Investigation of Lamp Damage in High-Mast Illumination Towers

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The present study was initiated to investigate the possible causes of excessive lamp failure rates on high-mast illumination towers in the state of Minnesota. This type of highway illumination system consists of high masts on which there are either three or four luminaires. The masts are 120 ft high and the luminaires are attached to a ring assembly that can be lowered for lamp replacement. The study consisted of two methods of analysis that were combined for obtaining information on the motion of the towers and the luminaires. The first step was to analyze numerically the complete system by employing a finite element code and to compute the motions of the complete system under typical wind conditions at the site (wind velocities between 5 and 60 mph). The numerical results provided information about lamp accelerations and the frequencies of motion. The second phase of the study consisted of an experimental effort during which one of the illumination towers was instrumented and motion data were collected for various wind conditions. The final phase of the study included the evaluation of the numerical as well as the experimental data that had been obtained from the preceding steps. This evaluation included the identification of modes of vibration in the frequency domain, filtering of data for assessment of modal effects, and comparisons of experimental and numerical results.

This paper is based on the results of a recent study performed at the University of Minnesota for the Minnesota Department of Transportation (MN/DOT) concerning excessive failure rates of high-pressure sodium lamps installed on high-mast towers (*1*). These failures consisted of fractured arc-tubes, broken arc-tube bases, and severely deformed support wires. Preliminary reviews of the issue by MN/DOT personnel determined that such failures were not caused by electrical or electromagnetic problems and that the issue of wind-induced vibrations needed to be investigated.

This study consisted of an analytical-numerical phase, an experimental phase, and a comparison and evaluation phase. The analytical-numerical part included implementation of the finite element method for the assessment of tower behavior under various static and dynamic loading conditions. In the experimental phase one of the towers was instrumented with 12 single-degree-of-freedom (SDOF) accelerometers that were placed on a tower ring assembly, on a luminaire, and on a lamp, and the recorded data were studied in both the time and frequency domains. The numerical and experimental results were compared at various stages of this study so as to obtain an accurate description of the wind-induced vibrations and the relationship with the observed bulb failures.

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Structural response to dynamic loads can be evaluated by employing either classical methods in the linear domain, or, when complex structures are considered, numerical techniques may have to be used for the analysis. For structural analysis in the nonlinear domain of behavior, a reliable numerical approach is almost always required for obtaining acceptable solutions that can be interpreted for engineering purposes. Newark and Rosenblueth (*2*) provide a substantial amount of information on dynamic structural analysis in the linear and nonlinear domains of behavior, but this work is primarily oriented toward addressing the problem of earthquake effects. Nevertheless, the analytical approach presented does apply to a wide range of structural dynamics problems. Also, there are similarities between the structural response of systems under the effects of dynamic loads transmitted through the ground and the behavior of structures under wind-induced loads as discussed by Cevallos-Candau and Hall (*3*).

A fundamental treatment of wind engineering is presented elsewhere (*4, 5*), and this information can be employed for the analysis of structural systems under the effects of wind loads. These texts also provide an abundance of information concerning wind forcing functions, drag coefficients, and vortex shedding effects. The issue of loading conditions associated with air flow has been studied quite extensively, and the information available from the literature was used in this study. Sachs (*4*) and Simiu and Scanlan (*5*) also contain several typical wind records with durations of about 2 min, and such instantaneous records were an essential part of the pretest numerical approach in which simulated wind loads had to be employed for assessing structural responses.

Cheung and Chiu (*6*) provided information concerning probabilistic determination of extreme winds from short-duration records, and other publications (*7–12*) contain relevant information on wind forces and corresponding structural response for various systems, primarily on towerlike structures, which are similar to the high-mast system under consideration.

There are many excellent references available on the finite element approach. Probably the most valuable for this study was written by Bathe (*13*) who also developed the finite element code ADINA (*14*) used to analyze the high-mast system. Other texts, such as that by Clough and Penzien (*15*), also provide relevant information and examples on implementation of the finite element method for dynamic analyses. Instrumentation and testing of structures and mechanical components have become increasingly popular and easy with the advent of personal computers and advanced software packages.

