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STRUCTURAL DESIGN ASPECTS OF HYPERBOLIC COOLING TOWERS

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SYNOPSIS

The present day hyperbolic cooling towers are exceptional structures in view of their sheer size and complexities. The towers involve considerable amount of design work on structural aspect. Besides providing suitable structural profile to meet the functional needs, the design requires consideration of external applied loadings, both static and dynamic. The Paper describes briefly salient structural features, and current practices adopted in the structural design of hyperbolic cooling towers.

1. INTRODUCTION

Over the past five decades, about 85 hyperbolic cooling towers have been built in the country at several thermal and nuclear power stations, and 12 more towers are under various stages of construction. The size of these towers has been on a steady increase from the very first 38m high towers at Sabarmati, built in 1934, (Fig.1) to the present time 141m high tower at Panipat which is the country's tallest, recently completed (Fig.2), and this trend for taller towers will continue to meet the growing demands of the industry.

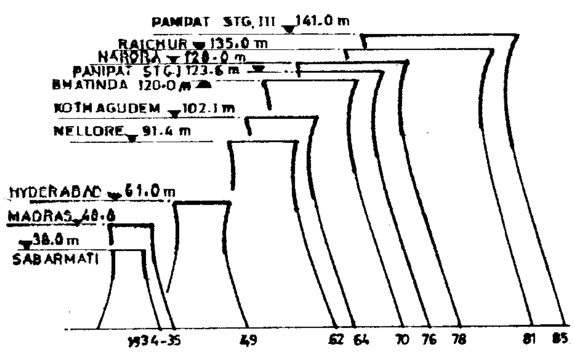

Figure 1 : Increase in Size of Hyperbolic Cooling Towers in India 1934-89.

These towers with very small shell thickness are exceptional structures by their sheer size and sensitiveness to horizontal loads. In this Paper an attempt has been made to describe salient structural features, and to review applied loadings, current design methods and specialised problems associated with the design of reinforced concrete hyperbolic cooling towers.

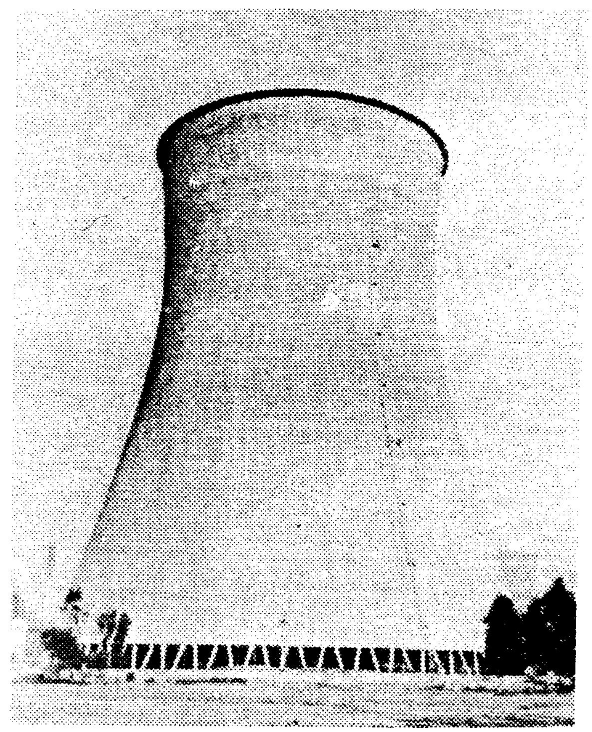

Figure 2 : 141m High Cooling Tower at Panipat Thermal Power Station Stage III, Tallest in the Country.

2. SALIENT FEATURE

The tower consists essentially of an outside hyperbolic shell, the principal function of which is to create a draught of air in a similar way to a chimney, an internal cooling fill at the bottom of the shell, a cold water basin into which the cold water falls from the fill and is stored for recirculation through the plant, and a water distribution system (Fig. 3). The shell diameter at base and throat, shell height and air-inlet height are governed by thermic design considerations. In regard to tower sizing for Indian climatic conditions, the design approach, i. e. the difference between the required cold water temperature and the wet bulb temperature, is only 4 deg. C to 5 deg. C as compared to an approach of 10 deg. C to 15 deg. C in the western countries. As a result, Indian towers are larger in size as compared to their western counterparts designed for the same quantum of heat removal.

