'I' supports and V' supports. The details in Figure 2 reveal that the 'V' support system is more flexible compared to 'I' support system.

2. With a view to achieve the thorough validation of the concept of the equivalent plate thickness; the radial deflections at the throat section were also compared as shown in Figure 3.

Elasto-plastic analysis

For both the towers the Elasto-plastic analysis is performed by adopting the equivalent plate idealizations developed above. For this the applied load is wind load along with the load due to self-weight. As the structural systems are huge it is considered impractical to present graphically the development of the phenomenon of plastic flow.

For 'I' type support system

The zone of plasticity starts with throat region. It further progresses in the downward direction and finally it flows in both upward and downward direction till the collapse load is reached. In Figure 4 percentage details are given.

For 'V' type support system

Similar response is also observed for this type of system. Table 4 indicates the collapse loads i.e. wind load multiplier factor for both the types of towers by 4 noded and 9 noded plate elements.

The results in Figures 4 and 5 reveals that between the 'I' and 'V' supports there is not much difference between the plasticity development however for the 'V' support the lesser load is required for the flow of plasticity in the region of 60% degree of plasticity and full collapse. In fact, this is expected because it has already been observed that the 'V' support system is more flexible. The basic phenomenon of development of the plastic flow remains similar to one observed with 4 noded plate formulations. The percentage development characteristics are as shown in Figure 6. However now no significant difference is observed between the 'I' support and 'V' support.

Influence of reinforcement

The incorporation of the reinforcement is achieved through equivalent steel plate pasted on the idealized system. Thus while maintaining the number of nodes same the number of elements get doubled. The thickness of the steel reinforcement is derived by assuming 2% of the average thickness of the tower, which comes about to be 10 mm. For this once more the Elasto-plastic analysis is conducted and the results are as shown in Figure 6.

From Table 5 and Figure 6, it may be concluded that both 'I' support and 'V' supports have identical response in case of the RCC formation. It means that the small difference which was earlier observed for the concrete sections has also now vanished. This may be termed as positive influence of incorporation of the steel reinforcement in the concrete components (Figures 7-11).

Table 3. Sway details by wind load.

		I–Support	V-Support
Wind Load Multiplying Factor	4 Noded Flat Plate Element	7	5
	9 Noded Flat Plate Element	7.5	7.5

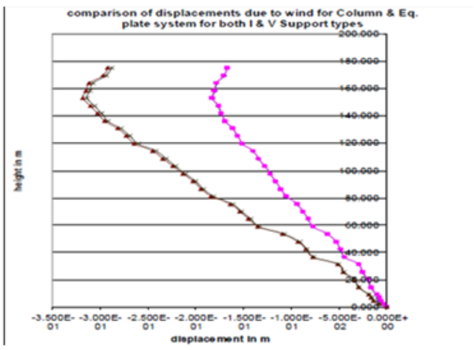

Figure 2. Comparisons of displacements due to wind. Note: (—) Wind load in meters, (—) Equivalent plate system, (—) Column plate system, (—) Displacement in meters.

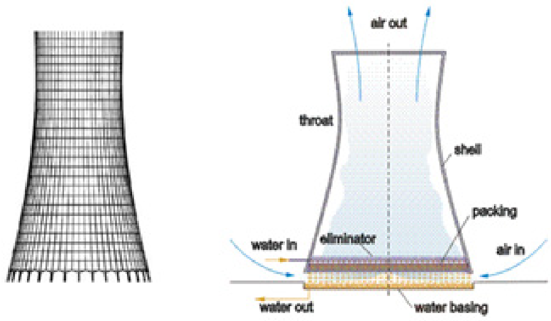

Figure 3. Schematic functional diagram of natural draught cooling tower.

Table 4. Wind load multiplying factor.

