Report on

Vibration Monitoring of Luminaires on the Burlington Cable-Stayed Bridge

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1. INTRODUCTION

1.1. Background

Construction on the Burlington cable-stayed bridge across the Mississippi River began in 1989 and the bridge was completed in 1994. From west to east the bridge consists of the following: (1) composite steel plate girder approach spans with exit and entrance ramps in Iowa, (2) 660-foot and 405-foot cable stayed spans, (3) a suspended span, and (4) prestressed concrete beam approach spans in Illinois (Petzold 1995). Forty-two light poles for roadway lighting were part of the bridge project and were installed in 1994 by a subcontractor to the primary contractor, Edward Kraemer and Sons.

1.2. Scope and Objective

Recently, attention has been drawn to failing luminaires on the Burlington Cable Stayed Bridge. Many light bulbs have failed prior to reaching their expected life. It was hypothesized that vibrations from vehicular traffic may have caused the premature failures. On November 15, 2001, with 10 mph winds and a 60-degree Fahrenheit temperature, several tests were conducted under different load conditions to study how traffic affects the light poles. The primary purpose of these tests was to understand the relationship between traffic induced vibrations and luminaire behavior and subsequent failures.

1.3. Literature Review

There was research interest in vibrations of roadway lighting during a 20-year period from the mid-1960s to the mid-1980s. During that time period manufacturers and departments of transportation initiated various studies to set standards for hardware and to solve field problems. Those studies are summarized below with respect to issues related to the Burlington bridge lighting.

1.3.1. Street Lighting Pole Vibration Research (and Discussion), Van Dusen and Wandler (1965)

In reporting the results of a pole vibration research project, the authors considered the following topics: service vibration frequencies, wind-vibration relationships, prediction of excessive vibrations, damping methods, and dynamic loads on luminaires. The authors tested aluminum and steel poles with a variety of luminaires with wind- and mechanically-induced vibrations. Within relatively steady state winds of 5 to 35 mph the authors measured peak luminaire accelerations of 1 g or less over a vibration frequency range of approximately 0.6 to 25 Hz. Within the envelope of vibrations, the authors suggested that a major source of excitation for higher modes was vortex shedding, and a source for first mode vibration was wind gusts.

Damping ratios for metal poles generally were in the 0.05% to 0.5% range. Motion between the pole foundation and unfrozen soil was a considerable source of damping and a reason why vibration problems were observed to be more severe for poles on concrete bridges. The authors determined that damping ratios less than 0.1% were ineffective but that ratios above 1% greatly reduced tendencies for high-amplitude oscillation. They suggested that lamps might be subjected to larger than 1-g forces because of relative lamp and socket motion. In summary, the authors stated that luminaires need not be designed for forces above 1 g and that occasional vibration problems be controlled with damping devices. The authors were unable to predict excessive vibrations.

Discussers welcomed the research as a first step in understanding vibrations in roadway lighting. One discusser described an installation with an extreme vibration problem due to wind. Vibration frequencies beginning with the first mode were 1.7 Hz, 10.9 Hz, 30 Hz, 59, Hz, and 90 Hz. Second mode accelerations at the luminaires were measured as high as 25 g. Evidence of failures was reported as early as 3 months after installation. Stiffer poles and dampers solved the problem.

Another discusser reported observing potentially damaging vibrations of the following types: vertical vibration caused by aerodynamic characteristics of the luminaire, vertical vibration caused by vertical vibration of a highway bridge, transverse vibration caused by aerodynamic characteristics of the pole, and fishtailing vibration caused by wind flow parallel with the luminaire support. Generally, the discusser observed that a pole would vibrate in the first mode without a luminaire, but in higher modes with an attached luminaire. In closing, the authors suggested that designing luminaires for more than 1 g may be warranted for severe conditions and they recommended an infinite number of cycles for fatigue design.

1.3.2. Street Lighting Luminaire Vibration (and Discussion), Van Dusen (1968)

In this second paper the author reported results of his luminaire vibration research. The author noted that wind generally excites a pole at a resonant frequency but that traffic may excite a pole and luminaire at non-resonant frequencies. Because there is little damping at the lower resonant frequencies of lighting structures, the amplitude of vibration can be quite large. Luminaires are usually mounted at a point of large displacement where acceleration is the significant measure of intensity. The resonant frequency of the pole will not necessarily be the same for wind- and traffic-induced vibration.

The author recommended the following two luminaire vibration test criteria: a simulation of an infinite number of cycles at 1 g (for performance in service) and a short duration, high intensity test at 4 g (for shipping). The author then discussed, at length, the mechanics of luminaire testing and the parameters for accelerated testing. During testing, typical observations of amplification at resonance ranged from 2 to 80, and fundamental frequencies were as low as 3 Hz but usually in the 10 to 15 Hz range. Fatigue cracking within the main load-bearing structure of the luminaire was a common vibration failure. In summary, the author recommended that testing should be completed on poles to ensure that there is no tendency for parts to loosen when a luminaire is subjected to 1 g vibration over a long time period and that the luminaire remain serviceable after a single 4-g event. Discussers again welcomed the research. The first discusser from a lighting manufacturer noted that vibration problems were likely to increase in the future because lighting was being used in open, rural environments with greater wind exposure, and weights of poles and luminaires were being reduced. The discusser mentioned that his company tested outdoor lighting to 1.5 g at 33 Hz and was having no vibration-related failures in the field.

Another discusser noted that the lamp base should be large with respect to the lamp itself so that the base would not be vulnerable to separation from the lamp. The discusser additionally suggested cushioned lampholders for areas of high vibration such as exposed bridges. In closing, the author suggested that cushioning be used with care. Although cushioning could isolate the lamp from high frequencies, the cushioning could have a low resonant frequency in the range of the light pole frequency.

1.3.3. Wind Induced Vibration in Light Poles, Ross and Edwards (1970)

With the support of the Texas Transportation Institute, the authors undertook work to go beyond the experimental work of Van Dusen by developing an analytical method for determining the response of a light pole to wind loading. The authors created a simple model of a typical light pole as an idealized structure of lumped masses interconnected by weightless springs in the plane of the pole and mastarm. The simple model allowed no out-of-plane vibration. Loading was limited to vortex shedding along a cylindrical pole. Although the model was limited, it was able to respond to the most severe vibrations that the authors observed for light poles in their natural environment. By experimenting with an actual pole, the authors determined a damping coefficient of 5% of critical damping for the first mode of vibration and used that value for their studies. In the analytical studies, the peak luminaire acceleration was slightly above 1 g. In general, the modal method of analysis described in the paper appeared to be promising as a means for solving for wind induced vibrations in light poles.

1.3.4. Luminaire Vibration Suppression Study, Burt and LeBlanc (1974)

A study was initiated by the Louisiana Department of Highways to find solutions to inadequate lamp life in luminaires located on bridges and overpasses. Low frequency vibrations were believed to cause lamps to loosen in their sockets and subsequently break. During the first testing phase, the authors measured traffic-induced accelerations on mercury vapor lamps in five luminaires located on overpasses in the Baton Rouge, Louisiana area. Accelerations measured on steel poles were 50 to 60% higher than comparable accelerations measured on aluminum poles. The largest measured, traffic-induced acceleration was 2.8 g on a steel pole placed near midspan of an overpass.

During the second testing phase, the authors systematically measured the performance of four pole-mounted inertial dampers in a laboratory. All of the dampers reduced duration and amplitude of low frequency vibrations reaching the lamps, but the heaviest damper, a

35-pound Alcoa Stockbridge type, had the best performance. The most efficient location for each damper was at the midpoint of the vertical portion of the pole.
In the final testing phase, the authors field-tested the dampers on the same poles monitored in the first phase. On average, the 35-pound Alcoa damper reduced vibrations 33%. In addition to the inertial dampers, the authors tested Fabreeka pads intended to isolate luminaires from high frequency vibrations that could cause breakage of internal lamp elements, but the pads had no significant damping effect. On the basis of the study the authors made three recommendations: (1) use only aluminum light standards, (2) record maintenance histories to identify locations of excessive lamp failures, and (3) install Alcoa dampers on poles with excessive lamp failures.