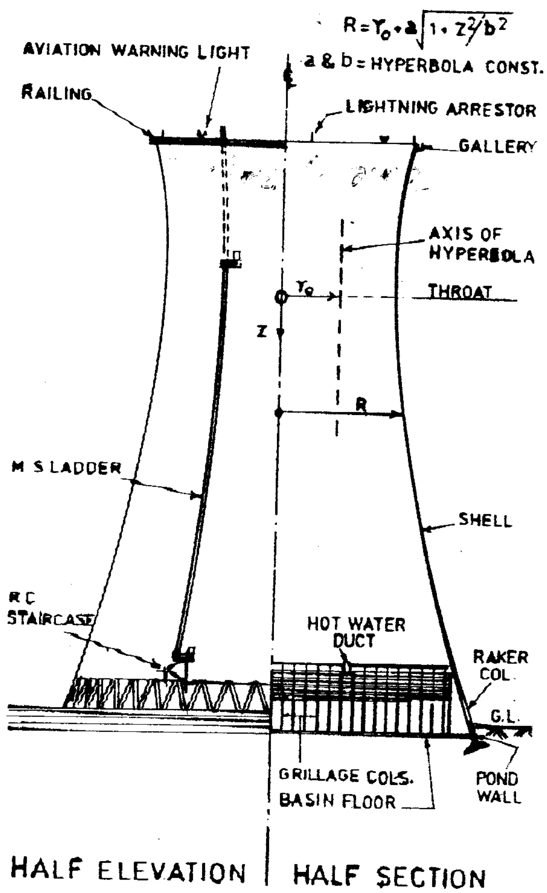

Figure 3 : General Arrangement

The ratio of tower height to base diameter varies between 1.48 and 1.15 depending on the tower height, the value of ratio decreasing with increasing tower height. The ratio of throat height from the top to overall height usually varies between 0.15 and 0.30. For the Indian towers built before 1970 when the tower height had just surpassed 100m, the shape of the meridian was based on a single hyperbola without any offset 'r_o' from the tower axis. An offset hyperbolic curve was used between the years 1970-1976 for the towers around 120m height. The towers built after 1976, the meridian is shaped by two offset hyperbolic curves - one below the throat and the other above. The offset distance 'r_o' influences not only stress distribution in the shell but also buckling load and natural frequency which are discussed later. As considered practical for construction, the slope at the shell bottom is usually limited to about 17 deg. from the vertical.

The shell is supported on RC Raker columns resting on pedestals which are integral with the peripheral wall of the cooled water basin. In order to make direct transference of shell loads, the raker columns and the basin wall are inclined in the same meridional plane of the shell. The tower foundation consists of a continuous ring foundation, resting either on soil or on raker piles where the bearing capacity of soil is poor.

The internal fill structure is independent of the outside shell tower, and the foundation for it is usually considered integral with the basin floor. As the internal fill structure comprises conventional column-beam framing with diagonal bracings, design aspects of this structure are not dealt in the Paper.

3. LOADS

The external applied loadings that affect cooling tower structure are :

(i) Wind forces
(ii) Earthquake forces
(iii) Temperature effect
(iv) Sun's radiation
(v) Soil pressure

3.1 Wind

3.1.1 Wind Pressures

Wind force forms the major external applied loading in the design of cooling towers, and it also provides the most common means of determining the degree of lateral strength required by the towers. Till the recent publication of the Indian Standard Code of Practice IS:875 (Part 3) -1987 in February 1989, the basic 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 which adopted wind pressure as static loads, the intensity of which varying with height and the zone at which the structure is located. The code gave basic pressure values of 100,150 and 200 kg/m^2 for three zones covering the country, starting with a uniform value upto 30m height and there above increasing in value with height upto 150m. The maximum pressures specified at 150m height for three wind zones were 138,207 and 276 kg/m^2 respectively. These pressures were based on a formula $P=0.006V^2$ where P is the pressure in kg/sq.m and V is the wind speed in km/h. This formula gave pressure value of about 25% more than those calculated by the theoretical pressure equation $P=1/2\rho v^2$, where ρ = density of air. This increase in pressure was to allow for the effect of topography, ground roughness, gust duration, etc.

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 10 m 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 K1, K2 and K3 related to probable life of structure, terrain, local topography and size of structure separately, and their combined effect is determined by multiplying the factors. The design wind pressure at height Z is given by : $P_z = 0.6\ (V_b .\ K1.\ K2.\ K3)^2$ in N/m^2, Where V_b is basic wind speed in m/sec.

