



Wind induced fatigue analysis of Lotus Tower Mast

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ABSTRACT

The design of the Lotus Tower in Sri Lanka, which is going to be the tallest tower in South Asia includes a slender concrete and steel antenna mast, approximately 88m in height. The design of the mast was mainly governed by wind-induced fatigue. A detailed fatigue analysis was carried out according to the requirements of AS4100. It was found that although the fatigue design actions are below the allowable limits, a strict inspection routine had to be implemented for possible fatigue cracks in the future. Investigations were also conducted to check whether this mast could be prone to significant wind-induced loading and response in both the along-wind and cross-wind directions. It was concluded that the predicted cross-wind actions generated primarily by vortex shedding, are significant and the possibility of fatigue damage exists if the fluctuating bending moments in the steel sections of the mast correspond to significant stress amplitudes.

1. Introduction

When completed in mid-2018, the Lotus tower in Sri Lanka will be the tallest tower in South Asia. The design of the Lotus Tower includes a slender concrete and steel antenna mast, approximately 88 m tall, extending from a level of approximately 262 m to 350m above ground level. The mast is designed for a design life of 50 years. During that time, the members are subjected to a very large number of cycles of fluctuating wind loads. Therefore the design of the mast was mainly governed by wind-induced fatigue. A detailed fatigue analysis was carried out by the authors according to the requirements of the Australian Steel Structures Code, AS4100. The details of the wind-induced fatigue analysis procedure are presented in this paper.

Investigations were also conducted to check whether this mast, treated as a separate structure, may be prone to significant wind-induced loading and response in both the along-wind direction (due to buffeting by atmospheric turbulence), and in the cross-wind direction (produced by vortex shedding or galloping). Long term reliable wind data is not available for Sri Lanka. Therefore, Along-wind response was calculated by the 'Equivalent static wind load' (ESWL) approach. The cross-wind response due to forces produced by vortex shedding was estimated using the procedure recommended in ESDU 90036. This allows calculation of vortex-induced cross-wind response for towers or masts with constant cross sections with sharp edges, including square cross sections, and polygonal shapes up to eight sides. For step-sided structures, such as the Lotus Tower's mast, ESDU 90036 suggests an approach in which 'the response is obtained by summing the variances of the response due to each parallel-sided section of the structure considered in isolation'. The latter approach was adopted for the Lotus mast. The results from this study will also be presented.

2. Description of the Structure

An elevation of the mast and recent photos of the Lotus Tower with the mast are given in Figure 1. The four sections of the mast (Figure 1a and Table 1) have square, octagonal, square and square cross-sections, respectively. Dimensions and other details of the mast are given in Table 1.

The dynamic analysis was conducted ignoring the effects of the building vibration. However a special study was conducted later to investigate the effects of the interaction between the mast and the building. These effects were found to be insignificant. Therefore only the isolated mast was considered in the fatigue analysis. First natural frequency of the mast was found to be 0.78 Hz.

