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WIND EFFECTS ON HYPERBOLIC COOLING TOWERS

N. Prabhakar

Technical Manager
Gammon India Limited
Prabhadevi
Bombay 400 025, INDIA

SYNOPSIS

Wind force forms the major external applied loading in the design of hyperbolic cooling towers. The paper reviews assessment of wind pressures acting on the towers with reference to Indian codes, and influence of meridional ribs on cooling tower shell with case studies. It also deals with sensitiveness of shell and steel reinforcement due to wind induced tension, and briefly on the dynamic effect in large cooling towers.

1. INTRODUCTION

Many hyperbolic cooling towers have been built in the country at several thermal and nuclear power stations. In view of their large size with very small shell thickness, they are very sensitive to horizontal loads such as wind. In this paper an attempt has been made to review applied wind loadings with reference to Indian codes, influence of meridional ribs on circumferential wind pressure distribution and its effect on the shell with case studies, and current design methods and specialised problems associated with wind effects on hyperbolic cooling towers.

2. WIND

2.1 Wind Pressure

Till the recent publication of the Indian Standard Code of Practice IS:875 (Part 3)-1987 [1] in February 1989, the design wind pressures on large number of cooling towers built since mid-1960s were calculated on the basis of the earlier code of practice IS:875-1964 [2] which adopted wind pressure as static loads, the intensity of which varying with height and the zone at which the structure is located. The new code IS:875 (Part 3)-1987 determines wind pressures based on peak wind speed of 3 second-gust with a return period of 50 years. The zones of basic wind speed at 10m above ground at speeds of 33, 39, 44, 47, 50 and 55 m/sec. are shown in the code on a wind map of the country. The design wind speed is calculated by considering factors related to probable life of structure, terrain, local topography and size of structure separately, and their combined effect is determined by multiplying the factors. Fig. 1 shows comparison of design wind pressures as per the old and new IS codes for cooling tower type structure in an open terrain. It is seen that the pressures at 39, 47 and 55 m/sec. wind speed more or less,tally with the pressures of the earlier code, and new wind zones at wind speed of 33, 44 and 50 m/sec. are introduced in the new code.

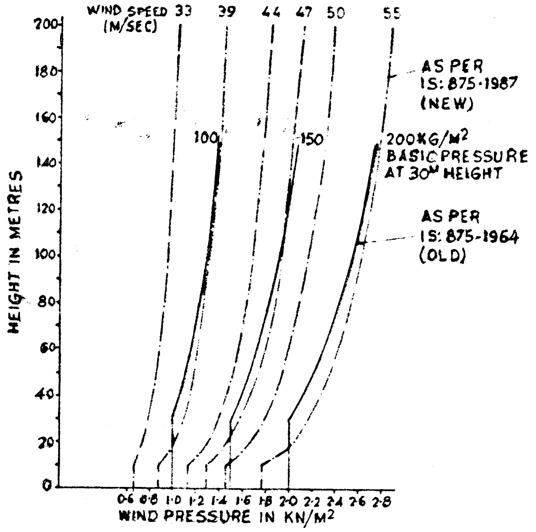

Figure 1: Design Wind Pressures Old and New IS Codes for Open Terrain, Class C Structures.

The Indian cooling towers built so far have been designed for peak-wind pressures of short duration by static method. It is very well established now that wind effects on the tower are characterised by the presence of a large steady-state component and a significant random component due to air turbulence. The response of the random component can be calculated in the frequency domain by spectral analysis. This component contributes strongly to the total response peaks at a rate of atleast 50%. Although this theory is well established in principle, it is not used for practical design of cooling towers as Niemann [3] has found that large amount of computations are to be made involving several factors in both meridional and circumferential directions at different elevations of the tower, for separate cases of tensile, compressive, shear forces and bending moments in the shell. The objective approach as adopted in many codes, has been to translate the loading and structural response into a quasi-static method by applying a factor, often called as the 'Gust-Factor', in the static analysis of the tower. It must be said however, that deficiencies if any, of the equivalent quasi-static load concept are balanced by a set of provisions such as minimum shell thickness and reinforcement, high buckling safety, etc. which are observed in the practical design.