		I–Support	V-Support
Wind Load Multiplying Factor	4 Noded Flat Plate Element	7	5
	9 Noded Flat Plate Element	7.5	7.5

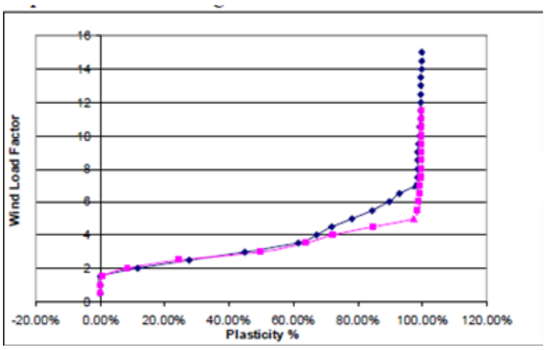

Figure 4. Percentage flow of plasticity of towers due to wind load by 4-noded plate element. **Note:** (—◆—) Wind load factor, (—■—) Plasticity.

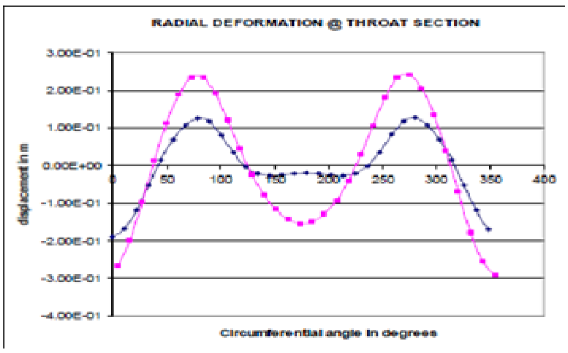

Figure 5. Once more the conclusions in 1and 2 above are also found to be valid for the throat section of the tower. **Note:** (—◆—) Displacement in meters, (—■—) Circumferential angle n degrees.

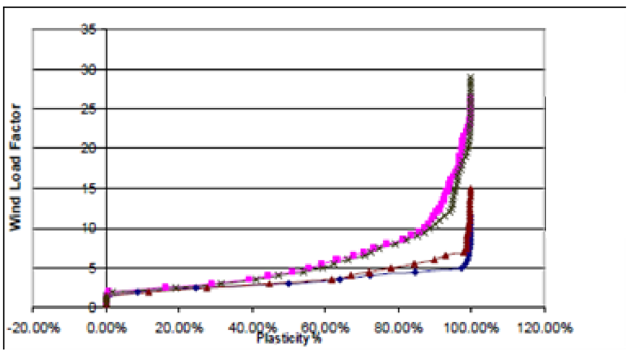

Figure 6. Percentage plasticity flow in towers due to wind by 4- noded element with and without reinforcement. **Note:** (—◆—) Wind load in factor, (—■—), Equivalent plate system, (—▲—) Column plate system, (—) Displacement in meters.

Table 5. Self-weight+wind load multiplyer.

Self-Weight+Wind Load Multiplyer	**I–Support**		**V–Support**	
	Concrete	Steel	Concrete	Steel
	7	9.5	5	8.5

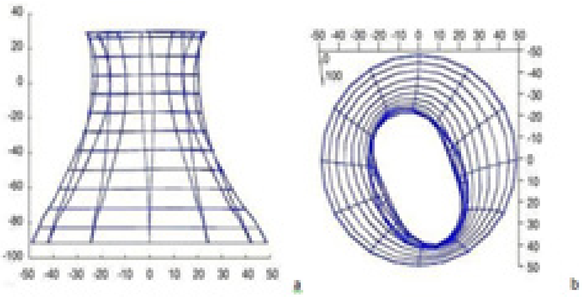

Figure 7. Drift measures under wind turbulence.

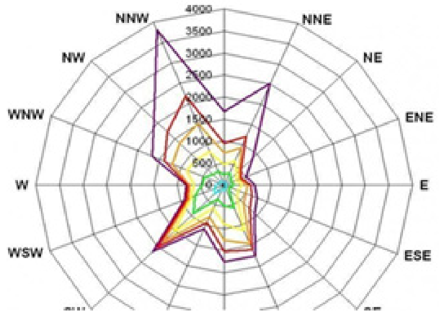

Figure 8. Model shows vortex shedding of shells under cross wind. **Note:** (——) Cross Wind, (——) Elasto-plastic, (——) Gravity of wind, (——) Wind analysis, (——) Concrete, (——) Shell thickness.