The Indian 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, mainly due to large amount of computations 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. The gust factor depends on the natural frequency in the fundamental mode, wind speed and size of structure. In view of large size of the structure, the peak response occurring in a time interval of 1 hour duration is considered as appropriate for the design of cooling towers.

The Gust Factor method given in the new IS Code IS:875 (Part 3)-1987 appears to be applicable for regular shaped slender structures such as cubes, cylinders with hardly any taper, and not for hyperbolic shaped cooling towers. The German VGB guidelines and IASS recommendations on cooling towers incorporate gust factor when working out the design wind pressures. 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 have to be observed within the design concept.

3.1.2 Distribution of Wind Around the Shell

The circumferential distribution of wind around the shell at any height is usually defined by normalising values at equal angle increments from the windward direction, and is represented by a Fourier series, $H = \sum A_n \cos n\theta$. Table I shows the wind presure coefficients specified by BS 4485 : Part 4, Niemann, Sollenberger - Scanlan-Billington, and Zerna.

TABLE - I

Fourier Coefficients 'An' for Circumferential Distribution of Wind Around the Shell.

Harmonic	BS:4485 1975	Niemann 1971	Sollenberger et al,1980	Zerna 68
0	-0.00071	-0.3923	-0.2636	0.128056
1	0.24611	0.2602	0.3419	0.435430
2	0.62296	0.6024	0.5418	0.511731
3	0.48833	0.5046	0.3872	0.372272
4	0.10756	0.1064	0.0525	0.104642
5	-0.09579	-0.0948	-0.0771	-0.045549
6	-0.01142	-0.0186	-0.0039	-0.027082
7	0.04551	0.0468	0.0341	0.018113

Note : 1. BS:4485 includes 0.4 internal pressure.
2. Niemann and Sollenberger et al exclude internal pressure.
3. Zerna includes 0.5 internal pressure.

The pressure coefficients suggested by Zerna and Niemann have been extensively used for Indian towers till the British Standard BS:4485-1975 was published in September 1975. The Indian Code IS:11504 - 1985 for natural draught cooling towers specifies the same coefficients as in BS:4485. The coefficients by Zerna and Sollenberger et al are based on measurements on a full-scale hyperbolic cooling tower having small vertical ribs projecting outwards from the shell surface. As it is seen in Fig.4 the variation in the coefficients are due to surface roughness of the tower.

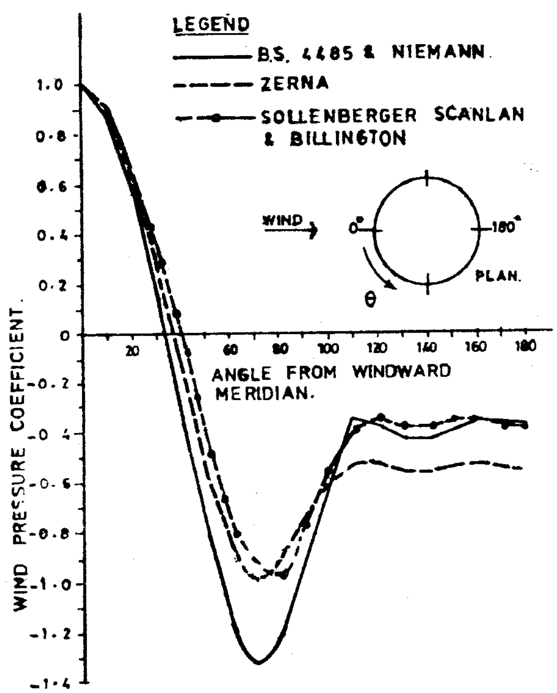

Fig. 4 : Wind Pressure Distribution Around Circumference of Shell (Excluding Internal Suction).

For the purpose of comparison, stress resultants for a cooling tower of the size given in Fig.5 are worked out for wind loads using wind pressure coefficients specified by BS:4485 and Zerna.