The gust factor depends on the natural frequency in the fundamantal mode, wind speed, terrain and size of structure. In view of large size of the structure, the peak response occuring in a time interval of 1 hour duration is considered appropriate for the design of cooling towers. The gust factor method given in the new IS code IS:875 (Part 3)-1987 is shown for regular shaped slender structures such as cubes, cylinders with hardly any taper. For hyperbolic shape, the diameter at the throat level is considered as the breadth of the structure, on a conservative approach. The gust factor is calculated by the following equation as per the code :

$$G = 1 + g_f r \sqrt{B(1 + \phi)^2 + SE/\beta}$$

where '$g_f r$' is a function of terrain and height of the structure, 'B' is background turbulence factor depending on terrain and size of the structure, 'ϕ' is usually zero as cooling towers are over 75 m height, and 'SE/β' is related to wind fluctuations near the natural frequency of the structure. It is found that for cooling towers, the 'G' value is governed by terms '$g_f r$' and 'B' only, and the other factors are very small and of little significance. The 'G' value usually varies between 1.6 and 2.2, the value increasing with smaller tower height and rough terrain. The gust factor given in IASS recommendations for cooling tower [4] for 'VH/(f a)' value of 0.8, 1.6 and 2.0 are 1.85, 2.0 and 2.15 respectively, where 'VH' is the mean-hourly wind at the top of tower, 'f' is the lowest mode frequency and 'a' is the throat radius. The 'G' values of IS Code and IASS recommendations are more or less close to each other. The gust factor given in German VGB guidelines [5] varies between 1.0 and 1.15, and in ACI-ASCE Report 334 [6] it is considered as 1.0, but these are for peak wind pressures instead of the mean-hourly wind pressure considered earlier. It is found that design wind pressures as calculated by the gust-factor method are more than those due to the peak-wind method by about 15 to 20%.

2.2 Distribution of Wind Around the Shell

The circumferential distribution of wind around the shell at any height is usually defined by normalising values of equal angle increments from the windward direction, and is represented by a Fourier series, $H = \Sigma An \cos N\Theta$. Table I shows the wind pressure cofficients 'An' which have been extensively used for the Indian towers. The IS code IS:11504-1985 [7] for natural draught cooling towers specifies the same cofficients as in BS:4485 [8].

TABLE - I

Fourier Coefficient 'An'

Harmonic	BS 4485 1975	Niemann 1971	Zerna 68
0	-0.00071	-0.3923	0.128056
1	0.24611	0.2602	0.435430
2	0.62296	0.6024	0.511731
3	0.48833	0.5046	0.372272
4	0.10756	0.1064	0.104642
5	-0.09579	-0.0948	-0.045549
6	-0.01142	-0.0186	-0.027082
7	0.04551	0.0468	0.018113

Note :

1. BS:4485 includes 0.4 internal pressure.
2. Niemann excludes internal pressure.
3. Zerna includes 0.5 internal pressure.

The coefficients by Zerna are based on measurements on full-scale hyperbolic cooling tower having small meridional ribs on shell. These ribs create surface roughness of the tower, and the effect of this is to reduce the suction on the sides of the tower. The roughness parameter is characterised by the ratio of K/S as shown in Fig. 2. The projection 'K' is usually 50 mm to 100 mm, and the spacing 'S' is about 2 m to 6 m. The width of the rib is taken between 2K and 5K, and this has no influence on pressure coefficients.