1.3.5. Vibration Testing of Luminaires, Van Dusen (1980)

In his third paper the author stated that fatigue effects were the leading cause of structural failure in outdoor lighting equipment. He noted that luminaire fatigue failures usually occur after luminaires have been in service one to six months and that luminaires accumulate millions of stress cycles per year. The author gave four criteria for a comprehensive luminaire vibration test: (1) a fatigue test to simulate an infinite number of cycles at 1 g, (2) a high intensity test of 1000 cycles at 4 g, (3) testing in each of the luminaire’s principal axes, and (4) testing at frequencies less than the fundamental frequency of the luminaire. After stating the criteria the author discussed details of testing and equipment. In closing he noted that testing is complex and must be conducted under carefully controlled conditions.

1.3.6. Dampening Vibration in Luminaire Structures, Keith and Edwards (1984)

After the opening of I-59 in Birmingham, Alabama, mastarms began failing due to traffic-induced vibrations. All of the mastarms were mounted on lightweight steel poles on continuous span bridges, and the poles with the most severe vibrations often were located within the center one-third of the bridge span.

The authors monitored a pole with severe vertical vibrations at the luminaire. An accelerometer mounted on the bridge deck did not give any indication of the vibration measured by an accelerometer on the luminaire. From the lack of correlation in the measurements the authors surmised that luminaire vibration was cumulative. Based on the earlier Louisiana study (Burt and LeBlanc, 1974), the authors experimented with two Alcoa 35-pound stockbridge type dampers mounted at various locations on the pole and mastarm. A single damper at 70% of the pole height provided the most damping, a reduction of 80 to 95% in duration and frequency of severe vibration. Conclusions of the research were the following: (1) Alcoa 35-pound dampers can be used to lessen vibrations, (2) a single damper should be mounted at 70% of pole height, (3) dampers attached with a mounting bracket should have a safety cable, and (4) lightweight pole structures should not be mounted on continuous span bridges.

1.3.7. Investigation of Lamp Damage in High-Mast Illumination Towers, Krauthammer and Rowekamp (1986)

After experiencing excessive failure rates for high pressure sodium lamps in 120-foot tall, high-mast illumination towers, the Minnesota DOT initiated a study to determine the cause of the failures. The lamp failures consisted of fractured arc-tubes, broken arc-tube bases, and severely deformed support wires. A preliminary evaluation of the problem indicated that the failures were not caused by electrical or related problems, and that wind-induced vibrations should be investigated.

Authors of the paper first studied the high-mast towers analytically by means of finite element analysis and then measured accelerations on a tower in Eagan, Minnesota. Both the analytical and experimental results were nearly identical for winds less than 30 mph. The tower vibrated primarily in the first mode, at 0.4 Hz, with accelerations less than 0.20 g and displacements less than 3 inches. It was clear that the accelerations and displacements should not have caused the excessive lamp damage. After examining the failed lamps the researchers decided that thermal effects due to switching lamps on and off caused buckling of lead wires inside the lamps, and the combination of deformed lead wires and low-level vibrations could lead to failure of the lamps. The researchers concluded that wind vibrations were not the primary cause of the lamp failures and that the lamps should be tested to study the combined effects of deformed lead wires and vibrations.

1.3.8. Fatigue Testing and Failure Analysis of Aluminum Luminaire Support Structures, Johns and Dexter (1998)

The authors' research was in response to fatigue failures at the bases of light poles along Route 147 in southern New Jersey. The failures involved two types of poles: straight aluminum light poles mounted on breakaway transformer bases along the highway near a bridge and aluminum light poles with cantilevered mastarms mounted on the bridge barrier rail. Because failures occurred both on and off the bridge the authors focused on wind vibrations.

The authors investigated two wind conditions: natural wind gusts and vortex shedding. For vibrations caused by vortex shedding it was possible to limit the range of wind velocities to 10 to 35 mph. Based on correlation of the velocities with natural frequencies the authors predicted that vortex shedding could excite vibrations in third through fifth modes for the straight poles and fifth through seventh modes for the poles with cantilevered mastarms. The manufacturer's internal damper in the straight poles was designed for the second mode and probably was ineffective at higher modes.

After fatigue testing and examination of specimens from the failed luminaire bases, the authors concluded that the stress ranges at the failures were above 12.2 ksi. For wind loads caused by natural gusts they recommended the latest AASHTO sign specifications and for vortex shedding they recommended a procedure involving finite element analysis. Using their recommendations for a straight pole, the authors calculated that the critical stress range was well above the allowable fatigue stress and concluded that the pole base should have failed.

1.3.9. Summary

Since the mid-1980s there has been focused research on vibration and fatigue of traffic signal and sign structures. At this time, it is recognized that both wind and traffic can cause vibration problems in roadway lighting and that the more severe problems usually are on bridges where both wind and traffic provide exciting forces. Lighting manufacturers are primarily concerned with damaging fatigue that can cause structural failure of luminaires, but departments of transportation also are interested in excessive lamp failures. Usually excessive lamp failures are found in bridge lighting installations for poles located away from bridge supports.

Primary vibrations of luminaire-pole structures are considered low frequency and generally have been measured in the 0.6 to 25 Hz range. Peak accelerations of luminaires and lamps usually are less than 1 g. Although short lamp life often is attributed to traffic vibrations, the short life also could be due to lamp and luminaire design. The usual manufacturer's vibration testing of luminaires considers fatigue of the luminaire but not lamp life associated with the vibration.

Studies of retrofit measures for bridge lighting installations with short lamp life have focused on the addition of dampers rather than other measures such as replacement of luminaires or poles. The dampers that have worked well are the inertial type (e.g., Alcoa stockbridge dampers in the 15- to 35-pound sizes). Optimal damper placement has been determined at 50 or 70% of the height of the pole, depending on the specific installation. In the one study summarized previously in which elastomeric isolation pads were tried, the pads proved to be ineffective. However, some departments of transportation now use the isolation pads to retrofit lighting installations with vibration problems and routinely specify the pads for new bridge lighting installations.

2. DESCRIPTION OF SUBJECT LIGHT POLE

The original Burlington Bridge lighting design placed the forty-two light poles for lighting efficiency but without regard to the locations of superstructure supports. Each of the forty-two light poles is anchor-bolted directly to the bridge or ramp barrier rail at 2.5 feet above the roadway and extends to the luminaire mounting height at an elevation of 40 feet. Each pole is a round steel tube, tapered and bent to form a mastarm, with a welded base plate and welded 2-3/8 inch diameter tube extension at the top for mounting the luminaire.

Because of the concern that typical high pressure sodium (HPS) lamps would attract mayflies and cause a slippery roadway surface during late spring and early summer, the bridge lighting designer selected low pressure sodium (LPS) lamps. LPS installations are unusual in Iowa except on Mississippi River bridges.

Each LPS luminaire is 65 inches long, weighs 52 pounds, and has an effective projected area of about 1.5 square feet. The luminaire has a slipfitter connection that fits over the tube extension at the top of the light pole. Inside the luminaire there is a socket for the LPS lamp and a wire loop and spring for supporting the loose end of the lamp. The lamp is secured by inserting its pins in the socket and twisting the lamp. A 180-watt LPS lamp is relatively long and heavy (larger and heavier than a baseball bat).

During November 1996 the Washington, Iowa maintenance office contacted the Office of Bridges and Structures regarding difficulty in maintaining the bridge lighting. At that time, it appeared that the major problem was a short tube extension on each pole. The extension was 8 inches long, but it needed to be at least 14 inches long for the two clamps in the luminaire slipfitter connection. The maintenance office indicated that light pole vibrations would shake the luminaires and damage the lamps. One luminaire and twenty-six lamps needed to be replaced during the year previous to November 1996. Because the problem appeared to be an installation error, the Southeast Iowa Transportation Center contacted Edward Kraemer and Sons, requesting that the situation be corrected.

Evidently, as a result of the 1996 request, the bridge contractor or subcontractor repaired the poles by increasing the lengths of the tube extensions and reinstalling the luminaires properly. However, problems with the lighting persisted. In 1999, many lamps were continuing to fail at an early age and, in addition, some sockets were failing due to arcing at the lamp pin connections. Although the problems may have been more severe on the ramps and Iowa approach spans, there were problems throughout the LPS bridge lighting. Unfortunately, there were no available records to indicate if there were specific locations with higher or lower lamp failure rates.