The specific towers dealt with on this project measure 120 ft high and taper in diameter from the base to the top. The average base diameter is approximately 24 in. and the average top diameter is approximately 7 in., as shown in Figure 1. The tower is actually composed of three sections of cold-formed

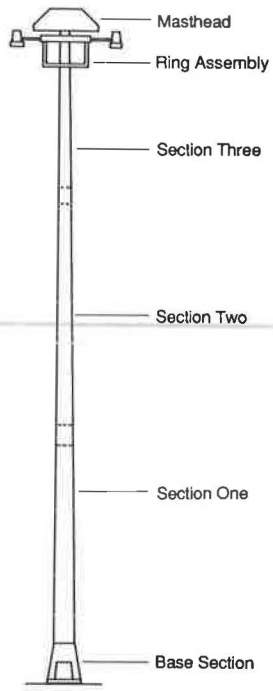

FIGURE 1 High-mast tower.

steel plate. Each section is approximately 40 ft high and is formed into a 16-sided cylinder fitted together to form the mast. All three tower sections have different plate thicknesses: the lower section is 5/16 in. thick, the middle section is 1/4 in. thick, and the top section is 3/16 in. thick. The material is high-strength, low-alloy steel per ASTM A588 with a 50,000-psi minimum yield strength and a 70,000-psi minimum tensile strength. The approximate weight of the shaft is 7,200 lb, and the section modulus at the base is nearly 190 in.^{3}, which results in a 100 percent yield moment of 785,000 ft-lb. At the base of the tower there is a 36-in.-high tube section made from folded plate that tapers from 39 in. in diameter at the bottom to 24 in. at the top. The top of this section is connected to the bottom of the lower section of the shaft with 100 percent penetration welds. This base section also has a door for access to the electrical connections and a steel cable winch for lowering the ring assembly and luminaires. The largest ring of the assembly is a 2.5-in. steel pipe that is cold formed into a 47.25-in.-diameter ring. Four smaller steel pipes extend from the ring and the luminaires are mounted on these smaller pipes. Each luminaire weighs approximately 65 lb and houses one 1000-watt high-pressure sodium bulb. These bulbs are cylindrical in shape, about 13 in. long, and 2.75 in. in diameter. When the winch at the tower base is unwound the ring assembly disengages from the masthead and is slowly lowered. The masthead assembly at the top of the tower houses several pulley mechanisms that thread the electrical and steel cables as the ring is raised or lowered. When the ring assembly is raised to its standard uppermost position it is locked into place by the masthead.

METHODOLOGY AND APPROACH

The purpose of the present study was to identify the effects of wind-induced vibrations on lamps in the high-mast illumination towers. Naturally, the main thrust was on obtaining experimental data from the site under consideration, but a significant amount of numerical preparation was required in order to plan and perform the tests and to assure the reliability of the data. The numerical studies had to be performed initially to derive predictions of expected tower and lamp vibrations and later for purposes of correlation with the available experimental information. Furthermore, it was important to be able to study a broad range of wind effects in order to identify which ones could be responsible for the observed damage, and such evaluations can be best performed either by well-controlled laboratory experiments or by numerical means. To obtain numerical results on the lamp motions, the entire tower needed to be analyzed because the wind forces cause the tower to vibrate and these vibrations are transferred to the lamps.

As briefly discussed earlier, the analysis of the high-mast illumination tower system consisted of an analytical-numerical phase and an experimental phase. The analytical study was carried out first to gain a better understanding of the tower's basic properties including stiffness, damping, and natural frequencies. When this information was available, simulated wind loads could be applied and the tower response observed (the motions computed in this study corresponded to the component of interest, such as the luminaires and lamps). These wind loads could be varied by changing the frequency and magnitude of the applied load and applying different loading patterns including uniform loads, concentrated loads, and mass proportional loads. This type of analysis should provide an adequate spectrum of expected values for the displacement velocities and accelerations of the luminaires and lamps. Because it is nearly impossible to exactly model the wind effects on the complicated head and ring assemblies, the second stage of the analysis included placing accelerometers on the ring assembly, luminaire, and lamp to observe the actual motion of the structure under field conditions.

In the analytical evaluation of the problem, use was made of the finite element method for the dynamic analysis of the structural system, and the program ADINA (*14*) was employed for this purpose. ADINA is a commercially available multipurpose finite element program for linear or nonlinear analysis of structural systems in static or dynamic domains of behavior in one-, two-, or three-dimensional space. The program can provide time histories of forces, strains, stresses, displacements, velocities, and accelerations, as well as natural frequencies and mode shapes of the structures.

In the following subsections several parameters that must be considered when analyzing a structure for wind-induced vibrations are discussed.