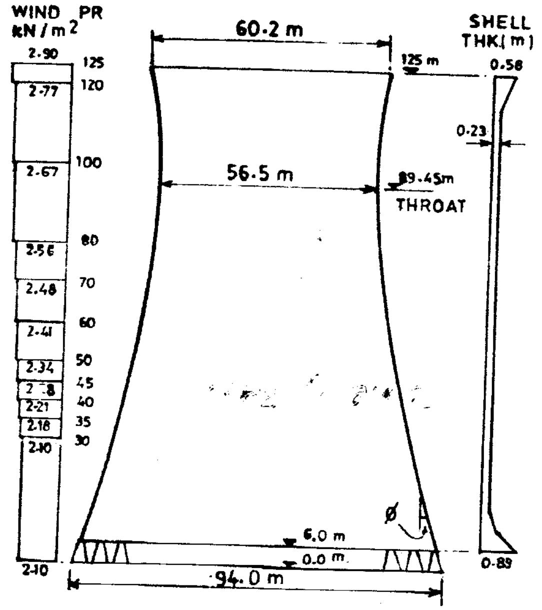

Figure 5 : Cooling Tower for the Purpose of Comparison.

In Figs. 6 and 7, it is seen that the stress resultants using cofficients of BS:4485 are higher than those of Zerna by as much as about 25 to 40% in the lower elevations which is due to smooth surface of shell considered in BS:4485 as compared to rough surface created by vertical ribs in the other case. The significance of this large variation in stress resultants has much effect on the requirement of shell thickness, reinforcement and tower foundation.

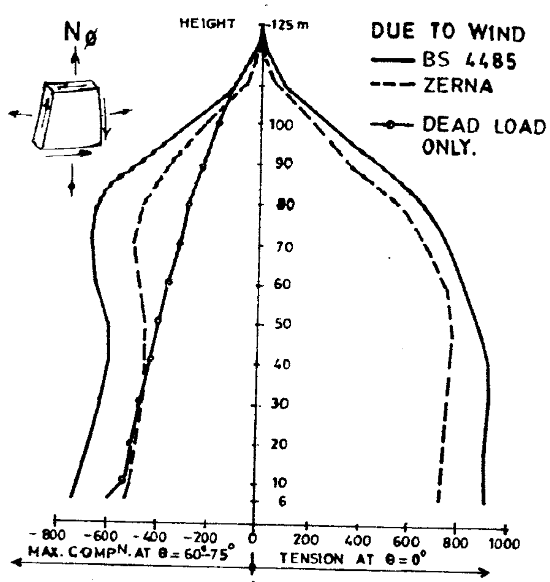

Figure 6:Meridional Stress Resultants in kN/m.

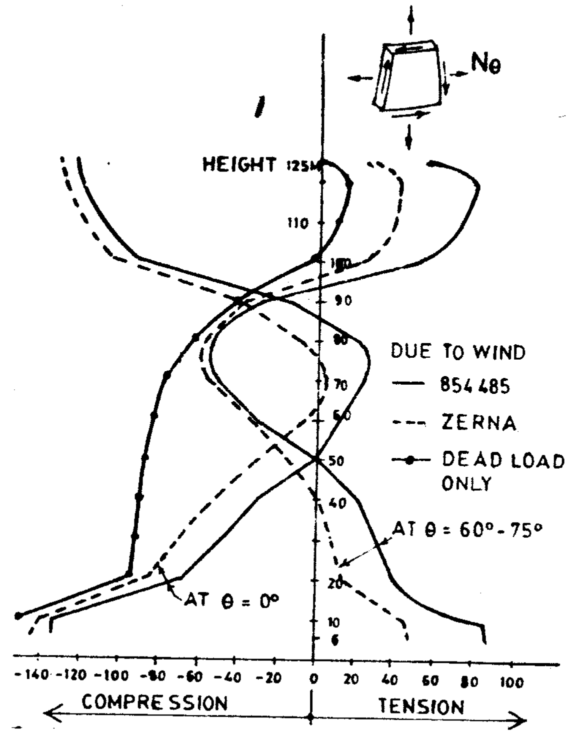

Figure 7 : Circumferential Stress Resultants in kN/m.

3.1.3 Internal Suction

The draught and the flow of air through the cooling tower create 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% of the forces due to wind, and corresponding reduction in the value of circumferential tensile forces 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.

3.1.4 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 3×10^5 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 tests 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 II.