Again maintenance personnel from southeastern Iowa requested help from the Office of Bridges and Structures. The request led to the vibration monitoring described in this report.

3. MONITORING PLAN

After a review of the literature, consideration of luminaire and pole manufacturers' advice, and consideration of the Iowa DOT's lighting consultant's advice, it was thought that the most promising options for retrofitting the Burlington bridge lighting were to replace the LPS luminaires with approximately equivalent but vibration resistant HPS luminaires and to attach dampers to the light poles as needed. Although replacing all of the luminaires would require changes to the two lighting circuits for the bridge, replacing a single luminaire on each circuit for vibration monitoring would not alter circuit performance sufficiently to cause problems. For test purposes the district maintenance officer agreed to purchase two HPS luminaires and two pole dampers.

To test the greatest variety of combinations within the limits of two HPS luminaires and two pole dampers, the following plans were developed.

- Replace a luminaire on the north lighting circuit and mount a damper on the same pole. The pole selected for the retrofit was on the curved exit ramp to Burlington.
- Replace a luminaire on the south circuit and mount a damper on an adjacent pole. The selected poles were on the cable-stayed span on the Iowa side of the main bridge tower.

The plan permitted the comparison of an existing pole and LPS luminaire with three retrofitted poles: a pole with a new HPS luminaire (POLE 2), a pole with both a new HPS luminaire and a damper (POLE 5), and a pole with an existing LPS luminaire and damper (POLE 3).

Based on manufacturer's and lighting consultant's advice, the replacement HPS luminaire had the following specifications:

- GE Lighting Systems, Inc. product M-400A with cutoff optics.
- 250 watts; HPS light source; 480 volts; HPF reactor or lag ballast.
- No PE function; non plug-in/none ignitor mounting.
- Flat glass lens type; medium, cutoff, Type III IES distribution type; fiber gasket filter.
- Suitable for severe vibration up to 3 Gs; ballast not mounted on Powr/Door module.
- 32 inches long (approx.); 33-39 pounds; 1.1 square foot maximum effective projected area.

From the three dampers available from the pole manufacturer, an external vibration damper was selected from Valmont Industries, Inc. (part number ABS3670). The damper is a capped, 2-3/8-inch diameter tube 12 inches long containing a loose 1-1/2-inch diameter steel rod with plastic end caps. A slotted aluminum extrusion bracket provides for a strap or hose clamp attachment to the side of the light pole.

4. TESTS AND TEST PROCECURE

Accelerometers for measuring vibration were installed at specified locations on the test poles prior to testing. A total of five light poles were instrumented. Notations for each general class of tests conducted (e.g., Mainline Test 1, Mainline Test 2 and Ramp Test) are given in Fig. 1. Accelerometers were typically installed at three locations on each of the poles and connected to a central data acquisition system. The accelerometers were placed on the luminaire casing, on the middle section of the pole approximately 28 ft from the base, and on the base of the pole as shown in Fig. 2. A typical photograph of the bridge is shown in Fig. 3. Also, typical photographs of the installment can be seen in Fig. 4.

Wind speeds in the morning, during testing on the ramp and approach span, were approximately 10 mph (+/- 3 mph). Wind speeds in the afternoon, during testing on the cable-stayed span, were approximately 14 mph (+/- 4 mph). Generally, winds were from the southwest but varied from west-southwest to south-southwest. The wind speed values are a combination of site-measured values and AccuWeather published values.

Tests could generally be divided into two categories: ‘Free Vibration’ and ‘Forced Vibration’. For the Free Vibration tests, data from the accelerometers were recorded when there was no traffic in the vicinity of the poles. For the Forced Vibration tests, data were recorded with traffic in the vicinity of the poles. Measurements were made under various traffic conditions (i.e., different vehicle types and combinations, etc.).

5. DATA ANALYSIS

The first step in the data analysis process consisted of a basic statistical analysis and graphical presentation using Microcal Origin. Accelerations obtained from each accelerometer were plotted with respect to time. From this, peak positive and negative accelerations were obtained from the test statistics. Fast Fourier Transform (FFT) analyses were then performed, after which frequency amplitudes were plotted with respect to frequency for each accelerometer. The FFT analysis employs the principle of the Fourier integral in discrete form to transfer a function or data from the time domain to the frequency domain. From the FFT results, the dominant vibration frequency from each test was determined by selecting the frequency that had the highest amplitude. Examples of acceleration and frequency content data are given in Figs. 5 and 6. It should be noted that the vertical scale in Fig. 6 varies. For example, the vertical scales used in graph (e) of Fig. 6 were from 0 to 80 while from 0 to 2.0 were used in graph in (f) in Fig. 6. This was necessary as the vertical scale is relative to a specific test and, as such, there is no relationship between tests.

6. TEST RESULTS

The data collected from these tests indicates that installing a damper on the exit ramp light pole tended to reduce the acceleration level. However, it was found that installing dampers on main span pole had no impact on acceleration levels. It should also be pointed out that it was also qualitatively noted during testing that vibration of the bridge, and also in the light poles, tended to continue for a relatively long duration after the test load had passed. Unfortunately, the testing procedure used, specifically the fact that the bridge was continuously open to traffic, did not allow for quantification of the length of these vibrations. Several general observations can be made from the data taken during the testing as follows:

- Peak accelerations at pole bases on the ramp and approach span were consistently higher than on the cable-stayed span (+/- 1.06 g vs. +/- 0.02 g to 0.03 g). Observations made from standing on the bridge decks during testing also indicated higher vibrations on the ramp and approach span. This suggests that traffic has more of an effect on the ramp and approach span than on the cable-stayed span.
- Peak accelerations on poles on the ramp and approach span were consistently higher than on the cable-stayed span (+/- 0.14 to 0.31 g vs. +/- 0.06 to 0.09 g). This suggests that the poles on the ramp and approach span are responding to the higher traffic vibrations.
- The luminaire, pole, and base measurements for peak acceleration for the LPS pole without a damper and the LPS pole with a damper were nearly the same.

This suggests that the damper was ineffective in reducing accelerations for the existing LPS liminaire and pole installation.

- The LPS luminaries on the cable-stayed bridge had lower peak accelerations than the LPS liminaire on the approach span (+/- 0.21 to 0.40 g vs. +0.68/-0.48 g). This suggests that the LPS luminaries on the approach span and ramps are responding to the higher traffic vibrations on the ramp and approach span than on the cable-stayed span.
- Peak accelerations and dominant frequencies on the LPS luminaries and bulbs were essentially identical. This suggests that the bulbs were not moving independently of the luminaries under the traffic vibrations on the cable-stayed span.

Table 1 shows typical data obtained from each accelerometer and Table 2 shows how the data were compared.

7. DISCUSSION

It was expected initially that the traffic-induced vibrations were the primary cause of the luminaire failures. However, due to the relatively low vibration levels obtained from the test, it appears that mechanical vibrations alone were not the cause of such failures. It is thought that wind-induced excitation, unlike the traffic-induced vibration monitored during this testing, might have a significant impact on the light system failure. Therefore, more study to understand the behavior of the light pole may be needed.

The most recent research by Johns and Dexter (1998), which follows the direction of the new AASHTO sign support specifications (2001), indicated that wind effects due to vortex shedding could be expected for wind velocities in the 10-35 mph. During testing, wind speeds were at the low end of that range, and therefore the effects of vortex shedding during the testing could have been minimal. Changes in wind velocity were gentle, and therefore the effect of wind gusts may have been minimal as well.

Johns and Dexter (1998) indicated that wind should excite higher pole vibration modes. The dominant modes measured on the Burlington Bridge ranged from low to high, but most were at lower modes. Van Dusen (1968) stated that wind generally excites a pole at a resonant frequency but that traffic excites poles at non-resonant frequencies. It appears that the frequency measurements indicate that traffic rather than wind was the driving force for the measured vibrations.