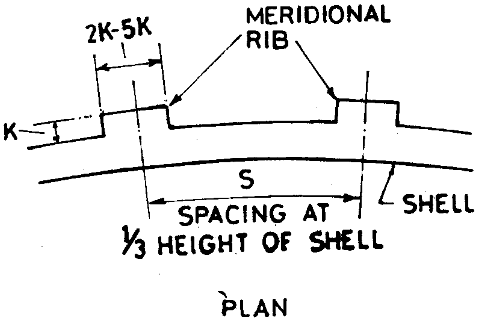

Figure 2: Roughness Parameters Ref. IASS [4] and VGB [5]

The IASS recommendations and the VGB guidelines give equations for the pressure coefficients around circumference of the shell for four different cases of rib projection, and these are given in Table.II. Graphical presentation of these coefficients are shown in Fig. 3.

TABLE - II

Coefficients for Circumferential Distribution of Wind Around Shell as per IASS Recommendations [4] and VGB Guidelines [5].

Roughness Parameter K/S	Curve	Coefficient		Zone III
		Zone I	Zone II	
0.025 to 0.100	K 1.0	$1-2.0(\text{SIN}\ \frac{90}{70}\ \theta)^{2.267}$	$-1.0+0.5(\text{SIN}(\frac{90}{21}(\theta-70)))^{2.395}$	-0.5
0.016 to 0.025	K 1.1	$1-2.1(\text{SIN}\ \frac{90}{71}\ \theta)^{2.239}$	$-1.1+0.6(\text{SIN}(\frac{90}{22}(\theta-71)))^{2.395}$	-0.5
0.010 to 0.016	K 1.2	$1-2.2(\text{SIN}\ \frac{90}{72}\ \theta)^{2.205}$	$-1.2+0.7(\text{SIN}(\frac{90}{23}(\theta-72)))^{2.395}$	-0.5
0.006 to 0.010	K 1.3	$1-2.3(\text{SIN}\ \frac{90}{73}\ \theta)^{2.166}$	$-1.3+0.8(\text{SIN}(\frac{90}{24}(\theta-73)))^{2.395}$	-0.5