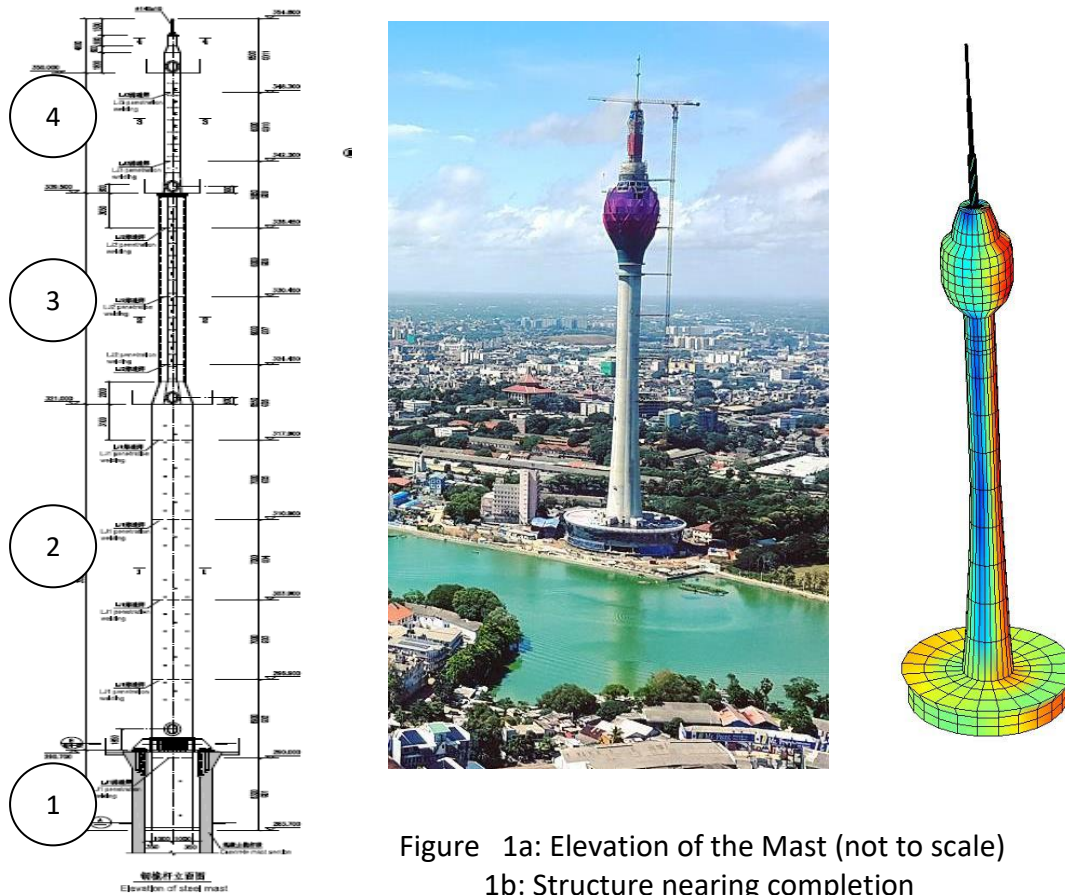


Figure 1a: Elevation of the Mast (not to scale)
 1b: Structure nearing completion
 1c: FEM model of the Tower

Table 1. Dimensions and other details of the mast

Section (as per Figure 1a)	Material	Shape	Top height (m)	Bottom height (m)	Length (m)	Width (m)	Mass/unit height (Kg/m)
1	Reinforced concrete	square	290.7	264.6	26.1	3.8	15576
2	Steel	square	321.0	290.7	30.3	2.0	2446
3	Steel	octagonal	339.5	323.0	16.5	1.2	1210
4	Steel	square	350.0	339.5	10.5	0.65	580

3. Wind-Induced Fatigue Analysis

The wind forces acting on wind sensitive structures depend on wind speed, terrain, dynamic response of the structure and cross-wind effects. The wind-induced fatigue occurs as a result of the along-wind and cross-wind response of the structure, and it governs normally in masts on top of towers (Mendis and Dean, 2000). The cross-wind response is mainly caused by vortex shedding and is a major problem for long slender solid towers or elements on towers (Mendis et al., 2007).

A detailed fatigue analysis was carried out according to the requirements of the Australian Steel Structures Code, AS4100-1998. The fatigue design rules in AS4100 are derived from the ECCS (1985) recommendations. Detail category (f_{rn}) is the designation given to a particular detail to indicate which of the S (Stress range)-N (Number of stress cycles) curves is to be used in the fatigue assessment. The detail category takes into consideration the local stress concentrations at the detail, the size and shape of the maximum acceptable discontinuity, the loading condition, metallurgical effects, residual stresses, the welding process and any post weld improvement. The detail category number (f_{rn}) is defined by the fatigue strength at 2×10^6 cycles on the S-N curve. S-N curves for normal stress are given in Figure 2. In this figure, the constant stress range fatigue limit (f_3) is the highest constant stress range for each detail category at which fatigue cracks are not expected to propagate. The cut-off limit (f_5) is the highest variable stress range for each detail category which does not require consideration when carrying out a fatigue analysis. The detail category was selected and the stress due to maximum wind speed was calculated. Fatigue design criteria are satisfied if the stress due to maximum wind speed was less than the constant stress range fatigue limit. A capacity reduction factor for thickness (for more than the 25 mm thick sections) and a factor for non-redundant paths (0.7) was applied according to Clause 11.1.6 of AS 4110-1998.

The three detail classifications used were, (i) Category 140 to consider stress ranges in the original plates to check stress concentrations at holes etc; (ii) Category 71 to consider stress ranges in direct transverse butt welds ($t > 8$ mm) (iii) Category 50 to consider stress ranges in butt welds to intermediate plates. The three allowable (constant stress) stress ranges obtained from Figure 11.6.1 of AS 4100-1998 are: (i) Category 140: $103 \times 0.623 = 64.2$ MPa; (ii) Category 71: $52 \times 0.623 = 32.4$ MPa; Category 50: $37 \times 0.623 = 23.1$ MPa.