Although not completed, with the data collected, tools could be developed to more fully understand the dynamic behavior of the light pole under traffic loadings for further retrofit design. For example, an envelope of dominant frequencies could be plotted to establish the expected behavior of the light pole. This envelope could serve as a guide for defining acceptable vibration limits. This information could be used by engineers to design a light pole system to accommodate and withstand the vibration intensities within the envelope.

Based on the test results, it is difficult to give a definitive answer about whether the new luminaires will or will not improve the situation. Although some useful information was obtained from the single-day test, the test results give no definitive basis for determining if the previous bulb problems have been caused by what could be described as global (e.g., pole and/or luminaire) vibration problems or local (e.g., bulb wiring, bulb connection, etc.) vibration problems. It is known that the 'g' levels measured on the luminaries were relatively low given the bulb specifications. However, it is possible that the excitation that has caused the previous problems were more extreme than the excitation created during this data collection. It should also be pointed out that the bulb did not experience differential vibrations relative to the luminaire. This would suggest that the bulb is not "bouncing around" in the luminaire.

8. RECOMMENDATION

It is recommended that complete records be kept on luminaire failures in the future. This would include noting when bulbs are replaced, the location of the failed bulbs, and the approximate duration the luminaire had been in service. Once more detailed records have been obtained, a more complete work plan could be developed to focus on locations more prone to failures. In addition, additional monitoring could allow the impact of wind loads to be assessed.

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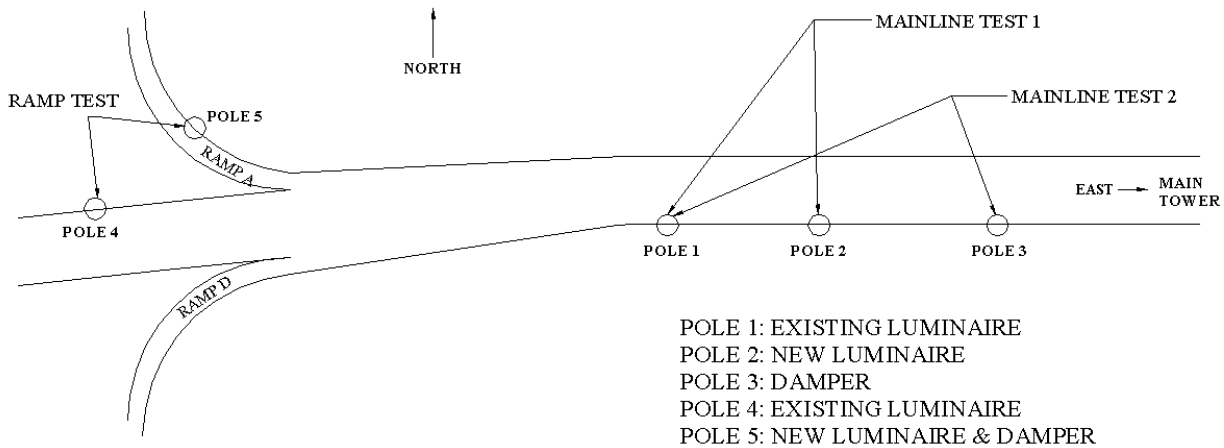


Figure 1. Condition and location of poles.

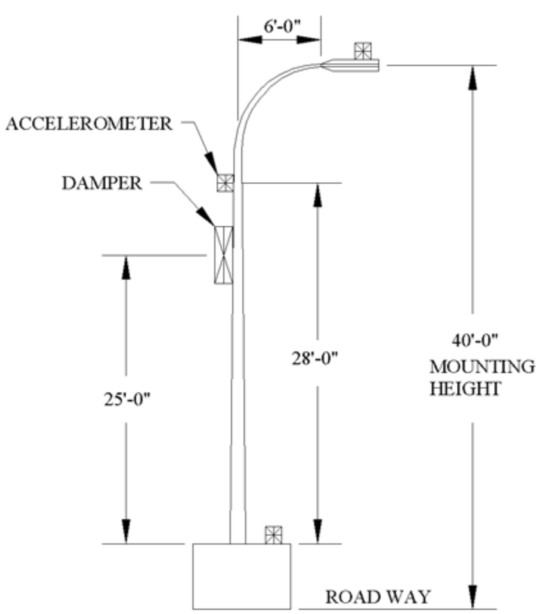


Figure 2. Location of accelerometer and damper.

Figure 3. Typical photograph of bridge (looking to the East).

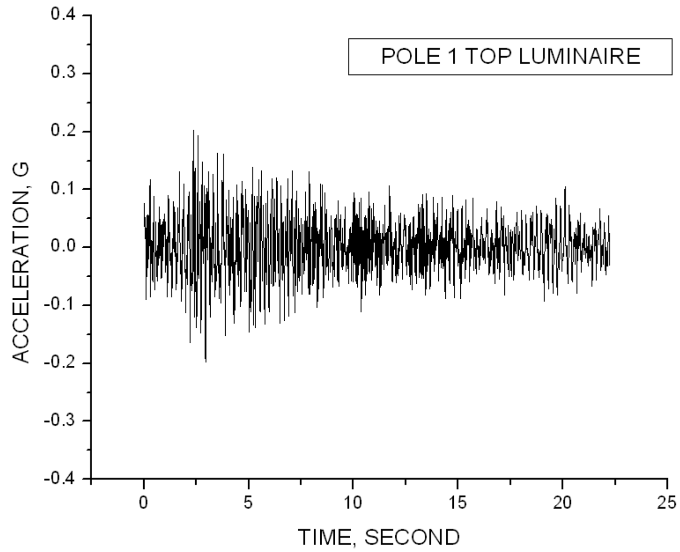
(a) Existing LPS Luminaire

(b) New HPS Luminaire

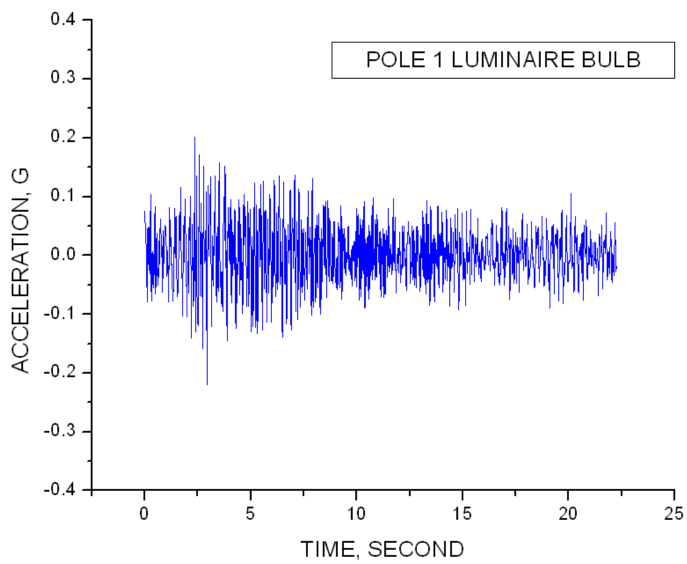
(c) Accelerometer

(d) Damper

Figure 4. Typical photographs of installment.