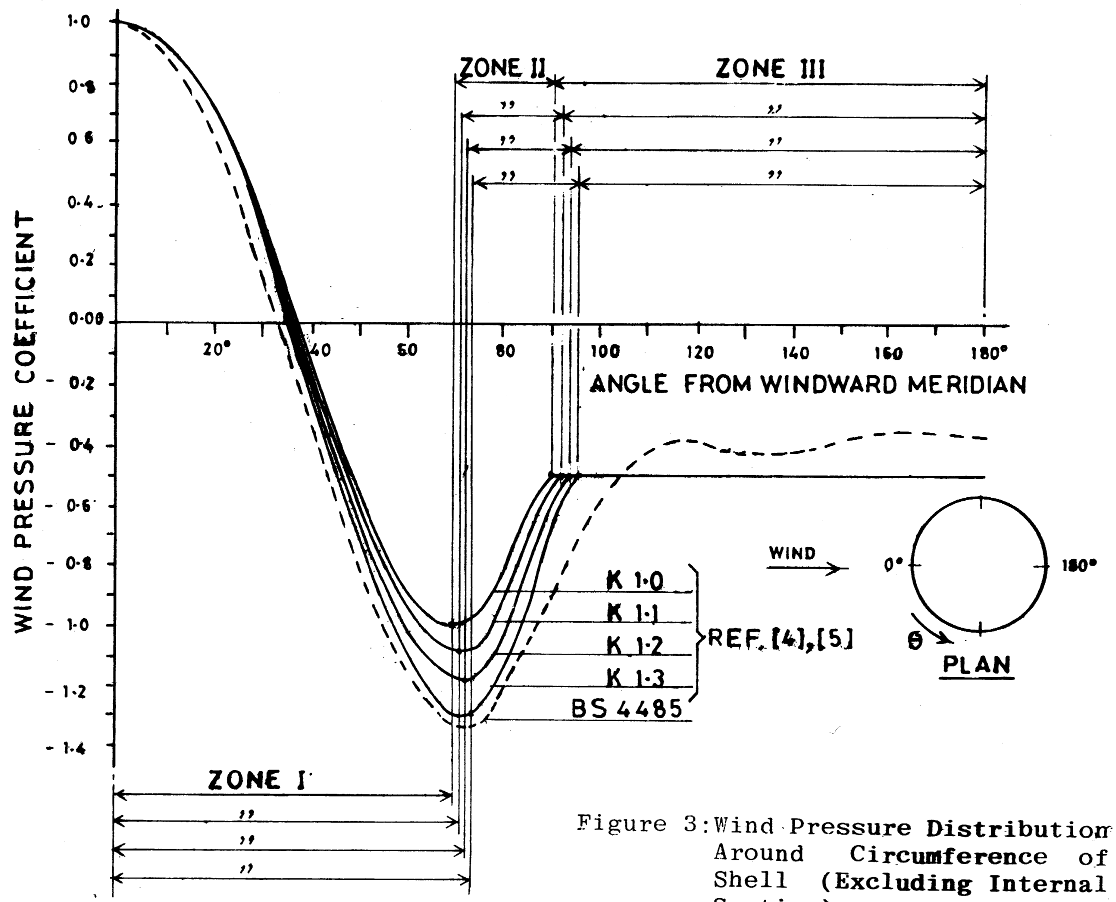

Figure 3: Wind Pressure Distribution Around Circumference of Shell (Excluding Internal Suction).

For the purpose of comparison of the effect of surface roughness, stress resultants for a cooling tower of the size given in Fig. 4 are worked out for the cases of smooth shell and four grades of roughness with meridional ribs. The shell thickness in each case is based on a minimum factor of safety of 5 against local buckling as per the IASS recommendations.

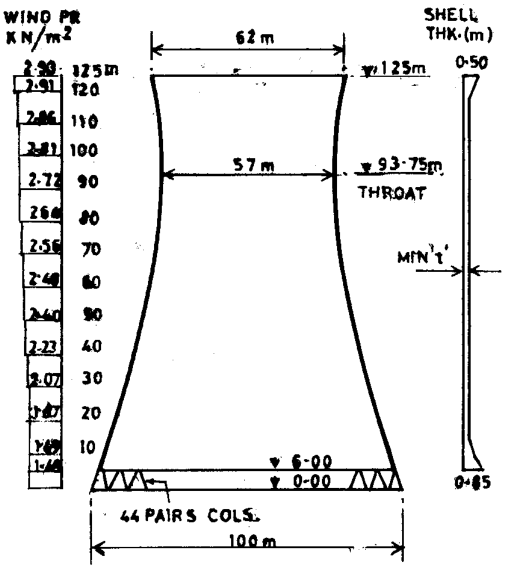

Figure 4:Cooling Tower for the Purpose of Comparison.

The results of meridional stress resultants in shell due to wind for the cases of maximum tension and compression are given in Tables III and IV. It is seen that the values of stress resultants in both the Tables are considerbly reduced at lower levels as the surface roughness increases. The significance of this reduction in the stress resultants has much effect on the requirement of shell thickness, quantity of shell concrete and reinforcement, and also on loads on raker columns and tower foundation, and these are given in Tables V, VI and VII. The maximum percentage of reductions are given below:

(i)	Shell concrete quantity	4.7%
(ii)	Shell reinforcement	16.4%
(iii)	Raker column load compn.	11.4%
	- do - Tension	24.2%
(iv)	Foundation load compn.	19.5%
	- do - Tension	68.0%

Undoubtedly, cooling tower shells with meridional ribs offer an economical solution, particularly for towers located in zones of high wind pressures.

TABLE - III

Meridional Stress Resultants due to Wind in kN/m, Tension at $\theta = 0$ Deg.

