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Study on Buckling Behaviour of Hyperbolic Cooling Towers

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ABSTRACT

Hyperbolic cooling towers are an essential part of thermal power plants. It is the most commonly used shape among natural draught cooling towers. Two cooling towers with different heights are analyzed. Both the towers have varying thickness throughout the height. ABAQUS 6.14 is used to conduct the finite element analysis. Towers are modeled as 3D shells of revolution with fixed support condition at the base. Static and buckling analyses are conducted. Stress concentration and buckling behavior of both towers are analyzed and compared.

Keywords :- ABAQUS, Shell, Hyperbolic cooling tower, Static, Buckling

I. INTRODUCTION

Hyperbolic cooling towers are large, thin shell reinforced concrete structures which contribute to environmental protection and to power generation efficiency and reliability. They are an imposing characteristic and integral part of thermal power plants all over the world. The purpose of a cooling tower is to re-cool the water used for condensation of steam in a thermal power plant. Cooling towers are divided into two main types, the first being named natural draught cooling towers and the second mechanical draught cooling towers. In natural draught tower, the circulation of air is induced by enclosing the heated air in a chimney which then contains a column of air which is lighter than the surrounding atmosphere. This difference in weight produces a continuous flow of air through the cooling tower. This upward flow of air is found to be easier to naturally sustain in towers having a hyperbolic shape. Cooling towers with other shapes such as cylindrical often have to be mechanical draught as the updraft of air inside the tower needs mechanical assistance to be sustained. Natural Draught cooling towers are most effective measures for cooling of thermal power plants. They are able to balance environmental factors, investments and operating costs with demands of reliable energy supply. Large reinforced concrete, natural draught cooling tower structures can be as tall as or even taller than many chimneys, however due to their design and function, they have a very much larger surface area, with a much lower mass to surface area ratio [10]. Cooling tower shell is usually supported by a truss or framework of columns.

Hyperbolic shape of cooling towers was introduced by two Dutch engineers, Van Iterson and Kuyper, who in 1914 constructed the first hyperboloid towers which were 35 m high. Soon, capacities and heights increased until around 1930, when tower heights of 65 m were achieved. The first such structures to reach higher than 100 m were the towers of the High Marnham Power Station in Britain. Today's tallest cooling towers, located at several nuclear power plants in France, reach heights of about 170 m.

Two cooling towers, one each from Tuticorin Power Plant and thermal power plant of Neyveli Lignite Corporation are selected for the analysis. Both the towers have varying thickness across its height.

In the study by Sachin Kulkarni et al [1], static and dynamic analysis of two existing cooling towers of different heights were chosen from Bellary Thermal Power Station (BTPS) as case study. The boundary conditions considered are top end free and bottom end fixed. The maximum principal stress for two existing cooling towers shows high value by using 4 nodded shell elements. Taller tower shows less maximum principal stress than the shorter tower. In buckling analysis, the buckling of shorter tower is found to be larger as compared to taller tower.

In the study by Sachin Kulkarni et al [3], two existing cooling towers of different heights were chosen from Bellary Thermal Power Station (BTPS) as case study. The seismic analysis of the towers showed larger stresses for larger thickness for the shorter tower and smaller stresses for the smaller thickness. In case of the taller tower the converse was found to be true.

In the study by A M El Ansary et al [5], minimum stress levels were found when the shell was optimised by reducing its thickness.

II. MODELLING IN ABAQUS

ABAQUS 6.14 is used to model the towers. The structural diagram of the shells is used to generate coordinates which are used to construct a series of nodes representing the vertical profile of the shell. The vertical profile is then revolved to form the three-dimensional shell structure. Thickness is assigned to the relevant regions of the shell. Shell elements with 4 nodes are used for meshing.

A. Geometry of Towers

Tower T1 from Tuticorin thermal power plant has the dimensions as follows:

- Top radius= 36.565 m
- Bottom radius= 56.1 m
- Throat radius=34.43 m
- Height of throat=116.275 m
- Height above ground= 147.7 m

The shell of the tower is supported on V shaped columns at a height of 7.8 m above ground.

Tower T2 from the power plant of Neyveli Lignite Corporation has the dimensions as follows:

- Top radius= 26.71 m
- Bottom radius= 41.75 m
- Throat radius=26.15 m
- Height of throat=71.215 m
- Height above ground= 105.5 m

The shell of the tower is supported on V shaped columns at a height of 5.4 m above ground.