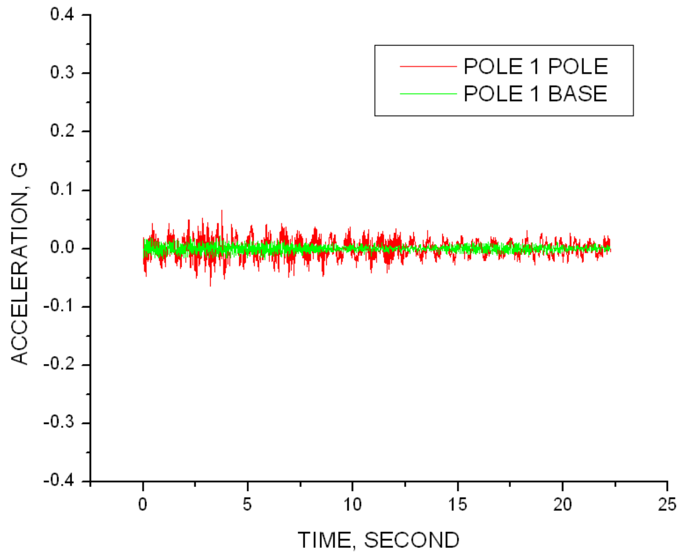


(a) Mainline Test 1, Pole 1 Top Luminaire

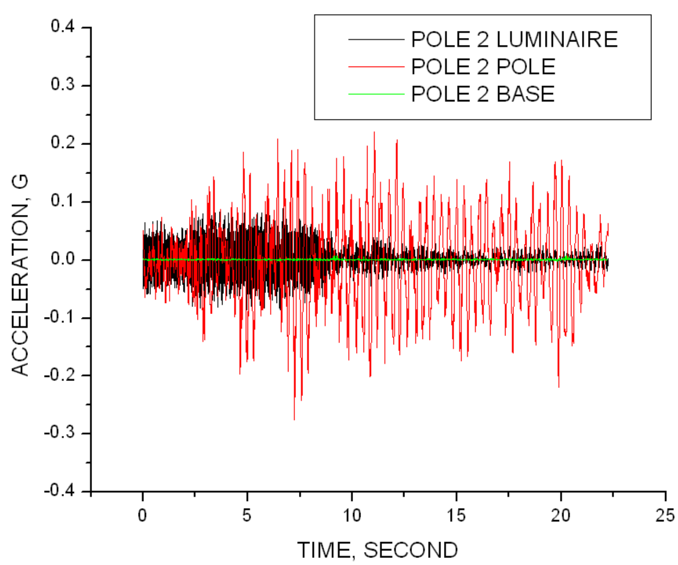


(b) Mainline Test 1, Pole 1 Luminaire Bulb

Figure 5. Typical acceleration data.

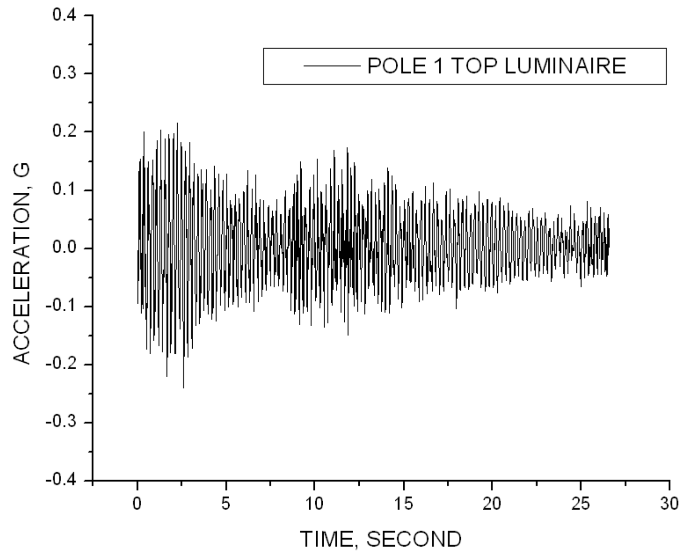


(c) Mainline Test 1, Pole 1 Pole and Base

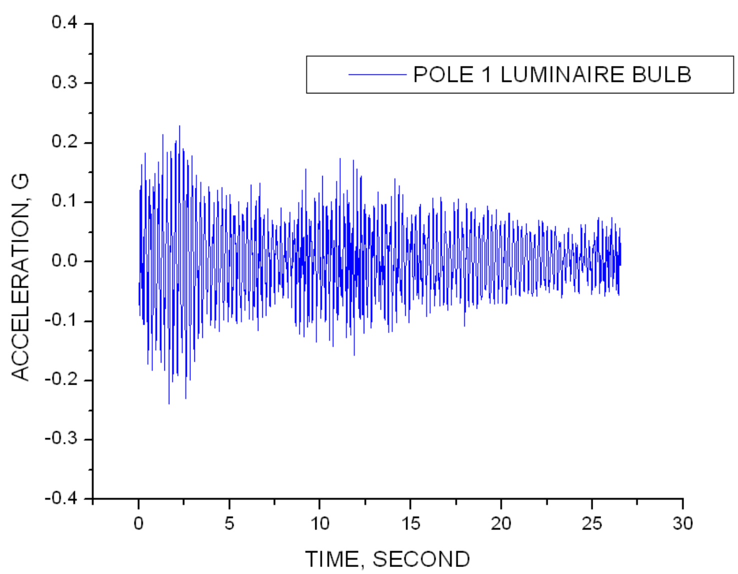


(d) Mainline Test 1, Pole 2

Figure 5. Typical acceleration data – continued.

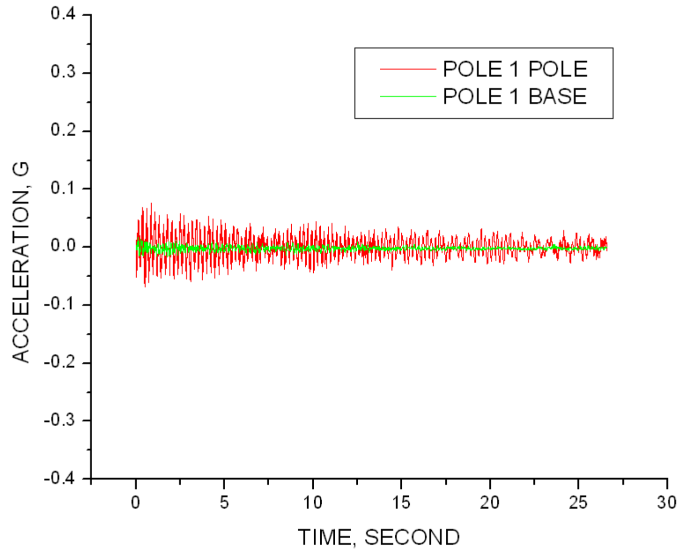


(e) Mainline Test 2, Pole 1 Top Luminaire

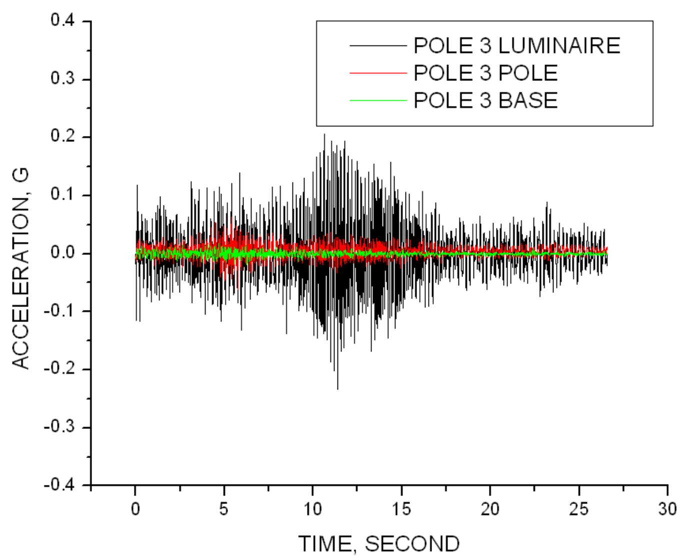


(f) Mainline Test 2, Pole 1 Luminaire Bulb

Figure 5. Typical acceleration data – continued.