TABLE - II

Sr. No.	Location	Basic wind pressure (kN/m^2) at 30 m 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

3.2. Earthquake

Earthquake force attracted by a structure is dynamic in nature and is governed by the ground motion and the properties of the structure itself. The basis for earthquake design in the country is IS:1893-1984 which gives two methods of finding earthquake forces on the structure, viz. Seismic Coefficient method and Response Spectrum method. The design seismic coefficient depends on several factors namely, soil-foundation system, importance factor, zonal factor, basic seismic coefficient depending on five differnt zones covering the country, and the spectral acceleration in the case of Response Spectral method which depends on natural periods of vibration of the structure. A critical damping factor of 5% is considered for concrete structures.

An earthquake ground motion can be represented by three orthogonal components, two horizontal and one vertical. It is normally sufficient to design the tower for only one horizontal component of the earthquake under which circular cross section of the shell deforms in the first circumferential mode only. The natural frequency corresponding to the lowest mode is generally in the range of 2 to 3 Hz. For statical method of analysis, the most severe design horizontal seismic coefficient, i.e. in zone V of the country, for vibration in the lowest mode does not exceed 0.12. In most cases, it is sufficient to make an approximate earthquake analysis through discretization of the shell into a beam model. If a more rigorous analysis by Response Spectrum method is adopted, the maximum response for each mode is considered, and the combined response is calculated by the square root of the sum of the squares of the values from the contributing modes. It is found in practice that wind loads usually affect the design of cooling towers more than the earthquake forces.

3.3 Temperature Effect

The temperature difference between the inside and outside faces of Indian cooling towers are usually of the order of 10 deg. C and my go upto 20 deg. C in the extreme circumstances which are relatively low as compared to the conditions in Europe and the USA. The temperature gradient in the shell as calculated by using the same methods as for industrial chimneys, works to about 5 deg. C, and in the extreme circumstances metioned above to 10 deg. C. It is found that these temperature gradients do not cause excessive tensile stress in the shell, but only increase meridional reinforcement in shell by about 10% than those calculated purely for self weight + wind load case.

3.4 Sun's Radiation

The effect of sun's radiation is to produce stresses in shell opposite to those arising from thermal gradients i. e., the internal cold face is subjected to tension. It is found that for the case when one side of the tower is exposed to the sun and the other in shade, with a temperature differnetial of 5 deg. C, the stresses in shell are quite small.

3.5 Soil Pressure

The cold water basin which is integral with pedestals supporting raker column is usually constructed below the finished grade level. The basin wall is designed for pressure due to retained soil for a case when the basin is empty. At some sites, where the ground-water table is high, the design requires consideration of additional pressures due to ground water on basin walls and uplift pressure on tower foundation. If vehicular movement close to cooling tower is expected to take place, soil pressure due to surcharge from the vehicle weight need to be considered in the design.

4. DESIGN ASPECTS

4.1 Structural Analysis

The analysis of these towers is an interesting challenge to any structural engineer in view of their shape and large size combined with non-axisymmetric horizontal loads. Fair share of research effort by mathematicians and practising engineers have been done over the last two decades, particularly after the failure of Ferrybridge cooling towers in UK in 1965. During the period upto 1972, most towers around the world were analysed based upon membrane theory only. It has been found that membrane analysis for shell is satisfactory for the purpose of design in most cases provided the shells are suitably thickened at bottom and top levels to account for local bending created by edge effect.

With the rapid development of computers and finite element method, analysis of shells for axisymmetric dead load and non-axisymmetric wind load can be carried out on the assumption that the reinforced concrete shell is isotropic, homogeneous and without cracks. The advantage of FE method is that the structural modelling can include raker columns basin walls, pedestals, tower foundation and soil-structure interaction. Notwithstanding the merit of sophisticated FE method, it must be said that the accuracy of the analysis by this method depends on the structural modelling of the tower, particularly requiring smaller size of the elements at lower and top elevations. It is found that stress resultant values for wind load as calculated by FE method using larger size elements, usually 5-10 m high, are less by about 10-15% of those calculated by membrane theory using 1.4m high elements.

The behaviour of natural draught 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, but the maximum meridional compression occurs at azimuths 65 deg. to 75 deg. from wind direction, and not at 180 deg. The meridional tensile force in shell due to wind in the upper elevation of the tower is usually balanced by compressive force arising from self weight. Circumferentially, self-weight produces tension above throat and compression below, and wind load causes moments throughout. The meridional shell moments are generally of the order of $\pm 0.0015\ pR^2$ where p is the wind pressure and R is the radius of the shell at the level being considered. Similarly, the magnitude of circumferential moment is about $\pm 0.005\ pR^2$. These moments are however of little significance in the design of shell.