Level (m)	Smooth Shell as per BS:4485	Shell with meridional ribs as per IASS			
		K 1.3	K 1.2	K 1.1	K 1.0
125	0	0	0	0	0
115	40	33	31	28	28
105	164	133	127	114	114
95	342	289	278	258	256
85	508	463	450	433	426
75	650	634	620	607	590
65	740	738	716	697	671
55	800	794	763	732	702
45	834	815	779	737	706
35	851	819	780	732	702
25	863	817	778	727	701
15	875	817	780	723	705
6	894	825	789	737	717

TABLE - IV

Meridional Stress Resultants due to Wind in kN/m, Compression $\theta=63-72$ Deg.

Level (m)	Smooth Shell as per BS:3385	Shell with meridional ribs as per IASS			
		K 1.3	K 1.2	K 1.1	K 1.0
125	0	0	0	0	0
115	- 43	- 41	- 36	- 32	- 29
105	-180	-169	-151	-135	-121
95	-358	-345	-312	-281	-253
85	-491	-486	-447	-409	-361
75	-558	-523	-488	-453	-417
65	-563	-514	-489	-458	-431
55	-558	-502	-480	-459	-438
45	-577	-516	-493	-467	-448
35	-615	-557	-526	-488	-462
25	-664	-601	-561	-514	-485
15	-708	-648	-600	-545	-509
6	-757	-700	-643	-581	-538

TABLE - V

Axial Load on Raker Column in kN, Dead + Wind Load Case

Case	Smooth Shell as per BS:4485	Shell with meridional ribs as per IASS			
		K 1.3	K 1.2	K 1.1	K 1.0
Max. Compn.	- 5548	- 5554	- 5356	- 5171	- 4981
Max. Tension	1621	1442	1370	1348	1305
% reduction over BS :4485					
Compression	-	-	3.6	7.3	11.4
Tension	-	12.4	18.3	20.3	24.2

TABLE - VI

Meridional Load on Tower Foundation in kN, Dead + Wind Case

Case	Smooth Shell as per BS:4485	Shell with meridional ribs as per IASS			
		K 1.3	K 1.2	K 1.1	K 1.0
Max. Compn.	- 9665	- 9212	- 8765	- 8184	- 8091
Max. Tension	2859	2362	2115	1800	1702
% reduction over BS:4485					
Compression	-	4.9	10.3	18.1	19.5
Tension	-	21.0	35.2	58.8	68.0

TABLE - VII

Quantities of Concrete and Steel Reinforcement in Shell

Material	Smooth Shell as per BS:4485	Shell with meridional ribs as per IASS			
		K 1.3	K 1.2	K 1.1	K 1.0
Min. shell Thickness (mm)	205	200	200	195	190
Concrete (cu.m)	6414	6306	6292	6184	6129
Reinforcement (M.T.)	510	490	472	452	438
% reduction over BS:4485					
Concrete	-	1.7	1.9	3.7	4.7
Reinforcement	-	4.1	8.1	12.8	16.4

2.3 Internal Suction

The draught and flow of air through the cooling tower creates an internal negative pressure or suction, and a value of 0.4 to 0.5 is usually considered in the design. The effect of the negative internal pressure results an increase in circumferential compressive forces to the extent of 40-50% of the forces due to wind, and corresponding reduction in the values of circumferential tensile force in the shell. The stress resultants in meridional direction are least affected. It may be prudent to consider the negative pressure for the purpose of calculating buckling safety, and ignore it for calculation of circumferential reinforcement in the shell.

2.4 Cooling Towers in Group

Where hyperbolic cooling towers are located in a group, the values of design wind pressures and pressure coefficients around circumference are much affected due to aero-dynamic interference effect depending on the spacing of towers or other structures of significant dimensions in the vicinity, and the angle of wind direction in relation to the axis of alignment of the towers. For such cases, in view of not many measured data being available on full-size towers, aero-elastic model testing in wind tunnel including all adjacent local topographical features, building and other structures is necessary although the test is valid for values of Reynolds number (Re) upto about 3 x 10^5 for laminar airflow as against Re of more than 10^8 in actual condition under turbulent wind flow.