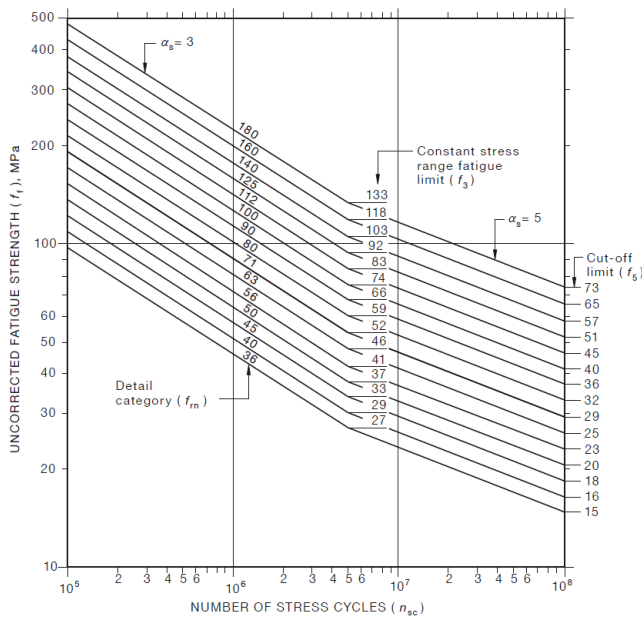


Figure 2: S-N Curves (AS4100-1998)

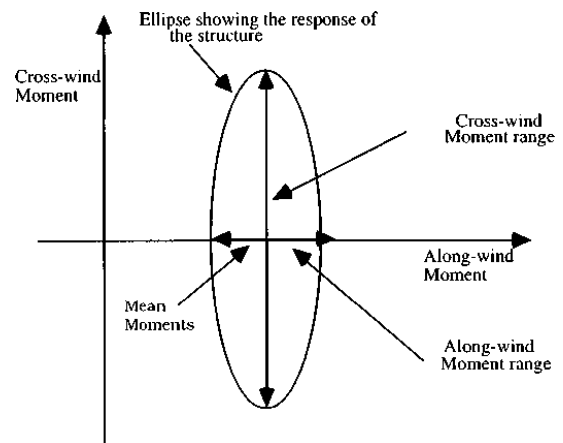


Figure 3: The response of the structure for a particular wind direction

The along-wind stress range is calculated for a 50 year return period mean hourly wind speed (at 10 m height) of 24.5 m/s; this corresponds to the 50 year 3s gust speed of 38 m/s derived by CSEC for structures in Colombo, Sri Lanka.

The maximum stress (f_{max}) and semi-variable stress range can be estimated by calculating a gust factor. It was calculated to be approximately 1.5. Then $f_{max} = f_{mean} (1+G) = f_{mean} (1.5)$ and the semi-variable stress is $0.5 f_{mean}$, meaning that the variable stress range (f_{var}) is numerically equal to f_{mean} . Then the stress range for the two critical sections were calculated to be 17.91 MPa (2 m rectangular at 291 m) and 14.27 MPa (1.2 octagonal at 323 m). These are within the allowed ranges.

The cross-wind response is obtained from the Holmes (2016) report. This gives the response as per ESDU 90036 (1993) as well as EN 1991-1-4 (2003). It should be noted that neither method gives a direct solution for a stepped mast, and approximations have been used in the results presented here. The critical tip deflection is 276 mm as per ESDU 90036 (1993) but only half that (i.e. 138 mm) as per EN 1991-1-4 (2003). EN 1991-1-4 (2003) result is used since the euro codes are traditionally used in Sri Lanka; it is also a more recent than the ESDU 90036 (1993) code. These results are given in the next section. Mode shape analysis is used to convert the deflection to obtain the stresses at the critical sections. This will give rise to bending stresses of 12.53 MPa (at 291 m) and 12.81 MPa (at 323 m). The stress range will in fact be double this – i.e. 25.06 MPa (at 291 m) and 25.62 MPa (at 323 m). These are within the allowed range.

4. Wind-Induced Response

4.1 Along-wind response

The methodology used for the calculation of along-wind response was the ‘Equivalent static wind load’ (ESWL) approach (Holmes, 1996; 2015). This method separately calculates equivalent static distributions of wind load, as a function of height, for: i) the mean or average wind load, ii) the ‘background’ quasi-static loading produced by turbulent gusting, and iii) the resonant dynamic loading resulting from the vibration of the structure. The latter is assumed (to a good approximation for low damping) as inertial loading – i.e. mass times acceleration at each height section considered. The load distributions are then combined into a single combined distribution. Different ESWL distributions are obtained for load effects (e.g. shear force or bending moments) at various height levels on the structure.

The ESWL approach for the along-wind response was applied to determine the bending moments at the bottom of each stepped section – i.e. at 264.6 m - 22.8 MNm, at 290.7 m -8.27 MNm, at 323.0 m -1.17 MNm, and at 339.5 m - 0.163 MNm.