4.2 Form of Shell Meridian

The shape of the shell meridian is of considerable significance in the value of stress resultants in the shell and in arriving at an economical tower. For the purpose of comparison, stress resultants are calculated for a similar tower to those given in Fig.5, but with a higher and lower throat depth from the top than shown. It is seen from Table III, the tower with lower throat depth is found to be more favourable than the other two types.

TABLE - III

Effect of Meridional Shape. Comparison of Meridional Stress Resultants due to wind, in kN/m.

Level	Tension $\theta = 0^\circ$ Type A	Type B	Type C	Max. compresion Type A	Type B	Type C
125	0	0	0	0	0	0
123	1	2	2	- 1	- 2	- 2
113	58	58	58	- 64	- 64	- 64
103	197	198	199	- 218	- 220	- 220
93	399	407	414	- 431	- 444	- 454
83	569	599	626	- 567	- 610	- 651
73	681	726	768	- 613	- 670	- 726
63	768	814	858	- 613	- 671	- 732
54	838	879	920	- 586	- 626	- 672
44	893	918	949	- 587	- 592	- 604
34	933	933	943	- 633	- 614	- 602
25	966	931	908	- 696	- 655	- 627
16	997	923	862	- 762	- 696	- 646
6	1037	927	829	- 838	- 746	- 673
Buckling safety factor (min)				6.519	6.025	5.643
Concrete quantity (cu.m)				6,773	6,727	6,686
Shell Reinforcement (T)				532	526	523

Note : 1. Throat depths from top :
Type A : 33.6 m
Type B : 35.55 m as in Fig. 5
Type C : 37.50 m
2. All other dimensions as in Fig. 5
3. Fourier Coefficents : as per BS:4485

4.3 Buckling of Shell

The concrete shell thickness is generally governed by buckling considerations 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 which is specified in BS:4485-Part 4, or alternatively by the inter-active formula developed as a result of experimental studies on local buckling by Kratzig, Zernä and Mungan at the University of Bochum, West Germany, which is specified in the German VGB guidelines and IASS recommendations on cooling towers.

It is also necessary to see that the tensile stress in concrete shell is limited to about 3.0 N/mm2 in order to consider the shell as an uncracked section which is the basis of the shell analysis.

4.4 Geometrical Imperfection

The collapse of a cooling tower at Ardeer in Scotland in 1973 due to geometrical imperfections in the construction of the shell, supplemented by vertical cracking of shell arising from thermal gradients, evoked considerable interest among researchers and engineers about the importance of construction tolerances and thermal cracking in shell. The investigation of the Ardeer failure showed that effects of imperfections induced tensile hoop membrane stresses and meridional bending which resulted yielding of circumferential steel in view of its low percentage provided, and it was aggravated by the existing meridional cracks, probably of thermal origin, which lead to inequilibrium and inevitable failure. The permissible tolerances in construction as specified in BS:4485 and IASS recommendations are given below :

BS:4485 :

(i)	Horizontally	: $\pm$ 15 mm measured on a chord of 3 m
(ii)	In the meridional plane	: $\pm$ 10 mm rotation measured over a height of 1 m.
(iii)	Thickness	: - 5 mm to + 10 mm.
(iv)	C.L. of base of shell considered radially: $\pm$ 40 mm.	

IASS :

(i)	Maximum error in the slope	: 1.5%
(ii)	Maximum error in the radius	: $\sqrt{Rt}/47.1$ and not more than 0.10m, where R and t being local value of minimum radius and shell thickness in metres.

4.5 Differential settlement

Where there are large irregular soil conditions around tower foundation, the effect of differential settlement need be considered. such settlements cause overloading on raker columns in the zone affected by the soil irregularities, but the shell is generally least affected in view of its rigidity in the meridional plane.

Where soil condition is irregular and also having poor bearing pressures, lower than about 100 kN/m2 (net), it would be prudent to provide pile foundation in such circumstances.

4.6 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 loadings 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, Part 4, or in accordance with IS:456-1978 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 shells are usually reinforced with two layers of high-yield deformed steel bars with a minimum percentage of 0.3% in both directions. Circumferentially, nominal reinforcement is adequate in most cases. With a minimum cover of 40 mm to reinforcement and two layers of steel, the minimum thickness of shell for practical considerations works to about 175 mm. In the upper elevations of the shell, the minimum percentage of meridional steel is adequate in most cases, but in the lower elevation additional meridional steel is usually required, the quantity of which depends on the tower profile and the wind loading.