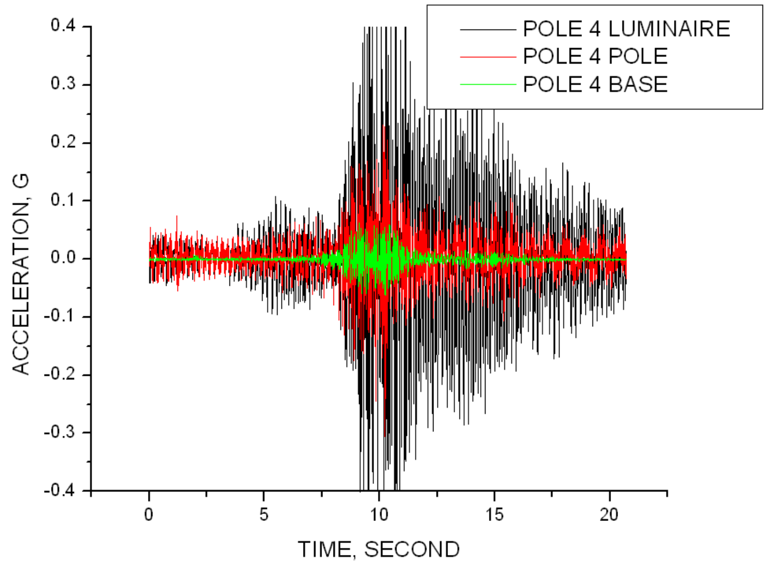


(g) Mainline Test 2, Pole 1 Pole and Base

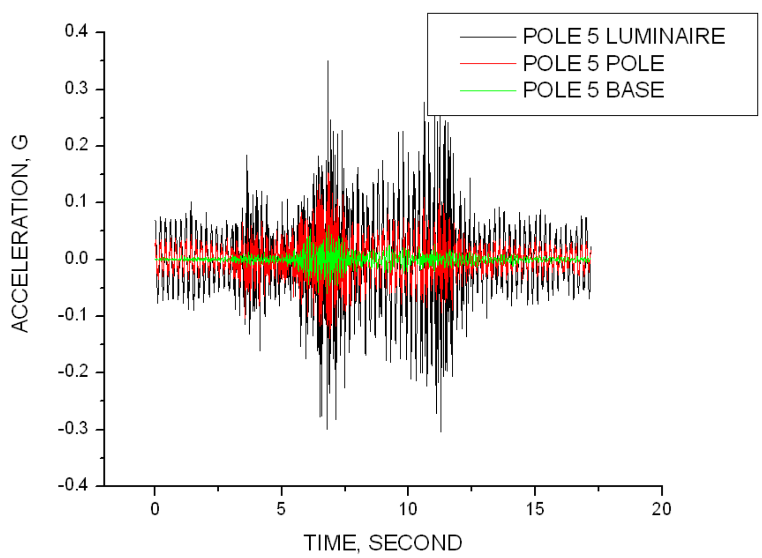


(h) Mainline Test 2, Pole 3

Figure 5. Typical acceleration data – continued.

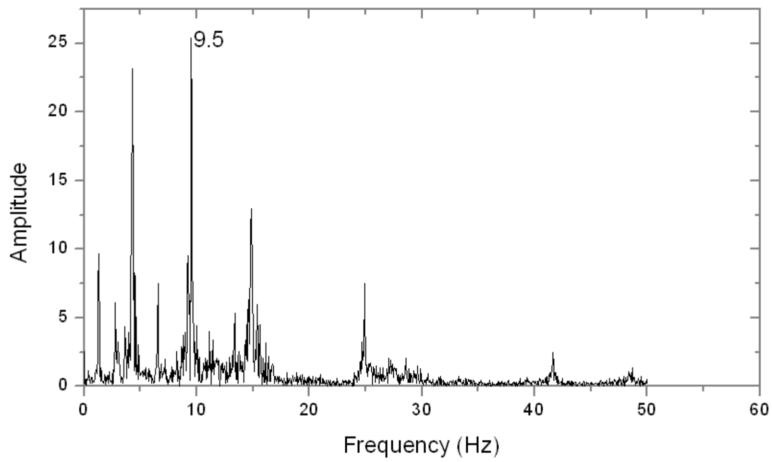


(i) Ramp Test, Pole 4

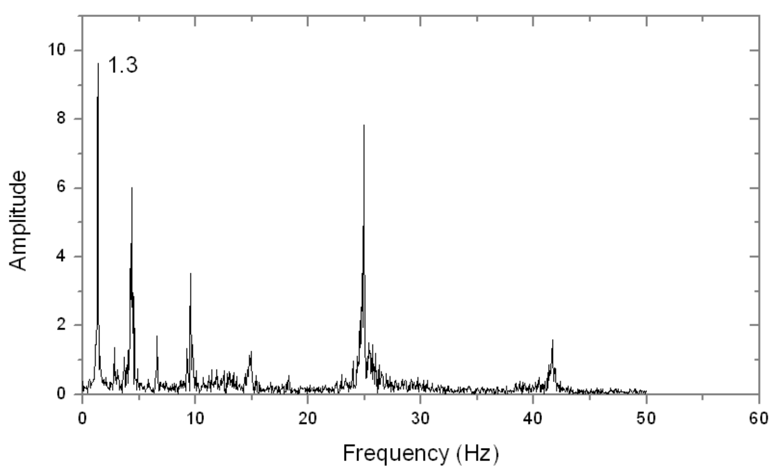


(j) Ramp Test, Pole 5

Figure 5. Typical acceleration data – continued.

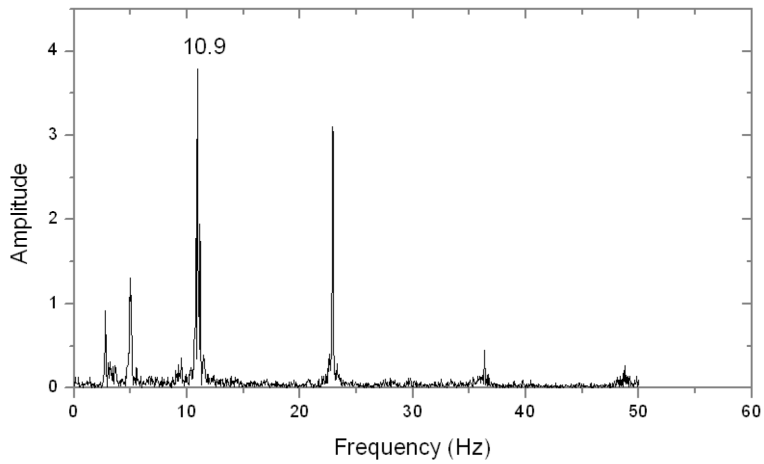


(a) Mainline Test 1, Pole 1 Luminaire

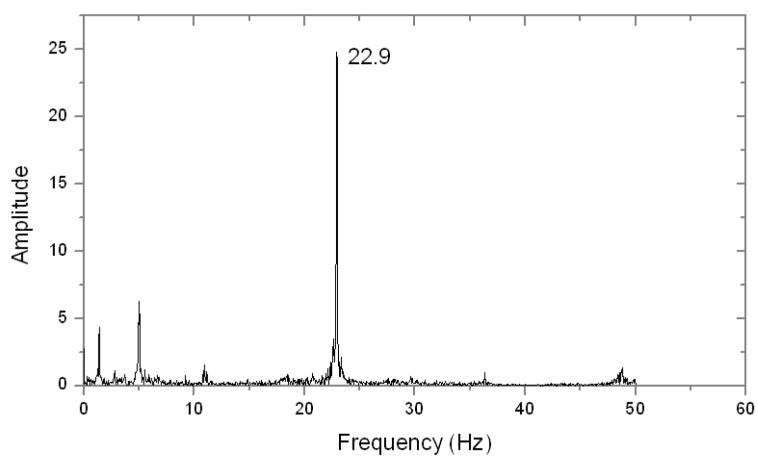


(b) Mainline Test 1, Pole 1 Pole

Figure 6. Typical frequency content data.

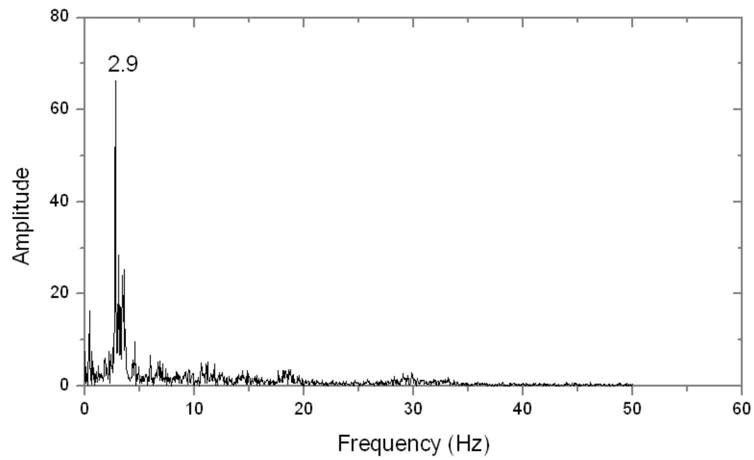


(c) Mainline Test 1, Pole 1 Base

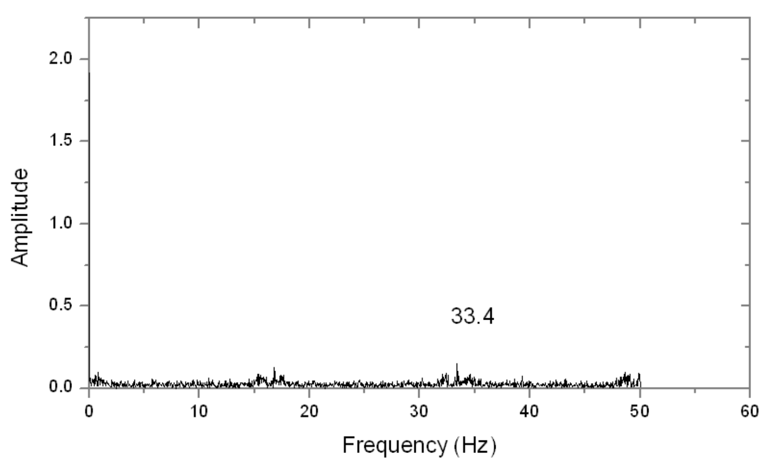


(d) Mainline Test 1, Pole 2 Luminaire

Figure 6. Typical frequency content data – continued.