Boundary Layer Effects and Changes in Wind Speed

When dealing with tall structures such as towers, an important consideration is the variation of wind speed with height. It is

well known that roughness of the terrain retards the wind near the ground. The lower layers of air then retard those above them, which results in different wind speeds from the ground level until the retarding forces are diminished to zero (*4*). Several approximate equations have been developed by various researchers to provide a better understanding of the wind gradient, and one such equation is the power law wind formula (*5*) that is used to calculate wind speed at a height other than that at which the wind is measured.

Reynolds Number

The Reynolds number is an index that helps to define what type of flow characteristics may be expected and is a function of the wind velocity, the diameter of the tower, and the kinematic viscosity of air (*4*, *5*). In the case of a tapered cylindrical tower, the Reynolds number will vary with height because the diameter changes with height, and so does the wind speed, as mentioned previously.

Force Coefficients

The force coefficient (or shape facor) is a dimensionless parameter, included in most wind pressure equations, that makes it possible to calculate forces on many different structural shapes using the same general equation that can be written as

$$\text{Wind force} = CAq \tag{1}$$

where

- C = shape factor or force coefficient,
- A = area upon which the wind force acts,
- q = $(1/2)\sigma V^2$ is the dynamic head of wind, and
- σ = mass density of air.

Furthermore, the shape factor or force coefficient can be divided into two different types, drag coefficient (C_d) and lift coefficient (C_k).

Vortex Shedding

Winds acting on a cylindrically shaped tower may induce forces in several different directions. Naturally, the tower is expected to deflect in a direction parallel to the wind. But it may also vibrate in a direction perpendicular to the wind; this is due to the phenomena of vortex shedding (*5*). Vortex shedding occurs when the airstream separates on each side of a structure and vortices or eddies are formed alternately at each separation edge. The formation and detachment of each vortex or eddy induces a suction force at the separation points that alternates back and forth between the points. When these across-wind vibrations act concurrently with the along-wind vibrations, the deflection of cylindrical shapes such as the high-mast towers can become rather complicated. For simplicity the across-wind and along-wind vibrations are usually handled separately and then superimposed to describe the final behavior. The present study deals with wind speeds that vary from 0 to 60 mph and corresponding Reynolds numbers that vary between 30,000 and 1,700,000.

Strouhal Number

The pronounced regularity of such vortex wake effects is describable in terms of a nondimensional number, known as the Strouhal number, that can be determined as follows (*4*, *5*):

$$S = f_v D/V \tag{2}$$

where

- f_v = frequency of full cycles of vortex shedding,
- D = dimension of the body projected on a plane normal to the mean flow velocity,
- V = velocity of the oncoming flow, assumed laminar, and
- S = Strouhal number.

The value of S takes on different characteristic constant magnitudes depending on the cross-sectional shape of the prism being enveloped by the flow. There has been some question as to what value of D, the diameter, should be input in this equation for tapered structures. Several authors including Sachs (*4*) have recommended that the diameter at the tip of tapered structures be input. For the high-mast towers the tip outside diameter is approximately 7 in. or 0.60 ft. Introducing this and S = 0.2 into Equation 2 results in

$$V = 3f_v \tag{3}$$

It was observed by Penzien (*11*) that the maximum deflection response from vortex shedding occurred when the frequency of vortex shedding equaled one of the structure's lowest natural frequencies. This condition is termed a resonance condition, and it will have a certain "critical" wind velocity associated with it as seen from Equation 3. This critical wind velocity can be calculated for various modes of vibration, and the critical wind velocity for the lowest natural frequency of the present tower is accordingly 0.82 mph. Hence, at this wind speed the shedding frequency will equal the structure's natural frequency and the tower will interact with the flow.

ANALYTICAL-NUMERICAL APPROACH

The numerical approach adopted for the present study was the finite element method, as discussed in the literature (*1*, *13*). The actual structure is simulated by an approximate model that consists of discrete elements to which the loads are applied, and for the present analysis it was decided to approximate the tower as an assemblage of beam elements. One restriction that could not be overcome by any of the element types was the

exact modeling of the nonprismatic shape of the tower. The lack of such an available element can be made up for by using a large number of prismatic members and gradually decreasing the outside diameter of each element from the bottom of the tower to the top. For the present analysis, 57 two-node beam elements were employed to approximate the tower shaft and 10 elements to model the ring assembly and masthead.

The elements describing the tower shaft were connected end to end and assigned an average diameter. The elements describing the ring assembly had a constant cross-sectional area and diameter and were placed end to end to form a 47-in.-diameter circular section. Because of the inability to model a circular surface with a small number of straight beam elements, isoparametric beam elements were used to model the ring's circular shape.