The shell reinforcement is very sensitive to wind loads, and Table IV 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%. This indicates that a proper assessment of wind speed is very much essential for the design.

TABLE - IV

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

4.7 Effect of 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 160m 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, 4 or 5 in numbers at equidistant spacing along the tower height, factor of safety against buckling could be provided without reducing the natural frequency. Such towers with stiffening rings have already been built in Germany and the USA.

4.8 Non-linear Behaviour

Failure of reinforced concrete cooling towers may be initiated by rapid propogation of cracks in tension zones, and finally by yielding of the reinforcement. As this stage is reached, the wind load can no more be increased whereas the membrane force acting on the cracked section drops abruptly. For an optimal design, the amount of reinforcement in the shell wall should be related to the tensile strength of the concrete section. Upto this range, the behaviour of the cooling tower shell is nearly linear. For this reason, it seems to be adequate to carry out an analysis according to the linear elastic shell theory, as being done in the design practice at present. There is also a close relation between a high wind load factor causing tensile failure and buckling safety factor of 5 normally considered in the design as the latter leads to the choice of a reasonable wall thickness against buckling failure.

4.9 Substructure

The raker columns are designed for compressive and tensile loads arising out of dead and wind loads, and the basis of design is IS:456-1978. The setting of the raker columns is a complicated geometrical configuration in order that the inclined plane of the columns is same as the meridional plane of the shell (Fig.8).

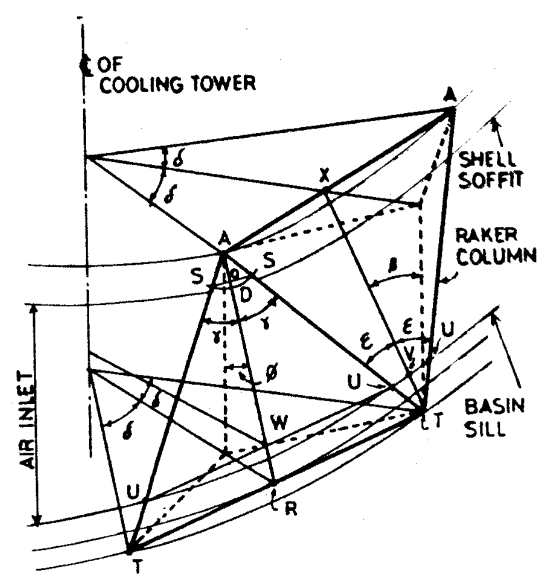

Figure 8 : Schematic view of Raker Columns

The basin wall which is integral with the thickened pedestals under raker columns is designed as an uncracked section as per IS:3370 for pressures due to water and retained soil. The size of tower foundation is based on dead loads from self weight of shell, raker columns, pedestals, basin wall, earth and water load on foundation, and wind compression or tension. Where ground-water table is high, additional uplift forces due to sub-soil water pressure need to be considered in the design of foundations, and in certain cases the thickness of the tower shell need to be increased to reduce the uplift forces. No uplift of foundation or tension in piles is allowed. The underside of the tower foundation is kept normal to the shell meridion, and in case of pile foundation,the rake of the piles is usually different from the shell meridion which results in unbalanced forces at the pile cap level, and these forces are resisted by passive resistance of soil and the friction of any mass concrete fill tieing the pile cap against soil.

4.10 Construction Materials

The grades of concrete generally used are M-25 for shell, M-35 for raker columns, and M-20 for the remaining R.C.Works. High yield deformed bars are used, with staggered laps in shell reinforcement. The internal surfaces of shell, raker columns and basin are usually painted with three coats of bituminous paint for durability.

5. CONCLUSION

Cooling towers are undoubtedly exceptional structures which require special expertise both to design and construct. Meridional form of the shell and proper assessment of wind loads are of considerable importance in arriving at stress resultant values and buckling safety factors. As the structure is sensitive to wind loads, shell reinforcement must be provided on the basis of limit state approach. In recent years lot of research has been carried out in regard to new design methods for the inclusion of non-linear material behaviour and cracking in computation for ultimate load, buckling and dynamic response. Evidence to-date indicates that there is yet ample scope for instrumentation of full-scale structures which may establish confidence in simplified methods of analysis as a basis of design.

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