B. Material Definition

M40 concrete is defined. Since only linear behaviour is investigated only elastic characteristics of the material is defined. Poisson's ratio of 0.15 and a Young's Modulus of 31GPa are defined. A unit weight of 25 kN/m^3 is assigned.

C. Thickness Definition

A section each is defined for each thickness of the shells. These individual sections are then manually assigned to the corresponding regions of the shell geometry.

D. Support Conditions

V shaped support columns have been found to mimic a shell base supported by fixed supports ([2],[7]). Hence, the base of the shells of the towers is modelled to have fixed support condition.

E. Loading

The shell is analysed under dead loads. Hence, gravity is defined with acceleration due to gravity as 9.8 m/s^2.

F. Meshing

Meshing is done by using shell elements with 4 nodes. Meshing with 4 nodded elements have been found to exhibit higher stresses ([1],[4],[6,][8],[9]).

III. ANALYSIS

The towers are subjected to static and buckling analysis.

Static analysis is performed under the influence of dead loads.

Buckling modes are investigated under the effect of dead loads. Subspace Eigen solver is used to generate 50 Eigen values and their corresponding buckling modes.

IV. RESULTS

G. Static Analysis Results

Stress distribution in both towers is obtained. Stress distributions plotted on deformed shapes are given below.

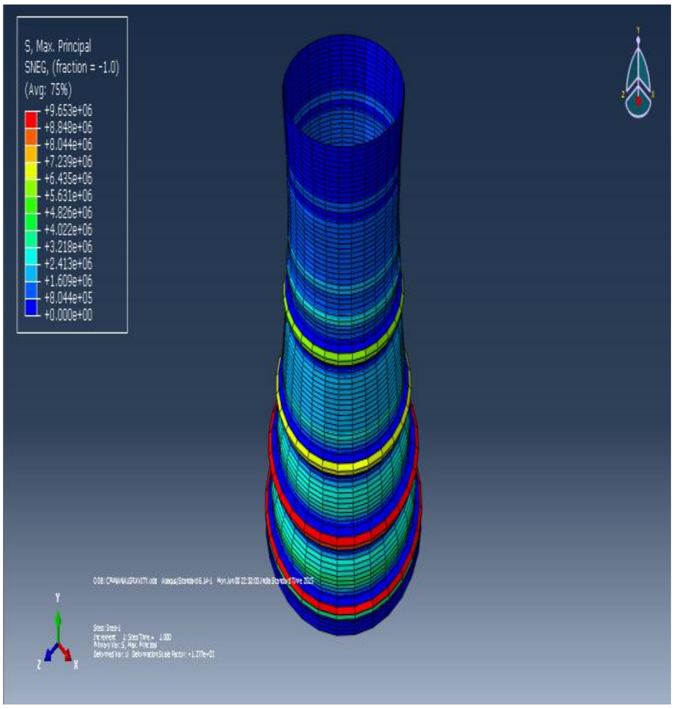

Fig. 1 Max principal stress distribution for tower T1

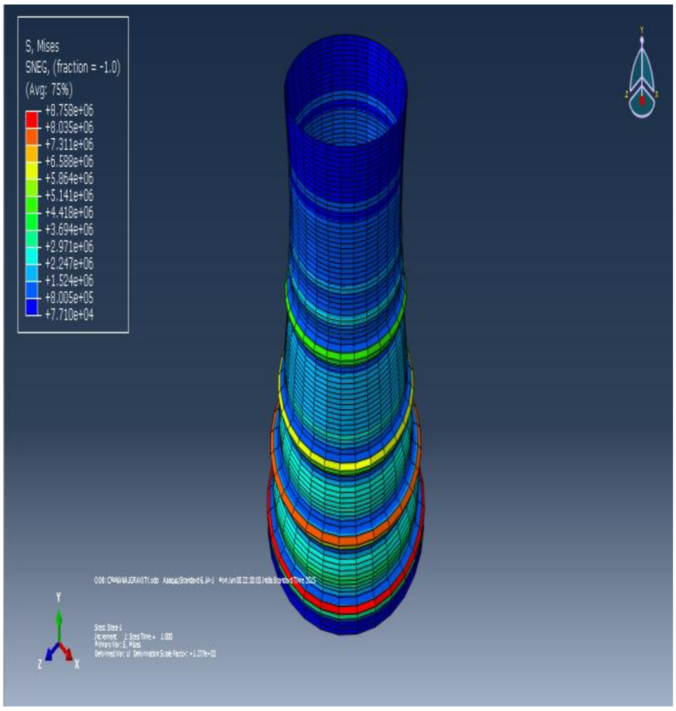

Fig. 1 Von Mises stress distribution for tower T1

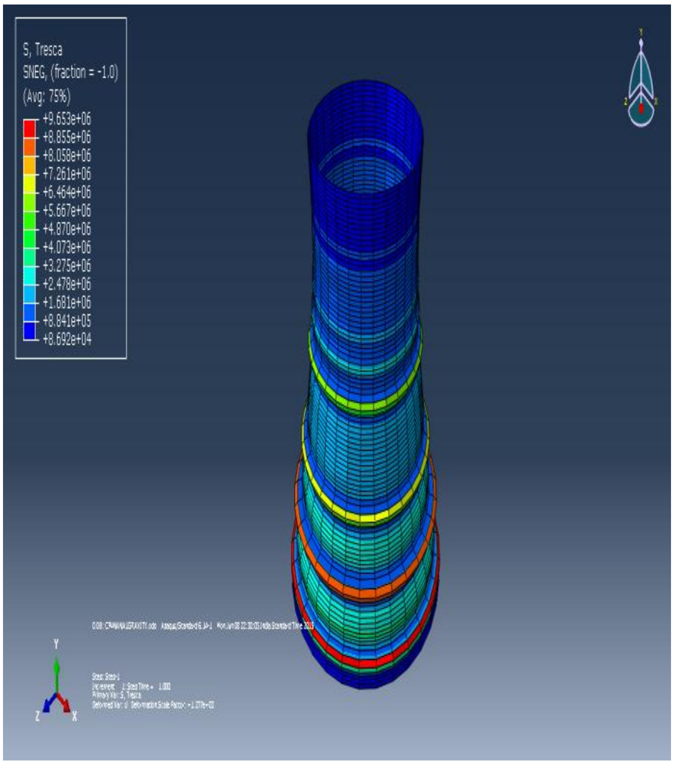

Fig. 3 Tresca stress distribution for tower T1

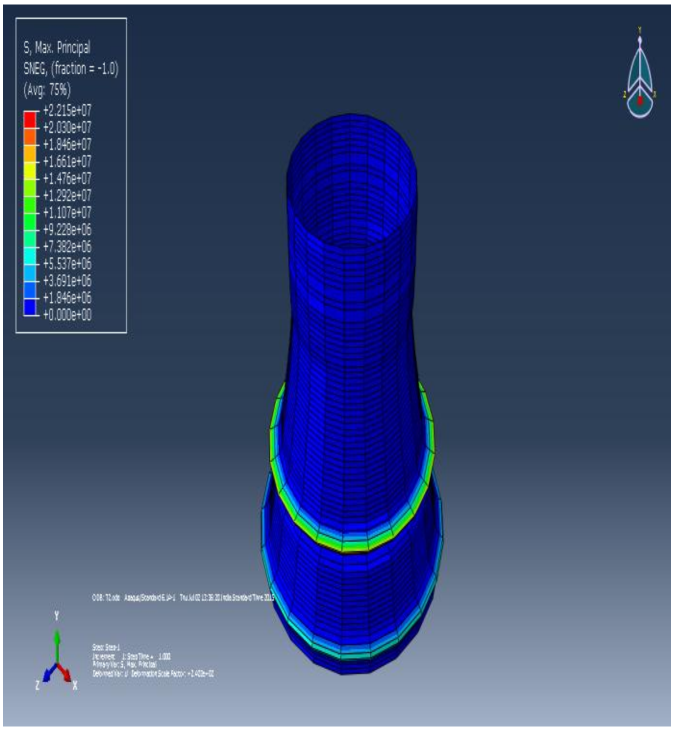

Fig. 4 Max principal stress distribution for tower T2

Max Mises stress for T1= 8.75791×10ˆ6 N/m^2

Max principal stress for T1= 9.65258×10ˆ6 N/m^2

Max Tresca stress for T1= 9.653×10ˆ6 N/m^2

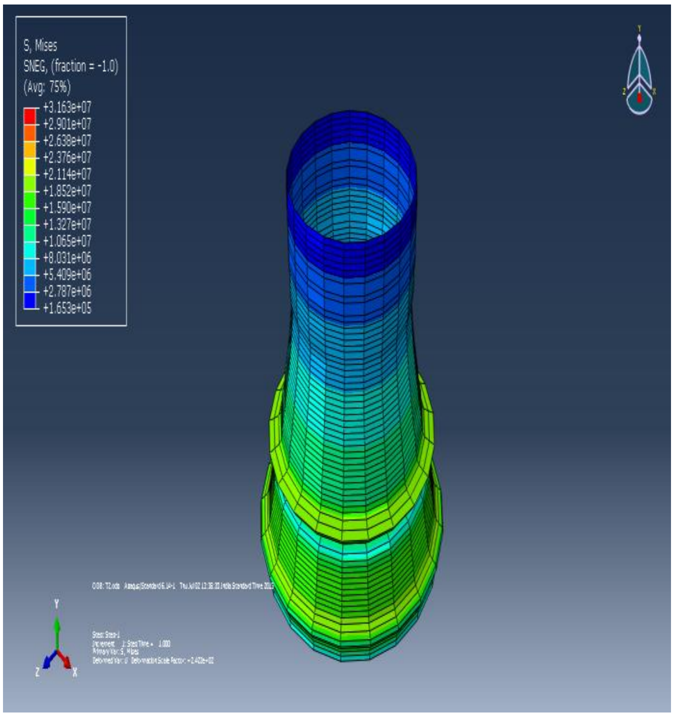