To properly calculate the dynamic response, the mass density of the structure must be included. Rolled steel weighs 490 lb/ft^3, which corresponds to a mass density of 0.000734 slugs per cubic inch. Another important consideration in the finite element analysis is the weight of the masthead and luminaires. The masthead was modeled as a 475-lb concentrated load at the top of the tower shaft. Each luminaire weighs 65 lb and a concentrated load was placed at each luminaire location to approximate the luminaire weight (*1*).

One of the restrictions of using the finite element code is that loads can only be applied at the structure's nodal points. Hence a continuous load due to wind must be broken down into equivalent nodal loads. In this case a program was written to compute the diameter at each node and then calculate the distance between adjacent nodes. When these data are available for the entire structure, the tributary area can easily be calculated for each nodal point. If the tributary area is then multiplied by the wind load per unit area, the result will be the equivalent nodal load.

The behavior of high-mast towers subjected to unsteady wind forces is obviously dynamic. Similarly, the structural response to a dynamic load (i.e., resulting motions and bending moments) is also dynamic. For this particular case the variation of wind force with time is assumed to be known. Although the force may be oscillatory or irregular, it still must be a predescribed dynamic load. The choice of a suitable dynamic load is an important step in the analysis, and if the load selected is not similar to the actual loads on the structure the numerical results may not resemble the actual behavior.

In general, structural response to any dynamic loading is described in terms of the displacements, velocities, accelerations, and frequencies of the structure. A deterministic analysis leads to such time histories that correspond to the prescribed loading history. Other aspects of the response including stresses and strains are then determined from the previously established response patterns. Further information on dynamic analysis can be found in texts on the subject, for example, Newmark and Rosenblueth (*2*) and Clough and Penzien (*15*). Most ordinary structures have a damping ratio of less than 5 percent. Many references on tower structures and steel shapes assume a damping ratio of 2 percent. For the numerical analysis it was assumed that the high-mast towers have a damping ratio of 2 percent, but computer runs were also made using 1 and 3 percent damping. For structures with such low ratios of critical damping, it is often assumed that the damped and undamped natural frequencies are equal (*2*).

EXPERIMENTAL APPROACH

The main objective of the experimental phase of the project was to instrument the high-mast tower system so actual acceleration measurements could be recorded. These measurements could then be compared with the predicted finite element response to better understand the behavior of the tower and lamps. The basic setup of the required instrumentation is shown in Figure 2 and includes accelerometers, which convert the measurand [measured quantity or property (i.e., acceleration)] into usable electrical output. The number of points and the sampling rate were determined by the user through input to the computer. After it exited from the analogue-to-digital converter the signal was processed through the computer, an IBM-PC in this study, and the data were stored on a hard disk.

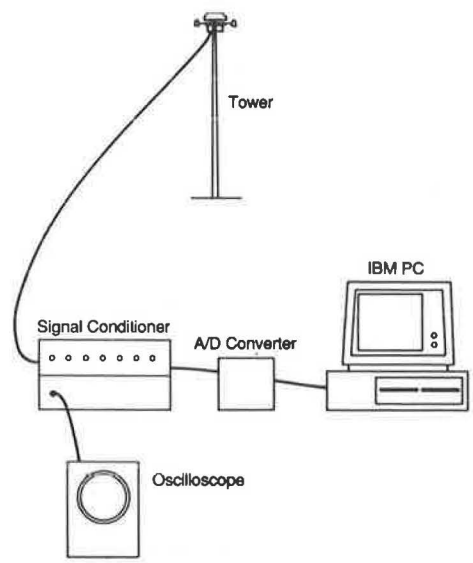

FIGURE 2 Experimental equipment configuration.

Each accelerometer of the type used in this study is capable of measuring acceleration in only one direction. Hence, to completely define three-dimensional motion at a point, three accelerometers must be used, one in each of the x, y, and z directions. Because of the tower height and the local topography it was decided not to place accelerometers on the tower shaft because that would have required special heavy equipment, and the points of interest were the lamps at the top of the tower. Fortunately, the ring assembly on which the lamps are located could be easily lowered to the ground and instrumented. The points of accelerometer installation were chosen such that they would provide the required experimental data and correspond with specific points identified in the finite element grid. Hence an appropriate correlation between the finite element model and the actual structure could be investigated. For ease of installation and better performance, the accelerometers were mounted in groups of three onto small aluminum plates and a total of four such devices were installed on the ring assembly, on a luminaire, and on the lower tip of one lamp. This approach also assured that three accelerometers on a plate were orthogonal to each other in each of the x, y, and z directions.