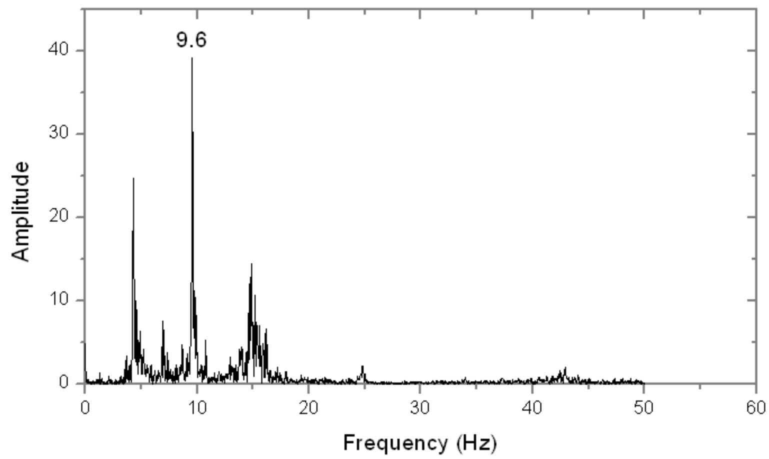


(e) Mainline Test 1, Pole 2 Pole

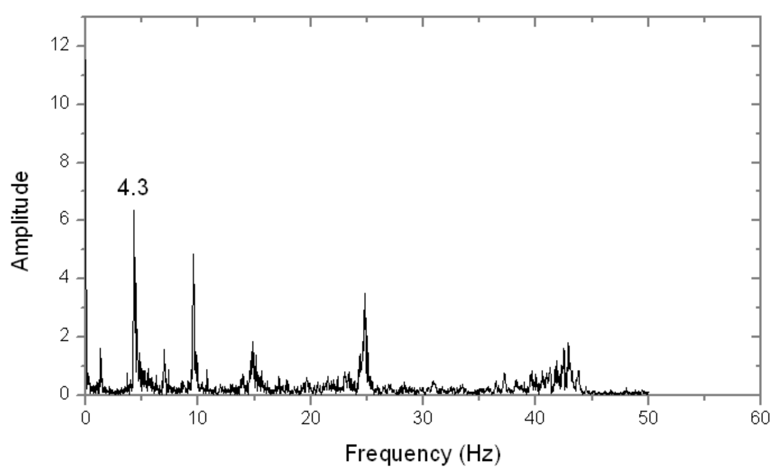


(f) Mainline Test 1, Pole 2 Base

Figure 6. Typical frequency content data – continued.

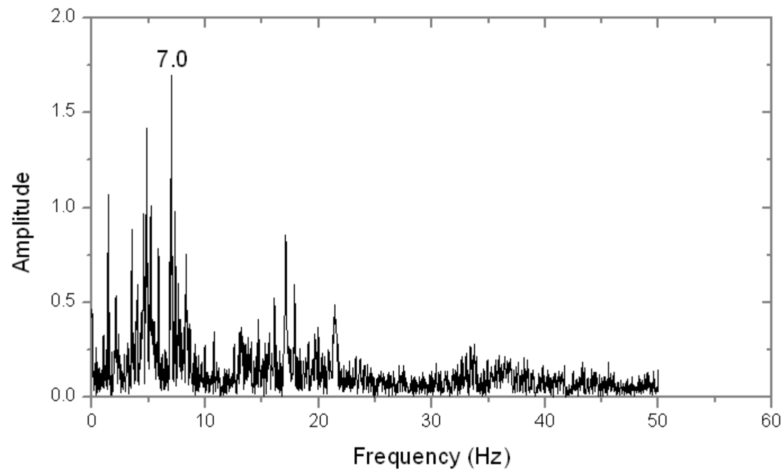


(g) Mainline Test 2, Pole 3 Luminaire

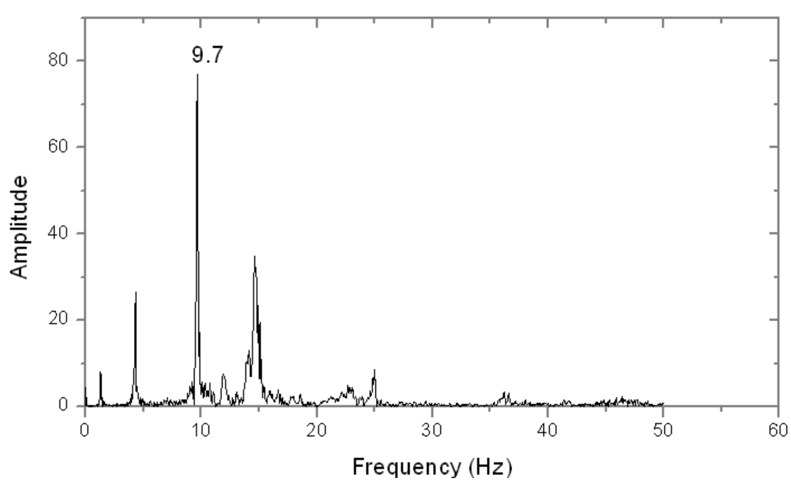


(h) Mainline Test 2, Pole 3 Pole

Figure 6. Typical frequency content data – continued.

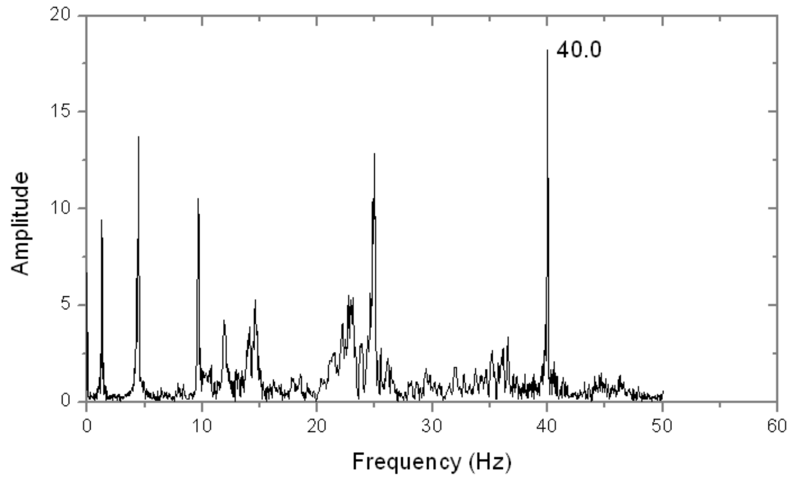


(i) Mainline Test 2, Pole 3 Base

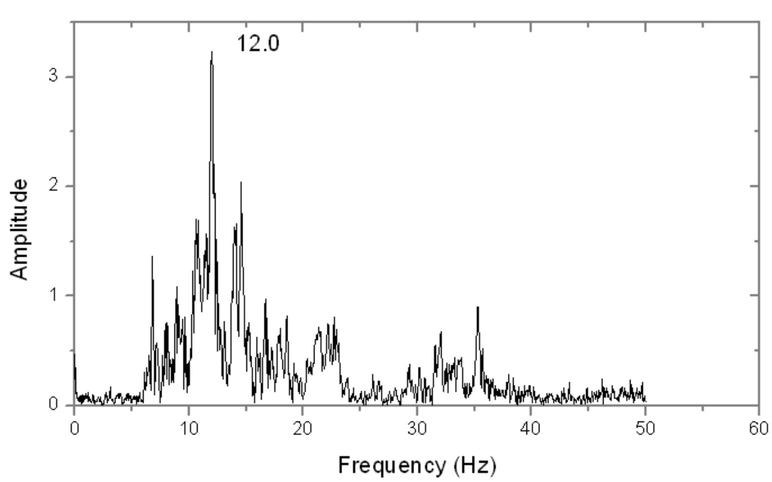


(j) Ramp Test, Pole 4 Luminaire

Figure 6. Typical frequency content data – continued.