Generally, a clear spacing of 0.5 times the base diameter is provided between the towers, and the wind pressures are enhanced between 10 and 40 percent when designing cooling towers in groups. For some of the Indian towers built in recent years, the design wind pressures are based on wind tunnel model test carried out at the Indian Institute of Science, Bangalore. The enhancement factors considered in some of the Indian Towers in groups are given in Table VIII.

TABLE - VIII

Sr. No.	Location	Basic wind pressure (kN/m^2) height	Enhancement Factor
1.	Wanakbori	1.5	1.33
2.	Neyveli Stage I	2.0	1.43
3.	Raichur	1.0	1.60
4	Kutch	1.5	1.35
5.	Panipat Stage III	1.5	1.50
6.	Kawas	1.47*	1.573

Note : * at 10 m height

3. DESIGN ASPECTS

3.1 Shell Thickness

The behaviour of hyperbolic cooling tower is quite different from that of a cantilever structure such as a chimney, in that maximum meridional tension in shell occurs at azimuth 0 deg. on the windward side and the maximum meridional compression occurs at azimuth 65-75 deg. from wind direction, following the same pattern as circumferential wind pressure distribution. Circumferentially, wind load produces compression and tension, and wind moments throughout. The magnitude of wind moments both in meridional and circumferential direction are quite small and are of little significance in the design.

The concrete shell thickness is generally governed by buckling consideration resulted by self weight and wind load, and a factor of safety of 5 is provided under service load condition. The buckling safety is calculated either by using equation derived by Der and Fidler for overall safety, based on wind tunnel tests, or alternatively by the inter-active formula developed as a result of experimental studies on local buckling by Kratzig, Zerna and Mungan at the University of Bochum, Germany.

The shell thickness is also governed by its tensile strength to avoid propagation of cracks in the tension zone, and for this reason, the tensile stress in concrete is limited to about 3.0 N/mm2. There is a close relation between a high wind load factor causing tensile failure and buckling safety factor of 5 as the latter leads to the choice of a reasonable wall thickness against tensile failure.

3.2 Shell Reinforcement

The shell reinforcement is usually governed by direct tension and bending moment acting on the section arising out of dead load + wind + temperature. The reinforcement is calculated on the basis of either factored loading of 1.4 for wind and 1.0 for dead weight at steel stresses limited to 87% of the yield stress of steel as per BS:4485, or in accordance with IS:456-1978 [9] by working stress method, but without considering 33% increase in permissible stresses in concrete and reinforcement, normally permitted under wind load case. It is found that the quantity of meridional reinforcement calculated by BS:4485 is generally greater than those by IS:456 by about 10%.

The shell reinforcement is very sensitive to wind loads, and Table IX shows how wind load factor drops rapidly with the increase in wind speed. For example, if a tower is designed for an under-estimated wind speed of 39 m/sec. and the shell is reinforced as per BS:4485, the wind load factor of 1.4 reduces to 1.0 if the wind speed increases to 46 m/sec. i. e. by 18%. Statistically it means that for a return period of wind of 50 years, the risk level increases from 0.63 to 0.97, or alternatively for a risk level of 0.63, the return period of wind reduces from 50 years to 14 years. This indicates that a proper assessment of wind speed is very much essential for the design.