4.2 Cross-wind response

The cross-wind response due to forces produced by vortex shedding was estimated using Data Item 90036 of ESDU International (1993). This allows calculation of vortex-induced cross-wind response for towers or masts with constant cross sections with sharp edges, including square cross sections, and polygonal shapes up to eight sides. For step-sided structures, like the Lotus mast, Section 4.3 of ESDU 90036 suggest an approach in which ‘the response is obtained by summing the variances of the response due to each parallel-sided section of the structure considered in isolation’.

The latter approach was adopted for the Lotus mast. Thus, the response was assumed to be composed of a combination of the response of an RC mast 26.1 m long, 3.8 m wide, with a square section, with that of a steel mast 56.4 m long, 2.0 m wide with a square section, a steel mast 74.9 m long, 1.2 m wide with an octagonal section, and a steel mast 85.4 m long, 0.65 m wide with a square section. The calculated peak deflections for these ‘notional’ masts, as a function of mean wind speed, were adjusted



Figure 4: Wind Tunnel Tests prepared for the Lotus Tower

to the top of the stepped mast, using correction factors for the generalized force and generalized mass based on the simple sinusoidal model of vortex-induced response

As a check, calculations of cross-wind response have also been made using ‘Method 2’ in the Eurocode for Wind Actions (2003). The background to this method with examples of the application were provided by Hansen (2007). This method is simpler to use as it gives a ‘closed-form’ solution, whereas the ESDU approach requires iterations to obtain a solution. However, the Eurocode method is more limited in its scope – for example, the response of the structure at ‘off-critical’ wind speeds is not calculated, and there is no indication of how to calculate the response of towers or masts with ‘stepped’ cross sections.

As a general comment, it should be noted that most developments of calculation methods for prediction of vortex-induced cross-wind response has taken place for chimneys and masts of circular cross sections. Although there has been a considerable amount of work done for tall buildings of square and rectangular cross sections, these are typically of relatively low height/width aspect ratio, and in city-centre locations with high turbulence intensities. Hence,

neither the ESDU nor the Eurocode methods are very well calibrated against experimental data, for slender towers or masts of square cross sections in low turbulence flow. Therefore, great accuracy should not be expected for either the ESDU or Eurocode methods for this mast, with the stepped cross sections adding an extra layer of complexity.

Table 2. Comparison of peak cross-wind deflections at top of mast

Mean wind speed at 350m (m/s)	Main excitation height range (m)	Cross-section	Peak deflection (mm) ESDU 90036	Peak deflection (mm) EN 1991-1-4
28.3	0 - 26.1	3.8 m square, RC	56	84
12.7	26.1 - 56.4	2.0 m square, steel	276	138
5.9	56.4 - 74.9	1.2 m-oct, steel	25	0.9
4.1	74.9 - 85.4	0.65 m, sq, steel	15	1.4

The peak bending moment at the base of the mast (i.e. at 264.6 m height) can be estimated by assuming an inertial loading distribution following the mode shape (Figure 2). The peak accelerations at the top of the mast were obtained from the maximum deflections shown in Table 2.

The maximum cross-wind base bending moment predicted by the ESDU 90036 method, corresponding to the peak deflection of 0.276 m is 14.4 MNm – about two-thirds of the predicted along-wind design bending moment at that height shown in Table 2. The Eurocode predictions give a lower value of 7.2 MNm.

5. Wind Tunnel Tests

Wind tests were conducted by China Academy of Building Research (CABR) in Beijing (Figure 4). The 1: 300 scale model test was carried out in CABR wind tunnel. This wind tunnel is an open-circuit wind tunnel with total length of 96.5 m and configured with two test sections. The test was carried out in the high-speed section with 4 m (W) * 3 m (H) * 22 m (L). The maximum wind speed is 30m/s. The maximum one direction cross-wind displacement amplitudes of the tower top and the mast top obtained is 283 mm, which is closer to the ESDU 90036 value of 276 mm.

6. Conclusions

1. Wind-induced fatigue governed the design of this mast. The paper presents a rational method for wind-induced fatigue analysis of a mast with a complicated shape. Generally, very little research work has been done to improve the knowledge on wind-induced fatigue damage.
2. Neither the ESDU nor the Eurocode methods are very well calibrated against experimental data, for slender towers or masts of square cross section in low turbulence flow. Further, stepped cross sections similar to the Lotus tower are not covered in these methods. More complex shapes are now introduced to masts, and there is an urgency to develop appropriate methods for these types of masts.
3. As mentioned before, detailed fatigue calculations are required when the stresses are higher than either the constant stress range limit or the cut-off limit. Use of Miner's rule for cumulative damage calculations is a tedious process. More reliable wind data, etc. are required to conduct this type of analysis.
4. As a result, although the fatigue design actions are just below the allowable limits, a strict inspection routine has to be implemented for possible fatigue cracks in the future.

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