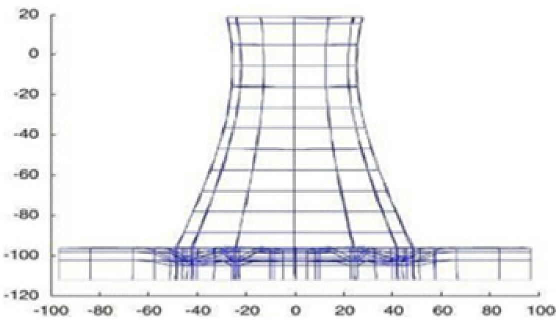

Figure 9. Deflection profile under gravity load and wind.

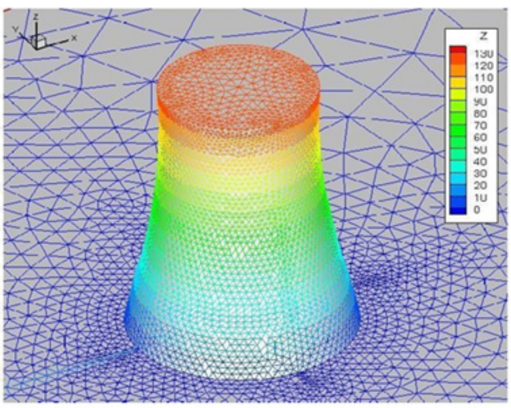

Figure 10. Unstructured tetrahedral mesh for natural draught cooling tower for dynamic wind analysis using ANSYS.

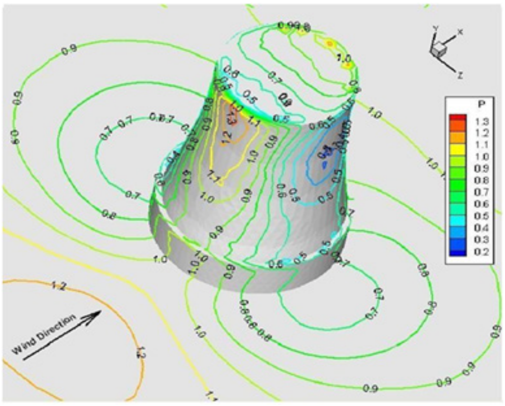

Figure 11. Upstream and downstream views of computed dimensionless wind pressure contours for cooling towers using ANSYS.

Results and Discussion

Data availability statement

Data available on request due to privacy/ethical restrictions Data subject to third party restrictions. The data that support the findings of this study are available from the author upon reasonable request over mail as mentioned in the manuscript.

Conclusions

(1) The main aim of the analysis work on the cooling tower has two folds:

a) To compare the structural behavior of the tower with different foundation supports such as 'I' type support and 'V' type supports.

b) To provide a rational basis for transforming each of these support types into equivalent shell surfaces, so that various software's employed in the basic investigation of the shells could be utilized.

(2) From figure. 3, it is observed that the equivalent shells provide identical deflected profiles for the application of the wind loads, as those due to actual supports.

(3) The 'V' supports create relatively more flexible structure compared to the one having 'I' supports. From fig. 3 and table 3, this is indicated by virtue of development of more sway in case of 'V' support with respect to 'I' support when the influence of the wind load is considered.

(4) From table 3, it is noticed that the 'V' supports give 73.6% more sway than 'I' supports in the case of column supports as well as equivalent plate system due to application of wind load.

[5] The progress of the development of the plastic zones has shown for both kinds of support systems initiation at the throat level and subsequently first progressing towards the downward direction over the height of the towers and then it progresses towards both downward as well as upward direction also.

(6) It is observed from fig. 5 that the collapse load pattern derived for 'I' support systems and 'V' support systems are fairly similar.

(7) It is observed that the collapse load in case of 'I' support system is having 40% higher value than in case of 'V' type support systems.

(8) It is clearly seen that the structure with the provision of reinforcement i.e. steel plate, can sustain almost 35 to 50% more collapse load than that of plain concrete.

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