TABLE - IX

Wind speed (m/sec)	Wind load Factor
39	1.400
40	1.331
42	1.207
44	1.100
46	1.006

3.3 Wind Induced Vibration

For large size cooling towers, the possibility of wind induced vibrations need to be investigated. The natural frequency is inversely proportional to the size, and it drops more rapidly due to increased shell thickness which is essential to provide the required factor of safety against buckling. For towers over 160 m height. the lowest natural frequency is generally below 1 Hz, and in such cases the design should take account of dynamical amplication factor for wind load based on aero-elastic model testing. To overcome this problem, it is found that by providing horizontal stiffening rings around shell, 4 or 5 in numbers along the tower height, the factor of safety against buckling could be provided without reducing the natural frequency. The ring stiffeners are located in the region of large buckling deformations of the unstiffened shell. The size of the rings is usually 5-6 times the shell thickness as the depth, and about 0.5m as the breadth. The shell around stiffening rings is designed for additional circumferential and meridional moments due to wind and temperature loading. Such towers with stiffening rings have already been built in Germany and the USA. Figure 5 shows the natural frequencies of a 165.5 m high tower of ISAR II nuclear power plant (Ref. [10]) in Germany, for both unstiffened shell and shell stiffened with 3 rings. It is seen that there is a marked improvement in the value of natural frequency with the ring stiffened shell. Figure 6 shows the mode shapes in buckling and vibration for the same tower.

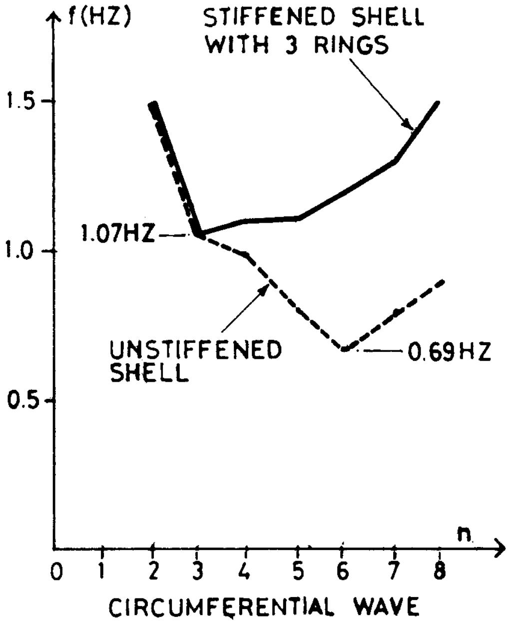

Figure 5: Natural Frequencies [10]

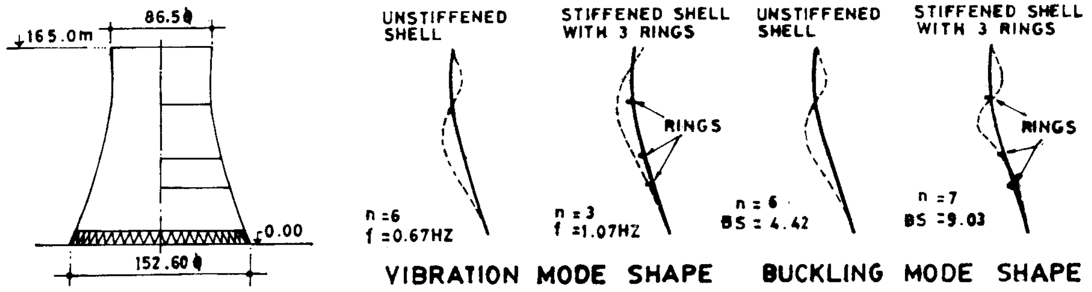

Figure 6: Vibration and Buckling Mode Shapes [10]

4. CONCLUSION

Cooling towers are undoubtedly one of the large civil engineering structures where wind forms the major applied loading in design. For analysing these structures, proper assessment of wind pressures and a clear understanding of the structural behaviour under asymmetric wind load are very much essential. The towers with increased roughness by providing meridional ribs, offer an economical solution, particularly in the high wind zones. The shell thickness should be based on its tensile strength against cracking due wind induced tension, in addition to satisfying the requirements for a high buckling safety. As the structure is sensitive to wind loads, shell reinforcement must be provided on the basis of limit-state approach. For large towers over 160 m height, shell stiffened with rings, offers a practical solution for problems of wind induced vibration. Evidence to-date indicates that there is yet ample scope for instrumentation of full-scale towers which may throw more light on the present knowledge of wind loads and structural behaviour of cooling towers.

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