Fig. 5 Von Mises stress distribution for tower T2

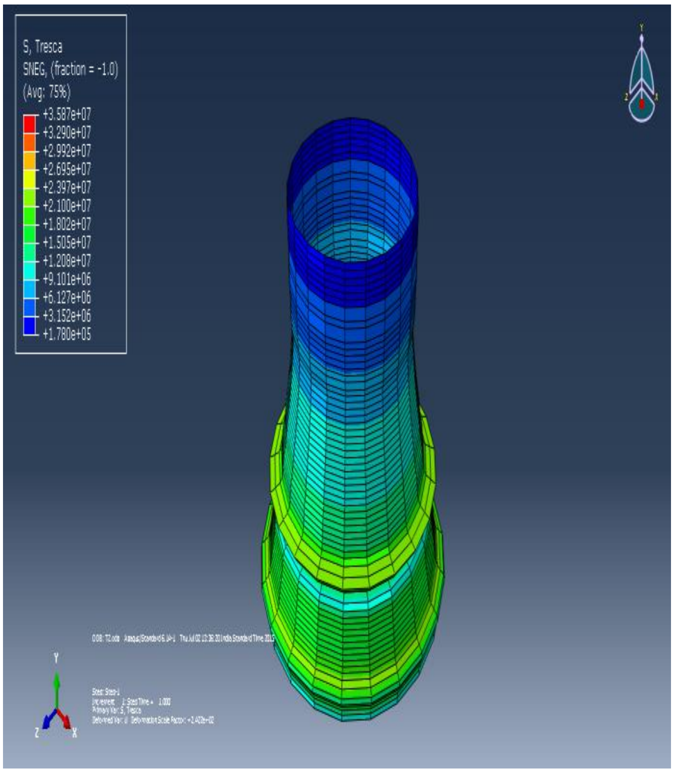

Fig. 6 Tresca stress distribution for tower T2

Max Mises stress for T2= 5.39737×10ˆ7 N/m^2

Max principal stress for T2= 2.21471×10ˆ7 N/m^2

Max Tresca stress for T2= 3.587×10ˆ7 N/m^2

H. Buckling Analysis Results

Subspace Eigen solver yielded 50 Eigen values for both the towers and their corresponding buckling modes.

TABLE I
EIGEN VALUES FOR TOWER T1

Mode No	Eigen Value	Mode No	Eigen Value
1	1.8508	26	3.7054
2	1.8508	27	3.7054
3	2.0816	28	3.7139
4	2.0816	29	3.7495
5	2.1645	30	3.7495
6	2.1645	31	3.7798
7	2.4582	32	3.7798
8	2.4582	33	3.8144
9	2.5762	34	3.8144
10	2.5762	35	3.8668
11	2.7038	36	3.8668
12	2.7038	37	3.9391
13	2.7846	38	3.9391
14	2.7846	39	4.0824
15	2.9291	40	4.0824
16	2.9291	41	4.0926
17	2.936	42	4.0926
18	2.936	43	4.2313
19	3.231	44	4.2313
20	3.231	45	4.3665
21	3.4304	46	4.3665
22	3.4304	47	4.3719
23	3.6083	48	4.4092
24	3.6083	49	4.416
25	3.6632	50	4.4161