RESULTS

Results of Numerical Study

After several simple checks were made to assure that the tower was modeled properly, the approximated structure was subjected to several simulated extremely windy and gusty conditions and several calm or mild wind conditions. A summary of the results from the analytical-numerical study for different wind conditions is given in Table 1. These results indicate that even for random loads of high magnitude the tower does not reach excessive levels of acceleration or extreme values of displacement. A review of all of the numerical results indicated that the maximum acceleration at the top of the tower induced by simulated wind loads did not exceed 0.15 *g*. The numerical displacements did reach as much as 14 in. when subjected to extreme winds in excess of 45 mph; however, for a 120-ft-high tower a tip displacement of 14 in. is still less than 1 percent of the tower height, which is quite small for a cantilever structure under extreme wind conditions. The moderate and calm wind conditions caused a maximum acceleration of less than 0.05 *g* and displacements of less than 3 in.

TABLE 1 A SAMPLE OF PEAK NUMERICAL RESULTS

Record	Peak Wind Velocity (mph)	Maximum Along-Wind Displacement (in.)	Maximum Along-Wind Acceleration (*g*)
Extreme	46	14	0.15
Moderate	25	3.8	0.07
Calm	<15	2.2	0.03

Experimental Results

The second phase of this study included mounting accelerometers on an existing high-mast tower in Eagan, Minnesota. A total of 12 accelerometers were mounted on the ring assembly of the tower, on a luminaire, and on a lamp, and the tower motions were monitored on 5 different days in late October and early November 1984. The average duration of each record was 7 min with a sampling rate of 10 readings per second. Several other records were also taken with sampling rates of 100 readings per second. After processing all the raw data into acceleration-time histories the next step in the analysis was to process the data through a double integration scheme in order to compute the corresponding deflections.

On November 9, 1984, afternoon wind gusts were measured in excess of 25 mph, and the resulting filtered tower accelerations showed that the maximum horizontal lamp acceleration was 0.08 *g* and the corresponding peak displacement for the lower end of the lamp was approximately 2 in. These accelerations and deflections compare quite closely with the response from five other data sets recorded on that day. Figures 3 and 4 show the horizontal and vertical acceleration-time records for the lamp, respectively. Comparing these plots shows that the vertical accelerations were approximately 50 percent less than those in the horizontal direction.

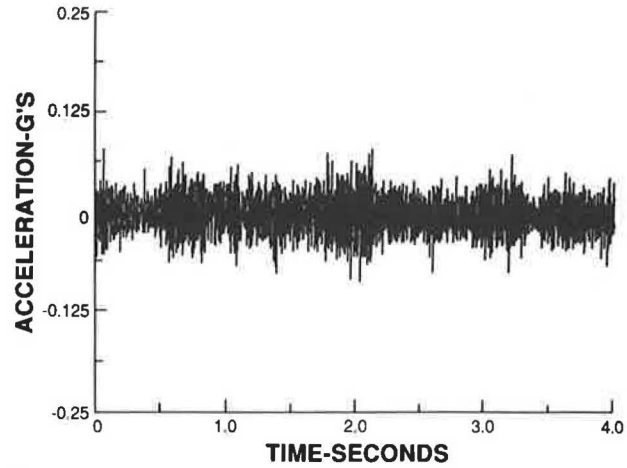

FIGURE 3 Horizontal acceleration-time history.

DISCUSSION OF RESULTS

Comparison of the results of the analytical-numerical study with the experimental data shows good agreement on almost all counts. The values calculated for the lowest natural frequencies of the structure were identical in both studies and the calculated values of maximum acceleration and displacement due to winds under 30 mph were also nearly identical. It appears quite likely that the motion of the tower is induced by a combination of both vortex shedding and along-wind vibrations, especially under fairly calm or steady wind conditions. An interesting point brought out in both phases of the study was the prevalence of first mode response. The tremendous fundamental mode response from the numerical study and the filtering of all frequencies except the fundamental in the experimental study also showed that almost all of the response occurred in the fundamental mode. A close examination of the numerical and experimental analyses results showed that even when the tower was subjected to very high winds the response was mainly at 0.4 Hz. The experimental displacement data indicated clearly that almost all of the horizontal response was in the first mode. Furthermore, both the analytical and the experimental results clearly show a level of acceleration of less than 0.20 *g* and displacements of less than 3 in. for a 30-mph wind.