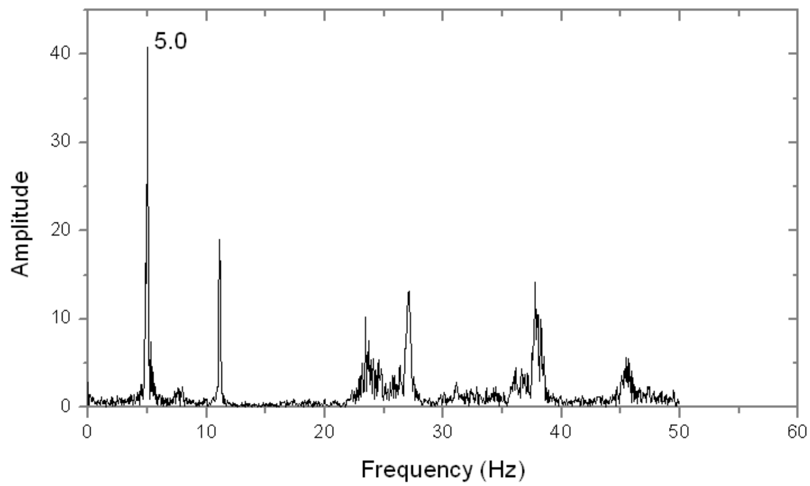


(k) Ramp Test, Pole 4 Pole

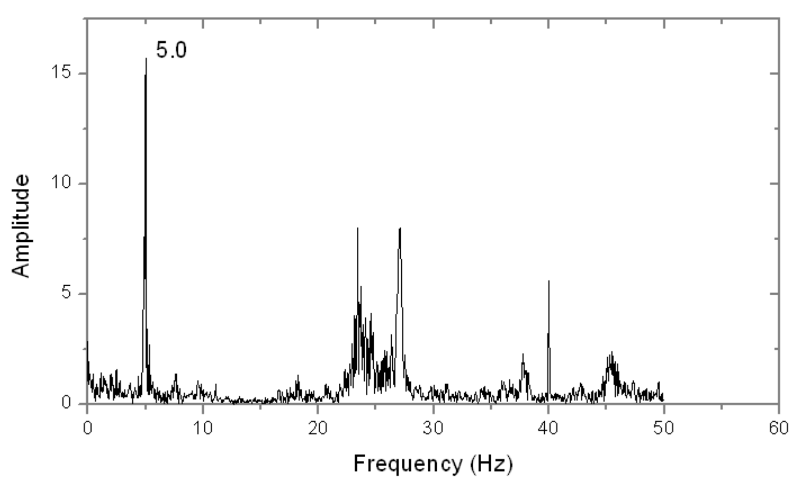


(l) Ramp Test, Pole 4 Base

Figure 6. Typical frequency content data – continued.

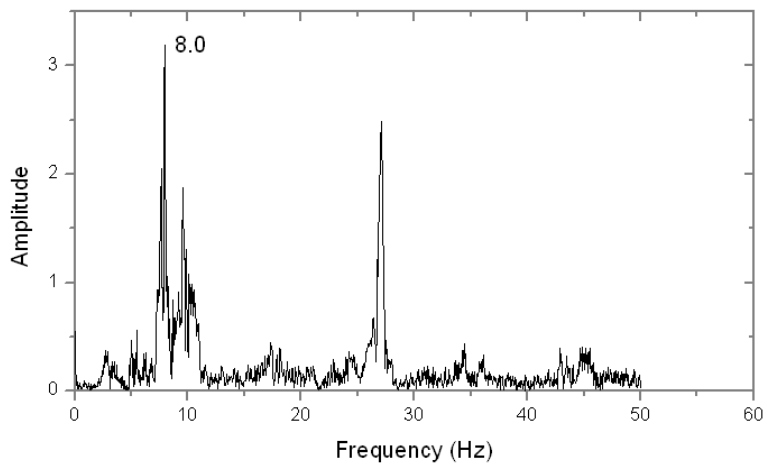


(m) Ramp Test, Pole 5 Luminaire



(n) Ramp Test, Pole 5 Pole

Figure 6. Typical frequency content data – continued.



(o) Ramp Test, Pole 5 Base

Figure 6. Typical frequency content data – continued.

Table 1. Typical Data Analysis

(a) Mainline Test 1

Accelerometer	Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 1 TOP	0.33	-0.40	9.5
POLE 1 BULB	0.32	-0.39	9.5
POLE 1 POLE	0.08	-0.09	1.3
POLE 1 BASE	0.03	-0.03	10.9
POLE 2 TOP	0.32	-0.30	22.9
POLE 2 POLE	0.09	-0.07	2.9
POLE 2 BASE	0.02	-0.03	33.4

(b) Mainline Test 2

Accelerometer	Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 1 TOP	0.21	-0.24	4.3
POLE 1 BULB	0.22	-0.24	4.3
POLE 1 POLE	0.07	-0.07	4.3
POLE 1 BASE	0.02	-0.02	3.5
POLE 3 TOP	0.20	-0.23	9.6
POLE 3 POLE	0.06	-0.06	4.3
POLE 3 BASE	0.02	-0.02	7.0

(c) Ramp Test

Accelerometer	Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 5 TOP	0.37	-0.30	5.0
POLE 5 POLE	0.15	-0.14	5.0
POLE 5 BASE	0.06	-0.06	8.0
POLE 4 TOP	0.68	-0.48	9.7
POLE 4 POLE (small)	0.23	-0.31	40.0
POLE 4 BASE	0.06	-0.06	12.0

Table 2. Typical Data Comparison

(a) Mainline Test 1

Accelerometer Location		Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 1 TOP		0.33	-0.40	9.5
POLE 2 TOP		0.32	-0.30	22.9
	Difference	0.01	-0.10	-13.4
POLE 1 POLE		0.08	-0.09	1.3
POLE 2 POLE		0.09	-0.07	2.8
	Difference	-0.01	-0.02	-1.5
POLE 1 BASE		0.03	-0.03	10.9
POLE 2 BASE		0.02	-0.03	33.4
	Difference	0.01	0.00	-12.5

(b) Mainline Test 2

Accelerometer Location		Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 1 TOP		0.22	-0.24	4.3
POLE 3 TOP		0.21	-0.23	9.6
	Difference	0.01	-0.01	-5.3
POLE 1 POLE		0.06	-0.07	4.3
POLE 3 POLE		0.06	-0.06	4.3
	Difference	0.00	-0.01	0.0
POLE 1 BASE		0.02	-0.02	3.5
POLE 3 BASE		0.02	-0.02	7.0
	Difference	0.00	0.00	-3.5

(c) Ramp Test

Accelerometer Location		Peak Positive Acceleration, g	Peak Negative Acceleration, g	Dominant Frequency, Hz
POLE 5 TOP		0.37	-0.30	5.0
POLE 4 TOP		0.69	-0.53	9.7
	Difference	-0.32	-0.23	-4.7
POLE 5 POLE		0.15	-0.14	5.0
POLE 4 POLE (small)		0.23	-0.31	40.0
	Difference	-0.08	-0.17	-35.0
POLE 5 BASE		0.06	-0.06	8.0
POLE 4 BASE		0.06	-0.06	12.0
	Difference	0.00	0.00	-4.0