Buckling Load Estimate = Eigen value × Load in buckle step
Load in buckle step=Weight of structure= 2.7575671×10ˆ8 N
Lowest Eigen value for T1 = 1.8508
Critical Buckling Load of T1= 510370518.9 N

TABLE III
EIGEN VALUES FOR TOWER T2

Mode No	Eigen Value	Mode No	Eigen Value
1	3.4955	26	6.1266
2	3.4955	27	6.1575
3	3.6334	28	6.1575
4	3.6334	29	6.4016
5	3.6362	30	6.4664
6	3.6362	31	6.4664
7	4.7926	32	6.5066
8	4.7926	33	6.5323
9	5.011	34	6.5323
10	5.011	35	6.6081
11	5.2733	36	6.6081
12	5.297	37	6.6358
13	5.297	38	6.6358
14	5.3369	39	6.9217
15	5.3886	40	6.9217
16	5.3886	41	7.0562
17	5.4095	42	7.0562
18	5.4095	43	7.5281
19	5.4528	44	7.6261
20	5.4528	45	7.644

21	5.7571	46	7.644
22	5.7571	47	7.6729
23	5.8229	48	7.6729
24	5.8229	49	7.6757
25	6.1266	50	7.6757

Buckling Load Estimate = Eigen value × Load in buckle step
Load in buckle step=Weight of structure= 1.0275282×10ˆ8 N
Lowest Eigen value for T2= 3.4955
Critical Buckling Load of T2= 359172482.3 N

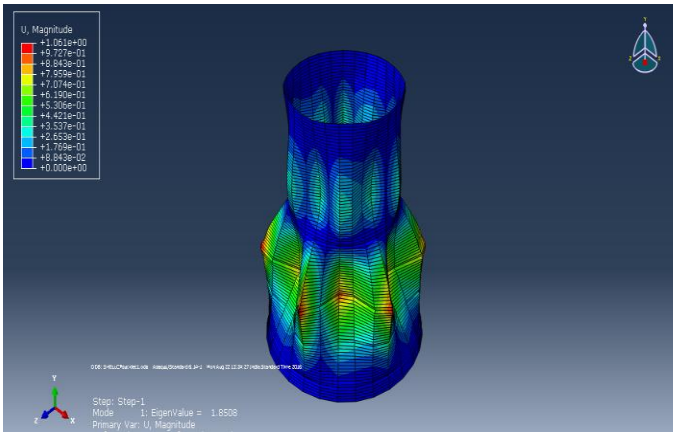

Fig. 7 Critical Buckling Mode for T1

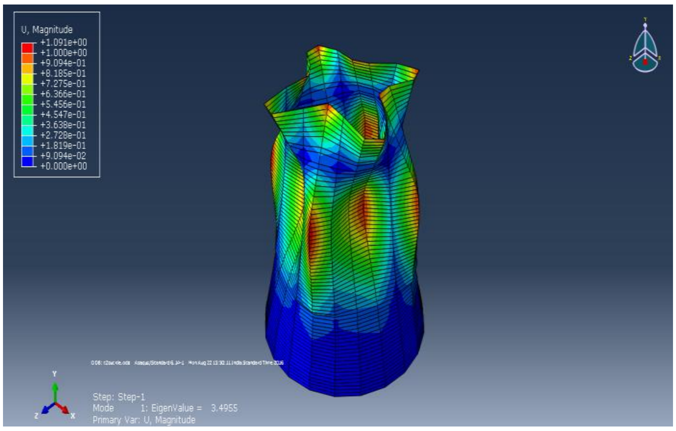

Fig. 8 Critical Buckling Mode for T2

V. CONCLUSIONS

Static and buckling analysis was done on two towers with different heights and varying thickness throughout their height. The following conclusions were arrived upon:

- Maximum stress occurs near the base, near the throat and near 1/6 th, 2/6 th the height from the base for the taller tower T1. This is due to large changes in slope of the vertical profile at these points.
- Maximum stress occurs near the base and near 2/6 th the height from the base for the shorter tower T2. This is due to large changes in slope of the vertical profile at these points.
- Both towers show the least stresses near the top.
- Both towers show relatively higher stresses near the throat.
- The taller tower T1 predictably has a higher critical buckling load than the shorter tower T2 on account of its larger shell thickness.

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