Several burned out lamps were examined, and noticeable

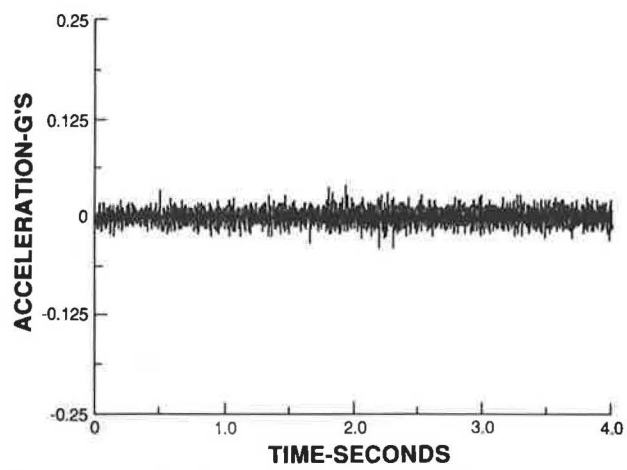

FIGURE 4 Vertical acceleration-time history.

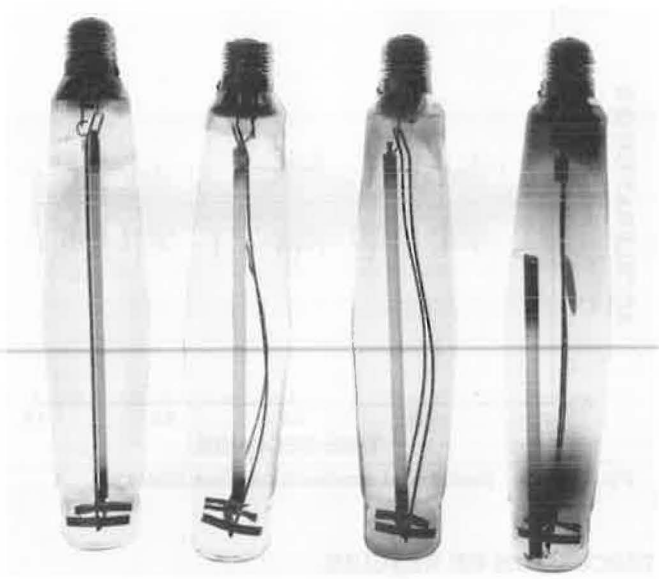

FIGURE 5 Damaged lamps.

nonrecoverable buckling deformations were very common among the small-diameter lead wires connected to the arc-tube inside the bulb, as shown in Figure 5, and it became clear that this buckling had been induced by thermal effects. When the 1000-watt lamps are lit they generate a large amount of thermal energy, and this heat will cause the small-diameter lead wires to expand and induce tensile stresses in the arc-tube. If the lamp is then subjected to a sudden decrease in temperature, when power to the lamp is terminated, the lead wires will contract, inducing compressive stresses in the arc-tube. These support wires inside the lamp are connected to the arc-tube by a pair of small metal cross-plates, and when the lead wires in the lamp buckle the cross-plates connected to them will rotate and may induce flexural stresses in the arc-tube. This combination of thermally induced flexural and axial stress cycling combined with low-level vibrations could lead to premature failure of the lamp by breaking the arc-tube or causing severe distortions at the arc-tube base, as seen in Figure 5. During the last few months an improved lamp has been introduced in which the two lead wires are placed in glass sleeves. At this time it appears that the performance of the lamp has been improved by the added stiffness of the glass sleeves.

CONCLUSIONS AND RECOMMENDATIONS

The present study was initiated to investigate possible wind-induced vibrations in lamps that are mounted on high-mast tower illumination systems. Initially, it was expected that such vibrations were the principal cause of excessive failure rates of the lamp. However, after preliminary data became available during the study, it appeared that mechanical vibrations could not be the only cause of such failures, and the following conclusions and recommendations were made.

1. As a result of this study it was concluded that the measured peak accelerations apparently were not the principal cause of the observed lamp damage.
2. No obvious problem was detected in regard to the tower design and its possible effect on lamp performance.
3. At this time it appears that a principal cause of severe stresses in the lamps is associated with thermally induced deformations in the lead wires that are located in parallel to the arc-tube. Nevertheless, it is quite reasonable to expect that low-magnitude vibrations would further enhance the failure rate because the dynamic effects will contribute to the state of stresses in the internal components of the lamp.
4. At this stage it is recommended to perform controlled experiments in the laboratory to investigate the separate effects of thermal cycling and mechanical vibrations and to determine more accurately their influences on the lamps, and also to study their combined influence on lamp performance.

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